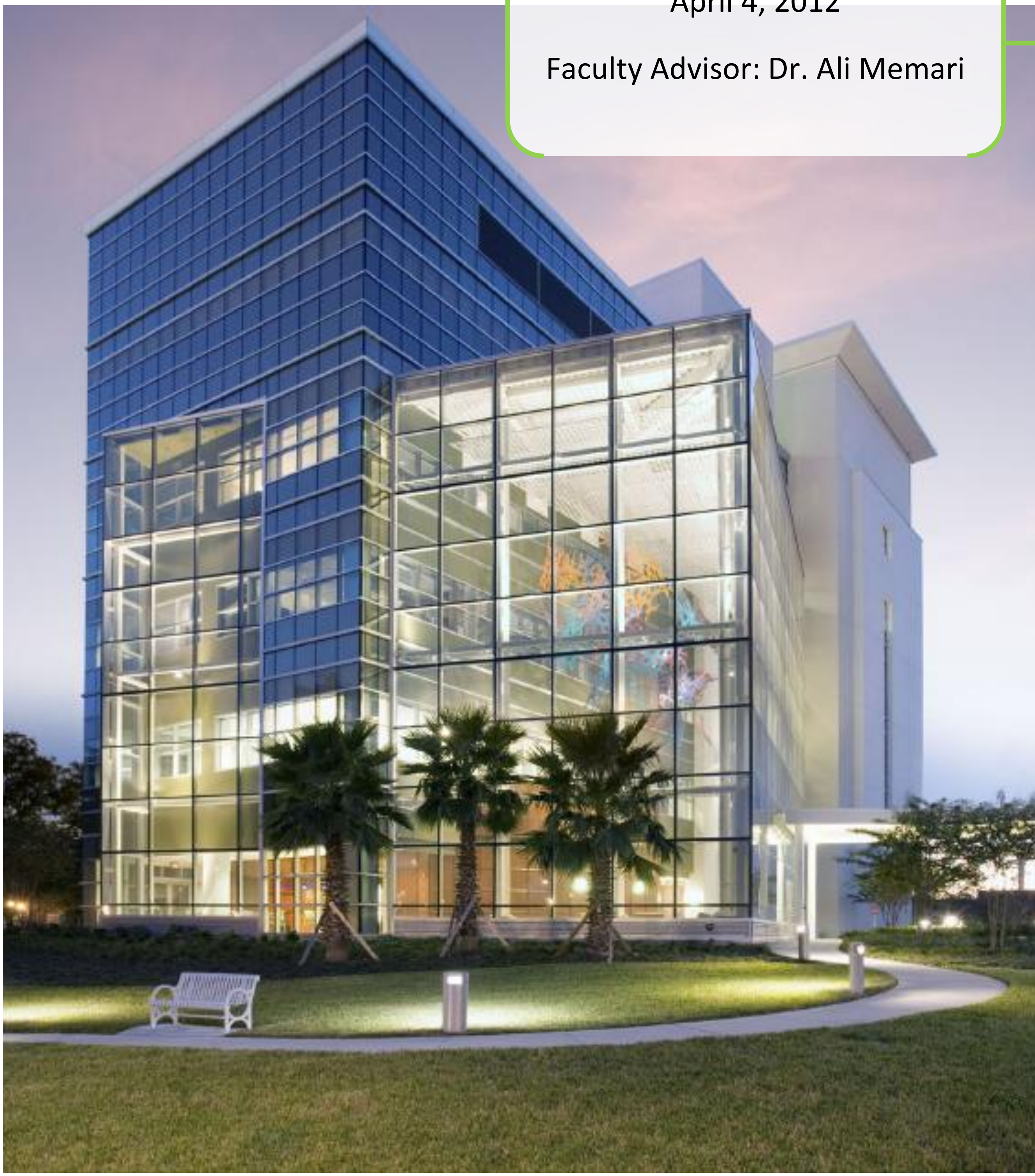
J.B. Byrd Alzheimer's Center & Research Institute

Tampa, Florida

April 4, 2012

Faculty Advisor: Dr. Ali Memari



Johnnie B.Byrd, Sr., Alzheimer Center & Research Institute

Tampa, Florida

General Information:

Occupant Type:	Office & Research Laboratories
Size:	108,000 SF
Construction Cost:	$23,600,000
Delivery Method:	Design-bid-build
Construction Schedule:	February 7, 2006 – July 9, 2007

Project Teams:

Occupant:	University of South Florida
Owner's Rep:	Ruyle, Masters, Hayes+Jennewein
Agency:	USF Facilities Planning & Construction
Architect:	HDR Architecture, Inc.
GC/CM:	Turner Construction
Structural:	HDR Engineering, Inc.
MEP/Landscape:	HDR Engineering, Inc.
Interior Design:	Elements

Lighting/Electrical:

- Fluorescent lighting used throughout
- Two main feed at 31 kV
- Two 4,000 kVA transformers feed
- 480/277 and 208/120 Panel boards

Architecture:

Cube:

60' high atrium structure symbolizing knowledge

Building Facade:

Cement plaster with the same curtain wall like glazing and decorative grille with louver at the top.

Curtain wall glazing: Clear Tempered, insulating laminated spandrel glass, clear insulating laminated glass, insulated fritted glass 30% & 50% silkscreen coverage pattern, sunscreens, and louvers.

Structural:

Building:

- Concrete frame & one way Slab
- Precast joist and soffit beams
- Spread footings & flat mat slab foundation
- 10" & 12" shear walls around elevators & stairs

Cube:

- Truss components made from structural tubes & pipes

Mechanical:

Building:

- Chilled water provided by 2 air cooled chillers & pumps
- Heating water provided by 2 gas fired boilers
- Medium pressure steam for use in laboratories
- 6 AHU total: 2 for each of labs, office, main

Cube:

- 1 AHU; fans vary to maintain duct static pressure

RAFFI KAYAT http://www.engr.psu.edu/ae/thesis/portfolios/2012/RAK5105 STUCTURAL

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Executive Summary

This is a fairly new, 108,000 square foot office and clinic building located on the University of South Florida's campus in the Tampa, FL. It has a construction cost of approximately $22 million, and has several unique architectural features, such as a curvy curtain wall façade and an atrium cube nipping the front entrance. The main gravity system consists of one way precast joists and soffit beams resting on cast-in-place columns, but the cube is constructed of steel trusses. The lateral system is a dual system of 11 shear walls and moment frames scattered throughout the building. The bulk of this report is comprised of two redesigns of the original structure. Because the existing structure was extremely efficient, the choice was made to attempt to design a viable alternative in California.

The scenario was then created in which the University of San Diego (USD) had commissioned the design of the building instead of the original owner. A geotechnical report was not located for the site on USD campus thus the same report of the original site was used. A one way cast-in-place system was designed for code minimum requirements to resist the gravity weights instead of the original design as it is specific to Florida. The resulting structure weighs approximately 20,000 k, a bit more than the original structure.

Furthermore, a high-performance design was investigated by producing two designs, one for "Prevention Collapse"S-5 criteria and the other as "Life Safety" S-3 criteria, as defined in ASCE's "Seismic Rehabilitation of Existing Buildings" (ASCE 41-05). The redesigns achieved this performance rating through the use of larger, stiffer shear walls and concrete moment frames. Then, the code-minimum frame was augmented with base isolators in order to achieve the higher performance requirement. This design was verified with Time History analysis using nonlinear properties for the isolators in ETABS. Master's level coursework was integrated throughout the report in the computer modeling of the structures (AE 597A) as well as a more direct application of earthquake design (AE 538). The hand calculations for these designs can be found in Appendices C, E and F.

To compare the structures, a construction management breadth was undertaken. This used quantity from the take-offs of both structural components and some additional architectural features which were considered to determine durations for activities. Then, the existing schedule was modified to remove the existing superstructure, and the new durations for the superstructure were added. These durations were used to calculate general conditions cost of the projects. The costs of the original structure and the two redesigned structures were calculated using a mix of square foot estimating, detailed estimating, and original cost data provided by HDR, Inc. This analysis found that the fixed base structure was 2% more expensive and the isolated structure was found 4.5% more expensive than the original. Also, both systems had 2-3 months (17% -20%) longer than the original schedule.

Finally, since the building was relocated to California, a sustainability breadth was undertaken to determine if a photovoltaic system which was not included on the original building would be viable at the new location. The system was designed and then evaluated with a lifecycle assessment and a payback period. It was deemed that the system is viable using the accelerated depreciation method.

Acknowledgement

I'd like to extend my gratitude to the following people and companies for their support during the completion of this report.

HDR Architecture, Inc. for providing the project and the owner permission form. For their swift responses to questions and their willingness to assist, I'd also like to specifically thank

Michael Paczak

Family and friends, for their unwavering support;

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The entire AE faculty, for 4 years of superb education, but specifically

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BISEM Inc for providing guidance and information on BIPV systems I'd also like to specifically thank

Nick Bagatelos

Building Introduction

The Johnnie B. Byrd, Sr. Alzheimer's Center & Research Institute or J.B Alzheimer's center is located in Tampa, Hillsborough, Florida in the University of South Florida's campus. It's located on the intersection of the orange lines on Fletcher Avenue and Magnolia Avenue (See Figure 1). Its occupant is the University of South Florida and it is a business occupancy used for offices and as a research facility. In fact, after its construction the Florida Alzheimer's center and Research facility became one of the largest freestanding facilities of its type in the world specifically devoted to this illness. It is designed to primarily function as a research unit with labs, a hub for clinic trials, and a data collection center for all Alzheimer facilities throughout the state of Florida. It is built on a 2.6 acres site and the size of the building is 108,054 sq ft, gross. It is 9 stories including a basement totaling a height of 107'. The actual building cost was $23,602,477. It has been LEED silver accredited after construction. From start to finish the construction dates were from February 7, 2006 to July 9, 2007 hence about a year and a half.

Figure 1- **Site Location on campus of USF**

The Owner/Client of the project is Johnnie B. Byrd Alzheimer's Center & Research Institute. The General Contractor + CM were Turner Construction Company. Everything else (i.e. Architecture, Structural Engineering, Mechanical & Electrical & Plumbing Engineering, Civil Engineering, Landscape Architecture, Security & Telecom) were handled by HDR Architecture, Inc. This project was delivered to the owner by a design-bid-build method.

The façade of the building is mainly divided into two parts. The east side consist of curtain wall glazing and Aluminum panels. The west side consists of cement plaster with the same curtain wall like glazing and decorative grille with louver at the top. As for the roof the use of Thermoplastic Membrane roofing was chosen with ¼"per foot slope with Aluminum parapet for architectural reasons.

Structural Overview

Basic construction materials of the building include stone column piers and a spread footing foundation system with below grade footing. The structure is composed of precast joist webs and soffit beam bottoms with concrete shear walls. Exterior walls are constructed of cement plaster and lath on steel stud back up framing. The curtain wall system has a kynar aluminum finish and integrates several glazing types. Mechanical systems include packaged air handlers, on-site chillers, and gas fired boilers.

Initially, HDR Architecture Inc. structural department had designed this building as a composite system composed of steel beams, flanges, columns and a concrete slab on metal floor deck. They had their system pre-designed with specifics. However, all these ideas got tossed away when the Owner and the Contractor decided to use a more economical and efficient concrete system with precast joist webs and soffit beams. The latter exists mainly in Florida. Hence, the use of it will be fairly new to others, which add uniqueness to this building and thesis.

The J.B. Byrd Alzheimer's Center & Research Institute rests on spread footings for columns and continuous strip footings for walls as well as a mat slab foundation system. This was advised by Nodarse & Associates, Inc. because the site lies on a potential sinkhole activity. The lower 7 floors utilize a one way concrete slab with precast joist ribs and soffit beam framing system for floor framing with cast in-place columns. Part of level 7 and level 8 still utilize the same floor framing but with larger spacing as well as concentrated reinforcing bars around roof anchors. The lateral system consists of moment frames with concrete shear walls around the main openings.

The importance factors for all calculations were based on Occupancy category II. This was chosen because the J.B A.C. & R.I. falls under office building.

Design Codes

According to sheet S001, the original building was designed to comply with the following major codes:

- 2001 Florida Building Code with 2003 updates
- 2001 Florida Building Mechanical Code with 2003 updates
- 2001 Florida Building Plumbing Code with 2003 updates
- 2001 Florida Building Fuel Gas Code with 2003 updates
- 2001 Florida Building Accessibility Code as Ch.11 and Energy Code as Ch.13
- 2000 National Fire Protection Association.
- Building code requirements for reinforced concrete (ACI 318)
- AISC Manual of Steel Construction, Allowable Stress Design 9th ED.
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD) 1st ED.
- American Welding Society (AWS), D1.1, D1.3, D1.4
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-98)
- Masonry Construction for Buildings (ACI 530-99 AND ACI 530.1-99)

These are also the codes used to complete this technical report:

- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building code requirements for reinforced concrete (ACI 318-08)
- 2006 International Building Code (IBC 2006)

Materials Used

Various materials were used on the structure of this project. Below are the main materials derived from Sheet S-001.

Concrete		
Usage	Weight	Strength (psi)
Spread footing	Normal	3000
Mat slab foundation	Normal	3000
Precast Joist Webs and soffit beams	Normal	5000
Cast-in-place slab	Normal	4000
Columns, typical	Normal	4000
Columns, as noted	Normal	6000
Precast Masonry Lintels	Normal	5000
Housekeeping Pads	Normal	4000
General Structure Concrete	Normal	4000
Note: Normal weight concrete is at 28 day compressive strength		

Steel		
Usage	Standard	Grade
Reinforcing Steel	ASTM A615	60
Reinforcing Steel (welded)	ASTM A706	60
Welded Wire Fabric	ASTM A185	70
Prestressing Tendons	ASTM A416	270
Wide Flange, S and Tee shapes	ASTM A992	50
Angles Channels and Plates	ASTM A36	36
Tubes	ASTM A500 B	46
Pipes	ASTM A53 B	35
Bolts	ASTM A325	36
Glavanized Roof deck	ASTM A653	33
Note: Welding Electrodes used were E70XX		

Masonary		
Usage	Standard	Strength (psi)
Concrete Masonary Units	ASTM C-90	f'_m= 1500
Mortar	ASTM C270, M	f'c= 2500
Mortar	ASTM C270, S	f'c= 1800
Grout	ASTM C476	f'c= 3000
Joint Reinforcement	ASTM A82, Truss Type	

Figure 2 - **Material Used in building: Concrete, Steel, Masonary**

Foundations

Nodarse & Associates, Inc prepared a report of Preliminary Geotechnical Exploration for this project. The subsurface exploration consisted of a Ground Penetrating Radar (GPR) survey on the site and eight Standard Penetration Test (SPT) borings to depths of 50 to 75 feet below existing site grades.

The borings encountered a relatively uniform subsurface profile consisting of the following respectively with depths: clean sands, medium dense clayey sands, very soft to stiff clays, and weathered to very hard limestone formation. There are indicators in the borings that correlate with the increased risk for sinkhole occurrence. These indicators consist of very soft soils or possibly voids. They estimated that sinkhole could range at the ground level from 10 to 25 feet across. A deep foundation system was not recommended due to the possibility of damage to other adjacent structures from pile-driving vibrations. Also, a cast-in-place deep foundations such as auger cast piles or drilled shafts are not recommended because the presence of joints, fissures, soft zones, and voids within the limestone formation and overburden soils will result in excessive overages of concrete and the need for permanent steel casing. In addition, The University of South Florida expressed concerns about this method as there is the potential of water contamination.

Hence, Nodarse & Associates, Inc recommended, based on their findings the use of a vibro-flotation/stone columns to improve soil conditions so that the building can be supported on a shallow foundation system such as footings and mat slabs (see figure 3 for shallow foundations used). The vibrating probe is intended to pre-collapse potential sinkholes (a total settlement of 1 inch or less) to reduce the possibility of future development. After the dry bottom, stone columns (42" +/-diameter) were installed to a depth of 25 feet. The stone columns were recommended to be crushed stone aggregate a similar gradation to FDOT No. 57 stone. Footings were then designed on a maximum allowable bearing pressure of 6,000psf. The allowable soil bearing capacity is 10,000 psf after soil improvement. Minimum footing widths for columns and wall footings of 36 and 24 inches respectively were used. Footings bear at least 36 inches below finished floor elevations to provide adequate confinement of bearing soils.

The ground water on this project site appears to be below a basement depth of 10 feet below existing grade, making a basement acceptable. Retaining Walls were also designed using a maximum allowable bearing pressure of 2,000 psi.

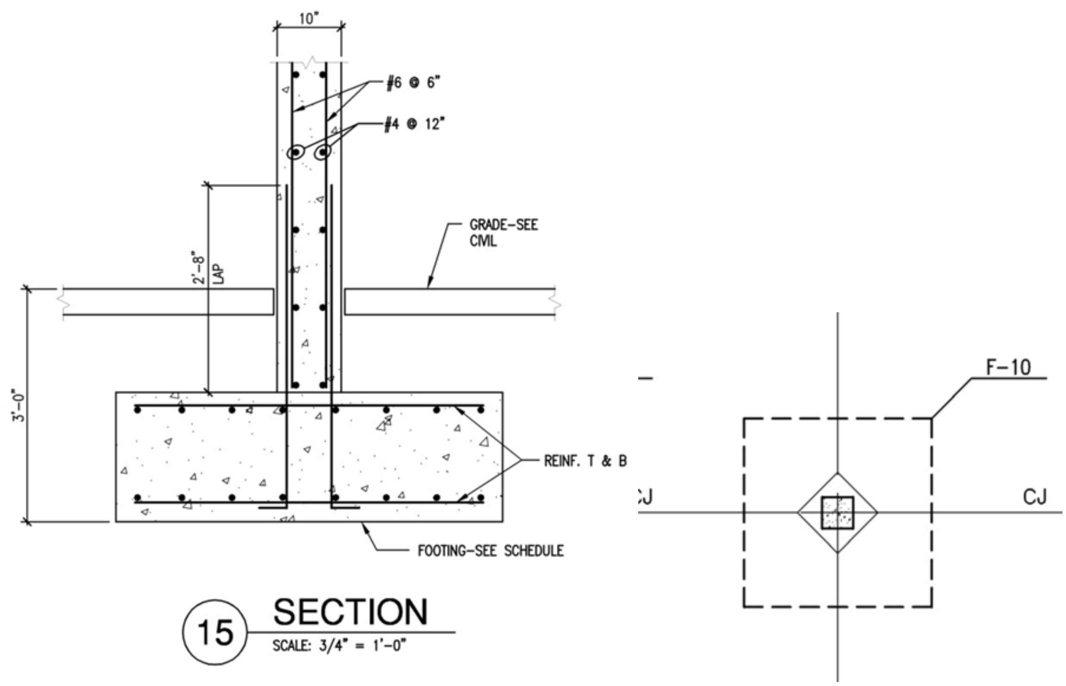


Figure 3- **Foundation section and plan showing footing-column connection and size**

Floor Systems

Even though this building is very architectural and seems like an irregular shape building with a complicated structure it can be divided into 4 simple sections. The sections also correspond to the different uses of the building. Figure 4 shows a typical floor plan with the different bay sizes highlighted with different colors.

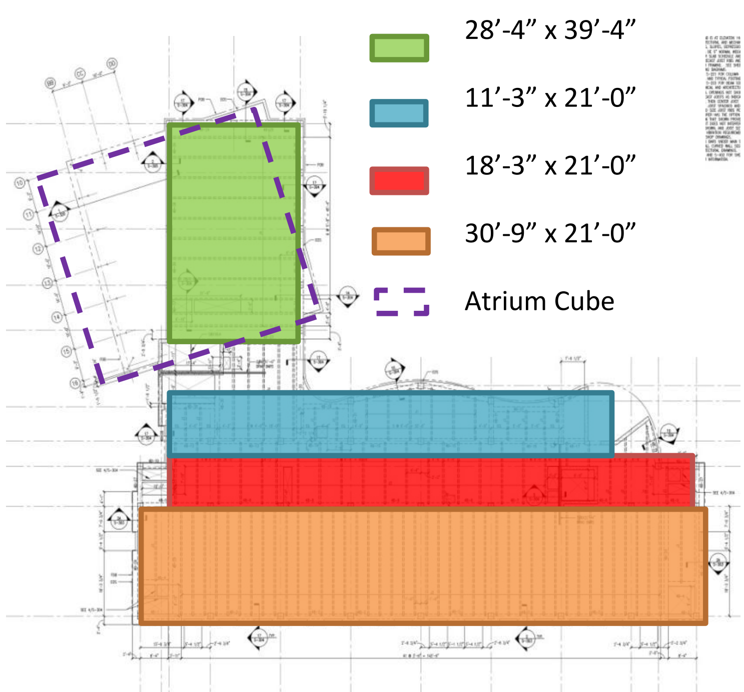


Figure 4- Floor plan showing different bay sizes

All the elevated floors of the J.B AC&RI are a hybrid system consisting of a precast joist ribs and soffit beam framing system with cast-in-place to unite the system. In fact, there are 5 main joists that have respectively the following depths: 8", 12', 16", 20", and 28". The entire precast joists and beam soffits are brought on site and lifted to the positions using scaffolding and then they are tied to the structure. Once the structure is erected, the formwork and the rebar reinforcing (if needed) are done then further a 5" concrete slab is casted in place to unite the system (see figure 6). As stated before, 5 different joist depths were used adequately depending on the required spans and uses. For the approximately 40' span, a 20" or J4 was used spaced at 5'-8". That area, corresponding to the green rectangle in figure 4 is typically an office area. For the orange rectangle, where the research labs reside, a J3 or 16" spaced at 5-6" was used for a span of 31'. However in the same area, J4 or 20" spaced at 3'-6" and J5 or 28" at 3'-2" were used to accommodate the PET scans and MRI components respectively (see figure 5).

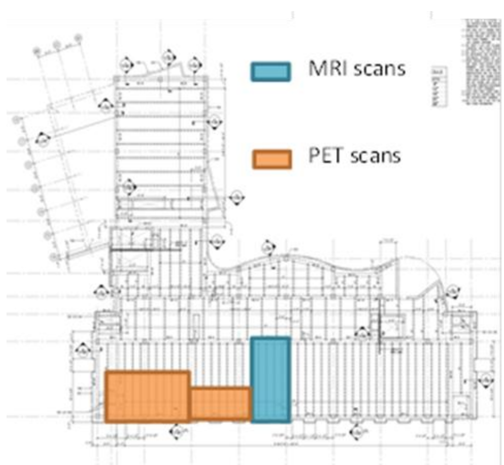
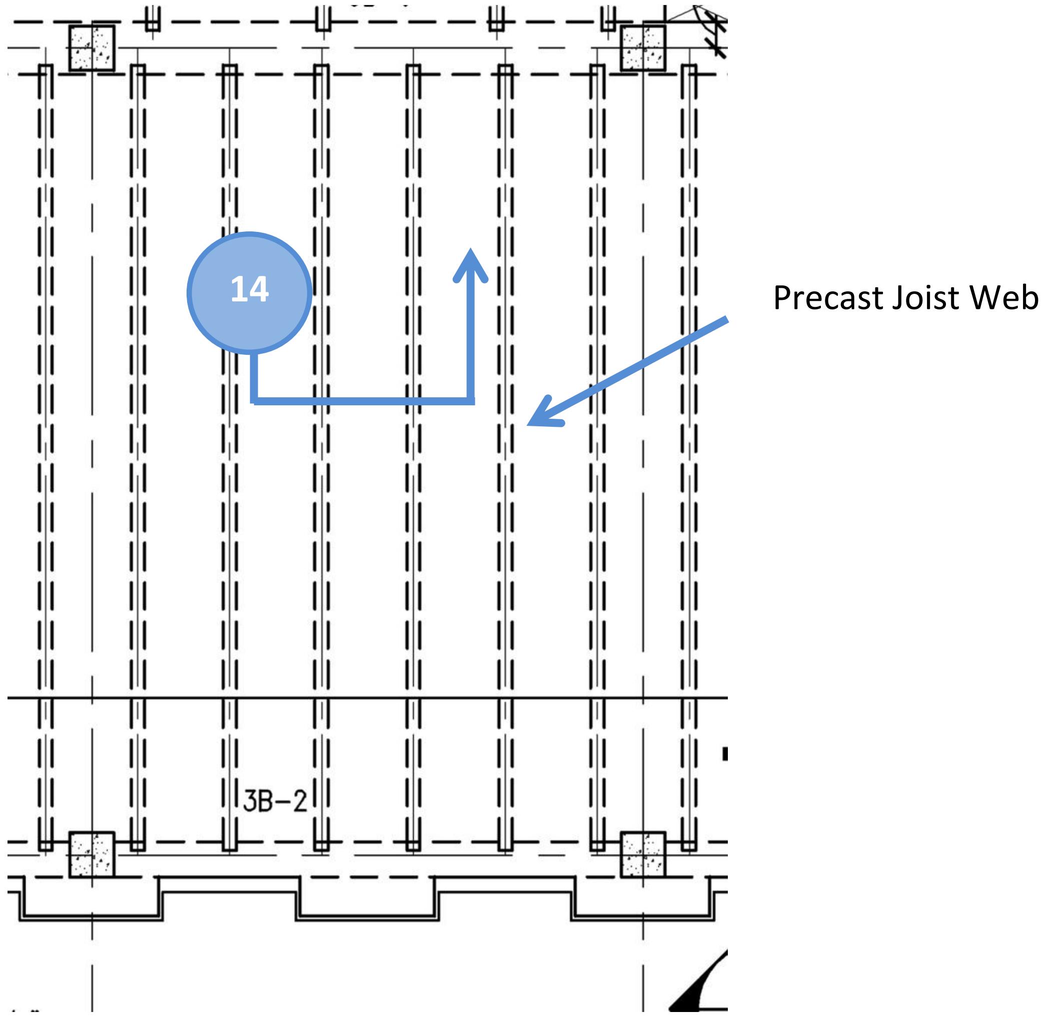


Figure 5- 2nd level floor plan showing MRI/PET scan location



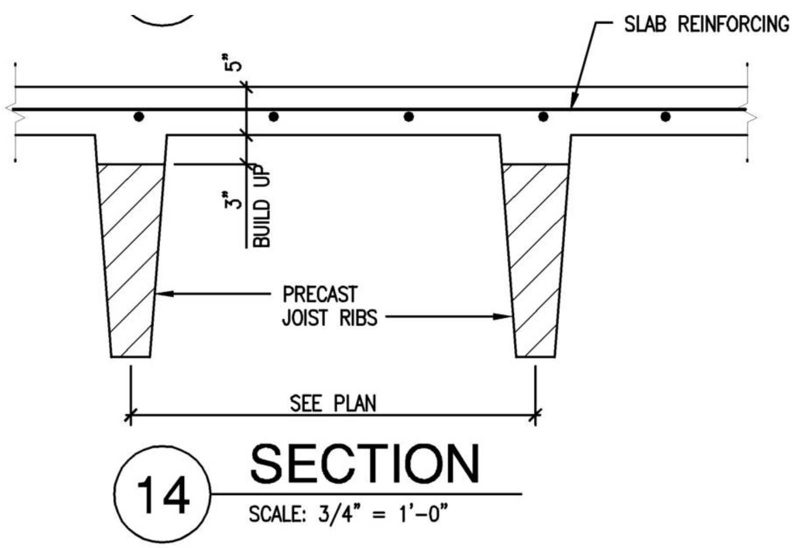


Figure 6- Plan and section of precast joists

Framing System

The columns in the lower 7 stories are all cast-in-place concrete. Most of the columns are square and have 4,000psi strength. However, the columns supporting the research labs where the heavy equipment exists and vibration criteria need to be attained a 6,000psi concrete columns were used at the basement and the first floor (see figure 7). All columns are about 20”x20” with reinforcing ranging from 4 to 8 bars except for a few exception that are 20”x30” with 16 bars.

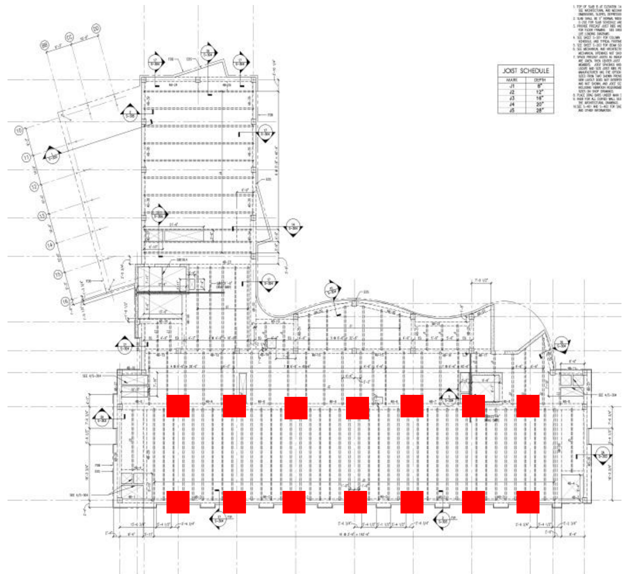


Figure 7- Floor plan showing the 6,000 psi column in basement and 1 floor

Lateral System

The lateral system is composed of concrete shear walls and moment frames. The shear walls are around the main vertical circulation at both ends of the building (see figure 8). They resist the N-S direction as well as E-W direction for best result and little torsion. All of these walls are cast-in-place and are 12” thick. All of them span from basement to the roof. They are anchored at the base by a mat slab foundation that is 3’-0” thick. An issue not investigated by this report is how much the moment frame resists the loading compared to the shear walls when loaded in both directions.

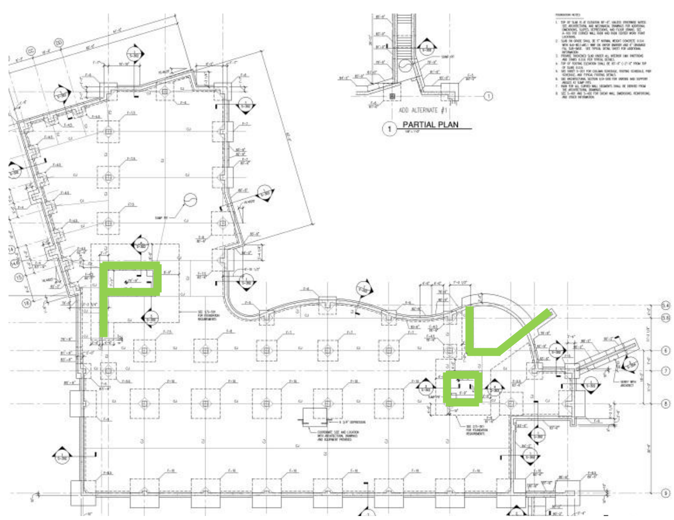


Figure 8- Floor plan showing shear walls

Atrium Wall Framing / Floor vibration Criteria

The atrium roof is approximately 60 feet above grade. Architectural trusses, approximately 36” deep are designed to support the exterior storefront glazing spanning this 60 feet. The trusses are designed to minimize deflections from hurricane force winds on this wall. The design wind speed for the area is 120mph which yields that the 50’- 60’ range was designed at 31.3 PSF. Truss components are made from structural tubes (ASTM A500, Grade B of Fy= 46Ksi) and pipes (ASTM A53,Grade B Fy= 35Ksi) in this highly visible part of the building.

The vibration control design interfaces with the design of structural, mechanical, architectural, and electrical systems in such a way that those systems do not generate or propagate vibrations detrimental to research activities of the Florida Alzheimer's Center & Research. Vibration criteria have been developed based upon examination of vibration requirements of planned or hypothetical equipment. General labs make up the research facility, and the structure will be designed for vibration amplitude of 2000-4000 µin/s. This accommodates bench microscopes at up to 400x magnification. This last will play a significant role in choosing the members of the system as well as the systems themselves.

Roof Systems

There are two different roof levels: one on the seventh floor and the other on the mechanical level on top of that (See Figure 9). The figure shows a height from level 1 that starts at 100'0" but for simplicity only the true height is shown. This two roof structure consists of the same material and system as the floor system as they hold a great deal of load (mainly mechanical that include packaged air handlers, on-site chillers, and gas fired boilers). However, the slabs were heavily reinforced around the roof anchors. Level 7 has joist spacing of 5'8" in the green section and 3'6" under the red section. On the mechanical level a spacing of 5'-6" is used as loads are minimal. There is also the roof of the atrium cube that is not shown on this figure. That last is at height of 153'-9"and consists of trusses, angles, C shape and HSS bars. In addition to the atrium roof, a canopy at the entrance hangs at a height of 114'-6" and consists of W shape with a 1½" 18 Gage galvanized metal roof deck.

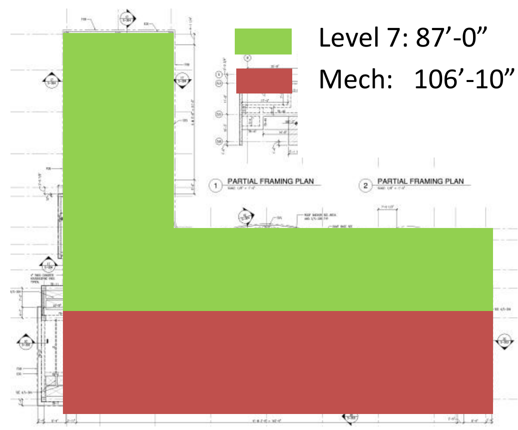


Figure 9- **Showing the different roof levels on the building**

Gravity Loads

Part of this technical report, dead and live loads were calculated and compared to the loads listed on the structural drawings. Snow loads however were not applicable for this project as this building exists in Tampa, Florida. Several gravity member checks were conducted.

Dead and Live Loads

The structural drawing S001 lists the superimposed dead loads to be used. This is summarized in figure 10. The SP for Ceilings, lighting, plumbing, fire protection, flooring, and HVAC for roof over mechanical levels is higher than usual because all the mechanical system that supplies the research labs that require special feed are situated in that area. These systems include packaged air handlers, on-site chillers, and gas fired boilers.

Also considered in the building weight calculation were the weights of the columns, shear walls, roofs, wall loads, precast joists and soffit beams.

SuperImposed dead loads	
Description	Load
Ceilings, lighting,plumbing, fire protection,flooring,and HVAC all	14 psf
Ceilings, lighting,plumbing, fire protection,flooring,and HVAC for roof over mechanical levels	40 psf
except mechanical	20 psf
allowance for roofing system	20 psf

Figure 10- **Superimposed Dead load on S-001**

The live loads listed below (figure 11) taken from S001 were compared to the live loads in Table 4-1 in ASCE 7-05 based on the usage of the spaces. The result came out to be the same or more than the expected minimum allowed by the code.

There was nothing about Alzheimer research labs or research labs in general hence the provision “Hospitals- Operating Rooms, Laboratories” was used for comparison. The same was done for high density file storage but with the use of two provisions one is based on "Storage-light/heavy" and the other is based on “Libraries-Stack rooms”. Both were in the range or more than the one designed with. That last one shows on the second level where the MRI and the PET scanner are located special loading was used. A 34kips MRI load distributed to 4 legs then each leg load to 2 joists spaced at 7’-6” apart, center in depression. Also, an 11k scanner load was considered as well as the access path to both the PET and MRI equipment.

One of the last discrepancies, the loadings on S-002 and S-003 are different than the ones stated in the table below. That is due to allow a more flexible building, more stable floors for the vibration and to take into effect the live load reductions.

Floor live loads may be reduced in accordance with the following previsions:

- For live loads not exceeding 100psf for any structural member supporting 150 sq ft or more may be reduced at the rate of 0.08% per sq ft of the area supported. Such reduction shall not exceed 40% for horizontal members, 60% for vertical members, nor R as determined by the following formula:
  R= 23.1 (1+D/L) where D=dead load and L=live load
- A reduction shall not be permitted when the live load exceeds 100psf except that the design live load for columns may be reduced by 20%.

Live Loads			
Area of the building considered	Design Load	ASCE 7-05 Live	Notes
Labratories	125psf	60 psf	Based on "Hospitals-Laboratories"
Offices	50 psf	50 psf	Based on "Office Bldg.-Offices"
Corridors, first floor	100 psf	100 psf	Based on "Office Bldg.-Corridors"
Corridors, above first floor	80 psf	80 psf	Based on "Office Bldg.-Corridors above"
Lobbies	100 psf	100 psf	Based on "Lobbies"
Storage areas	125 psf	125-250 psf	Based on "Storage- light/heavy"
High density file storage	200 psf	125-250 psf	
Mechanical spaces	150 psf	N/A	
Stairs	100 psf	100 psf	Based on "Stairs
Roof	20 psf	20 psf	Based on "Roof- Sloped"

Figure 11- **Live Load comparison to ASCE 7-05**

Snow Loads

No snow load was applicable for this project as it is located in Tampa, Florida. From this following figure 12 taken from ASCE 7-05, the ground snow loads equal zero lb/ft2.

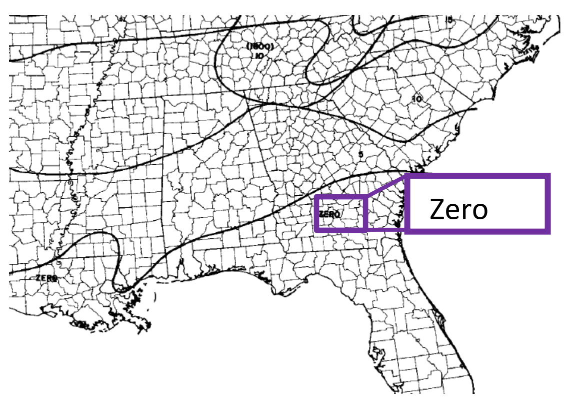


Figure 12- **Diagram showing the ground snow load for Florida**

Lateral Loads

In order to better understand the lateral systems, wind loads and seismic loads were calculated for this technical report. These were calculated by hand, and then applied to a lateral model of the structure created in ETABS. The hand calculations for the wind loads can be found in Appendix B, and the hand calculations for the seismic loads can be found in Appendix C.

Wind Loads

In Technical Report 1, “Existing Conditions and Design Concepts,” wind loads were calculated with method 2 Main Wind Force Resisting System (MWRFS) procedure identified in ASCE 7-05 Chapter 6. In order to be able to use this procedure, several simplifying assumptions had to be made. First, the building was modeled with a single roof height of 107’. Next, the surface areas were projected onto North-South (N-S) and East-West (E-W) axes and the projected lengths were used to calculate wind pressures. Using these projected lengths for the calculation of L and B would be conservative. Also, since the new projected shape looks like an L shape, it is assumed that there wouldn’t be a buildup in pressure where the void in the L-shape exists. The same forces were used in this technical report.

From technical report 1, it was found that wind loads were greater than seismic by a factor of about 3.6 in the East-West direction and 2.5 in the North-South direction. The design base shear in the North-South direction was calculated to be 682kip, and in the East-West direction was calculated to be 892 kip. Thus, it is expected that wind will control over seismic however this still needs to be checked due to the different load combinations and factors that exist in ASCE 7-05.

Most calculations were performed using Microsoft Excel to simplify a potentially repetitive process. Wind pressures, including windward, leeward, sidewall, and internal pressure were found. These were then used to calculate the story forces at each level. It should be noted that the story forces include windward and leeward pressures, but not internal pressure, because internal pressure is effectively self-cancelling.

The wind loads on this building are collected by the curtain wall glazing and cement plaster walls on the exterior of the building. The walls and the glazing in return transfer these loads to the slabs that they are anchored to. This then transfers the loads into the slabs, which then carry the load to the shear walls and moment frames in relative to their stiffness. These return the loads to their foundations which are mat slabs and footings respectively.

For this technical report, accidental moments were also calculated. This was achieved through the use of the four load cases for torsion due to wind, given in Figure 6-9 of ASCE 7-05 and included as Figure 13. This was done due to the nature of the geometry of the building (L-shaped) that is susceptible to torsion and may control.

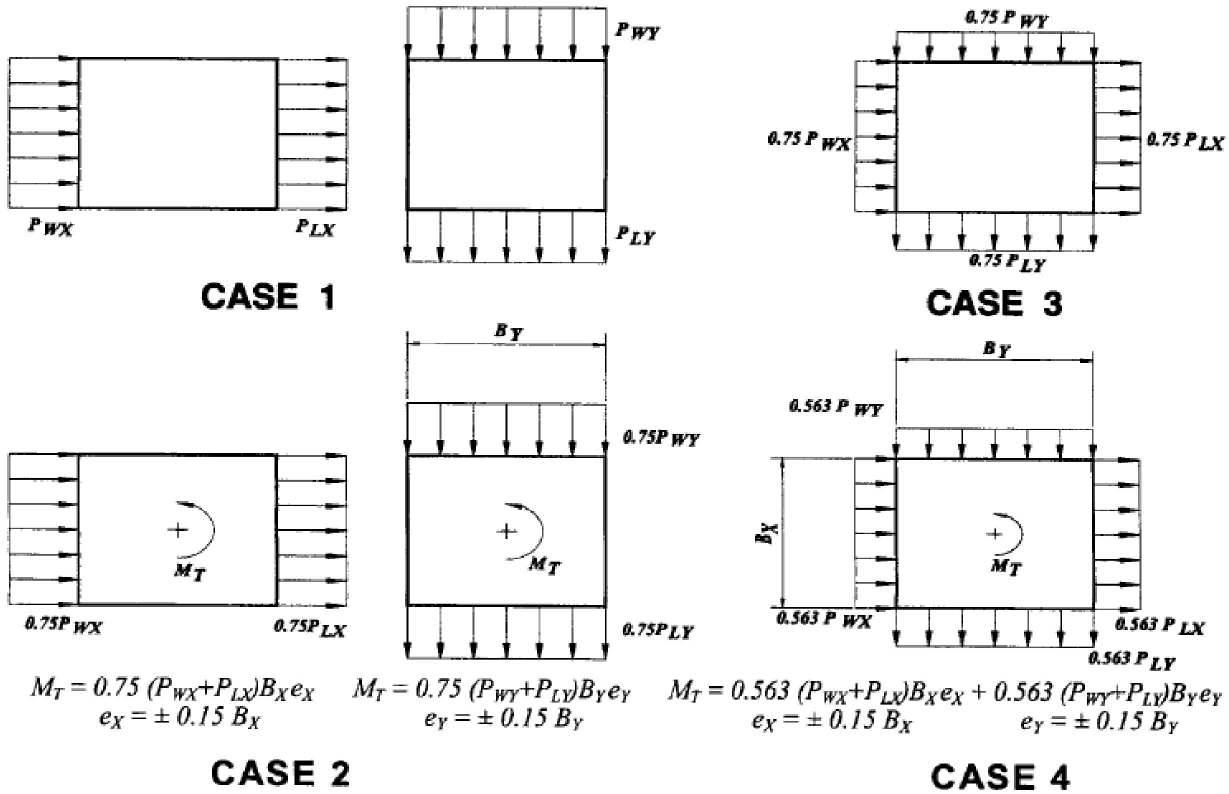


Figure 13- **Figure 6-9 in ASCE 7-05 showing all the torsional wind load cases**

For simple and not iterative process, each load case was represented and labeled differently. They were entered into the model in four basic static load cases: wind forces in the N-S direction (WX), wind forces in the E-W direction (WY), accidental moments due to the N-S loads (WXMX), and accidental moments due to the E-W loads (WYMY). After establishing the formulas and retrieving the corresponding MT, a total of 11 wind cases were established and reported in figure 14. These were then taken as serviceability loads (no factor was incorporated) and analyzed to acquire drifts.

This was done as a first step to determine which of the cases controlled in each direction and in return are then compared to the earthquake loads. This methodology came from the fact that the load factor of wind in ASCE 7-05 is 1.6 much greater than the 1.0 factor used for earthquake meaning the wind forces are magnified. Thus, a simple serviceability comparison would yield the controlling case since the wind forces are greater than earthquake load in both directions. This reasoning produced 13 load combinations detailed in figure 14 (11 with wind and 2 with earthquake).

Serviceability using a factor of 1.0

Load combinations		
Wind (total of 11 cases)	Case 1 (2)	$P_{Wx}+P_{Lx}$
		$P_{Wy}+P_{Ly}$
	Case 2 (4)	$.75P_{Wx}+.75P_{Lx} \pm M_T$
		$.75P_{Wy}+.75P_{Ly} \pm M_T$
	Case 3 (1)	$.75\ (P_{Wx}+P_{Lx}) + .75\ (P_{Wy}+P_{Ly})$
	Case 4 (4)	$.563\ (P_{Wx}+P_{Lx}) + .563\ (P_{Wy}+P_{Ly}) + M_T$
Earthquake (total of 4)	Case 1 (2)	$1.0\ E_x \pm M_{zx}$
	Case 2 (2)	$1.0\ E_y\ \pm M_{zy}$

Legend	
Eccentricity	$e_x = \pm 0.15B_x$
	$e_y = \pm 0.15B_y$
where M_T=	$0.75(P_{Wx}+P_{Lx})B_xe_x$
	$0.75(P_{Wy}+P_{Ly})B_ye_y$
Bx= width of building in x-direction By= width of building in y-direction	
where M_T=	$\pm 0.563(P_{Wx}+P_{Lx})B_xe_x \pm 0.563(P_{Wy}+P_{Ly})B_ye_y$

Figure 14- **The 11 cases retrieved from figure 6-9 ASCE 7-05 and inputted in ETABS to acquire drifts.**

"Px" or "Py" are the story force at a given level in the direction under consideration and Bx or By are the building dimension in the direction under consideration. The subscripts "W" and "L" represent windward and leeward pressures. The accidental moments are shown under M_T and are shown how they are calculated in the legend of figure 14.

The wind pressures in the N-S direction are listed and diagramed in Figure 15. These were resolved into wind forces in the N-S direction, which are listed and diagramed in Figure 16. The resulting base shear is 682k.

In addition, the wind pressures in the E-W direction are listed and diagramed in Figure 17. These were resolved into wind forces in the E-W direction, which are listed and diagramed in Figure 18. The resulting base shear is 892k.

Wind pressures calculated were able to be compared with the engineer's calculations. In fact, discrepancies of windward and leeward calculations were only 5%. This minor difference was due to the fact that the engineer had used a larger leeward pressure at the altitude of 120'. This height is higher than the building and did not take a simplified roof like it was done in this report.

To see the engineer's calculations and diagrams to compare please refer to pages 38-39.

Desgin wind pressure for MWFRS in N-S Direction						
Type	Level	Height / distance	qz/ qh	Wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
Windward walls	1	0'	21.0	14.3	-6.1	34.7
	2	14'-6"	21.0	14.3	-6.1	34.7
	3	29'	25.5	17.3	-3.1	37.8
	4	43'-6"	28.7	19.5	-0.9	39.9
	5	58'	31.0	21.1	0.7	41.5
	6	72'-6"	33.2	22.6	2.1	43.0
	7	87'	35.1	23.8	3.4	44.3
	Roof	107'	37.1	25.3	4.8	45.7
Leeward walls	All	All	37.1	-13.8	-34.3	6.6
Sidewalls	All	All	37.1	-22.1	-42.5	-1.7
Roof		0-53.5	37.1	-29.9	-50.4	-9.5
		53.5-107	37.1	-27.7	-48.1	-7.2
		107-214	37.1	-16.5	-37.0	3.9

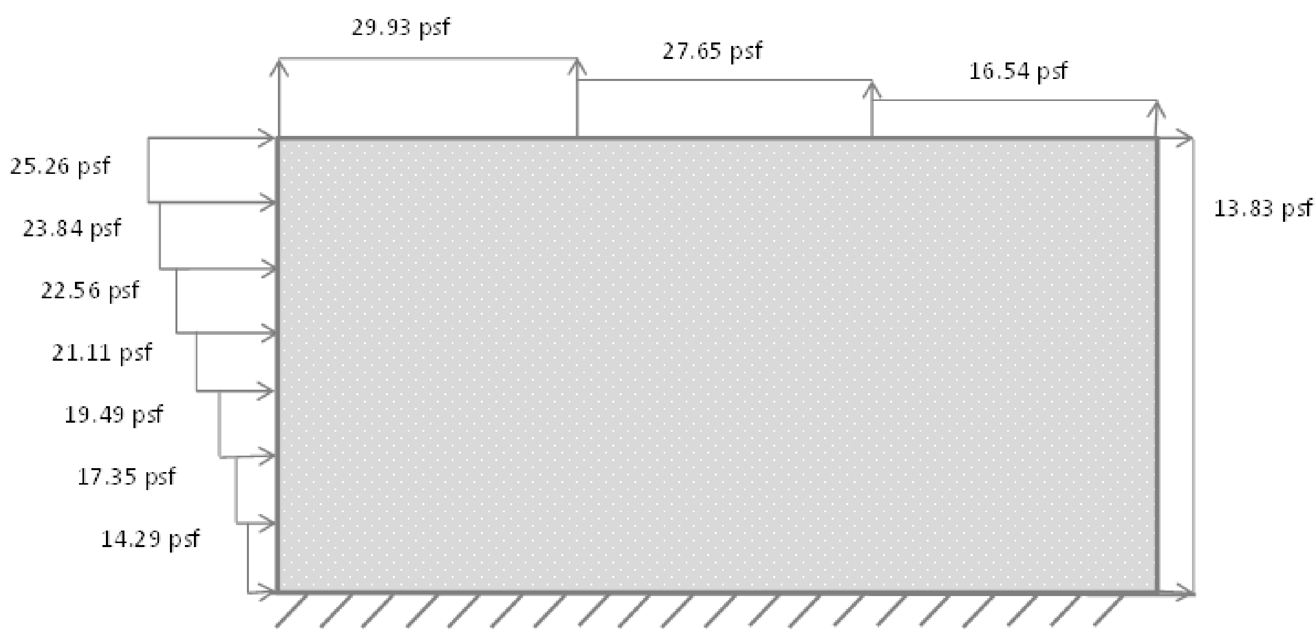


Figure 15 - **List and diagram showing the wind pressure on the building in N-S direction**

Wind Forces- N-S Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
1	0'	N/A	0	8	1095	73	682	0
2	14.5	7	1022	8	1095	77	609	1111
3	29	7	1022	8	1095	82	532	2383
4	43.5	7	1022	8	1095	86	450	3748
5	58	7	1022	8	1095	89	364	5186
6	72.5	7	1022	8	1095	92	274	6693
7	87	7	1022	8	1095	115	182	10020
Roof	107	10	1460	10	1460	67	67	7137
Total base shear=								682 k
Total overturning Moment=								36276 k

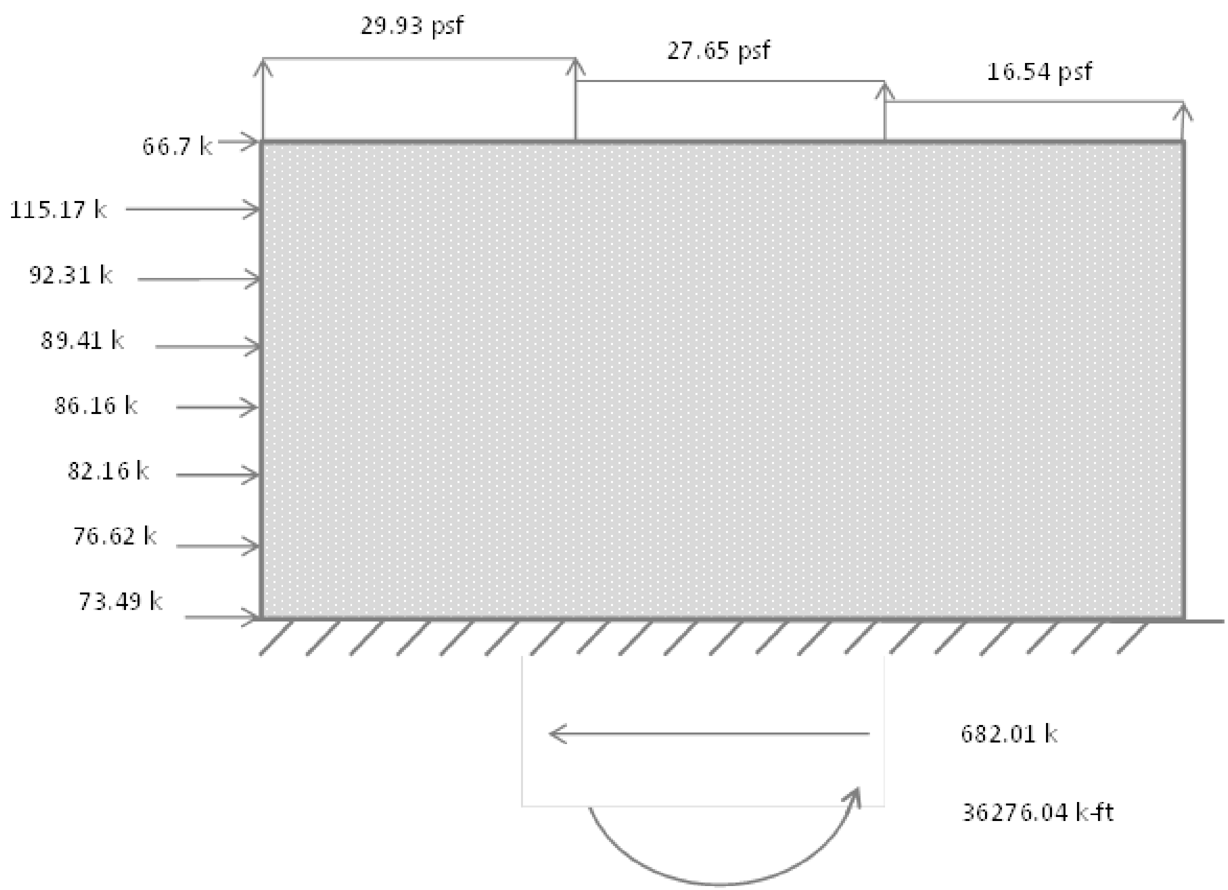


Figure 16 - **List and diagram showing the wind forces on the building in the N-S direction**

Desgin wind pressure for MWFRS in E-W Direction						
type	Level	Height / distance	qz/ qh	Wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
Windward walls	1	0'	21.0	14.3	-6.1	34.7
	2	14'-6"	21.0	14.3	-6.1	34.7
	3	29'	25.5	17.3	-3.1	37.8
	4	43'-6"	28.7	19.5	-0.9	39.9
	5	58'	31.0	21.1	0.7	41.5
	6	72'-6"	33.2	22.6	2.1	43.0
	7	87'	35.1	23.8	3.4	44.3
	Roof	107'	37.1	25.3	4.8	45.7
Leeward walls	All	All	-16.5	-15.8	-36.2	4.6
Sidewalls	All	All	37.1	-22.1	-42.5	-1.7
Roof		0-53.5'	37.1	-34.2	-54.6	-13.8
		53.5'-107'	37.1	-25.5	-45.9	-5.1
		107'-214'	37.1	-18.7	-39.1	1.7

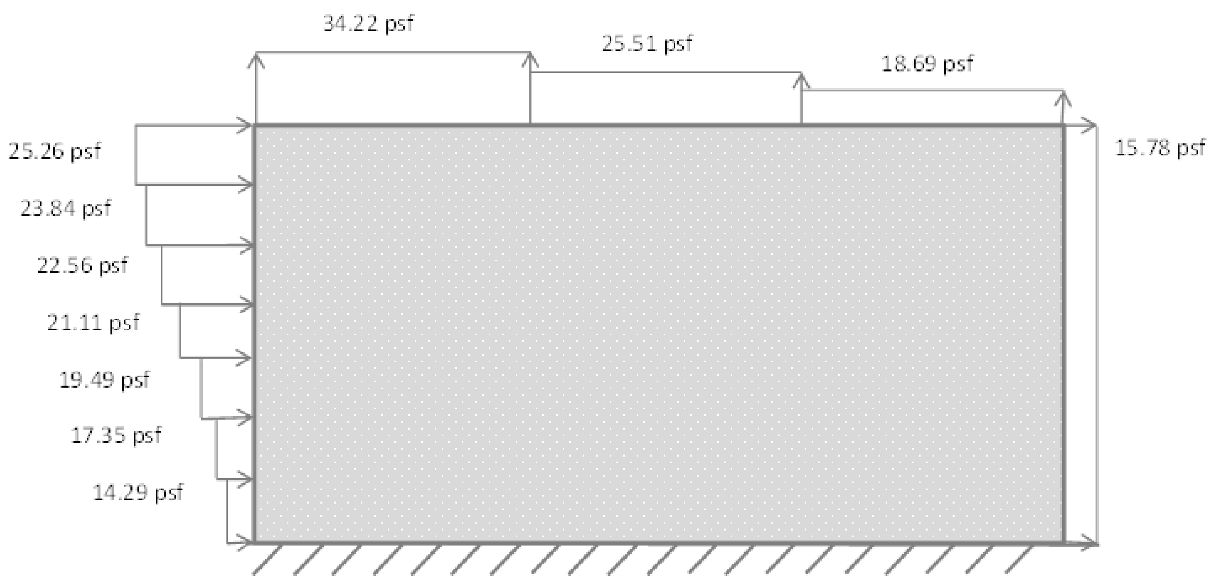


Figure 17 - **List and diagram showing the wind pressure on the building in E-W direction**

`Wind Forces - E-W Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
1	0'	N/A	0	8	1433	96	892	0
2	14.5	7.00	1337	8	1433	100	796	1453
3	29	7.00	1337	8	1433	107	696	3117
4	43.5	7.00	1337	8	1433	113	588	4903
5	58	7.00	1337	8	1433	117	476	6784
6	72.5	7.00	1337	8	1433	121	359	8755
7	87	7.00	1337	8	1433	151	238	13108
Roof	107	10.00	1910	10	1910	87	87	9336
							Total base shear=	892 k
							Total overturning Moment=	47457 k

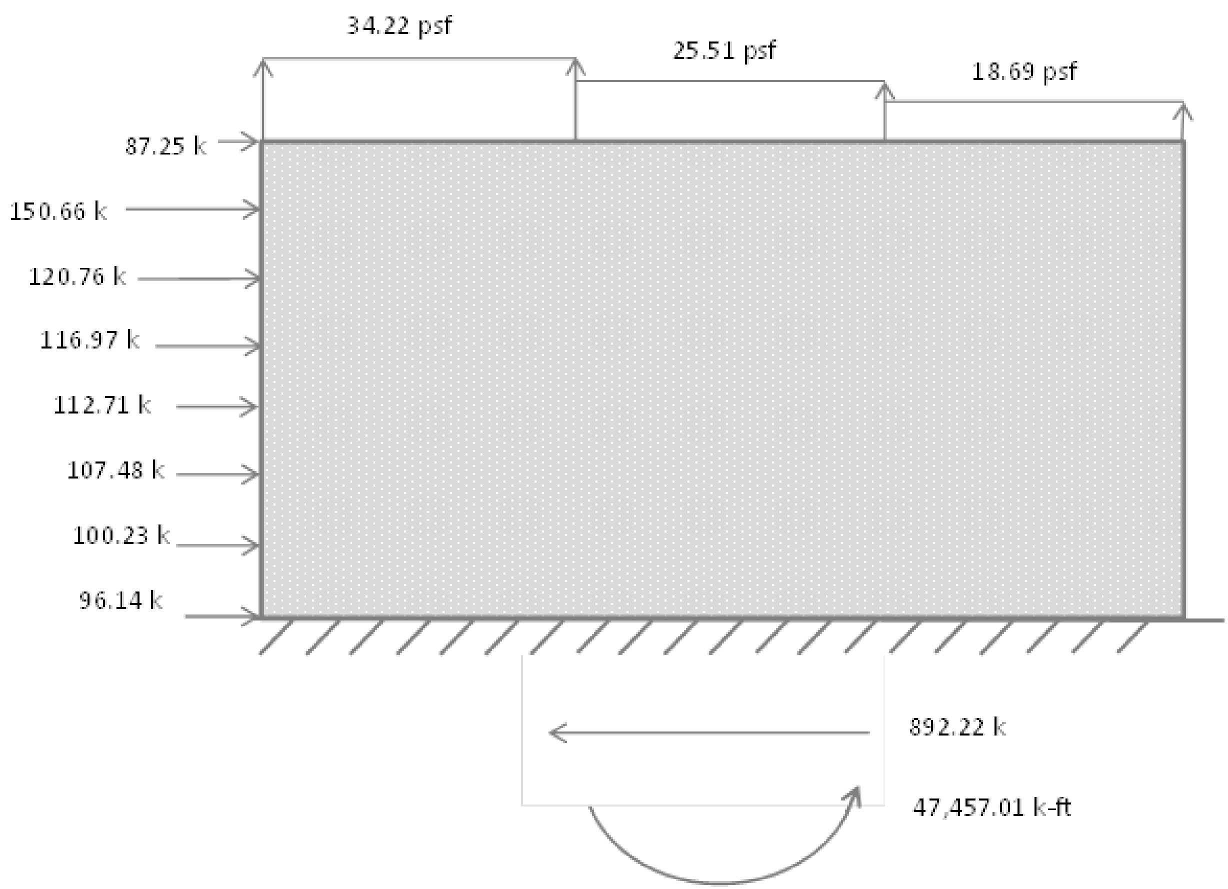


Figure 18 - **List and diagram showing the wind forces on the building in the E-W direction**

Seismic Loads

The engineers who designed this building did not analyze the building for seismic forces as wind always controls in Tampa, Florida. However, Seismic loads were still calculated to check that statement.

Seismic loads were calculated with the Equivalent Lateral Force procedure outlined in Chapters 11 and 12 of ASCE 7-05. This procedure also assumes a simple building footprint. In fact when calculating the weight of the building, 3 sections were considered to simplify the different floor joists system used. Also, an average size of beam of 24"x24"was taken to represent all sizes to simplify the calculations of each weight of the beams.

The loads from seismic forces originate from the inertia of the structure itself, which is related to the mass of the structure. Most of the mass of the structure is locked in the slabs, beams, joists, and columns which are connected to the shear walls. When seismic loads are generated by a ground motion, the slabs transfer the loads directly into the shear walls, which then carry the loads down to the foundations and therefore to grade.

It was assumed that the site is classified as site class E or stiff soil. After calculating the SMs, and S1, the SD1 and SDM were computed which lead to a design category for this structure A. This means that each lateral force at every floor is the weight of the floor multiplied by 0.01.
Seismic forces in the N-S direction are listed and diagramed in Figure 21. The resultant base shear in this direction is 193 k and the overturning moment was 10,819 k-ft. The calculations cannot be compared to those of the engineer's as no analysis was done.

Furthermore, to follow the ASCE 7-05 and get more accurate loading on the building an accidental moment was computed. In order to compute those moments, a 5% of the building's length in each direction was taken as eccentricity. Those loads that represent Mzx and Mzy in the load combinations found in figure 14 of the report. In return, the force was multiplied by the eccentricity and a torsional amplification factor, Ax. In fact, that factor is initially assumed to be equal to 1.0 in order to get max and min drifts on each level and recalculate its true value. The maximum and minimum drift per level and Ax were derived according to the figure 12.8-1 from ASCE 7-05 found on figure 19 below.

Seismic Forces - N-S Direction					
Level	Story weight, w_x	height (ft), h_x	Story force (k) Fx=0.01, w_x	Story Shear (k)	Overturning moment (k-ft)
2	2895	15	29	193	420
3	2893	29	29	164	839
4	2893	44	29	135	1258
5	2893	58	29	106	1678
6	2944	73	29	77	2134
7	3133	87	31	48	2726
8	1648	107	16	16	1764
Total=	19299	Base Shear =			193
		Total Overturning moment=			10819

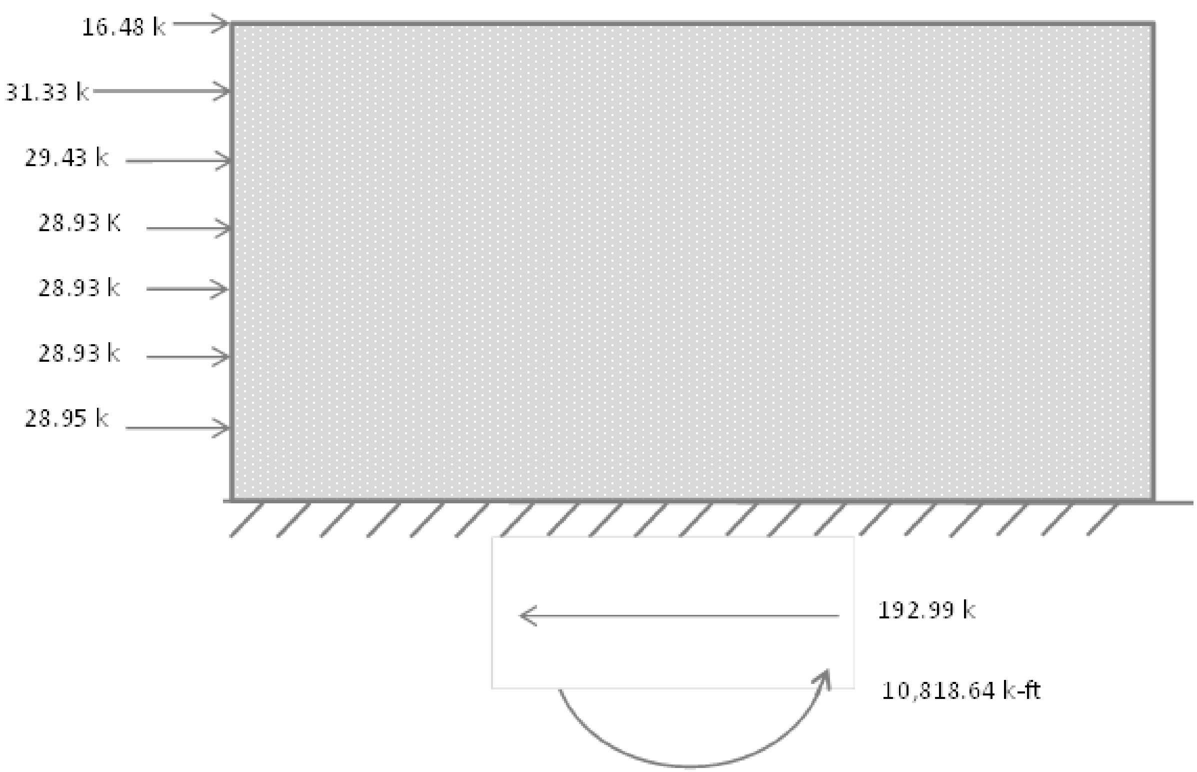


Figure 19 - **List and diagram showing the Seismic forces on the building in the N-S direction**

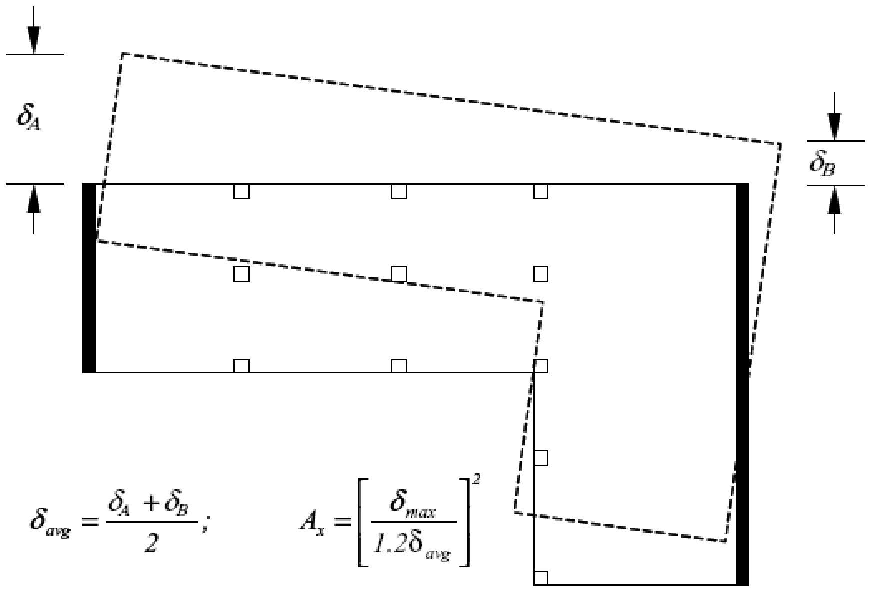


Figure 20 - Figure showing max/min drift and Torsional Amplification factor, Ax from ASCE 7-05

After retrieving the true value of Ax, a comparison was made to determine if the building is torsional irregular. Even though, seismic category A does not require this, it was chosen to be completed due to the irregular shape of the building. If Ax was found above 1.2 then it is type 1-a irregular and if Ax is in between 1.2 and 1.4 respectively then it is type 1-b irregular. From table 12.3-1 of ASCE 7-05, type 1-a is torsional irregularity and type 1-b is extreme torsional irregularity. The results came that the building is not torsional irregular in the X-direction however is extreme torsional irregular in the Y-directions. These table and calculations can be found in further details in appendix C.

The story drift was determined according to section 12.8.6 “Story drift determination” in ASCE 7-05. See figure 20.

$$\delta_x = \frac{C_d\,\delta_{xe}}{I} \qquad (12.8\text{-}15)$$

where

C_d = the deflection amplification factor in Table 12.2-1
δ_{xe} = the deflections determined by an elastic analysis
I = the importance factor determined in accordance with Section 11.5.1

Figure 21 - Story drift determination

The “I” factor was taken 1.0 and “Cd” was retrieved from table 12.2 -1 as 4 . This amplified the drifts in each direction by 4.0 but it was still under the code allowance of .01hsx. To see in details these calculations please refer to appendix C.

Problem Statement

Since it is well designed, there is not much that could be done to the J.B Byrd Center that would lead to major improvements. The structural system is suitable in strength, cheap and is equitable in comparison to typical alternatives. In fact, the only two realistic replacements would be cast-in-place concrete or a steel frame building as noted in Tech 2 but even then only minimal differences are produced. Thus, redesigning the building other than the systems noted previously would produce a non-feasible solution especially in its current location.

Furthermore, as the author is interested in seismic design a scenario has been created in which an identical Alzheimer's Center and Research Institute to the J.B Byrd. Center is being requested to be built in San Diego, California. To be more specific, the University of San Diego (USD) will be taken as the new campus of this building. This change in location will alter the wind and seismic forces imposing the design to be controlled by seismic instead of wind. The same geotechnical report will be used for unavailability of (USD) campus's geotechnical report and for being conservative as the current soil properties are poor.

Moreover, the scenario chosen is in contemporary with major seismic events that happened in 2011 all over the world. The earthquakes in Chile, New Zealand and Japan made engineers more ardent in averting catastrophes in the future. Typical materials and construction method will be used to design the structure for the new building.

Therefore, a reasonable system must be designed to provide sufficient strength and serviceability to prevent the building from collapse after a major seismic event. The new design will be able to resist all dead, live, wind and seismic loads with little impact to architecture in order to satisfy the new owner. Nonetheless, the cost and schedule is likely to increase.

Also, in the last decade, a major movement in building design has been to improve the performance of buildings above the minimum design requirements without significant cost impacts. Therefore, the owner in the proposed scenario has requested the building be designed for an ASCE Structural Performance Level of "S-3 Life Safety" to target life safety following an earthquake with some damage to structural components and "S-5 Prevention Collapse" targeting life safety following an earthquake with severe structural damage. A comparison of the requirements for S-3 requirements and the minimum S-5 requirements can be found in table C1-3 below, taken from FEMA 356.

Table C1-3 Structural Performance Levels and Damage[1, 2, 3]—Vertical Elements (continued)

		Structural Performance Levels		
Elements	**Type**	**Collapse Prevention S-5**	**Life Safety S-3**	**Immediate Occupancy S-1**
Concrete Walls	Primary	Major flexural and shear cracks and voids. Sliding at joints. Extensive crushing and buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element stress, including limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing, but concrete generally remains in place.	Minor hairline cracking of walls, <1/16" wide. Coupling beams experience cracking <1/8" width.
	Secondary	Panels shattered and virtually disintegrated.	Major flexural and shear cracks. Sliding at joints. Extensive crushing. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Minor hairline cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience cracks <1/8" width. Minor spalling.
	Drift	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligible permanent
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent

Table C1-3 Structural Performance Levels and Damage[1, 2, 3]—Vertical Elements

		Structural Performance Levels		
Elements	**Type**	**Collapse Prevention S-5**	**Life Safety S-3**	**Immediate Occupancy S-1**
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints <1/16" width.
	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent

Figure 22 - **Comparison of performance requirements for different structural performance levels for Concrete Walls (this is similar to table C1-3 of ASCE 41)**

Proposed Solution

Two concrete systems will be compared in this proposal. As this is a different construction type than the original design, the gravity system will be re-designed first then the lateral system will follow according to ASCE 7-05. Once all the system is designed, base isolators placed above the basement walls will be placed creating a different system. Thus, the following structural systems will be compared:

- A Cast-in-Place Dual System (Moment Frames and Shear Walls)
- A Cast-in-Place Dual System (Moment Frames and Shear Walls) with Base Isolators

The first solution will be a cast-in-place system chosen to fit typical construction in San Diego. The concrete system will be designed to support the existing gravity loads. It will consist of typical sizes already used in the building using the same layout. An increase in member sizes may be needed if an alternative layout was determined to be used to fit the building's structural needs. However, the shear walls arrangement will stay intact so the architectural layout is not changed.

The second solution will be the same system as above but with base isolators above the foundations. Base isolators are a collection of structural elements which should substantially decouple a superstructure from its substructure resting on a shaking ground thus protecting the building structure's integrity. Base isolation is one of the most powerful tools to protect a building from a potentially devastating earthquake. In fact, the isolators allow the structure to respond much more slowly than it would without them, resulting in lower seismic demand on the structure. They will help on lowering deflections and cracks to both structural and non-structural components in the building. There are main categories of base isolators: Rubber, Lead, and Steel. In this proposal, the type of base isolator is lead. To view such a type, please refer to the image below.

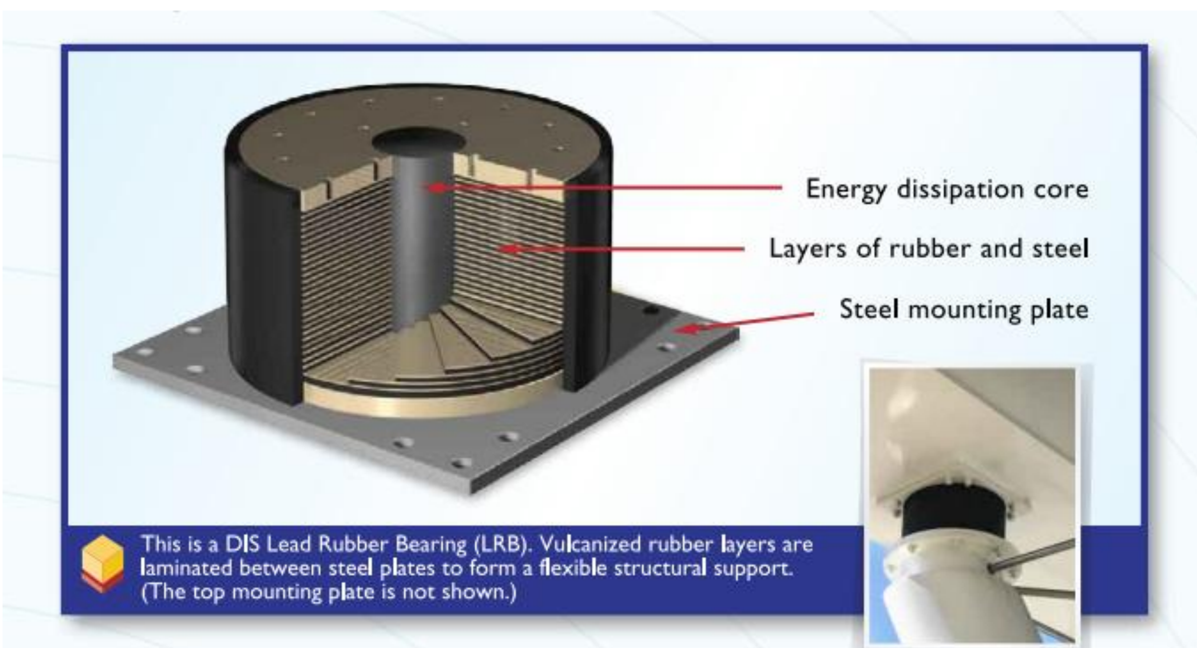


Figure 23 - **Lead Rubber bearing base isolators taken from Teratec Dynamic Isolation Systems Brochure**

Lead was chosen because of its plastic property. In fact, while it may deform with the movement of the earthquake, it will revert to its original shape, and it is capable of deforming many times without losing strength. During an earthquake, the kinetic energy of the earthquake is absorbed into heat energy as the lead is deformed.

MAE Material Incorporation

Much of the calculation of the proposed redesign will draw upon material learned in MAE courses. Computer modeling techniques taught in AE 597A or Computer Modeling will be an integral tool in the completion of this redesign. Concepts such as insertion points, rigid diaphragm constraints, panel zone modeling, property modifiers, and modal analysis results determination were taught for ETABS and SAP 2000. These skills will be applied to ETABS and potentially SAP.

The design of the concrete moment frames and base isolators will rely heavily on material presented in AE 538 - Earthquake Design. The limitations and requirements for concrete moment frame and the procedures used to implement performance-based design will be of particular use. However, even though base isolation wasn't covered in depth, the design of such system will be done through faculty advice and the author's own research and knowledge.

Breadth Studies

To address the integrated nature of the Architectural Engineering program, two breadth studies are also included as a part of this report. The first is a construction management breadth, which uses quantities of superstructure components and data from RS Means to determine the duration and cost of each structure. This was used to help compare the designs to determine the relative efficiency of each.

The second breadth study attempts to determine if a curtain wall photovoltaic system would be viable now that the building is in California. This technology was not included in the original building and is being considered as a sustainable design.

The BIPV (or Building Integrated Photovoltaic) has the potential to earn the building additional LEED points. To fully capture the viability of the system, a life cycle assessment and payback period will be evaluated for the design. Also, an architectural detail using Google Sketchup will be done to determine the aesthetic change of the façade where the PV will be applied. After evaluating the viability of the proposed BIPV, it should be possible to conclude if this system will be more beneficial to the J.B Byrd Alzheimer's Center.

Structural Depth

Two concrete systems will be compared in this proposal. As this is a different construction type than the original design, the gravity system will be re-designed first then the lateral system will follow according to ASCE 7-05. Once all the system is designed, base isolators placed above the basement walls will be placed creating a different system. Thus, the following structural systems will be compared:

- A Cast-in-Place Dual System (Moment Frames and Shear Walls)
- A Cast-in-Place Dual System (Moment Frames and Shear Walls) with Base Isolators

California Site Overview

As mentioned before the University of San Diego (USD) is requesting an identical Alzheimer's Center and Research Institute to the J.B Byrd Center to be built on their campus. The same geotechnical report as the original location will be used for the unavailability of the USD campus geotechnical report. This is a conservative approach as the current soil properties are poor. As can be seen below, the site is large enough to accommodate the building's footprint. The building's nature fits with the surrounding buildings as Science, Health and Technology. The orientation of the building will change considering the unique architecture of the USD campus. Since this is a modern building, the white plaster façade will face the existing buildings to respect the traditional Spanish architecture. In fact, this will orient the curtain wall façade towards the South where the BIPV will be most productive. The landscape of the proposed site will be changed to accommodate the entrances of the buildings. This will not be discussed as it was not part of the original proposal.

Figure 24 - **Image from Google Maps showing the site selected on the University of San Diego (USD) campus. The approximate footprint of the Alzheimer's Center is shown in green.**

Since the geotechnical report is assumed to be the same the site class revealed was Class D, just like the Tampa, FL site, which is the most crucial parameter for the production of the designs in this report. This is a conservative assumption since the soils at the California site are of a much higher quality. Secondly, it is assumed that ground water is a not an issue on this site, as opposed to the potential sinkhole site in Tampa, FL. Both of these differences would be important in redesigning the below-grade portions (the basement and the foundations) of the J.B Byrd Alzheimer's Center. However, these portions of the building were not included in the redesign, and therefore these conditions can be neglected.

Fixed Cast-in-Place One Way Slab CA (S-5)

It was proved in previous report that a one way slab would be the most viable option for a gravity redesign in California. This system was chosen over the existing system of the precast joist and soffit beams due to its lack of presence and cost in California compared to Florida. The redesigns were done sequentially as mentioned in the proposed solution.

Gravity system

First, the structure was redesigned as a one way pure gravity system. This was accomplished by selecting a typical laboratory lab with high loads shown in the figure below and then designing the slab, infill beams, girders and column.

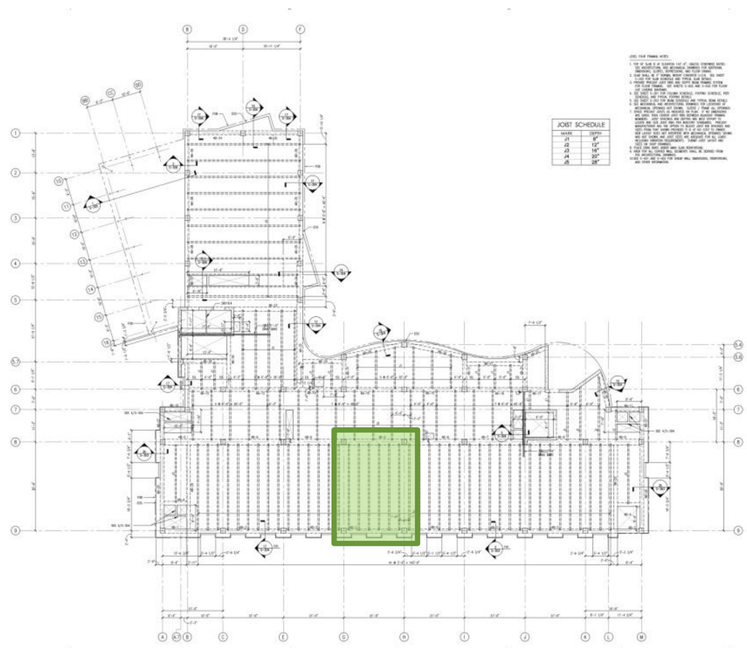


Figure 25 - **Floor plan indicating the typical bay used for preliminary hand calculations for the one way gravity design.**

The layout on figure 26 was chosen to minimize the slab thickness in order to minimize the weight, and minimize any architectural or mechanical differences. A total of 5"thick slab was chosen for vibration, fire proofing requirements and according to minimum slab thickness table 9.5 (a) in ACI 318-08. The slab lies on top of 16" wide by 24" deep joists and 20"wide by 24" deep for moment resisting beams and girders. Both have equal depth for formwork and constructability reasons to reduce costs. The beams are spaced at 7'-0" to conform with the original layout as no architectural changes were desired. Everything has a concrete strength of 4,000 psi normal weight and 60,000 psi for steel reinforcements.

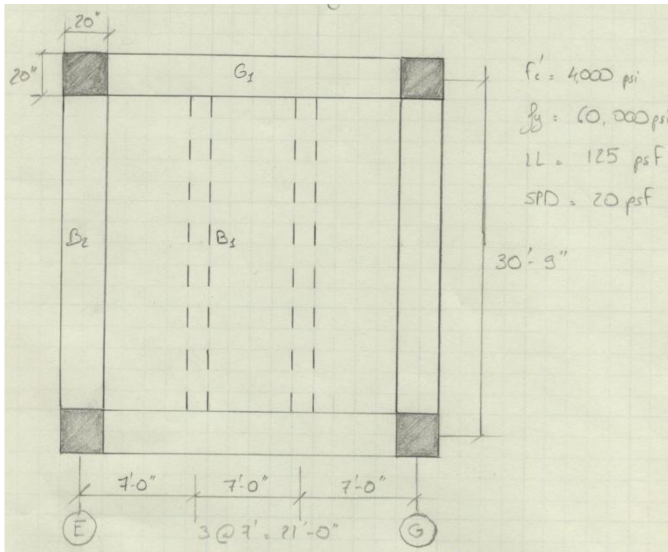


Figure 26 – **Layout chosen for one way cast-in-place design.**

All structural members were designed using the ACI coefficient from ACI 318-08. Please note for the simplicity of the calculations that last was used even though not all of the requirements were satisfied. Upon completion of the design calculations it was determined that the slab was designed to have #4 at 12" on center for flexure, shrinkage and temperature. The beam spanning the 30'-9" had large negative moments which required more reinforcements. Also, since the bay is at the edge of the building the beam was analyzed at the supports and mid-span totaling 3 zones. The following reinforcements were designed starting from the edge going to the interior of the building: (2) #9, (3) #9 and (4) #9. The girder had (2) #6 at mid-span and (4) #9 at the supports. All of the members had a # 4 stirrup. The detailed calculations for the one-slab system can be found in Appendix D.

After all the hand calculations were computed a check was done using RAM Concept V8i. The layout and the sizes of beam and girders were kept the same.

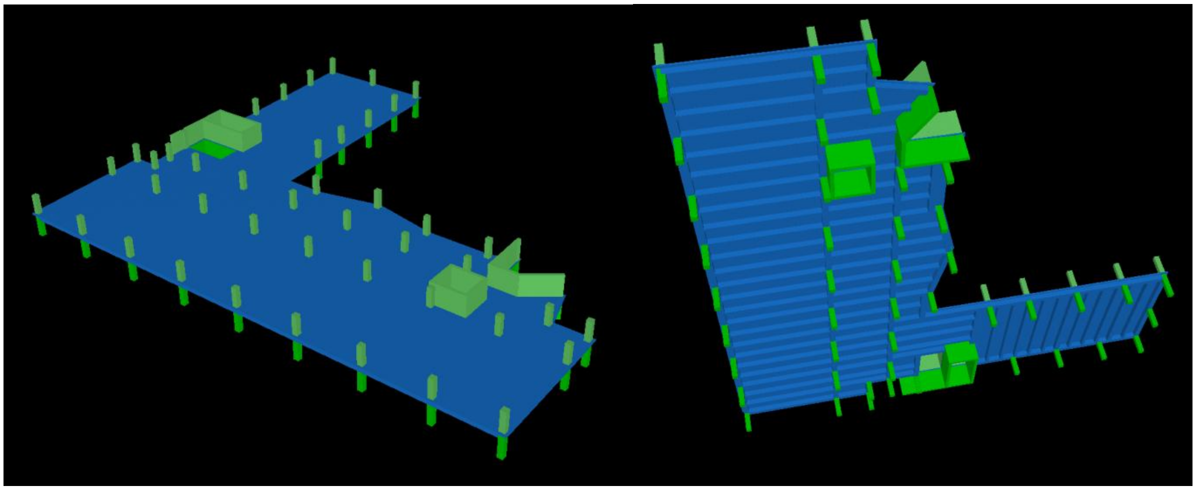


Figure 27 - **Screen shots taken from RAM Concepts to analyze the one way slab system proposed. The picture on the right shows an aerial view of the top of a typical floor and the picture on the right shows a worm's eye view of the bottom of a typical floor.**

After modeling the elements with the same assumptions used in the hand calculations a discrepancy of 7% was found. Ram Concepts in fact had 7% more reinforcements (30.5 tons) than what was calculated (27.8 tons). This is a reasonable difference due to the fact that the bays change in overall thus the assumptions made by hand were to simplify the calculations. To see the details of the hand calculations please refer to Appendix D.

Lateral system

The lateral system was chosen to be a dual system of concrete shear walls and moment frames. This design is an effective lateral system and comparatively cheap compared to others. It is the same system as the original to minimize architectural discrepancies, but enhanced to resist the greater earthquake loads in California. The dual lateral system requires that in both directions the moment frames needs to resist at least 25% of the lateral forces. The layout of the shear walls was kept as the building located in Florida in order to keep the functionality of the building the same. This caused the inability to avoid torsional problems that needed to be addressed in the design. Before the iteration design method was done to compute the right lateral system, wind and seismic loads at the California site were calculated.

Wind Loads

It was necessary to calculate wind loads for this structure to verify they did not exceed the seismic loads in California which were used for design. The basic wind speed for the California site is 85 mph, as opposed to 120 mph at the Tampa, FL site. This required the wind loads to be recalculated for the California site. The assumptions made for the calculation of wind loads at the Tampa site were also applied to the California site (see the "Wind Loads" subsection of the "Lateral Loads" section for a discussion of what these assumptions were). The gust factor was calculated for the building as it was considered a flexible structure with a period above 1.0 second. The full set of parameters used for the calculation of these wind loads can be found in Appendix B.

The wind pressures in both directions are listed in the Tables below. The N-S direction pressures were resolved into wind forces in the same direction, which are listed and diagramed in Figure 28. The resulting base shear is 340 k when the 1.6 load factor is considered. The E-W pressures were resolved into wind forces in the E-W direction, which are listed and diagramed in Figure 29. The resulting base shear in this direction is 448 k when the 1.6 load factor is considered. The factored base shears were used to compare to the seismic loads for each design to verify that the lateral design was controlled by seismic forces.

Desgin wind pressure for MWFRS in N-S Direction						
type	Level	Height / distance	qz/ qh	wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
windward walls	Roof	107'	18.63	12.67	2.42	22.92
	7	87'	17.59	11.96	1.71	22.21
	6	72'-6"	16.65	11.32	1.07	21.57
	5	58'	15.57	10.59	0.34	20.84
	4	43'-6"	14.38	9.78	-0.47	20.03
	3	29'	12.80	8.70	-1.55	18.95
	2	14'-6"	10.54	7.17	-3.08	17.42
	1	0'	10.54	7.17	-3.08	17.42
leeward walls	All	All	18.63	-6.94	-17.19	3.31
sidewalls	All	All	18.63	-11.09	-21.34	-0.84
Roof		0-53.5	18.63	-15.02	-25.26	-4.77
		53.5-107	18.63	-13.88	-24.12	-3.63
		107-214	18.63	-8.30	-18.55	1.95

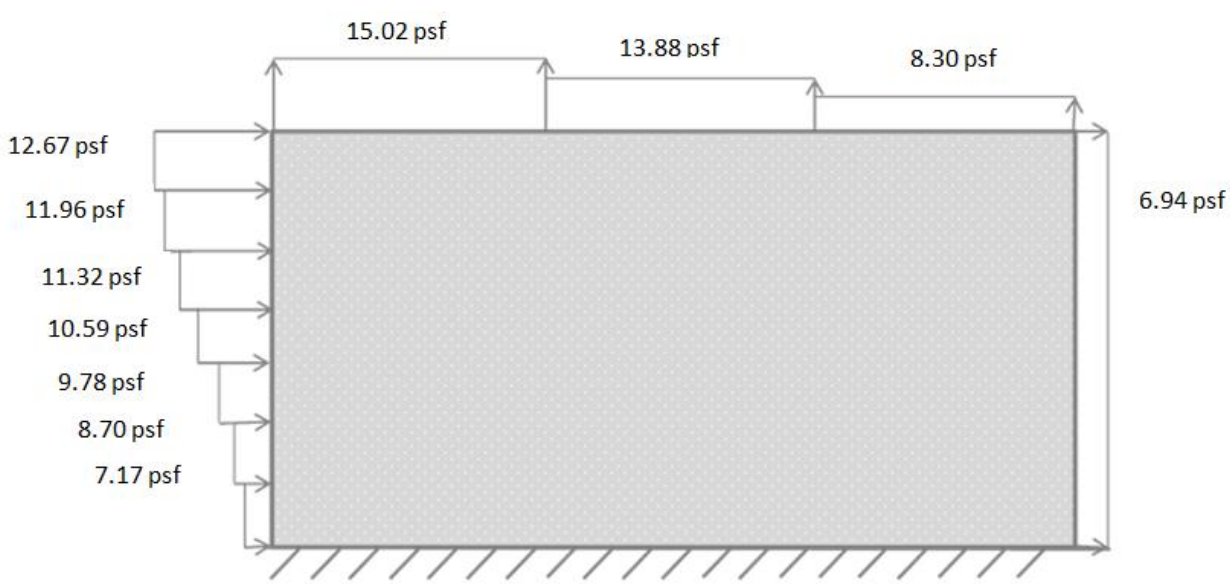


Figure 28 - **List and diagram of the wind pressures on the building in the N-S Direction at the California site.**

Desgin wind pressure for MWFRS in E-W Direction						
type	Level	Height / distance	qz/ qh	wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
windward walls	Roof	107'	18.63	12.67	2.42	22.92
	7	87'	17.59	11.96	1.71	22.21
	6	72'-6"	16.65	11.32	1.07	21.57
	5	58'	15.57	10.59	0.34	20.84
	4	43'-6"	14.38	9.78	-0.47	20.03
	3	29'	12.80	8.70	-1.55	18.95
	2	14'-6"	10.54	7.17	-3.08	17.42
	1	0'	10.54	7.17	-3.08	17.42
leeward walls	All	All	18.63	-7.92	-18.17	2.33
sidewalls	All	All	18.63	-11.09	-21.34	-0.84
Roof		0-53.5'	18.63	-17.17	-27.42	-6.92
		53.5'-107'	18.63	-12.80	-23.05	-2.55
		107'-214'	18.63	-9.38	-19.63	0.87

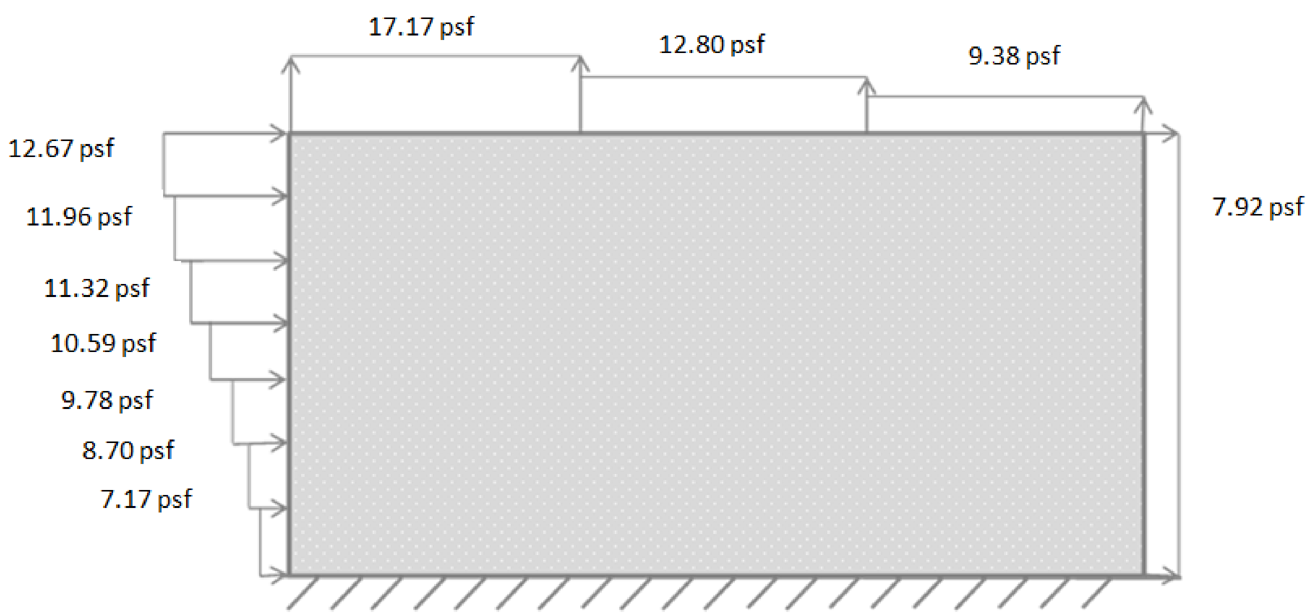


Figure 29 - **List and diagram of the wind pressures on the building in the E-W Direction at the California site.**

Wind Forces- N-S Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Roof	107	10.0	1450.0	10.0	1450.0	33.2	33.2	3556
7	87	7.0	1015.0	7.5	1087.5	57.4	90.6	4993
6	72.5	7.0	1015.0	7.5	1087.5	46.0	136.6	3335
5	58	7.0	1015.0	7.5	1087.5	44.6	181.2	2584
4	43.5	7.0	1015.0	7.5	1087.5	42.9	224.1	1868
3	29	7.0	1015.0	7.5	1087.5	40.9	265.0	1187
2	14.5	7.0	1015.0	7.5	1087.5	38.2	303.2	554
1	0'	N/A	0.0	7.5	1087.5	36.6	339.8	0
							Total base shear=	340
							Total overturning Moment=	18076

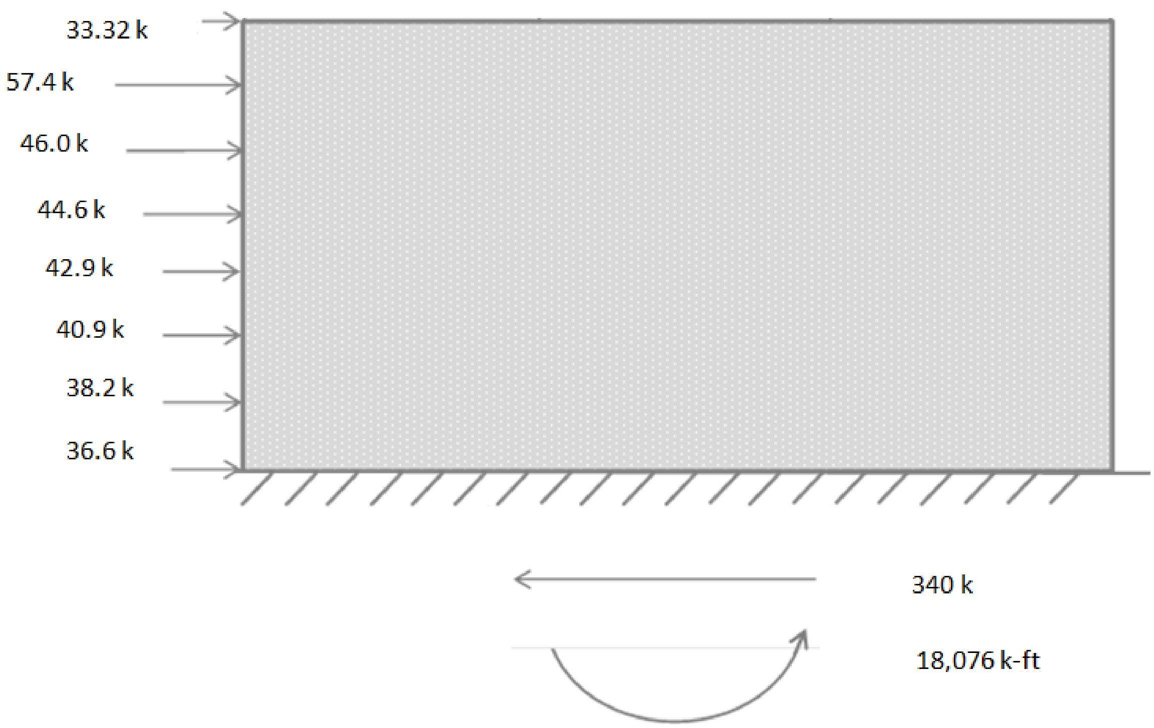


Figure 30 - **List and diagram showing the wind forces on the building in N-S direction at the California site. Note: all forces include the 1.6 factor per ASCE 7-05 Chapter 2.**

Wind Forces- E-W Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Roof	107	10.0	1910.0	10.0	1910.0	43.8	43.8	4684
7	87	7.0	1337.0	7.5	1432.5	75.6	119.4	6577
6	72.5	7.0	1337.0	7.5	1432.5	60.6	180.0	4393
5	58	7.0	1337.0	7.5	1432.5	58.7	238.7	3404
4	43.5	7.0	1337.0	7.5	1432.5	56.6	295.2	2460
3	29	7.0	1337.0	7.5	1432.5	53.9	349.1	1564
2	14.5	7.0	1337.0	7.5	1432.5	50.3	399.4	729
1	0'	N/A	0.0	7.5	1432.5	48.2	447.7	0
							Total base shear=	448
							Total overturning Moment=	23811

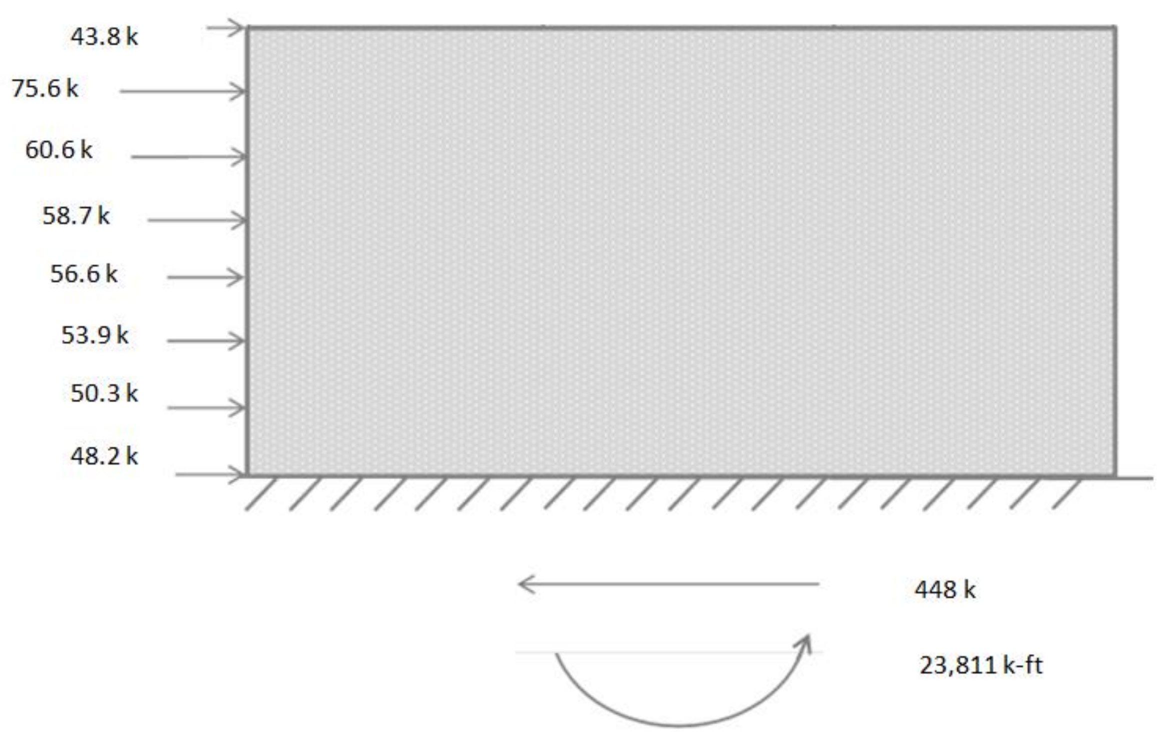


Figure 31 - List and diagram showing the wind forces on the building in N-S direction at the California site. Note: all forces include the 1.6 factor per ASCE 7-05 Chapter 2.

<u>Seismic Loads</u>

Seismic loads were calculated with the Equivalent Lateral Force procedure outlined in Chapters 11 and 12 of ASCE 7-05. This procedure assumes a simple building footprint. Also, an average size of beam of 24" wide by 24"deep was taken to represent all sizes to simplify weight calculations. It appeared that the total weight of the one way slab system used and original precast joists and beam soffit were approximately the same. The total weight of the structure used is 20,000 kips.

The loads from seismic forces originate from the inertia of the structure itself, which is related to the mass of the structure. Most of the mass of the structure is locked in the slabs, beams, joists, and columns which are connected to the shear walls. When seismic loads are generated by a ground motion, the slabs transfer the loads directly into the shear walls, which then carry the loads down to the foundations and therefore to grade.

The best system chosen for seismic category D was E-2 according to table 12.2 -1 in ASCE 7-05 as shown in the figure below. The table states E-2 as a dual system with intermediate moment frames capable of resisting at least 25% of prescribed seismic forces and special reinforced concrete shear walls. This system was chosen due to the nature of the original building, to resist the loads of a seismic category D and a height less than 160 feet. It has a response modification factor R of 6 ½ and a deflection amplification factor Cd of 5.

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS (continued)

Seismic Force-Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (ft) Limit[c] Seismic Design Category B	C	D^d	E^d	F^e
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Special steel concentrically braced frames[f]	14.1	6	2½	5	NL	NL	35	NP	NP[h,k]
2. Special reinforced concrete shear walls	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear	14.4	3½	3	3	NL	NL	NP	NP	NP

Figure 32 - **Table 12.2-1 taken from ASCE 7-05 showing the system used for seismic category D for the California site.**

Furthermore, to follow the ASCE 7-05 and get more accurate loading on the building an accidental moment was computed. In order to compute those moments, a 5% of the building's length in each direction was taken as eccentricity. Those loads that represent Mzx and Mzy in the load combinations found in Appendix C. In return, the force was multiplied by the eccentricity and a torsional amplification factor, Ax. In fact, that factor is initially assumed to be equal to 1.0 in order to get max and min drifts on each level and recalculate its true value. ELF seismic forces are listed in the figure below.

Seismic Forces							
Level	Story weight, w_x	Story height (ft), h_x	$w_x.h_x^K$	C_{vx}	Story force (k)	Story Shear (k)	Overturning moment (k-ft)
8	1648	121.5	564183.7326	0.16183	383.2	383.2	46559
7	3133	101.5	861718.7654	0.24718	585.3	968.5	59407
6	2944	87	671221.7942	0.19254	455.9	1424.4	39663
5	2893	72.5	528510.0985	0.1516	359.0	1783.4	26025
4	2893	58	402933.6984	0.11558	273.7	2057.0	15873
3	2893	43.5	284012.5809	0.08147	192.9	2249.9	8391
2	2895	29	173634.6248	0.04981	118	2367.9	3420
1	2895	14.5	74747.71396	0.02144	51	199338.2	736
$\sum$	22194	$\sum w_i h_i^K$=	3486215.295			**Base Shear =**	**2368 kip**
						Total Overturning Moment =	**199338 kip**

Figure 33 - **Seismic forces in N-S direction using the ELF method at the California site.**

The equivalent lateral force analysis was performed for the current location. However, due to the torsional irregularity and the seismic design class of D for the high seismic region, a modal response spectrum analysis had to be performed for the current location to check the values from the equivalent lateral force analysis. After the lateral was established, base shears were found again using the Modal Response Spectrum Analysis (MRSA) procedure on a finite element model constructed in ETABS with the cracked section properties modeled by a 50% reduction on the modulus of elasticity for all concrete materials. This involves calculating a Cs-like quantity using the modal periods for sufficient modes to obtain 90% mass-participation in two orthogonal translational directions. This base shear is typically lower than that calculated by the ELF procedure. However, it is limited by an absolute minimum of 85% of the base shear calculated by ELF. The equations for this process follows section 11.4.5 from ASCE 7-05 and can be seen in Appendix E.

Envelope of T for each mode		Sa calculation
A*	for T<To	Sa= 0.4+0.6 (T/To)
B*	for To ≤ T ≤ T_S	Sa= S_{DS}
C*	for Ts ≤ T ≤ T_L	Sa= S_{D1}/T
D*	for T > T_L	Sa= SD1*T_L /T^2
with	To= 0.2 S_{D1}/S_{DS} =	0.12 sec
	Ts= S_{D1}/S_{DS} =	0.59 sec
	T_L=	8.00 sec
*: Envelope type created by the author for ease of identification		

Figure 34 - **Table showing the period envelopes in order to calculate the design response Spectrum.**

The base shear in both directions was controlled by 85% minimum, and was therefore found to be 2013 k. The MRSA method was determined using the original design. These seismic forces are listed and diagrammed in Figure 36.

Modal Information									
Mode	Period	UX%	UY%	Envelope	Sa	S_a/(R/I)	Cm,i	$(Cm,i*UX\%)^2$	$(Cm,i*UY\%)^2$
1	1.844882	0.0848	60.7839	C	0.350158	0.05387	0.05387	2.08686E-09	0.001072206
2	1.497176	49.564	3.6154	C	0.431479	0.066381	0.066381	0.001082494	5.75977E-06
3	1.150446	23.3993	8.9318	C	0.561521	0.086388	0.086388	0.000408612	5.95366E-05
4	0.404201	2.0679	10.9419	B	1.091	0.167795	0.167795	1.20397E-05	0.000337088
5	0.37798	10.5813	5.326	B	1.091	0.167795	0.167795	0.000315236	7.98656E-05
6	0.252525	6.6079	2.0383	B	1.091	0.167795	0.167795	0.000122937	1.16975E-05
								$C_{m,x}=SQRT(\sum(C_{m,i}*UX\%)^2)$=	0.044060424
								$C_{m,y}=SQRT(\sum(C_{m,i}*UY\%)^2)$=	0.039574653
								0.85Cs=	0.090686413

Figure 35 - **Modal Information used to find Cm, which was used to calculate MRSA seismic forces**

Seismic Forces							
Level	Story weight, w_x	Story height (ft), h_x	$w_x.h_x^K$	C_{vx}	Story force (k)	Story Shear (k)	Overturning moment (k-ft)
8	1648	121.5	564183.7326	0.16183	326	325.7	39575
7	3133	101.5	861718.7654	0.24718	497	823.2	50496
6	2944	87	671221.7942	0.19254	388	1210.7	33714
5	2893	72.5	528510.0985	0.1516	305	1515.8	22121
4	2893	58	402933.6984	0.11558	233	1748.5	13492
3	2893	43.5	284012.5809	0.08147	164	1912.4	7133
2	2895	29	173634.6248	0.04981	100	2012.7	2907
1	2895	14.5	74747.71396	0.02144	43	169437.4	626
$\sum$	22194	$\sum w_i h_i^K$=	3486215.295			**Base Shear =**	**2013 kip**
					Total Overturning Moment =		**169437 kip-ft**

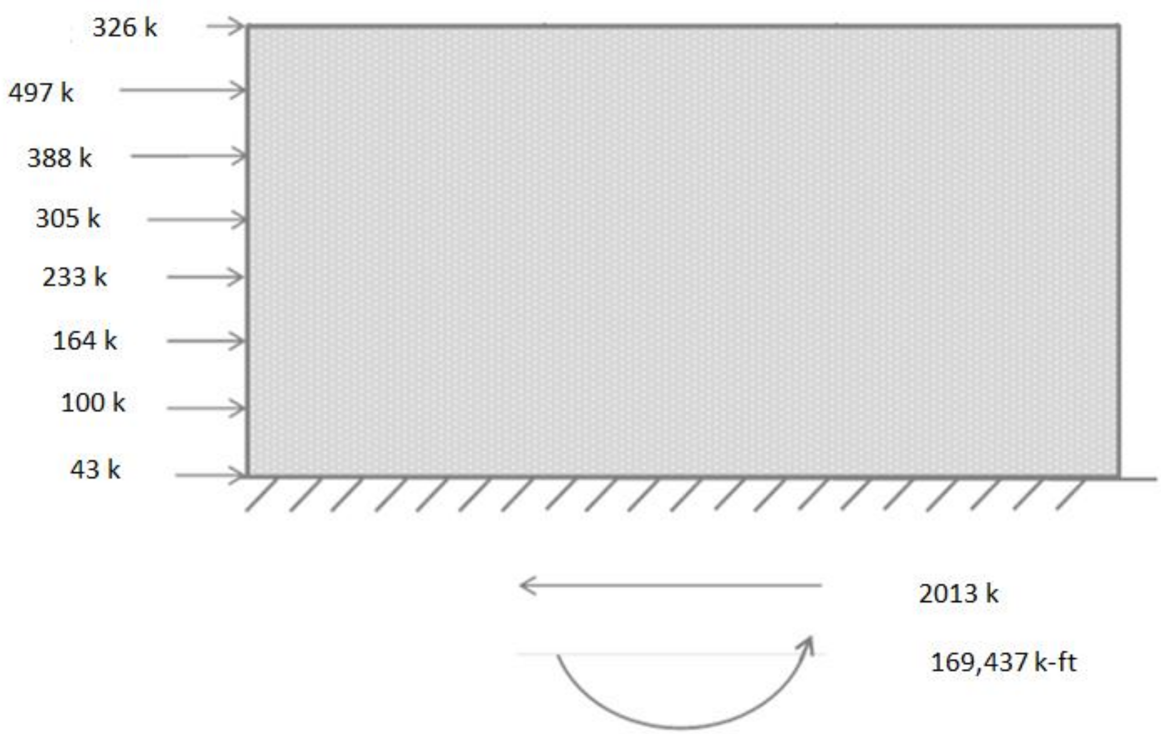


Figure 36 – **List and diagram of seismic forces for the CA-S5 design in the N-S direction, found with the MRSA procedure. These represent the forces in both directions.**

Earthquake thus controls by a factor of by almost 6 in the N-S direction and 4.5 in the E-W direction. After finding all the lateral forces a torsional check needed to be done for seismic category D. This was done by taking the max inter-story at one end ΔA and one adjacent to it as ΔB and computing their average. Then, the max inter-story was divided by the average to compare it to two different numbers:

1.2 times the average and 1.4 times the average. If the max over the average was determined to be in between 1.2 and 1.4 times the average then the diaphragm was determined to have a type 1-a horizontal torsional irregularity. If the max over the average was computed to be more than 1.4 times the average then the diaphragm was determined to have a type 1-b extreme torsional irregularity. The results of the calculations can be seen in the tables below. For further details of these calculations please refer to Appendix E.

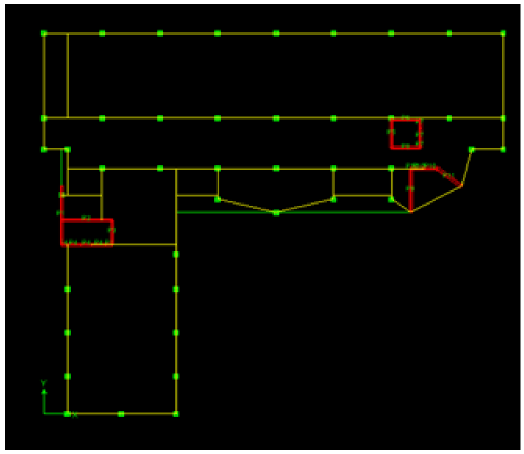
Story level	Earthquake Loads X-direction			
	$\Delta max/\Delta avg$	Type 1a - 1b	type	No torsional Irregularity
8	1.09	1.2 - 1.4 Δavg	none	
7	1.13	1.2 - 1.4 Δavg	none	
6	1.15	1.2 - 1.4 Δavg	none	
5	1.16	1.2 - 1.4 Δavg	none	
4	1.16	1.2 - 1.4 Δavg	none	
3	1.15	1.2 - 1.4 Δavg	none	
2	1.11	1.2 - 1.4 Δavg	none	
1	1.06	1.2 - 1.4 Δavg	none	

Story level	Earthquake Loads Y-direction			
	$\Delta max/\Delta avg$	Type 1a - 1b	type	Type 1-a torsionally Irregular
8	1.24	1.2 - 1.4 Δavg	1-a	
7	1.26	1.2 - 1.4 Δavg	1-a	
6	1.29	1.2 - 1.4 Δavg	1-a	
5	1.29	1.2 - 1.4 Δavg	1-a	
4	1.29	1.2 - 1.4 Δavg	1-a	
3	1.29	1.2 - 1.4 Δavg	1-a	
2	1.27	1.2 - 1.4 Δavg	1-a	
1	1.35	1.2 - 1.4 Δavg	1-a	

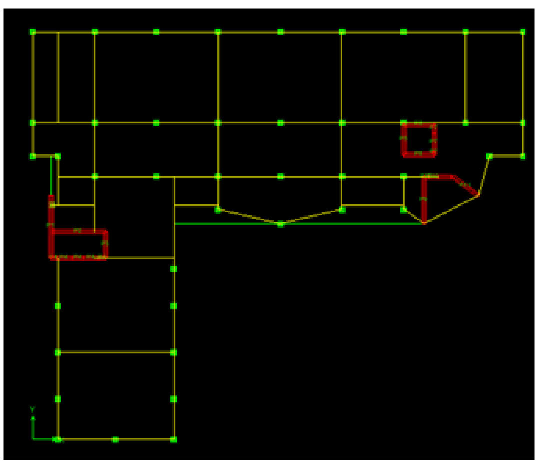
Figure 37 - **Tables showing the structure is torsional irregular in the Y-direction under the CA-S5 lateral system.**

Thus, after running all the calculations, it was found that the structure is torsional irregular (type 1-a) in the Y-direction. This will cause the strength design method of structural members to be multiplied by a redundancy factor, ρ =1.3. This factor will be used for the spot checks.

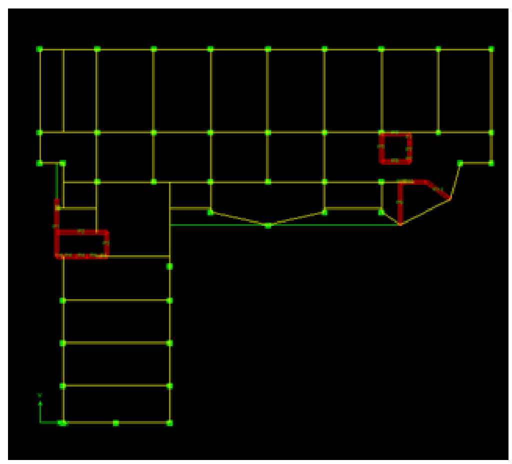
Subsequently, after torsional irregularity has been determined a lateral system needed to be determined to resist the earthquake forces accordingly. Since it was assumed that the shear walls were not to be re-arranged, and a dual system was chosen, the only option left was to determine moment frame layout. Thus, two proposed moment frames layout were investigated to be able to resist the lateral loads. The building was torsional irregular in the Y-direction an increase of stiffness of the moment frames in that direction was considered. Also, there was no need to compute relative stiffness for the new system since an increase in the moment frames will only confirm the 25% stiffness already found in the original structure (Relative stiffness calculations can be seen in Appendix C for original model). The figure below on the left shows layout A and the figure below on the right shows layout B studied as compared to the original placed above them.



(i)



(ii)



(iii)

Figure 38 - **Screenshots taken from ETABS showing the different concrete moment frame layouts to resist the lateral loads in California starting with, (i) the original layout of the J.B Byrd Center, (ii) Layout A with a slight moment frame increase, (iii) Layout B with maximum possible moment frames.**

In fact, layout A was chosen as a first iteration step in order to resist the loads however was proven to be inefficient. Layout B was then created to be able to minimize inter-story drift in able to pass the designed requirements. After viewing layout B, the possibility of a two-way slab instead of a one way was investigated. It was found that according to the minimum thickness of slab from table 9.5 (a) and (b), the two way slab would require a thickness of 14" which would have greatly increased the weight of the building hence unviable.

The sizing of the system was chosen after several iterations in ETABS of different combinations of shear wall thickness, beam sizes and moment frame layouts. The iterations were compared to see if they meet S-5 or S-3. The max inter-story drifts according to FEMA for permanent S-5 was 2% or 3.48 inches for a typical height of 14.5 feet and 1 % for S-3 or 1.74 inches in each the Y and X directions. The results can be found in the table below:

Original design								
Period (sec)	total deflection in Y (inch)	total deflection in X (inch)	Max drift in Y (inch) between 5-6	Max drift in X(inch) between 3-4	S5= 2%= 3.48"	S3= 1%= 1.74"	S5= 2%= 3.48"	S3= 1%= 1.74"
1.841	37.74368	26.559545	5.500085	3.91366	NG	NG	NG	NG

Layout A- Added moment frames on (C,G,I 9-6)(K,9-6)(3,B-F) without base isolation										
Wall size	Beam size	Period (sec)	total deflection in Y (inch)	total deflection in X (inch)	Max drift in Y (inch) between 5-6	Max drift in X (inch) between 3-4	Y -direction		X-direction	
							S5= 2%= 3.48"	S3= 1%= 1.74"	S5= 2%= 3.48"	S3= 1%= 1.74"
12"	20x24	1.737	32.21	26.29	4.686	3.875	NG	NG	NG	NG
	20x28	1.633	28.23	22.97	4.098	3.374	NG	NG	OK	NG
	20x32	1.553	25.25	20.65	3.660	3.022	NG	NG	OK	NG
	20x36	1.489	22.97	18.93	3.324	2.762	OK	NG	OK	NG
16"	20x24	1.622	27.98	23.37	4.072	3.441	NG	NG	OK	NG
	20x28	1.533	24.84	20.65	3.603	3.034	NG	NG	OK	NG
	20x32	1.463	22.42	18.71	3.249	2.742	OK	NG	OK	NG
	20x36	1.406	20.53	17.25	2.971	2.522	OK	NG	OK	NG

Layout B - Added moment frames on (C,E,G,H,I, 9-6)(K,J,9-6)(2,3,4,B-F) without base isolation										
Wall size	Beam size	Period (sec)	total deflection in Y (inch)	total deflection in X (inch)	Max drift in Y (inch) between 5-6	Max drift in X (inch) between 3-4	Y -direction		X-direction	
							S5= 2%= 3.48"	S3= 1%= 1.74"	S5= 2%= 3.48"	S3= 1%= 1.74"
12"	20x24	1.687	29.42	26.09	4.271	3.844	NG	NG	NG	NG
	20x28	1.581	25.58	22.77	3.703	3.343	NG	NG	OK	NG
	20x32	1.501	22.78	20.45	3.290	2.992	OK	NG	OK	NG
	20x36	1.439	20.68	18.73	2.980	2.733	OK	NG	OK	NG
16"	20x24	1.582	25.82	23.20	3.744	3.416	NG	NG	OK	NG
	20x28	1.491	22.73	20.48	3.290	3.008	OK	NG	OK	NG
	20x32	1.421	20.42	18.54	2.950	2.716	OK	NG	OK	NG
	20x36	1.366	18.65	17.08	2.690	2.496	OK	NG	OK	NG
20"	20x36	1.307	17.05	15.77	2.460	2.305	OK	NG	OK	NG
24"	20x36	1.258	15.75	14.67	2.272	2.145	OK	NG	OK	NG
	20x42	1.204	14.22	13.40	2.049	1.955	OK	NG	OK	NG
	24x42	1.184	14.22	13.40	2.049	1.955	OK	NG	OK	NG
28"	20x42	1.165	13.291	12.602	1.916	1.839	OK	NG	OK	NG
32"	24x42	1.113	12.808	12.301	1.847	1.794	OK	NG	OK	NG
	24x48	1.077	11.847	11.473	1.708	1.670	OK	OK	OK	OK

Figure 39 - Different iterations ran in ETABS to determine the best lateral system to resist S-5 for a fixed base design at the California site.

After running all the iterations, it was found that the best system to use to meet S-5 was a dual system composed of 16 inches thick specially reinforced concrete shear walls and 20 inches wide by 28 inches deep intermediate concrete moment frames using layout B. The slight 4 inches increase of the shear walls helped reduce the inter-story drift in the X direction and the 4 inches depth increase of the moment frames using Layout B helped reduce the inter-story drift in the Y direction. Also, note that another efficient system of 12 inches shear walls and 20 inches wide by 32 inches deep moment frames using layout B could have been chosen but was opted out due to the nature of the building. In fact, the 14.5 feet height story has a plenum space of 5 feet for mechanical, lighting, plumbing, fire system and other to accommodate all the labs and their different functionalities.

The system highlighted in bleu in the figure above has a period of 1.491 sec for mode 1 thus new forces using the MRSA method needed to be calculated. Using the new design, it was found as seen on the table below that 0.85Cs controlled again thus there was no need to change the forces that are seen in figure 36.

Modal Information									
Mode	Period	UX%	UY%	Envelope	Sa	$S_a/(R/I)$	Cm,i	$(Cm,i*UX\%)^2$	$(Cm,i*UY\%)^2$
1	1.491	0.0848	60.7839	C	0.433266	0.066656	0.066656	3.19503E-09	0.001641573
2	1.062	49.564	3.6154	C	0.608286	0.093583	0.093583	0.002151404	1.14473E-05
3	0.701156	23.3993	8.9318	C	0.921336	0.141744	0.141744	0.001100056	0.000160283
4	0.3204	2.0679	10.9419	B	1.091	0.167795	0.167795	1.20397E-05	0.000337088
5	0.3013	10.5813	5.326	B	1.091	0.167795	0.167795	0.000315236	7.98656E-05
6	0.1561	6.6079	2.0383	B	1.091	0.167795	0.167795	0.000122937	1.16975E-05
								$C_{m,x}$=SQRT(∑$(C_{m,i}$*UX%$)^2$)=	0.060841401
								$C_{m,y}$=SQRT(∑$(C_{m,i}$*UY%$)^2$)=	0.047349274
								0.85Cs=	0.090686413

Figure 40 - Modal Information used to find Cm for the CA-S5, which was used to calculate MRSA seismic forces. To see what each envelope type refers to please refer to figure 34.

Now that all the forces were calculated, accidental moments were also considered for all seismic forces using the prescribed procedure for this given in section 12.8.4.2 of ASCE 7-05. This requires accidental torsional moments induced by the story force multiplied by an accidental eccentricity equal to 5% of the dimension of the building perpendicular to the forces applied. For ease of manipulation, seismic loads were entered into the model in four basic static load cases: seismic forces in the N-S direction (EX), seismic forces in the E-W direction (EY), accidental moments due to the N-S loads (EMX), and accidental moments due to the E-W loads (EMY). The amplification factor Ax was needed to be recalculated in order since the building by the L-shape nature has torsional issues. This calculation can be found in Appendix C. The results of the re-calculated forces are in the figures below.

Seismic Forces															
Level	Story weight, w_x	Story height (ft), h_x	$w_x.h_x^K$	C_{vx}	Story force (k)	Story Shear (k)	Overturning moment (k-ft)	Bx (ft)	5% Bx	Ax	M_{zy} (k-ft)	By (ft)	5% By	Ay	M_{zx} (k-ft)
8	1648	121.5	564183.7326	0.16183	326	325.7	39575	145	7.25	1.0	2361	191	9.55	1.1	3558
7	3133	101.5	861718.7654	0.24718	497	823.2	50496	145	7.25	1.0	3607	191	9.55	1.2	5490
6	2944	87	671221.7942	0.19254	388	1210.7	33714	145	7.25	1.0	2809	191	9.55	1.2	4311
5	2893	72.5	528510.0985	0.1516	305	1515.8	22121	145	7.25	1.0	2212	191	9.55	1.2	3406
4	2893	58	402933.6984	0.11558	233	1748.5	13492	145	7.25	1.0	1687	191	9.55	1.2	2602
3	2893	43.5	284012.5809	0.08147	164	1912.4	7133	145	7.25	1.0	1189	191	9.55	1.2	1839
2	2895	29	173634.6248	0.04981	100	2012.7	2907	145	7.25	1.0	727	191	9.55	1.2	1139
1	2895	14.5	74747.71396	0.02144	43	169437.4	626	146	7.3	1.0	315	192	9.6	1.3	529
$\sum$	22194	$\sum w_i h_i^K$=	3486215.295			**Base Shear =**	**2013 kip**			ΣM_{ZY} =	**14907 k-ft**			ΣM_{ZX} =	**22874 k-ft**
					Total Overturning Moment =		**169437 kip-ft**								

Figure 41 - New Seismic forces calculated due to the increase of the amplification factor of type 1-a irregularity in the Y-direction.

Once all the loads were calculated with the appropriate amplification factor the drift were obtained using the equation 12.8-15 in ASCE 7-05. The "I" factor was taken 1.0 and "Cd" was retrieved from table 12.2 -1 as "5" for E-2. This amplified the drifts in each direction by 5.0 but it was still under the code allowance of .02hsx. The results can be seen on the two tables below. To see in details these calculations please refer to appendix C.

		Earthquake story drift		Earthquake drift		Earthquake interstory drift	
Earthquake Loads Y-direction	Story level	δx	δy	Cd. dx / I	Cd.dy/I	Δx	Δy
	8	-0.25	5.01	-1.26	25.05	-0.16	2.90
	7	-0.22	4.43	-1.10	22.15	-0.21	3.15
	6	-0.18	3.80	-0.89	19.00	-0.23	3.25
	5	-0.13	3.15	-0.66	15.75	-0.23	3.40
	4	-0.09	2.47	-0.43	12.35	-0.20	3.05
	3	-0.05	1.86	-0.23	9.30	-0.12	3.00
	2	-0.02	1.26	-0.10	6.30	0.02	2.80
	1	-0.02	0.70	-0.12	3.50	-0.12	2.95

		Earthquake story drift		Earthquake drift		Earthquake interstory drift	
Earthquake Loads X-direction	Story level	δx	δy	Cd.dx/I	Cd.dy/I	Δx	Δy
	8	4.44	-0.13	22.20	-0.64	2.84	0.00
	7	3.87	-0.13	19.36	-0.64	3.08	-0.13
	6	3.25	-0.10	16.27	-0.52	3.21	-0.20
	5	2.61	-0.06	13.06	-0.31	3.26	-0.24
	4	1.96	-0.01	9.80	-0.07	3.15	-0.22
	3	1.33	0.03	6.66	0.15	2.80	-0.13
	2	0.77	0.05	3.85	0.27	2.16	0.09
	1	0.34	0.04	1.70	0.19	1.70	0.19

Figure 42 - Tables showing the inter-story drifts of the CA-S5 design chosen above with a Cd=5 and a story requirement of 2% or 3.74".

Foundation Impact

The structural redesign of the J.B Byrd Center was focused on the superstructure but impacts on the existing below grade foundation. The column layout of the redesign was based on the existing locations so the columns and the spread footings would not need to be altered. The increase in weight due to a smaller addition to the dead load associated with the cast-in-place one way slab redesign of the J.B. Byrd is little compared to the increase of the new base shear at the California site. Thus, despite an overall increase of weight of the structure, a great width to height ratio of the structure, and better soil condition for improved foundations an overturning moment analysis would need to be considered. This was not pursued by the author; however, a small hand calculation was performed to check the overturning moment of the building created by the new base shear. It was found that the resisting moment overpassed the overturning making the building stable. To view the detailed hand calculations, refer to Appendix D.

Nevertheless, the foundation design would have to be adjusted for the larger earthquake forces that the building will experience.

Isolated One Way Slab Cast-in-Place CA (S-3)

Moreover, from the iterations of the different systems using ETABS, it was concluded that an unviable system of 32 inches thick specially reinforced shear walls and 24 inches wide and 48 inches deep intermediate concrete moment frames need to be used. This will decrease greatly the architectural space and the plenum space not to mention the increase in cost of the system. Hence, a more viable design following the interest of the author was used by utilizing base isolators.

This design uses the CA S-5 design as a baseline structure, and then proceeds to simply add base isolators to the frame in an effort to reduce building drifts below 1%, which is the allowable drift for a Life Safety occupancy structure as given in ASCE 41-05 (see Figure 22). After the basics of the structure are designed an optimization of the system will follow.

<u>Seismic and Wind Loads</u>

Seismic loads were not used for base isolation. Dampers are designed for a target damping percentage rather than a specific force.

<u>Earthquake Ground Motion History Record and Scaling</u>

In order to perform a time history analysis with base isolators with nonlinear properties to conduct a preliminary design, earthquake ground motion history records had to be selected and scaled. According to the code the use of three records can be considered but the maximum envelope of the histories must be used. Due to the irregularities of this structure, motions had to be applied to multiple directions simultaneously. Therefore, a total of 6 acceleration records were selected. In order to simplify the selection of the records, recommended records from FEMA P695 were chosen. Due to the proximity of the structure to the Northridge Fault line and San Andreas Fault line (seen in the figure below), near and far field records were chosen.

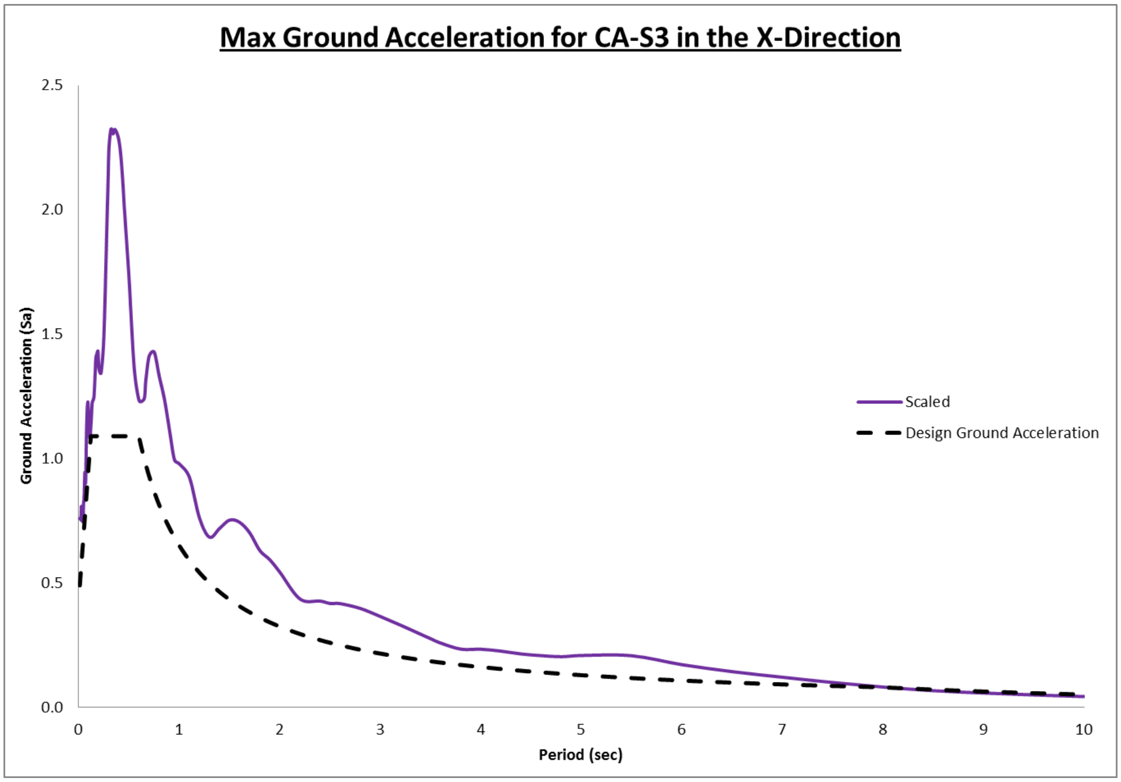
Figure 43 - **Picture taken from Google maps showing the proximity of San Andreas fault line as well as Northridge earthquake compared to the proposed site**

The ground acceleration histories for these records were then retrieved from the PEER NGA website, which is a database for ground motion records. The records for each direction were graphed, and these plots can be found in Appendix G. The three earthquakes chosen in each direction are listed in the table below.

Direction of Earthquake	Earthquake	Station	Magnitude
X-Direction	Imperial Valley	El centro 7	6.5
	Northridge-01	Sylmar - Olive View	6.7
	Chi Chi, Taiwan	TCU065	7.6
Y-Direction	Imperial Valley	Chihuahua	6.5
	Northridge-01	Northridge - Saticoy	6.7
	Chi Chi, Taiwan	TCU067	7.6

Figure 44 - **Table showing the chosen earthquakes for Time History analysis with their corresponding station and magnitude.**

Furthermore, response spectra for each ground motion were taken from PEER NGA as well as the scale factors according to the proposed site. The maximum envelope of the three ground motion history was used and compared to the code-required design response spectrum in each direction. Then, the maximum was scaled to the proposed site location according to the scale factors given by PEER NGA for each ground motion. The maximum Sa (g) in both directions are shown below.



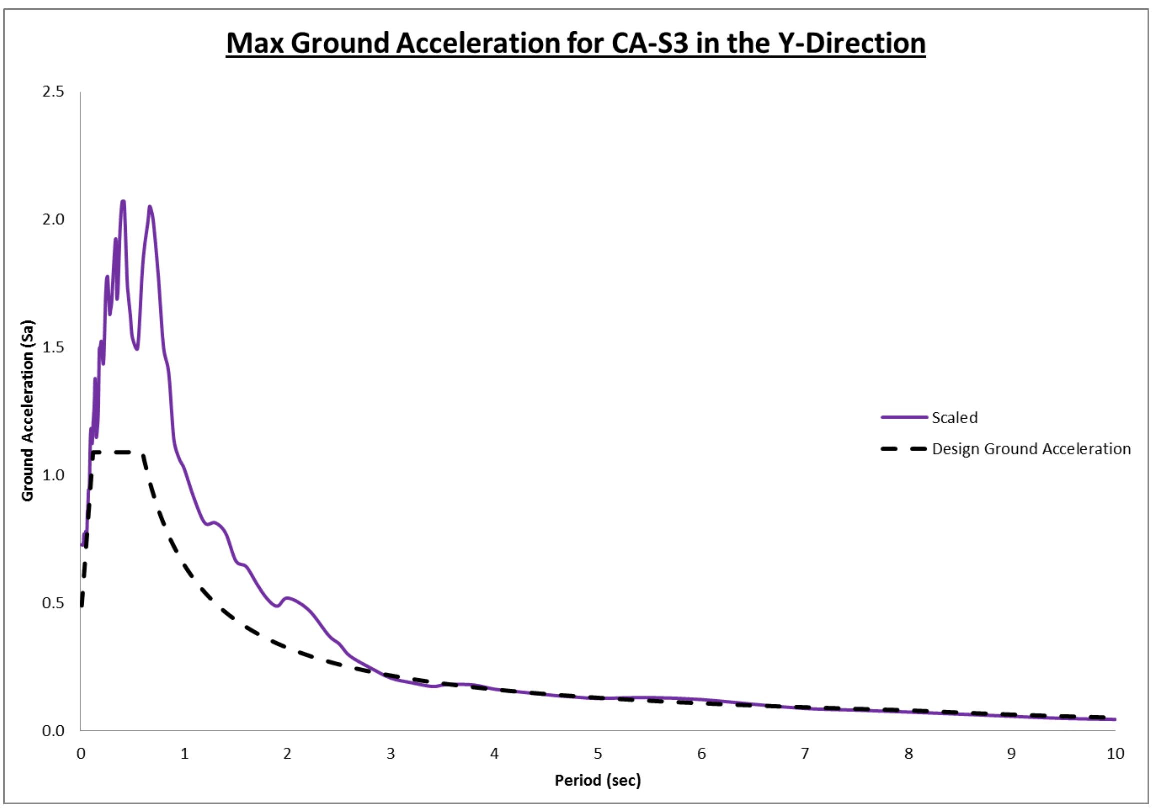


Figure 45 – **Plots of the maximum scaled envelope of the three ground motion history in both the X and Y direction. The solid purple line is the maximum scaled response spectrum and the dotted black line is the code response spectrum.**

Base Isolation (Time history analysis with nonlinear isolator properties)

Stiff buildings under seismic loading experience high floor acceleration but flexible buildings experience large inter-story drifts. Thus, to reduce such effects, base isolation concept was introduced. The effect is that most of the ground movement will not be transmitted to the building. Therefore, the building will experience smaller acceleration and inter-story drifts. Also, the building will experience reduced seismic forces in return reducing damaged to structural and non-structural components that will enhance life safety.

For base isolation to be effective a damping need to be present in the horizontal direction to reduce amplitude of motion isolator and vertical stiffness provided by steel plates within the rubber bearing as seen in figure 23. Also, a flexible bearing is needed to lengthen the period of vibration to reduce the forces. An example figure showing an actual stiff structure with a low period T1 and a base isolated building with an extended period T2 are depicted in the figure below:

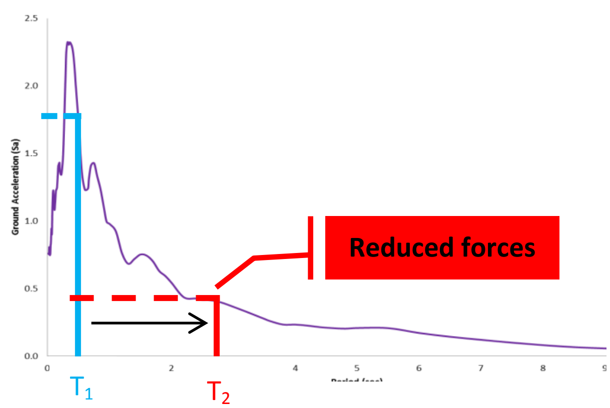


Figure 46 – Plot of Sa (g) vs Time (sec) to show how a base isolated structure would reduce the forces by extending the period of the building.

In order to achieve the period shift shown above, devices such as Lead-rubber bearing isolators need to be sized and used. Since the period is now larger an increase of the total displacement of the structure will occur. In fact, the added displacement needs to be calculated according to ASCE 41. In case the displacement is high, damping of the isolators can be increased to reduce the displacement and forces the building experiences. The damping increase is done through energy dissipation and is unique to each type of isolator. For the lead rubber base isolator used in CA-S3, the damping is achieved through the yielding and plastic deformation of the mild steel and lead.

Preliminary sizing of the base isolator was achieved through hand calculation following FEMA and ASCE 41 provisions. It was found that the design displacement of the structure is 34.9 inches, the maximum displacement is 30.5 inches and the total maximum displacement is 43 inches. The detailed calculation of the minimum design displacement and maximum displacement can be found in Appendix F.

After preliminary sizing, a more refined sizing of the isolators was done using ETABS. Base isolators were modeled using “example O” from SAP 2000 and manufacturer’s guidelines as a reference. They were modeled as a link support element with no restraints in the horizontal direction. The link properties were taken from Teratec, the manufacturer chosen for the isolators. All the sizes with the different properties can be seen in Appendix F. The layout of the 66 isolators used in the J.B Byrd building can be seen in the figure below.

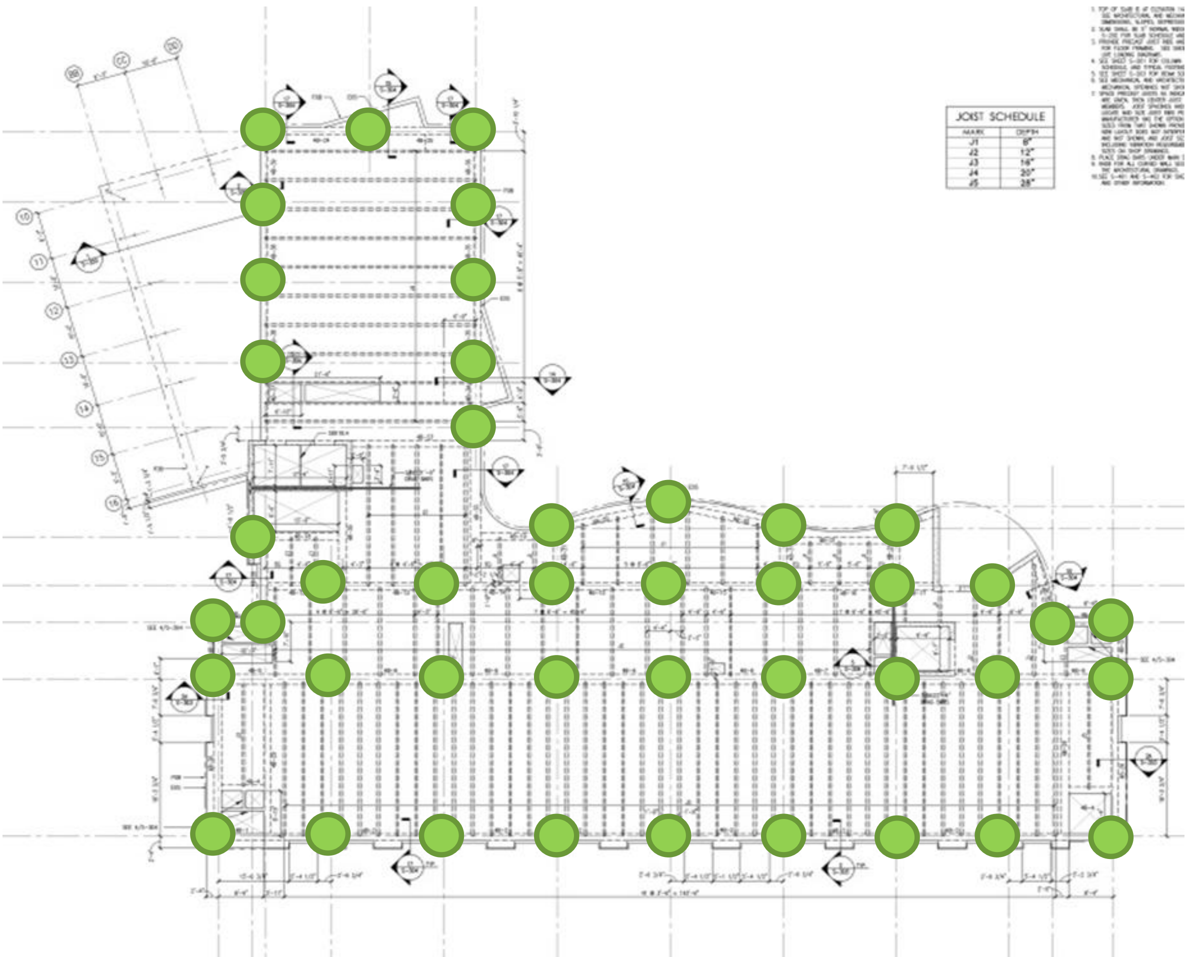
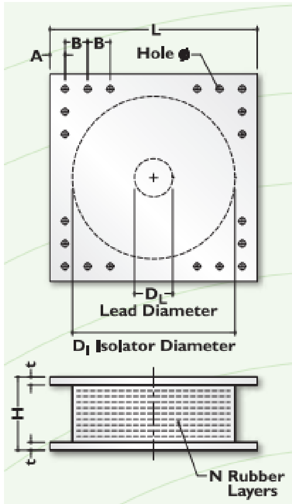


Figure 47 - **Plan showing the base isolator layout at the base of the first floor above the basement.**

After several iterations to resist the all the earthquakes, and optimization of the structural system, the appropriate size of the isolator was determined. With an axial capacity of 1500 kips (greater than the 1400 kips of the column at the base calculated), it has a diameter of 37.5 inches and a maximum displacement, D_{max} of 24 inches. A yielded stiffness K_d of 4 kips per inch and a strength of 110 kips was chosen. The isolators have a damping value of 15% in order to reduce the forces and displacement of the structure.



Isolator Dimensions	
DI (in)	37.5
H (in)	23
N	40
DL (in)	11
L (in)	39.5
t (in)	1.5
Hole Qty	12
Hole D (in)	1 5/16
A (in)	2.5
B (in)	3.75

Figure 48 - **Detail and dimensions for the isolator chosen for the CA-S3 isolated structure.**

The 37.5 inch isolator will be installed under the optimized lateral system. In fact, using the isolator, a more efficient and economical system was used. The dual system now consists of a 12 inch shear walls as found in the original design with 20 inches wide by 28 inches deep beams for intermediate concrete moment frame utilizing layout A. This a slight increase as compared to the original structure in Florida where the forces on the structure were 2 to 3 times less. Hence, base isolators are really effective in seismic regions.

Moreover, to determine the lateral displacements and inter-story drifts the following link properties were used in ETABS:

Isolator Properties	
Linear Properties	
Effective Stiffness	4
Effective Damping	0.15
Nonlinear Properties	
Stiffness	40
Yield Strength	110
Post Yield Stiffness Ratio	0.2

Figure 49 - **Link element properties used in ETABS to model the base isolator.**

After imputing all the earthquakes and appropriate link properties an analysis to compute all the inter-story drifts and maximum displacement was done. The period of the structure was 4.041 seconds, thus 2 to 3 times the period of the fixed structure CA-S5. This is a reasonable result as noted in FEMA and other base isolation references cited in References.

Additionally, it was found that the controlling earthquake was Northridge Olive View station in the X-direction even though irregularities were found in the Y-direction. This is possible as the stiffness in the Y-direction was increased compared to the X-direction. In fact, most of the moment frames added and increased in member sizes were in the Y-direction. The results of the drifts and displacements can be seen in the tables below.

Table 1								
Direction of Earthquake	Earthquake	Station	Scale Factor	Magnitude	Peak time in X (sec)	Peak time in Y (sec)	Max Displacement (inch)	
							X	Y
X-Direction	Imperial Valley	El centro 7	525	6.5	5.48	11.27	16.38	1.88
	Northridge-01	Sylmar - Olive View	441	6.7	4.82	14.60	21.22	1.76
	Chi Chi, Taiwan	TCU065	312	7.6	5.42	12.37	9.20	1.50
Y-Direction	Imperial Valley	Chihuahua	1018	6.5	32.41	14.91	1.51	9.23
	Northridge-01	Northridge - Saticoy	579	6.7	7.31	4.07	1.22	16.56
	Chi Chi, Taiwan	TCU067	451	7.6	44.27	30.94	1.57	18.74

Table 2										
Direction of Earthquake	Earthquake	Station	Max interstory drift		Max interstory drift location	S5= 2%= 3.48"		S3= 1%= 1.74"	
			X	Y		X	Y	X	Y
X-Direction	Imperial Valley	El centro 7	1.458	0.334	Story 1-Story 2	OK	OK	OK	OK
	Northridge-01	Sylmar - Olive View	1.729	0.789	Story 1-Story 2	OK	OK	OK	OK
	Chi Chi, Taiwan	TCU065	1.032	0.277	Story 1-Story 2	OK	OK	OK	OK
Y-Direction	Imperial Valley	Chihuahua	0.164	0.734	Story 1-Story 2	OK	OK	OK	OK
	Northridge-01	Northridge - Saticoy	0.161	1.321	Story 1-Story 2	OK	OK	OK	OK
	Chi Chi, Taiwan	TCU067	0.177	1.493	Story 1-Story 2	OK	OK	OK	OK

Figure 50 - **Tables summarizing the results of the isolated structure CA-S3. Table 1: summarizes the maximum displacement of the structure; Table 2: summarizes the maximum inter-story drifts of the structure. Note: The controlling earthquake is highlighted in red.**

System Finalization

Once the drifts were found to be adequate to the structure, calculations were performed to size a typical interior column under high loads. The column I-8 was chosen for the sample calculation and its location can be found in the typical plans in Appendix A. The design process was using spColumn under the loads calculated by hand (axial) and the loads provided by ETABS (lateral). In fact, the load combination used to the design the column using the LRFD method was 1.2D+1.0L +1.0E +0.2S.

It was found that the column needed to be having a higher strength on the bottom compared to the top. Also, a bigger column size with heavier reinforcement was needed to be able to resist the moment induced the earthquakes. A detailed design was done going through all the floors to be able to size and reinforce the column. The results of spColumn can be seen in Appendix G. Note that since the structure used has specially reinforced shear walls and exist in a high seismic region further stirrup detailing need to be done and added to the structure. However, due to time restraints the author did not investigate the detailing of column, beam and shear walls.

Interior Column I-8 designed to resist the Olive View- Northridge Earthquake						
Story	Designed Section	f'c (ksi)	designed Reinforcement	Spacing req.		
				actual	min	actual>min
8	22x22	5	12 #9	4.83	1.692	OK
7	22x22	5	12 #9	4.83	1.692	OK
6	22x22	5	12 #9	4.83	1.692	OK
5	22x22	5	16 #9	3.34	1.692	OK
4	22x22	5	12 #11	4.45	2.11	OK
3	22x22	5	12 #11	4.45	2.11	OK
2	26x26	5	20 #11	2.91	2.11	OK
1	26x26	5	16 #11	3.99	2.11	OK
Base	30x30	7	28 #11	2.25	2.11	OK

Figure 51 – **Table summarizing the design of column I-9 at each story level.**

System Comparison/Summary

This section seeks to provide a concise summary of the results of the designs which are important to comparing the overall efficiency of the structures. Final efficiency determinations are made in the "Construction Management Breadth: Cost and Schedule Analysis" section.

Lateral System Summary

Two different lateral systems were designed to resist the same seismic loading. However, one was designed to meet a structural performance level and damage of S-5 "Collapse Prevention" and the other S-3 "Life safety". After several iterations, it was found that the structure to meet S-3 would require an unviable lateral system due to the horizontal irregularity of the building. Thus, an isolated system was used following the author's interest. The table below shows the summary of both systems.

Structure:	Fixed CA-S-5	Isolated CA-S3
Moment frame		
Layout	B	A
Size	20" x 28"	20" x 28"
Shear Wall		
Layout	same as original	
Thickness	16"	12"

Figure 52 - **Table summarizing the two different lateral systems used according to their performance requirements.**

System Drifts Summary

The drifts for all of the designs are for the X-Direction and the Y-Direction, respectively. These are compared to the allowable drifts for each design type. As can be seen in figure 53, all drifts are below the allowable, and by far the most efficient structure in terms of deflection is the CA S-3 with base isolation structure. This is the result that was expected as we were designing for smaller inter-story drifts.

Isolated CA-S3			
Direction of Earthquake	Earthquake	Max interstory drift (inch)	
		X	Y
X-Direction	Imperial Valley	1.458	0.334
	Northridge-01	1.729	0.789
	Chi Chi, Taiwan	1.032	0.277
Y-Direction	Imperial Valley	0.164	0.734
	Northridge-01	0.161	1.321
	Chi Chi, Taiwan	0.177	1.493

Fixed CA-S5 max inter-story drift				
	Story level	Δy		Δx
Earthquake Loads Y-direction	8	2.90	Earthquake Loads X-direction	2.84
	7	3.15		3.08
	6	3.25		3.21
	5	3.40		3.26
	4	3.05		3.15
	3	3.00		2.80
	2	2.80		2.16
	1	2.95		1.70

Figure 53 - **Tables showing the max inter-story drifts for each lateral system according to their designed performance. Performance levels: S-3=1%=1.74" and S-5=2%=3.48".**

Construction Management Breadth: Cost and Schedule Analysis

The purpose of this breadth was to investigate how the changes to the superstructure will alter the building construction schedule and cost. Thus, a simplified cost estimate was created to compare the materials used in the existing structural system, the One Way Slab fixed base structure and the One Way Slab isolated structure. Material, labor, and equipment costs were taken from the RS Means Cost Data 2011 and were used to create a cost estimate summaries for both systems.

Cost

Detailed structural takeoffs were performed for the design portion of the building for both designs. Concrete takeoffs and steel takeoffs were taken from the RAM model and hand calculations. More detail takeoffs of the structures can be found in Appendix H. Using the take-offs, RS Means 2012 data could also be used to produce the costs of each structure. First, it was attempted to replicate the original costs of the building. The only information which HDR was able to provide directly in relation to the super structure is summarized in Table 54.

Original design of the building	
Description	Cost
Foundation	$682,261
Concrete	$2,248,708
Steel	$642,094
Masonary	$584,694
Total superstucture	$2,890,802
Total Cost of building	$21,620,193

Figure 54 - **Table summarizing the cost of the original structural system provided by HDR.**

Fixed Base CA-S5

After the completion of the gravity and lateral system designs, the expenses of the redesigned structures were tabulated, using RSMeans Building Construction Cost Data. This guaranteed a level comparison between the concrete designs. The simplified breakdown of costs for the concrete system is shown in the table below. This breakdown includes the cost of the 5,000 psi concrete that was used in the columns, slabs, and shear walls. The total tonnage of reinforcing for the slab was determined from RAM Concepts that was used to model the one way cast-in-place. To check the accuracy of the weight of reinforcing steel taken from RAM, a simplified hand calculation was done. Formwork was assumed to be used several times to save expenses as it would be done in the field, and it was expected that placing the concrete would be done by pump. A sample calculation of a ground and upper floor can be found in Appendix H.

Fixed CA-S5 Cost Summary	
Description	Cost
Foundation	$682,261
Concrete	$2,656,186
Steel	$706,303
Masonary	$584,694
Total super structure	$3,362,489
Total cost of building	$22,091,880

Figure 55 - **Table summarizing the total cost of CA-S5.**

As the table shows, the estimated cost for the concrete structure is $22,091,880 which represents a 14% increase in comparison of the superstructure and a 2.13 % increase in comparison of the total building cost. Another concern to take into account is the increase in steel tonnage to account for the increase in stirrups for the specially reinforced, columns and beams. However, the author did not analysis that addition. Note that Masonry and foundation as well as similar materials in the systems were omitted in the cost estimate.

Isolated base CA-S3

The isolated system considering only the superstructure was cheaper since the design loads were reduced due to the isolators. However, base isolators are expensive according to industry professionals. In fact, base isolator costs between $8,000 and $22,000 each. The one chosen to be used for this building located in California was $14,245. Also, assuming that one crane can install 6 isolators per day, it was found that a crane would cost $45,650 for 11 days. Thus, the total cost of the 66 isolators is $940,170. This is considered an expensive addition to the superstructure however with a cheaper structure on top of them the systems are not far off.

Isolated CA-S3 Cost Summary	
Description	Cost
Foundation	$682,261
Concrete	$2,302,165
Steel	$666,303
Isolators	$985,820
Masonary	$584,694
Total super structure	$3,954,288
Total cost of building	$22,683,679

Figure 56 – **Table summarizing the total cost of CA-S3**

Conclusion

Upon evaluating the cost of the existing and redesigns structural systems, it is clearly evident that the isolated is the most expensive system. Also, it was expected that the isolated system was more expensive than the fixed base one due to the fact that it had to meet a higher structural performance. Given the fact that the superstructure is cheaper for the isolated system, the isolators alone increased the price by 30%. This totaled to an increase of $591,800 compared to the fixed base system. Hence, it depends on the owner's choice of structural performance. The overall results are shown below.

	Original	Fixed CA-S5	Isolated CA-S3
Superstructure	$2,890,802	$2,656,186	$2,302,165
Isolators	$0	$0	$985,820
Total Cost	$21,620,193	$22,091,880	$22,683,679
Difference to original	-	+ $471,687	+ $1,063,486

Figure 57 - Table showing the final costs of each system used.

Schedule

Construction Schedule of Existing Structural System

The existing structural system of the J.B.Byrd Alzheimer's Center & Research Institute was scheduled to begin on May 24, 2005. The entire structure was estimated to take approximately 11 months, but being completed on August 18, 2006. A schedule for the construction of the structural system coordinates the erection of concrete shear walls, floor slabs, precast joists and beam soffits, and masonry veneer. A detailed construction schedule of the existing construction schedule is provided in Appendix H.

Construction Schedule of Redesigned CA-S5 Structural System

The redesigned fixed base structural system will have the same start date of May 24, 2005. The one way cast-in-place system was estimated to take approximately 13 months, being completed on October, 2006. By modifying the structural system to a one way cast-in-place, a small amount of the construction time was added. Ignoring the construction of the façade, it took 324 days to erect the existing system, as opposed to 380 days to erect the redesigned cast-in-place system. A mock construction schedule for the redesigned structural system was created. Please refer to Appendix H for a detailed construction schedule of the structural system.

Construction Schedule of Redesigned CA-S3 Structural System

The redesigned isolated system will have the same start date of May 24, 2005. Since this is the same system as the previous only the base isolators were considered for the comparison. It was found from industry professionals that it would take about 15 weeks or 105 days for the ordering and shipping of the isolators. Also, it was found that it would take 11 days to install the 66 isolators assuming that 6 can be installed in one day.

Furthermore, it was assumed that the request of the isolators was done during the design stage thus the 105 days of manufacturing and shipping would not delay the schedule. Only the installation phase after the basement walls were casted would postpone the work. Please refer to Appendix H for a detailed construction schedule of the isolated structural system.

Schedule Summary		
System	# days	Extra to original
Original design	324	-
One way cast-in-place	380	56
Isolated one way cast-in-place	391	67

Figure 58 - **Summary table showing the number of days scheduled to complete the structures.**

Sustainability Breadth: Sustainability Viability Study

This viability study attempts to address the differences between the various ways of evaluating sustainable technology and determine the viability of incorporating a photovoltaic system in the curtain wall of the J.B Byrd Center. This technology was not included in the original design of the building, however now that the building is in San Diego, California, it may be deemed viable.

The system was evaluated based upon two different criteria. The first was a life cycle assessment, which incorporates the cost to produce, install, and maintain. The next was a payback period, which attempts to determine how long (typically in years) it will take for the system cost to be counteracted by how much it saves the owner in comparison to the baseline since the product was not incorporated. The variety of evaluations sought to provide a full profile of the true sustainability and effectiveness of the BIPV system.

Building Integrated Photovoltaic Curtain Wall

There are many different kinds of solar cell modules which have a variety of efficiencies. Most traditional photovoltaic (PV) systems are mounted on racks to angle them to catch more sun, and therefore produce more power. These panels which can be directly adhered to building surfaces are known as building-integrated photovoltaics (BIPV). For the Byrd Center, it was impractical to attempt to design a PV system which would power the entire building due to the electricity consumption requirements of the building. Therefore, it was decided that a solar shading study should be conducted to determine if the location or locations on the curtain wall would be effective for the placement of the panels. Then, a layout would be created to accommodate the available space, and the system could be designed for the layout.

The system chosen for the J.B Byrd building were Abound Solar thin film module 72W panels provided by BISEM Inc. The panel size is 33” by 33” which fits exactly the existing unitized curtain wall on the façade. These were selected for the quality of the panel (the solar cells are monocrystalline silicon, with an efficiency of slightly less than 14%) and the ease of placement (They can be easily switched between existing panels without changing the mullions).

In order to determine where to place the panels, a solar shading study was conducted. This consisted of constructing a model of the building and the surrounding buildings at the USD campus site in Google

Sketchup. The same program was then used to create images of the building at sunrise, sunset, and 12:00 PM (the two extreme cases of shadows and the shadows at peak production time) for an equinox and the winter and summer solstices .A sample of this can be seen in Figure 59.

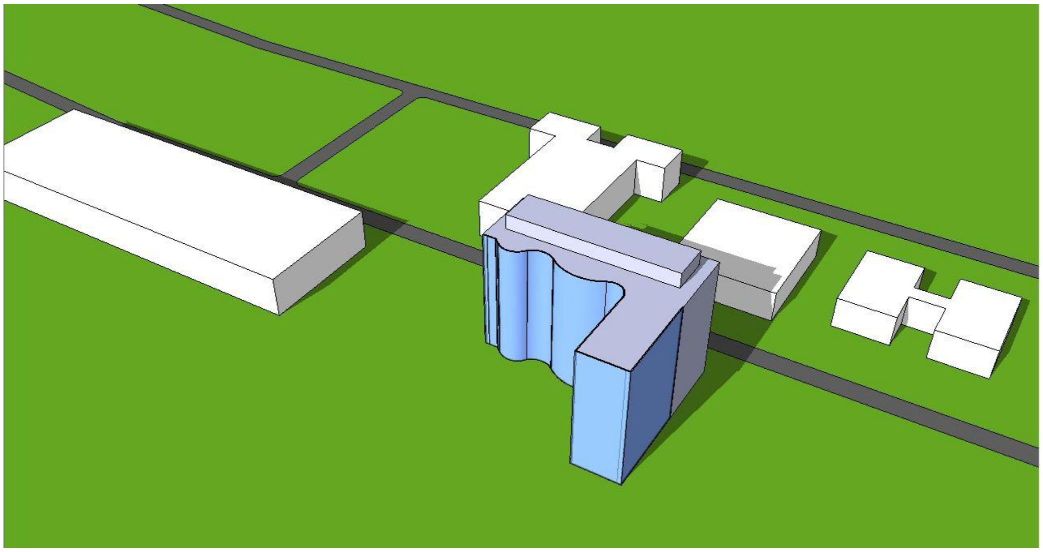


Figure 59 - Solar shading study from December 21 at 12:00pm. Note that the curtain wall is highlighted in bleu.

After modeling the building in Sketchup, it was then input in ShadowAnalysis where a more detailed shading study was performed. The program in fact reports all the shaded surfaces and the period of time it is shaded during the day (7:00 am to 5:00pm). A sample of this can be seen below.

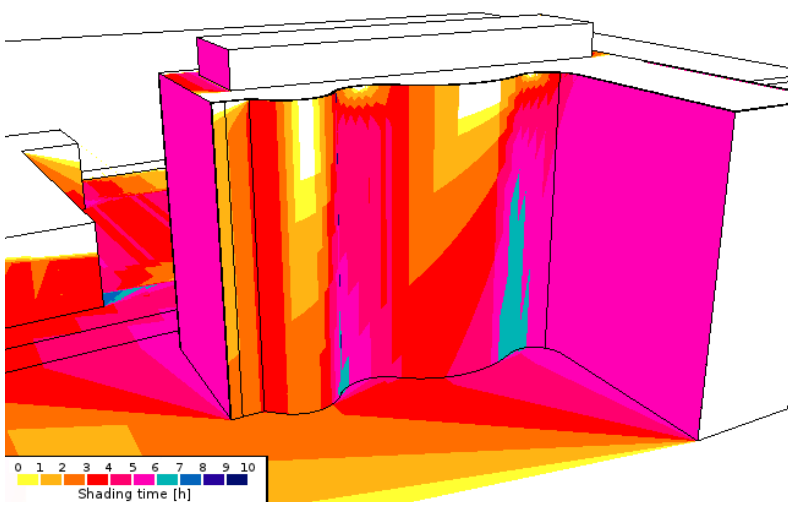


Figure 60 – Sample screenshot from ShadowAnalysis solar shading from June 6th.

Due to the nature design of the curtain wall façade only 46% of the area was utilized for BIPV. In fact, the solar panels were placed on the spandrel glass and the 30% silkscreen glass. The panels were laid out in that space in order not to reduce the vision glass. However, due to the nature color of solar panels the aesthetics of the façade will be affected. The olive color of the spandrel and silkscreen will have to change to dark green as the manufacturer can accommodate for a bit of color instead of the usual dark purple solar panels. An image of what a set of BIPV panels might look like once fully installed compared to the original is included below.

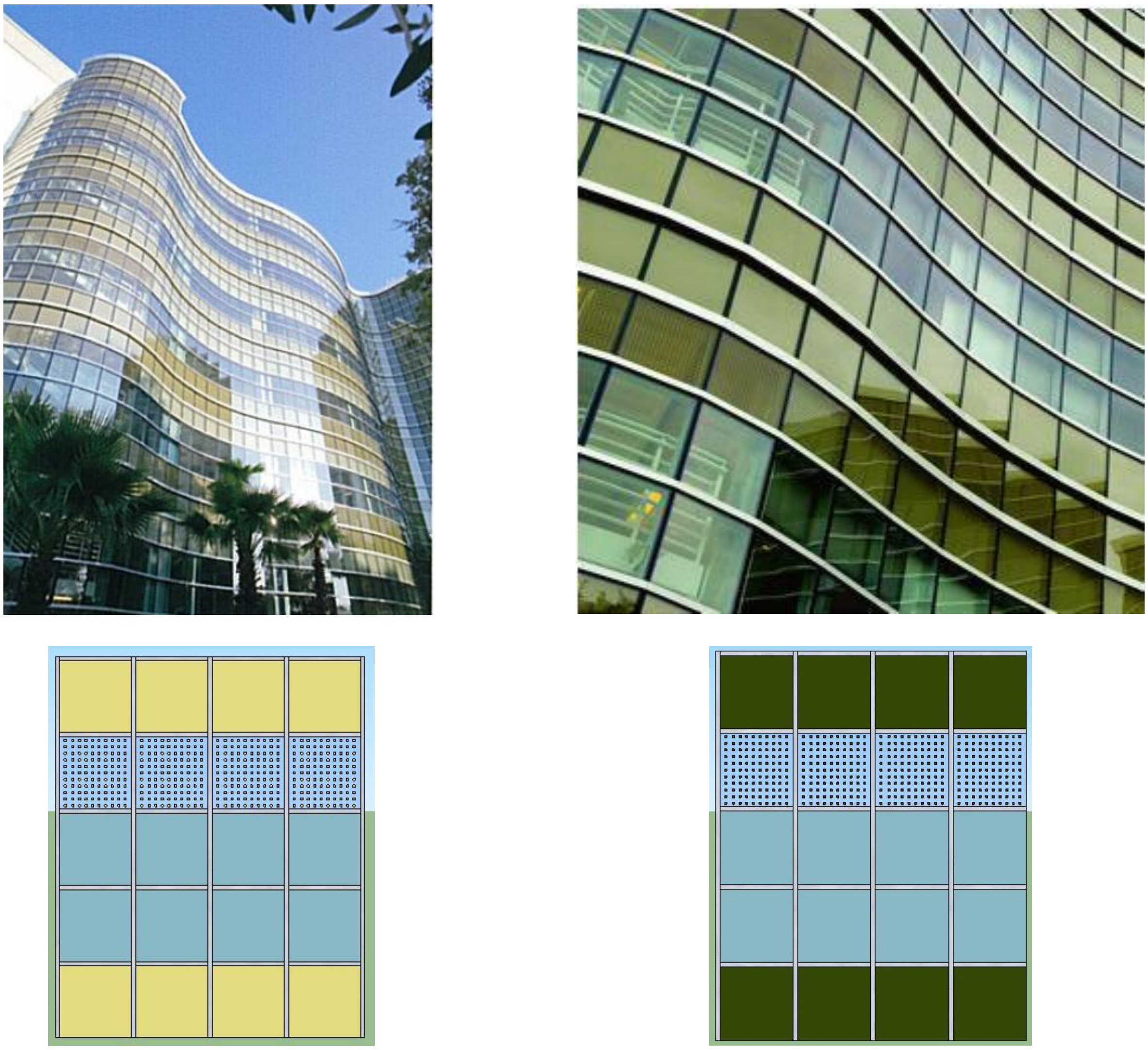


Figure 61 – The image on the left is a photo showing the actual façade of the J.B Center. The screenshot below it represents the actual façade modeled in Sketchup. The image on the right represents on how the façade would look like in real life. The screenshot below that represents the new curtain wall with BIPV panels.

Life Cycle Assessment

As previously mentioned, the life cycle assessment of a product incorporates the cost to produce, transport, install, maintain, and replace (if necessary).For the purposes of this report, a life span of 10 years was chosen. The installed cost of the system (which incorporates production, transportation, retrofit and installation) was given by BISEM Inc to be $190/sf. However, the federal government gives a tax incentive for 30% of the costs of a photovoltaic system. Therefore, this was deducted from the costs. In terms of maintenance, a PV system has to be inspected yearly for defects. The panels chosen are warrantied to produce peak power for 25 years, and therefore do not cost the owner to replace unless some form of damage occurs to the panels (as this cannot be accurately foreseen or predicted, the possibility of damage to the panels was neglected). However, the inverters are only warrantied for 10 years, and therefore the cost to replace the inverters at 10 years was incorporated. Finally, the salvage value of the system of the previous curtain wall system was $78/sf. thus $112/sf. net for retrofit.

	Qty	Price Per panel	Total
	SF	Price/SF	Total
New Curtain Wall			
South Wall	10028	$ 78.00	$ 782,184.00
North Wall	2284	$ 78.00	$ 178,152.00
West Wall	4885	$ 78.00	$ 381,030.00
East Wall	2541	$ 78.00	$ 198,198.00
Total	**19738**		**$ 1,539,564.00**
	SF	Price/SF	Total
Add for BiPV Retrofit			
South Wall	7212	$ 112.00	$ 807,699.20
West Wall	4038	$ 112.00	$ 452,298.00
East Wall	1956	$ 112.00	$ 219,072.00
Total	**13206**		**$ 1,479,069.20**
Total Base:	**$**	**1,539,564.00**	
Add BiPV:	**$**	**1,479,069.20**	Added value for the PV panels
Total Base:	**$**	**3,018,633.20**	
This assumes thin film modules to be installed at all spandrel areas. The panels will be Abound Solar 72W			

Figure 62 - **Total life cycle cost including the old curtain wall system. Thus, for a BIPV retrofit a fee of $1,479,069 is applied.**

<u>Payback Period</u>

The payback period of the PV system was determined using the power rates taken from Form EIA-826, Monthly Electric Sales and Revenue Report with State Distributions Report. An average value of 12.97cents/KW was taken for high season (June-September) and a low season (October-May). Within each season, it has high peak hours (Monday-Friday, 1PM-5PM), low peak hours (Monday-Friday, 10AM-1PM and 5PM-8PM), and finally base rate hours (Monday-Friday 8PM-10AM, Saturday all day, and Sunday all day). Since the rate is per kWh, the number of kWh of AC power had to be determined. A crude estimate of this was determined using recommendations from BISEM Inc with 80% system performance. This was given per year, which enabled the calculation of AC power produced by the entire year. It was found that a savings of electricity of $18,985 per year were made. This calculation can be found in Appendix I.

Next, the MACRS depreciation value or tax deduction needed to be calculated. As mentioned before a 30% investment credit for the business energy goes to 12/31/16. Second, a depreciation value of the system over 39 years was done to be at $25,641 per year. However, there is a state depreciation of 10% of the MACRS depreciation value that equals $21,831.

The payback period was then determined through detailed calculation that can be found in Appendix I. The resulting payback period was 3 years, and the rest is a Return-On-Investment (ROI) the calculation for which can be seen in figure 63.

95% Payback in 36 Months

Assumption: South, East & West Elevation of the curtain wall is 13,206 square feet. The federal tax credit for the BIPV curtain wall is 30% in the first year. There is also a state and federal accelerated depriciation, MACRS. This allows the BIPV curtain wall to be deducted over 5 years, rather than 30 years. So, by the end of the second year, you will have paid for the premium for the BIPV thinfilm addition. The next three years of accelerated depriciation become an ROI.

			Cost		
Standard Curtain Wall:	13,206	$ 78	$1,030,068		
BiPV Curtain Wall Premium:	13,206	$ 112	$1,479,072		
Total Taxable BiPV:			$2,509,140		
Federal Tax Credit 30% of total BiPV in First Year:			$ 740,857		
MACRS Depreciation Year One:			$ 189,758		
Local Utility Rebate:			$ 94,925	=	$18,925 per year for 5 years
MACRS Depreciation Federal/State Year Two:			$ 189,758		
MACRS Depreciation Federal/State Year Three:			$ 189,758	95%	Payback 36 Months
MACRS Depreciation Federal/State Year Four:			$ 189,758	13%	ROI
MACRS Depreciation Federal/State Year Five:			$ 189,758	13%	ROI

Figure 63 - **Table summarizing the calculations done is Appendix I to determine the payback period.**

Summary

Using the assumptions that the South, East & West Elevation of the curtain wall is 13,206 square feet, the federal tax credit for the BIPV curtain wall is 30% in the first year and there is also a state and federal accelerated depreciation, MACRS. This allows the BIPV curtain wall to be deducted over 5 years, rather than 30 years. So, by the end of the second year, the owner will have paid for the premium for the BIPV thin film addition. The next two years of accelerated depreciation become an ROI.

Total BIPV curtain wall	Sell exisiting Panels	Addition for BiPV	Deduct for Tax Credit and MACRS	Local Utility Savings	Savings after 5 years
$2,469,522	$1,030,068	$1,479,072	$1,689,647	$94,925	$305,500

Figure 64 - **Table showing the total cost and savings of the BIPV retrofit done on the J.B Byrd Center with BISEM Inc.**

Conclusion

Two designs were undertaken in concrete to depict real life construction that would have been viable at the proposed location. The redesigns also grasp what costs are associated with moving from a low seismic region to a high seismic region, how much cost is associated with designing for higher performance criteria, and which of two alternative designs, one traditional and one high-tech is the more efficient for achieving a higher performance.

It was found that the penalty to move that structure to a high-seismic region was an increase in weight and cost. In order to increase the performance of the structure in the traditional method, the structure increases in weight by 4% and in cost by 2% over the basic structure in a high seismic region. The fixed base structure utilized 16 inch special reinforced shear walls with 20" by 28" intermediate moment frames to achieve "S-5 Collapse Prevention". However, in order to increase the performance of the structure using the high-tech method, the structure increased in weight in 2% and in cost by 4.5% over the basic structure in a high-seismic region. The isolated base structure utilized 12 inch special reinforced shear walls with 20" by 28" intermediate moment frames to achieve "S-3 Life Safety". It was therefore determined that it was the owner's choice according to the performance level needed.

These designs were created using a mix of hand calculations, spreadsheets, RAM Concepts, ETABS, and SAP 2000. This design process integrated master's level coursework in the modeling of the structures (AE 597A), and the earthquake design (AE 538).

The costs and schedule durations of the designs were found to constitute a construction management breadth. Using the original schedule and original cost data provided by HDR.Inc, quantity take-offs for the superstructure, and data from RS Means schedules and their associated costs were developed for each design. This was used to help compare the designs.

Finally, a sustainability breadth was undertaken to determine the viability of a retrofit of including a curtain wall photovoltaic (BIPV) system on the building once it was relocated to California. The system was designed with industry support using BISEM Inc, and then evaluated based on a life-cycle assessment and payback period. The assumptions that the federal tax credit for the BIPV curtain wall is 30% in the first year and that state and federal were accelerated depreciation, MACRS, allowed the BIPV curtain wall to be deducted over 5 years, rather than 30 years. Under those norms, it was found that the payback period of the BIPV curtain wall was 36 months and created 2 years return on investment of 13%.

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Appendices

Appendix A: Typical Plans

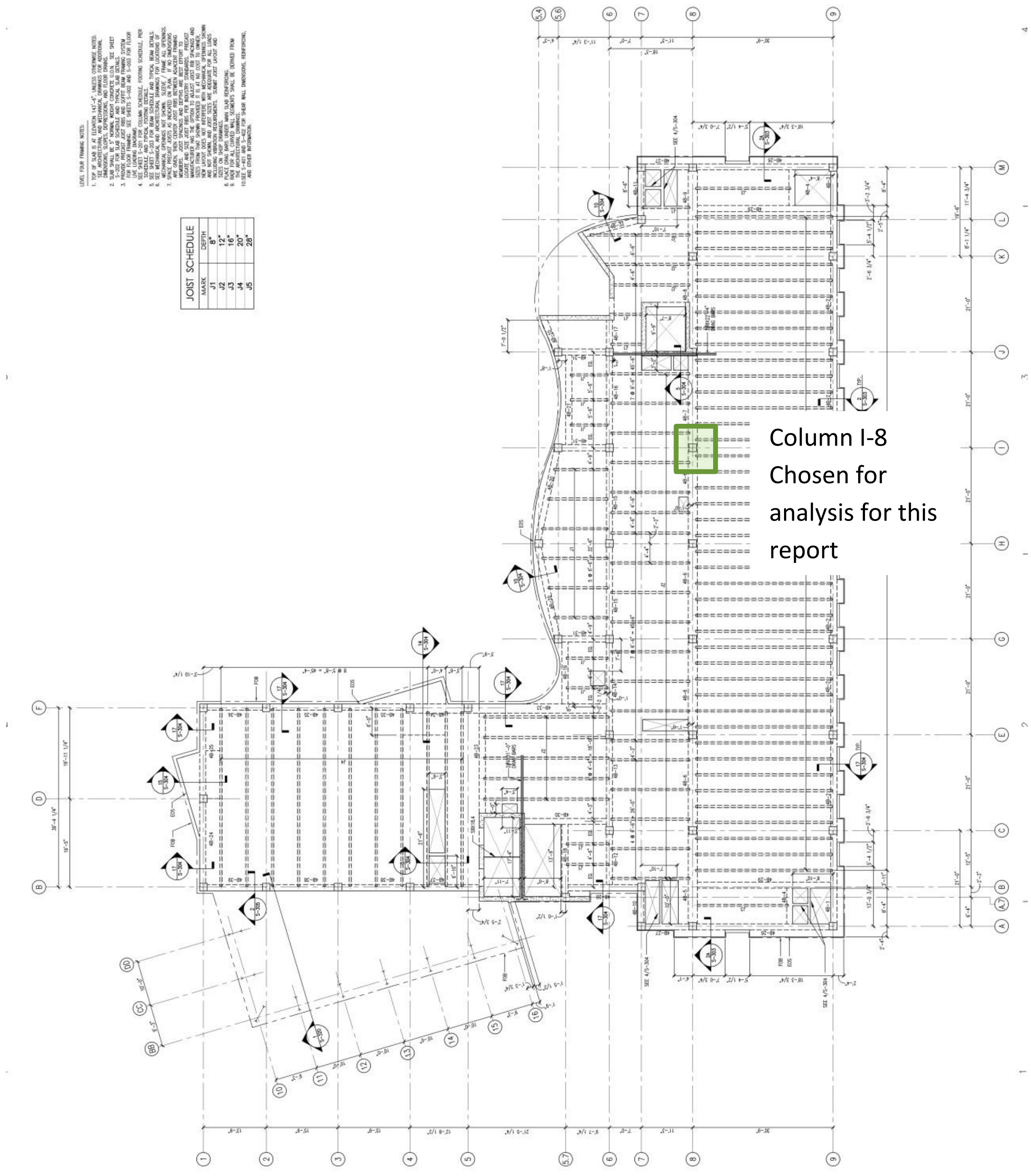


Figure 65 - **Typical floor plan taken from S-104 N-S**

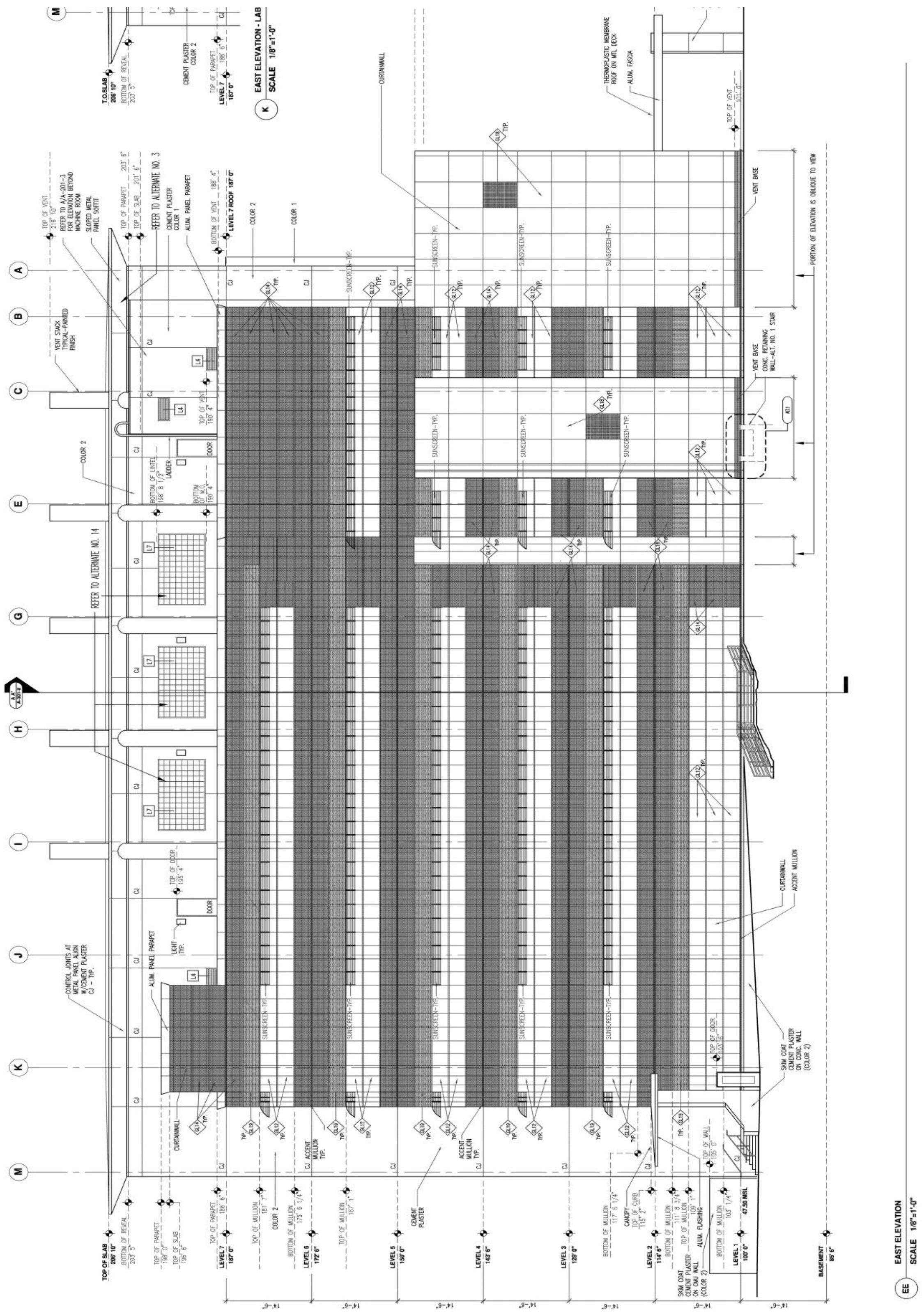


Figure 66 - **Elevation of the building showing the different floor heights from A -201- 0**

Appendix B: Wind Load Calculations

Calculating the Gust Factor

Exposure	α	zg (ft)	â	b^	$\bar{\alpha}$	$\bar{b}$	c	L (ft)	∈	zmin
B	7	1200	0.142857	0.84	0.25	0.45	0.3	320	0.333333	30

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} \qquad (6\text{-}5)$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \qquad (6\text{-}6)$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} \qquad (6\text{-}7)$$

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

g_Q and g_v shall be taken as 3.4 and g_R is given by

$$g_R = \sqrt{2\ln(3{,}600 n_1)} + \frac{0.577}{\sqrt{2\ln(3{,}600 n_1)}}$$

R, the resonant response factor, is given by

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

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

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} \qquad (6\text{-}12)$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) \quad \text{for } \eta > 0 \qquad (6\text{-}13a)$$

$$R_\ell = 1 \quad \text{for } \eta = 0 \qquad (6\text{-}13b)$$

where the subscript ℓ in Eq. 6-13 shall be taken as h, B, and L, respectively, where h, B, and L are defined in Section 6.3.

n_1 = building natural frequency
$R_\ell = R_h$ setting $\eta = 4.6 n_1 h/\bar{V}_{\bar{z}}$
$R_\ell = R_B$ setting $\eta = 4.6 n_1 EB/\bar{V}_{\bar{z}}$
$R_\ell = R_L$ setting $\eta = 15.4 n_1 L/\bar{V}_{\bar{z}}$
β = damping ratio, percent of critical
$\bar{V}_{\bar{z}}$ = mean hourly wind speed (ft/s) at height $\bar{z}$ determined from Eq. 6-14.

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V \left(\frac{88}{60}\right) \qquad (6\text{-}14)$$

In SI: $\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{\alpha}} V$

where $\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 6-2 and V is the basic wind speed in mi/h.

z-=	0.6h=	64.2	>	zmin=30
X			Y	
h=	107		h=	107
Bx=	145		By=	191
Qx=	0.82		Qy=	0.810098
Lz-=	399.48		Lz-=	399.4757
Iz-=	0.27		Iz-=	0.268504
gr=	4.17		gr=	4.173315
V-z-=	66.25		V-z-=	66.25492
N1=	5.63		N1=	5.634929
Rn=	0.05		Rn=	0.047022
nh=	6.94		nh=	6.942881
Rh=	0.13		Rh=	0.13366
nb=	9.41		nb=	12.39337
Rb=	0.10		Rb=	0.077433
nL=	41.49		nL=	31.49828
RL=	0.02		RL=	0.031244
R=	0.19		R=	0.162812
T=1.64sec	0.61		T=1.64sec	0.609756
n1=	0.93		n1=	0.934579
	0.70			0.700935
	2.33			2.294925
	2.55			2.551954
Gf=	0.84		Gf=	0.831835

Desgin wind pressure for MWFRS in N-S Direction						
type	Level	Height / distance	qz/ qh	wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
windward walls	Roof	107'	18.63	12.67	2.42	22.92
	7	87'	17.59	11.96	1.71	22.21
	6	72'-6"	16.65	11.32	1.07	21.57
	5	58'	15.57	10.59	0.34	20.84
	4	43'-6"	14.38	9.78	-0.47	20.03
	3	29'	12.80	8.70	-1.55	18.95
	2	14'-6"	10.54	7.17	-3.08	17.42
	1	0'	10.54	7.17	-3.08	17.42
leeward walls	All	All	18.63	-6.94	-17.19	3.31
sidewalls	All	All	18.63	-11.09	-21.34	-0.84
Roof		0-53.5	18.63	-15.02	-25.26	-4.77
		53.5-107	18.63	-13.88	-24.12	-3.63
		107-214	18.63	-8.30	-18.55	1.95

Wind Forces- N-S Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Roof	107	10.00	1450.00	10.00	1450.00	33.24	33.24	3556.15
7	87	7.00	1015.00	7.50	1087.50	57.39	90.62	4992.80
6	72.5	7.00	1015.00	7.50	1087.50	46.00	136.62	3334.94
5	58	7.00	1015.00	7.50	1087.50	44.55	181.18	2584.18
4	43.5	7.00	1015.00	7.50	1087.50	42.93	224.11	1867.55
3	29	7.00	1015.00	7.50	1087.50	40.94	265.05	1187.24
2	14.5	7.00	1015.00	7.50	1087.50	38.18	303.23	553.60
1	0'	N/A	0.00	7.50	1087.50	36.62	339.85	0
							Total base shear=	339.85 k
							Total overturning Moment=	18076.44 k-ft

Desgin wind pressure for MWFRS in E-W Direction						
type	Level	Height / distance	qz/ qh	wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
windward walls	Roof	107'	18.63	12.67	2.42	22.92
	7	87'	17.59	11.96	1.71	22.21
	6	72'-6"	16.65	11.32	1.07	21.57
	5	58'	15.57	10.59	0.34	20.84
	4	43'-6"	14.38	9.78	-0.47	20.03
	3	29'	12.80	8.70	-1.55	18.95
	2	14'-6"	10.54	7.17	-3.08	17.42
	1	0'	10.54	7.17	-3.08	17.42
leeward walls	All	All	18.63	-7.92	-18.17	2.33
sidewalls	All	All	18.63	-11.09	-21.34	-0.84
Roof		0-53.5'	18.63	-17.17	-27.42	-6.92
		53.5'-107'	18.63	-12.80	-23.05	-2.55
		107'-214'	18.63	-9.38	-19.63	0.87

Wind Forces- E-W Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Roof	107	10.00	1910.00	10.00	1910.00	43.78	43.78	4684.30
7	87	7.00	1337.00	7.50	1432.50	75.59	119.37	6576.72
6	72.5	7.00	1337.00	7.50	1432.50	60.59	179.97	4392.92
5	58	7.00	1337.00	7.50	1432.50	58.69	238.65	3403.98
4	43.5	7.00	1337.00	7.50	1432.50	56.55	295.21	2460.01
3	29	7.00	1337.00	7.50	1432.50	53.93	349.13	1563.88
2	14.5	7.00	1337.00	7.50	1432.50	50.29	399.42	729.22
1	0'	N/A	0.00	7.50	1432.50	48.24	447.66	0
							Total base shear=	447.66 k
							Total overturning Moment=	23811.04 k-ft

Relative Stiffness of Original structure:

Distribution of forces in shear walls under a 100 Kip load in X-Direction at the center of rigidity in percentage (%)								
Floor Level	P-2	P-4	P-6	P-8	P-10	P-11	P-11 in X	Total of walls
8	0.0	29.7	6.0	0.0	15.8	14.2	11.5	63.0
7	49.5	13.1	5.8	4.6	22.6	14.4	11.6	107.3
6	46.4	4.2	5.4	1.9	20.2	16.2	13.2	91.3
5	45.1	4.1	5.3	3.8	21.4	17.5	14.2	93.9
4	44.2	4.5	6.3	4.9	23.5	20.8	16.9	100.2
3	37.2	4.3	5.8	1.6	24.8	36.3	29.4	103.1
2	30.8	4.5	7.0	-1.5	16.7	69.5	56.4	113.9
1	29.6	4.3	6.7	-1.5	19.4	66.5	54.0	112.6

Distribution of forcesin the moment frames under a 100 Kip load in X-Direction at the center of rigidity in percentage (%)						
Floor Level	PF-6	PF-7	PF-8	PF-9	PF-1	Total of frames
8	0.0	16.0	12.8	12.7	0.0	41.5
7	17.9	6.8	20.6	16.7	4.5	66.6
6	12.0	5.2	15.7	13.1	2.7	48.6
5	10.7	4.6	13.7	11.5	2.4	42.9
4	8.8	3.8	11.3	9.4	2.0	35.2
3	6.5	3.1	8.7	7.2	1.8	27.3
2	3.2	0.5	1.5	1.2	0.4	6.8
1	0.4	0.4	1.3	1.0	0.3	3.4

Summary of distribution of forces in Kips				
Floor level	Walls	Frames	Total	Other members*
8	63	41	104	2
7	107	67	174	10
6	91	49	140	8
5	94	43	137	6
4	100	35	135	2
3	103	27	130	1
2	114	7	121	2
1	113	3	116	2
*:Members ignored in calculations for simplicity				

For relative stiffness in Y direction

Summary of Distribution of forces in Kips				
Floor level	Walls	Frames	Total	Other members*
8	78	27	105	1
7	78	94	173	-8
6	83	52	135	-10
5	95	41	135	-5
4	99	33	132	-3
3	94	29	123	-6
2	117	6	123	3
1	113	5	118	3
*:Members ignored in calculations for simplicity				

Summary of Relative Stiffness in %				
Floor level	Walls	Frames	Total	Other members*
8	75	26	101	-0.7
7	43	52	95	4.5
6	57	36	93	6.9
5	68	29	96	3.5
4	73	25	98	2.2
3	73	23	96	4.4
2	97	5	102	-2.1
1	98	5	102	-2.3
*:Members ignored in calculations for simplicity				

<u>See below for detailed distribution</u>

Distribution of Forces based on Relative stiffness of the shear walls in the Y-Direction (Kips)									
Floor Level	Force	P-1	P-3	P-5	P-7	P-9	P-11	P-11 in Y	Total of Walls
8	105	46.7	0.0	12.9	0.0	18.6	0.5	0.3	78.4
7	181	32.0	10.0	12.5	8.2	10.9	8.3	4.9	78.4
6	145	32.1	6.8	15.3	2.9	25.3	1.0	0.6	83.0
5	140	40.3	10.9	15.3	2.3	24.5	2.8	1.6	94.8
4	135	41.9	11.4	17.0	2.3	25.0	2.4	1.4	98.9
3	129	43.0	10.0	15.8	1.8	22.0	2.2	1.3	93.8
2	120	57.4	12.2	15.7	1.5	18.4	19.8	11.6	116.8
1	115	55.7	11.7	15.0	1.4	17.6	19.0	11.1	112.6

Distribution of Forces based on Relative stiffness of the frame piers chosen in the Y-direction (Kips)													
Floor Level	Force	PF-A	PF-B	PF-C	PF-E	PF-F	PF-G	PF-H	PF-I	PF-J	PF-K	PF-M	Total of frames
8	105	6.2	0.0	2.2	1.7	0.0	1.8	1.6	1.9	3.6	1.1	6.8	27.0
7	181	5.8	19.4	7.2	5.6	19.8	9.1	4.9	10.4	1.8	2.8	7.4	94.2
6	145	4.9	21.7	1.9	0.0	12.8	2.6	0.0	2.3	1.8	-0.1	4.1	52.0
5	140	4.4	8.6	2.5	1.0	11.6	3.2	0.5	3.1	1.4	0.3	3.9	40.6
4	135	3.7	8.0	1.9	0.5	9.7	2.5	0.1	2.4	1.3	0.2	3.2	33.4
3	129	3.3	7.0	1.8	0.8	8.6	2.3	0.3	2.2	0.2	0.2	2.8	29.5
2	120	0.8	1.5	0.3	0.0	2.3	0.3	0.0	0.4	0.2	0.0	0.4	6.1
1	115	0.4	1.1	0.2	0.1	1.4	0.3	0.0	0.2	1.2	0.0	0.3	5.4

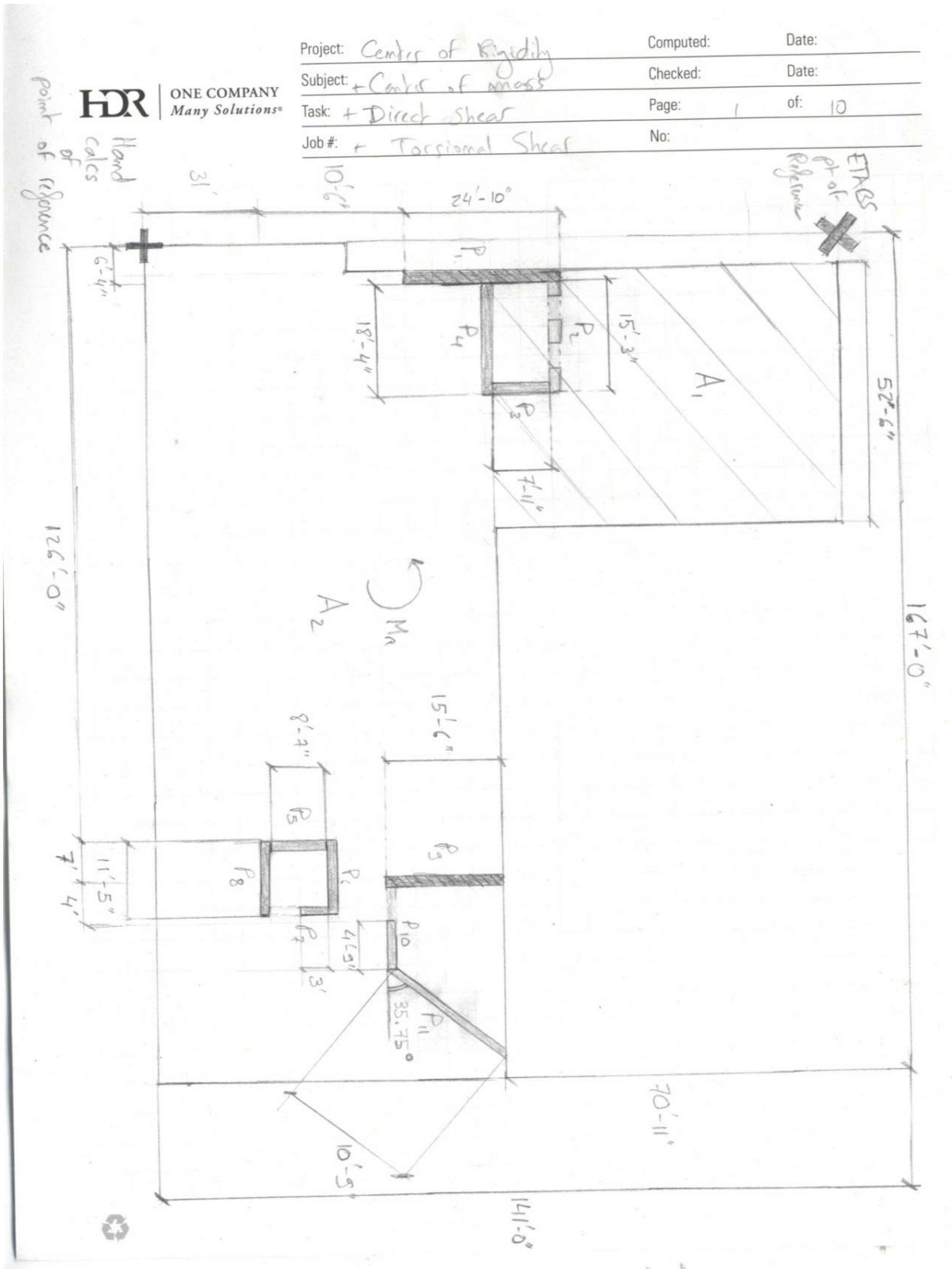
Relative Stiffness in Y direction:

Relative stiffness of the shear walls in the Y-direction under a 100 Kip load at the center of rigidity. Reported in percentage (%)								
Floor Level	P-1	P-3	P-5	P-7	P-9	P-11	P-11 in Y	Total of walls
8	44.6	0.0	12.3	0.0	17.7	0.5	0.3	74.9
7	17.7	5.5	6.9	4.5	6.0	4.6	2.7	43.4
6	22.2	4.7	10.5	2.0	17.5	0.7	0.4	57.3
5	28.7	7.7	10.9	1.6	17.4	2.0	1.2	67.6
4	31.0	8.4	12.5	1.7	18.5	1.8	1.1	73.2
3	33.3	7.8	12.2	1.4	17.0	1.7	1.0	72.7
2	47.7	10.2	13.0	1.2	15.3	16.5	9.6	97.1
1	48.2	10.2	13.0	1.2	15.3	16.5	9.6	97.6

Relative stiffness of the moment frames in the Y-direction under a 100 Kip load at the center of rigidity. In percentage (%)												
Floor Level	PF-A	PF-B	PF-C	PF-E	PF-F	PF-G	PF-H	PF-I	PF-J	PF-K	PF-M	Total of frames
8	6.0	0.0	2.1	1.6	0.0	1.7	1.6	1.8	3.4	1.1	6.5	25.8
7	3.2	10.7	4.0	3.1	10.9	5.0	2.7	5.8	1.0	1.6	4.1	52.1
6	3.4	15.0	1.3	0.0	8.8	1.8	0.0	1.6	1.2	0.0	2.8	35.9
5	3.2	6.1	1.8	0.7	8.3	2.3	0.3	2.2	1.0	0.2	2.8	28.9
4	2.8	6.0	1.4	0.4	7.2	1.8	0.0	1.8	0.9	0.1	2.4	24.7
3	2.6	5.5	1.4	0.6	6.6	1.8	0.2	1.7	0.1	0.2	2.2	22.9
2	0.7	1.3	0.2	0.0	1.9	0.2	0.0	0.3	0.2	0.0	0.3	5.1
1	0.4	1.0	0.2	0.1	1.2	0.3	0.0	0.2	1.1	0.0	0.3	4.7

Appendix C: Seismic Load Calculations

Center of Rigidity of shear walls



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Center of mass: Area × thickness × weight (pcf) x-value y-value

Slabs

Slab A_1: $(70.92')(52.5')\left(\frac{5''}{12''}\right)(150) = 232.71\,K$ 26.25' 105.5'

Slab A_2: $(70.08')(167)\left(\frac{5''}{12''}\right)(150) = 731.46\,K$ 83.5' 35'

Walls (length × t × weight × height) x-value y-value

$P_1 = (24.83')\underbrace{(1')(150)(14.5')}_{=2,175\,plf} = 54\,K$ 5.58' 64.1'

$P_2 = (15.25)(2.175) = 33.17\,K$ 13.93' 77.1'

$P_3 = (7.92)(2.175) = 17.23\,K$ 24.2' 70.1'

$P_4 = (18.3)(2.175) = 39.8\,K$ 13.93' 66.1'

$P_5 = (8.58)(2.175) = 18.66\,K$ 126.5' 35'

$P_6 = (11.42)(2.175) = 24.84\,K$ 131.7' 40'

$P_7 = P_5 = 18.66\,K$ 137' 35'

$P_8 = P_6 = 24.84\,K$ 131.7' 31'

$P_9 = (15.5)(2.175) = 33.71\,K$ 133.5' 55.25'

$P_{10} = (8.75)(2.175) = 19.03\,K$ 139.4' 49'

$P_{11} = (10.75)(2.175) = 23.38\,K$ 150.5' 52.1'

total = 1271.5 K

X-Direction

$$\bar{X} = \frac{(232.71)(26.25)+(731.46)(83.5)+(54)(5.58)+(33.17)(13.93) + (17.23)(24.2)+(39.8)(13.93)+(18.66)(126.5)+[(24.84)(131.7)]2 + (18.66)(137)+(33.71)(133.5)+(19.03)(139.4)+(23.38)(150.5')}{1271.5}$$

$$\Rightarrow \bar{x} = \frac{91070.2}{1271.5} = 71.62' \quad \therefore \quad \text{ETABS: } 71.61'$$

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Y- direction :

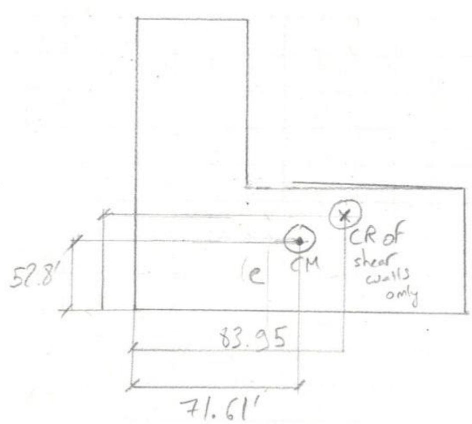
$$\bar{y} = \frac{(232.71)(105)+(731.46)(35)+(54)(64.1)+(33.17)(74.1)+(17.23)(70.1)+(39.8)(66.1)+(18.66)(35.3)+(24.84)(40)+(24.84)(31)+(33.71)(55.75)+(15.03)(58.5)+(23.38)(52.65)}{1271.5}$$

$\Rightarrow \bar{y} = \frac{67098}{1271.5} = 52.8' \Rightarrow 141' - 52.8' = 88.2'$

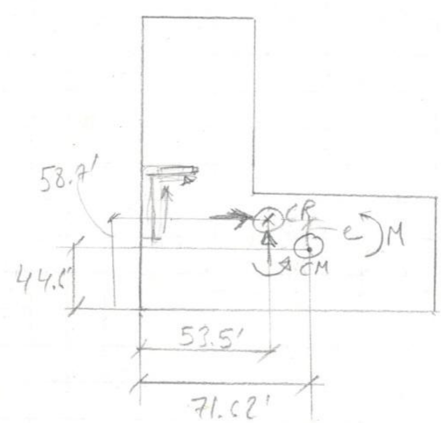
$\therefore$ ETABS $= 96.4'$

$\Rightarrow$ Difference of $8.2'$

$\frac{8.2'}{141'} = 5.8\%$ difference



Calculated



ETABS

the center of Rigidity of shear walls is calculated on the next page.

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Center of Rigidity: (Shear walls only)

R:

		X-value:	Y-value:
ρ_1:	.222	5.92'	64.1'
ρ_2:	.314	13.93'	77.1'
ρ_3:	.047	24.2'	70.1'
ρ_4:	.028	13.93'	66.1'
ρ_5:	.105	126.5'	35.13
ρ_6:	.037	131.7'	40'
ρ_7:	.02	137'	35.3'
ρ_8:	.013	131.7'	31'
ρ_9:	.175	133.5'	55.25'
ρ_{10}:	.137	139.4'	58.5'
ρ_{11} in x:	.089	150.5'	52.1'
ρ_{11} in y:	.004 ≈ 0		

$$X_r = \frac{(.222)(5.92)+(.047)(24.2)+(.105)(126.5)+(.02)(137)+(.175)(133.5)+(.089)(150.5)}{(.222+.047+.105+.02+.175+.089)}$$

$X_r = 83.95$

∴ ETABS: $X_r = 53.478'$

$83.95 - 53.478 = 30.47'$ $\qquad 30.47/167 = 18.25\%$

from ETABS total wall contribution to resistance is 57.3%

Thus the frames take ≈43% of the force in the x-direction

Hence, moment frames in this system are heavily relied on due to the building's shape and wall placement.

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$$y_r = \left[(.314)(77.1) + (.028)(66.1') + (.037)(40) + (.013)(31) + (.137)(58.5')\right] / (.314 + .028 + .037 + .013 + .137)$$

$y_r = 67.97'$

∴ ETABS $Y_r = 82.32'$ $141 - 82.32' = 58.68$

$67.97 - 58.68 = 9.29'$ $9.29/141 = 6.6\%$

Thus, the walls contribute more to the center of rigidity in the Y-direction than in the x-direction this is due to the lack of big moment frames resisting in that direction as opposed to the X-direction

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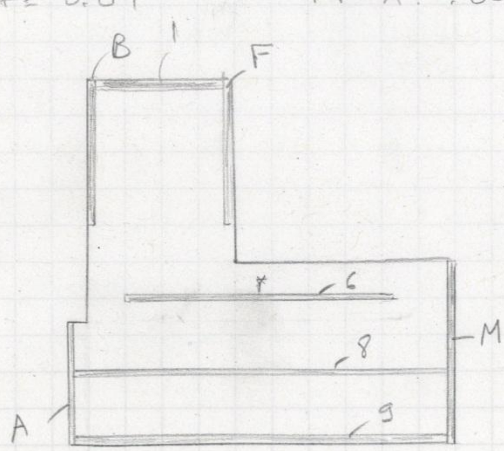
Torsional Rigidity : $J = \Sigma R_i d_i^2$ (d_i: distance from CR to wall i)

Using ETABS's $x_r = 53.5'$ / $y_r = 58.7'$

R

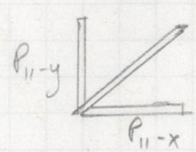
PF-6 : .081 PF-8 : .106 PF-F : .088
PF-1 ≈ 0.02 PF-9 : .089 PF-B : .15
PF-7 = 0.04 PF-A : .034 PF-M : .028



d_i of B = 47'
d_i of F = 7.75'
d_i of A = 53.5'
d_i of M = 113.5'
d_i of 6 = -9.7'
d_i of 8 = -28'
d_i of 9 = -58.7'
d_i of 1 = 82.3'
d_i of 7 = 16.7'

d_i of P_1 = 47.5' d_i of P_5 = 73' d_i of P_9 = 80'
d_i of P_2 = 22.3' " P_6 = 18.7' " P_{10} = 9.7'
" P_3 = 23.7' " P_7 = 83.5'
" P_4 = 14.3' " P_8 = 27.7'

Decomposing the slanted wall into two walls for simplicity



d_i of P_{11-x} = 9.7'
d_i of P_{11-y} = 89'

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$$J = RP_1(47.5)^2 + RP_2(22.3)^2 + RP_3(29.7)^2 + RP_4(14.3)^2 + RP_5(73)^2 + RP_6(18.7')^2 + RP_7(83.5')^2 + RP_8(27.7')^2 + RP_9(80)^2 + RP_{10}(9.7)^2 + RP_{11-x}(9.7)^2 + RP_{11-9}(89')^2 + RPF6(9.7)^2 + RPF8(28)^2 + RPF9(58.7)^2 + RPF\text{-}A(53.5)^2 + RPF\text{-}B(47)^2 + RPF\text{-}F(7.75)^2 + RPF\text{-}M(113.5)^2 + RPF_1(82.3)^2$$

$$J = 3938\ (K/in)(ft^2)$$

Direct shear: 93.2 in x-direction using worst case

$$V_{P_2} = .32\,(93.2) = 29.82$$
$$V_{P_4} = .028\,(93.2) = 2.6$$
$$V_{P_6} = .037\,(93.2) = 3.45$$
$$V_{P_8} = .013\,(93.2) = 1.21$$
$$V_{P_{10}} = .137\,(93.2) = 12.77$$
$$V_{P_{11-x}} = .09\,(93.2) = 8.4$$
$$V_{PF-6} = .081\,(93.2) = 7.55$$
$$V_{PF-8} = .11\,(93.2) = 10.3$$
$$V_{PF-9} = .09\,(93.2) = 8.4$$
$$V_{PF-1} = .02\,(93.2) = 1.86$$
$$V_{PF-7} = .04\,(93.2) = 3.73$$

0.97 — 90.1 K

All forces are acting in this direction (←)

this not equal to 1 as there are the other members not taken into consideration

A 3.1 K difference is due to the minor contributing frames not considered

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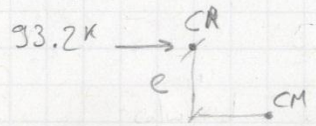
Torsional Shear in Walls :



$e_y = 58.7' - 44.6' = 14.1$ using ETABS

$$V_{P_8} = \frac{(93.2)(14.1)(27.7)(.013)}{3538} = .12\ K \leftarrow$$

$$V_{P_2} = \frac{(93.2)(14.1)(22.3)(.314)}{3538} = 2.33\ K \rightarrow$$

$$V_{P_4} = \frac{(93.2)(14.1)(14.3)(.028)}{3538} = .14\ K \rightarrow$$

$$V_{P_6} = \frac{(93.2)(14.1)(18.7)(.037)}{3538} = .25\ K \leftarrow$$

$$V_{P_{10}} = \frac{(93.2)(14.1)(9.7)(.137)}{3538} = .44\ K \rightarrow$$

$$V_{P_{11}} = \frac{(93.2)(9.7)(14.1)(.09)}{3538} = .3\ K \rightarrow$$

$$V_{PF-6} = \frac{(93.2)(14.1)(9.7)(.08)}{3538} = .26\ K \leftarrow$$

$$V_{PF-8} = \frac{(93.2)(14.1)(28)(.106)}{3538} = 1.0\ K \leftarrow$$

$$V_{PF-5} = \frac{(93.2)(14.1)(58.7)(.09)}{3538} = 1.81\ K \leftarrow$$

$$V_{PF1} = \frac{(93.2)(14.1)(82.3)(.02)}{3538} = 0.55\ K \rightarrow$$

$$V_{PF7} = \frac{(93.2)(14.1)(16.7)(.04)}{3538} = .22\ K \rightarrow$$

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Total shear: Direct shear + torsional shear:

$V_{P_2} = 29.82 - 2.33 = 27.49^k$
$V_{P_4} = 2.6 - .14 = 2.46^k$
$V_{P_6} = 3.45 + .25 = 3.7^k$
$V_{P_8} = 1.21 + .12 = 1.33^k$
$V_{P_{10}} = 12.77 - .44 = 12.33^k$
$V_{P_{11-X}} = 8.4 - .3 = 8.1^k$
$V_{PF-6} = 7.55 + .26 = 7.81^k$
$V_{PF-8} = 10.3 + 1 = 11.3^k$
$V_{PF-9} = 8.4 + 1.81 = 10.21^k$
$V_{PF-1} = 1.86 - .55 = 1.31^k$
$V_{PF-7} = 3.73 - .22 = 3.52^k$

P_2 is worst case in x-direction $V_{P_2} = 27.49^k$

Y-direction:

$P = 145^k$ $M_T = 3630$
$e_x = 18.12'$ from ETABS

e, M_R, V

Direct shear:

One of these two walls will control in Y direction

$V_{P_1} = (.222)(145) = 32.2^k$

$V_{P_9} = (.175)(145) = 25.4^k$

Torsional Shear:

$$V_{P_1} = \frac{[(145)(18.12) + 3630](47.5)(.222)}{3938} = 16.76^k$$

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$$V_{P_9} = \frac{[(145)(18.12) + 3630](80)(.175)}{3938} = 22.25^{K} \downarrow$$

Total shear

$$V_{P_1} = 32.2 - 16.76 = 15.44 \downarrow$$

$$V_{P_9} = 25.4^{K} + 22.25 = 47.65^{K} \downarrow$$

Worst case in Y-direction is wall P_9 $V_{P_9} = 47.65^{K} \downarrow$

From ETABS, for the same load case, this wall yielded a value of -47.71^{K} thus a difference of only 0.13%.

This same shear wall was then checked through Excell under ACI-318-08.

Spreadsheet is shown on the page after.

Center of Mass and Rigidity of CA-S3 taken from ETABS

Etabs Results for Center of Mass and Rigidity (in)						
Floor level	Center of mass		Center of rigidity		Eccenticity (in)	
	XCM	YCM	XCR	YCR	ex	ey
8	999	1403	726	956	273	447
7	825	1097	721	960	104	137
6	825	1097	712	962	113	134
5	825	1097	708	956	117	141
4	825	1097	703	952	122	144
3	825	1097	697	957	128	140
2	825	1097	684	976	142	121

Etabs Results for Center of Mass and Rigidity (ft)						
Floor level	Center of mass		Center of rigidity		Eccenticity (ft)	
	XCM	YCM	XCR	YCR	ex	ey
8	83	117	61	80	23	37
7	69	91	60	80	9	11
6	69	91	59	80	9	11
5	69	91	59	80	10	12
4	69	91	59	79	10	12
3	69	91	58	80	11	12
2	69	91	57	81	12	10

: not accurately modeled in ETABS for simplicity of rigid diaphragms

Step 1															
Seismic Forces															
Level	Story weight, wx	Story height (ft), hx	wx.hxK	Cvx	Story force (k)	Story Shear (k)	Overturning moment (k-ft)	Bx (ft)	5% Bx	Ax	Mzy (k-ft)	By (ft)	5% By	Ax	Mzx (k-ft)
8	1648	121.5	564184	0.2	325.7	325.7	39574.8	145	7.25	1	2361	191	9.55	1	3111
7	3133	101.5	861719	0.2	497.5	823.2	50495.6	145	7.25	1	3607	191	9.55	1	4751
6	2944	87.0	671222	0.2	387.5	1210.7	33713.8	145	7.25	1	2809	191	9.55	1	3701
5	2893	72.5	528510	0.2	305.1	1515.8	22121.4	145	7.25	1	2212	191	9.55	1	2914
4	2893	58.0	402934	0.1	232.6	1748.5	13492.2	145	7.25	1	1687	191	9.55	1	2222
3	2893	43.5	284013	0.1	164.0	1912.4	7132.6	145	7.25	1	1189	191	9.55	1	1566
2	2895	29.0	173635	0.0	100.2	2012.7	2907.1	145	7.25	1	727	191	9.55	1	957
1	2895	14.5	74748	0.0	43.2	169437.4	625.7	146	7.3	1	315	192	9.6	1	414
∑	22194	∑wihiK=	3486215			Base Shear =	2013			∑MZY =	14907			∑MZX =	19635.4
						Total Overturning Moment =	169437								

Step 2									
Earthquake Loads Y-direction			Earthquake drift		Earthquake interstory drift		Axy		
Story level	Ey (k)	Mzy (k-ft)	δA	δB	δmax	δavg	Ax calculated	Ax min\max	Ax used
8	326	2361	5.93	3.31	5.93	4.62	1.14	1\3	1.14
7	497.5	3607	5.18	2.85	5.18	4.02	1.16	1\3	1.16
6	387.5	2809	4.39	2.39	4.39	3.39	1.16	1\3	1.16
5	305.1	2212	3.55	1.92	3.55	2.74	1.17	1\3	1.17
4	232.6	1687	2.69	1.45	2.69	2.07	1.17	1\3	1.17
3	164.0	1189	1.86	1.00	1.86	1.43	1.17	1\3	1.17
2	100.0	727	1.11	0.59	1.11	0.85	1.19	1\3	1.19
1	43.0	315	0.52	0.25	0.52	0.39	1.28	1\3	1.28

Step 3									
Earthquake Loads X-direction			Earthquake drift		Earthquake interstory drift		Axx		
Story level	Ex (k)	Mzx (k-ft)	δA	δB	δmax	δavg	Ax calculated	Ax min\max	Ax used
8	326	3111	4.44	4.44	4.44	4.44	0.69	1 \ 3	1.00
7	497.5	4751	3.87	3.87	3.87	3.87	0.69	1 \ 3	1.00
6	387.5	3701	3.25	3.25	3.25	3.25	0.69	1 \ 3	1.00
5	305.1	2914	2.61	2.61	2.61	2.61	0.69	1 \ 3	1.00
4	232.6	2222	1.96	1.96	1.96	1.96	0.69	1 \ 3	1.00
3	164.0	1566	1.33	1.33	1.33	1.33	0.69	1 \ 3	1.00
2	100.0	957	0.77	0.77	0.77	0.77	0.69	1 \ 3	1.00
1	43.0	414	0.34	0.34	0.34	0.34	0.69	1 \ 3	1.00

Step 4															
Seismic Forces															
Level	Story weight, wx	Story height (ft), hx	wx.hxK	Cvx	Story force (k)	Story Shear (k)	Overturning moment (k-ft)	Bx (ft)	5% Bx	Ax	Mzy (k-ft)	By (ft)	5% By	Ax	Mzx (k-ft)
8	1648	121.5	564184	0.2	325.7	325.7	39574.8	145	7.25	1.14	2361	191	9.55	1	3111
7	3133	101.5	861719	0.2	497.5	823.2	50495.6	145	7.25	1.16	3607	191	9.55	1	4751
6	2944	87.0	671222	0.2	387.5	1210.7	33713.8	145	7.25	1.16	2809	191	9.55	1	3701
5	2893	72.5	528510	0.2	305.1	1515.8	22121.4	145	7.25	1.17	2212	191	9.55	1	2914
4	2893	58.0	402934	0.1	232.6	1748.5	13492.2	145	7.25	1.17	1687	191	9.55	1	2222
3	2893	43.5	284013	0.1	164.0	1912.4	7132.6	145	7.25	1.17	1189	191	9.55	1	1566
2	2895	29.0	173635	0.0	100.2	2012.7	2907.1	145	7.25	1.19	727	191	9.55	1	957
1	2895	14.5	74748	0.0	43.2	169437.4	625.7	146	7.3	1.28	315	192	9.6	1	414
∑	22194	∑wihiK=	3486215			Base Shear =	2013			∑MZY =	14907			∑MZX =	19635.4
						Total Overturning Moment =	169437								

Earthquake Loads Y-direction			Earthquake drift		Earthquake interstory drift					
Story level	Ey (k)	Mzy (k-ft)	ΔA	ΔB	Δmax	Δavg	Δmax/Δavg	Type 1a - 1b	type	
8	326	2361	0.75	0.46	0.75	0.60	1.24	1.2 - 1.4 Δavg	1-a	Type 1-a torsionally Irregular
7	497.5	3607	0.79	0.46	0.79	0.62	1.26	1.2 - 1.4 Δavg	1-a	
6	387.5	2809	0.84	0.47	0.84	0.66	1.29	1.2 - 1.4 Δavg	1-a	
5	305.1	2212	0.86	0.47	0.86	0.66	1.29	1.2 - 1.4 Δavg	1-a	
4	232.6	1687	0.83	0.45	0.83	0.64	1.29	1.2 - 1.4 Δavg	1-a	
3	164.0	1189	0.75	0.41	0.75	0.58	1.29	1.2 - 1.4 Δavg	1-a	
2	100.0	727	0.59	0.34	0.59	0.46	1.27	1.2 - 1.4 Δavg	1-a	
1	43.0	315	0.50	0.24	0.50	0.37	1.35	1.2 - 1.4 Δavg	1-a	

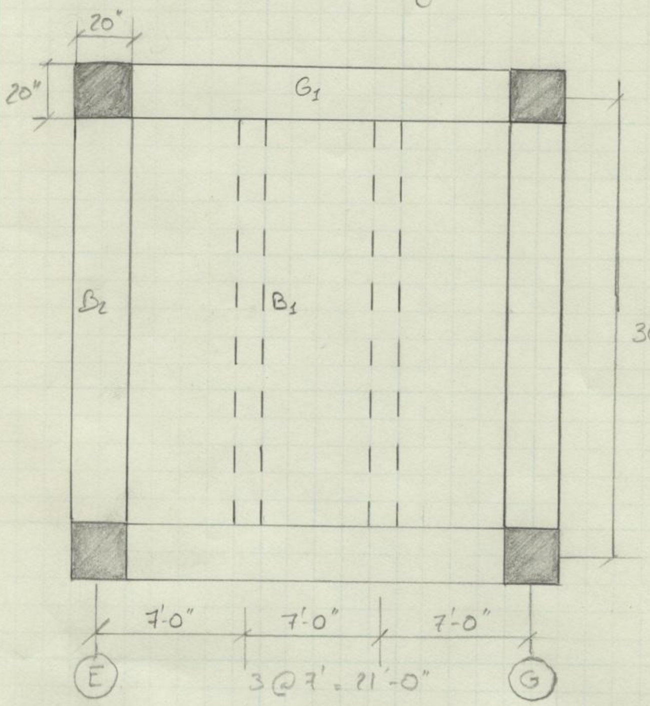
Earthquake Loads X-direction			Earthquake drift		Earthquake interstory drift				
Story level	Ex (k)	Mzx (k-ft)	ΔA	ΔB	Δmax	Δavg	Δmax/Δavg	Type 1a - 1b	type
8	326	3111	0.57	0.48	0.57	0.52	1.09	1.2 - 1.4 Δavg	No torsional Irregularity
7	497.5	4751	0.62	0.48	0.62	0.55	1.13	1.2 - 1.4 Δavg	
6	387.5	3701	0.64	0.48	0.64	0.56	1.15	1.2 - 1.4 Δavg	
5	305.1	2914	0.65	0.47	0.65	0.56	1.16	1.2 - 1.4 Δavg	
4	232.6	2222	0.63	0.45	0.63	0.54	1.16	1.2 - 1.4 Δavg	
3	164.0	1566	0.56	0.41	0.56	0.49	1.15	1.2 - 1.4 Δavg	
2	100.0	957	0.43	0.35	0.43	0.39	1.11	1.2 - 1.4 Δavg	
1	43.0	414	0.33	0.29	0.33	0.31	1.06	1.2 - 1.4 Δavg	

Earthquake Loads Y-direction	Story level	Earthquake story drift		Earthquake drift		Earthquake interstory drift	
		δx	δy	Cd. dx / I	Cd.dy/I	Δx	Δy
	8	-0.25	5.01	-1.26	25.05	-0.16	2.90
	7	-0.22	4.43	-1.10	22.15	-0.21	3.15
	6	-0.18	3.80	-0.89	19.00	-0.23	3.25
	5	-0.13	3.15	-0.66	15.75	-0.23	3.40
	4	-0.09	2.47	-0.43	12.35	-0.20	3.05
	3	-0.05	1.86	-0.23	9.30	-0.12	3.00
	2	-0.02	1.26	-0.10	6.30	0.02	2.80
	1	-0.02	0.70	-0.12	3.50	-0.12	2.95

Earthquake Loads X-direction	Story level	Earthquake story drift		Earthquake drift		Earthquake interstory drift	
		δx	δy	Cd.dx/I	Cd.dy/I	Δx	Δy
	8	4.44	-0.13	22.20	-0.64	2.84	0.00
	7	3.87	-0.13	19.36	-0.64	3.08	-0.13
	6	3.25	-0.10	16.27	-0.52	3.21	-0.20
	5	2.61	-0.06	13.06	-0.31	3.26	-0.24
	4	1.96	-0.01	9.80	-0.07	3.15	-0.22
	3	1.33	0.03	6.66	0.15	2.80	-0.13
	2	0.77	0.05	3.85	0.27	2.16	0.09
	1	0.34	0.04	1.70	0.19	1.70	0.19

Appendix D: Typical Concrete Cast-in-place One Way Slab

Final Report One Way Slab 1/11



$f'_c = 4{,}000$ psi

$f_y = 60{,}000$ psi

LL = 125 psf

SPD = 20 psf

Minimum thickness of slab from table 9.5 (a) ACI 318-08

Exterior Bay ; one end continuous : $\frac{\ell}{24} = \frac{(7')(12'')}{24} = 3.5''$

⇒ use a thickness of 5" for constructibility, vibrations and fire proofing

d : Assume #4 bars to calculate d with a clear cover of 3/4"

$d = h - 3/4'' - d_b/2 = 5'' - 3/4'' - \frac{0.5''}{2} = 4''$

Load : $w_D = \left(\frac{5}{12}\right)(150) = 62.5$ psf

Superimposed load = 20 psf

$w_{D\,total} = 82.5$ psf

$w_L = 125$ psf unreducible

$w_u = 1.2D + 1.6L = 1.2(82.5) + 1.6(125) = 299$ psf ≈ 300 psf

Assume the section is tensioned controlled ⇒ $\phi = 0.9$

use $b_1 = 12''$ for constructibility and formwork costs

$b_2 = 20''$ " " " " to fit the columns

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$w_L = 125 < 3\,w_D = 82.5 \times 3 = 248$ psf $\Rightarrow$ use ACI Moment Coefficient

Exterior Bay: $l_n = 84 - 10 - 6" = 68"$ $\qquad l_{n\,avg} = 70"$ $\qquad$ Interior Bay: $l_n = 84" - 6" - 6" = 72"$

Note: For simple calculations and preliminary schematic design, this method will be used even though the spans in the E-W direction are not the same. However, only the one bay will be used

To be conservative use higher $l_n = 72"$

Mu: First Interior Support: $M_u = -\dfrac{w_u l_n^2}{10} = -\dfrac{.3\,(70/12)^2}{10} = -1.02$ k-ft

Second Interior Support: $M_u = \dfrac{-w_u l_n^2}{11}$

$$M_u = -\frac{.3\,(72/12)^2}{10} = -1.55 \text{ k-ft} \Rightarrow \text{max controls}$$

Reinforcement: One Way normally have low reinforcement ratio, ρ thus assume $jd = 0.95d$

$$A_s \geq \frac{M_u}{\phi f_y \left(d - \frac{a}{2}\right)} \cong \frac{M_u}{\phi f_y (jd)} = \frac{1.55 \times 12"}{0.9(60)(.95)(4)} = 0.091 \text{ in}^2/\text{ft}$$

$$a = \frac{A_s \cdot f_y}{0.85 f'_c\, b} = \frac{0.091 \times (60)}{0.85\,(4)\,(\underbrace{12"}_{\text{per foot}})} = 0.134 \text{ in}$$

$a \ll \frac{3}{8}d$ thus $c = a/\beta_1 \ll \frac{3}{8}d \Rightarrow \phi = 0.9$

$$A_s \geq \frac{0.091 \times 12}{0.9\,(60)\left(4 - \frac{0.134}{2}\right)} = 5.14 \times 10^{-3} \text{ in}^2/\text{ft}$$

$$\rho = \frac{A_s}{b\,d} = \frac{5.14 \times 10^{-3}}{(12)\,(4)} = 1.07 \times 10^{-4}$$

$$A_{s,min} = 1.07 \times 10^{-4} \times b \times h = 1.07 \times 10^{-4} \times 12 \times 5 = 0.0064 \text{ in}^2/\text{ft}$$

Shear Check: Using ACI coefficient § 8.3.3 shear @ exterior face of first interior support

$$V_u = \frac{1.15\, w_u\, l_n}{2} = \frac{1.15\,(300)\,(72/12)}{2} = 1035 \text{ lb/ft}$$

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$\phi V_c = 0.75\left(2\lambda\sqrt{f'_c}\, b_w d\right) = 0.75(2)\left(\sqrt{4000}\right)(12)(4)$ (λ = 1.0)

$\phi V_c = 2277\text{ lb} > V_u = 1035\text{ lb/ft} \Rightarrow$ OK!

Design of Reinforcement:

Description	External Support	Exterior Midspan	First Interior	Interior Midspan
1. l_n (in)	72"	72"	70"	68"
2. $w_u l_n^2$ (K-ft/ft)	10.8	10.8	10.21	9.63
3. M coefficient	-1/24	1/14	-1/10	1/14
4. M_u (K-ft/ft)	-0.45	0.77	-1.02	0.69
5. As required (in²/ft)	3.14×10^{-4}	4.15×10^{-3}	5.14×10^{-3}	4.0×10^{-3}
6. As, minimum (in²/ft)	0.0064 →			
7. Bars selected	No. 4 @ 12" →			
8. Final As				

Check for spacing: $s = 15\left(\frac{40{,}000}{2/3(60{,}000)}\right) - 2.5\left(3/4\right) < 12\left(\frac{40{,}000}{40{,}000}\right)$ (1.0, 1.0)

$= 13.125 > 12" \Rightarrow$ OK!

Transverse direction

As for shrinkage and temperature $= 0.0018(12)(5)$
$= 0.108$

Maximum spacing: $\leq 5h = 25"$
$\leq 18 \rightarrow$ controls

$$\boxed{\text{Slab detail: 5" slab} \quad \text{No. 4 @ 12" for top and bottom steel; No. 4 @ 18" for transverse steel}}$$

Beam Design: B2:

$b = 16"$ $\quad h \approx \frac{l}{12}$ to $\frac{l}{18}$ $\quad$ 30.75" to 20.5" $\quad$ select $h = 24"$ (between two values)

$w_{beam} = \frac{(24-5)(16)}{144} \times 150 = 317\text{ plf} = .317\text{ K/ft}$

$w(\text{slab and SDL}) = (82.5 \times 12) = 990\text{ lb/ft} = .99\text{ K/ft}$

$W_u = \begin{cases} 1.2D + 1.6L = 1.2(.99 + .317) + 1.6(1.5) = 3.97\text{ K/ft} \\ 1.4D = 1.4(.99 + .317) = 1.72\text{ K/ft} \end{cases}$

First ext: $M_u = \frac{-w_u l_n^2}{24} = \frac{-3.97(30.75-1.5)^2}{24}$, $M_u = -142$ K-ft

Interior: $M_u = \frac{w_u l_n^2}{14} = \frac{+3.97(29.25)^2}{14}$, $M_u = 243$ K-ft

First Interior: $M_u = \frac{-w_u l_n^2}{10} = \frac{-3.97(29.25)^2}{10}$, $M_u = -335$ K-ft

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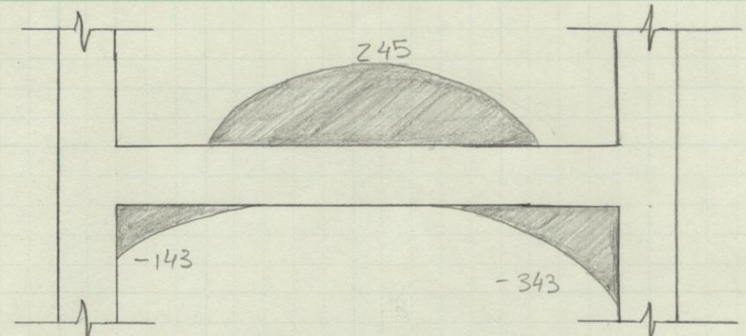

b_{eff} = 1/4 span length = 1/4 (30.75)(12) = 92.25"
$b_w + 16 h_f$ = 20 + 16 (5) = 100"
$b_w + 2$ (1/2 clear distance) = 20 + 2(1/2 (7'-1.5')) = 86"
b_{eff} = 86"

Assume f'_c = 4,000 psi use #4 stirrups

Description	First Exterior	Midspan	First Interior
1. M_u (K-ft)	-142	243	-335
2. $A_s = \frac{M_u}{4d}$	1.7	2.90	3.96
3. A_s chosen	(2)#9 = 2.0	(3)#9 = 3.0	(4)#9 = 4.0
4. $\rho = \frac{A_s}{bd} \leq .0125$	0.0059	0.0089	0.012
5. $d = 24 - 1.5 - \frac{1.128}{2}$	21.44	21.44	21.44

6. $M_u = \phi .85 f'_c b_{eff} h_f (d - \frac{h_f}{2})$ = 2077'-K > M_u treat as rectangular beam

8. $a = \frac{A_s F_y}{0.85 f'_c b}$	2.21	3.31	4.41
9. $c = a/\beta_1$	2.6	3.89	5.19
10. $\varepsilon_s = \frac{(d-c)}{c} \varepsilon_u$	0.0217 ⇒ ϕ = 0.9	0.0135 ϕ = 0.9	0.009 > .005 ϕ = 0.9
11. $\phi M_n = \phi A_s F_y (d - \frac{a}{2})/12$	183'-K > M_u ⇒ OK	267'-K > M_u ⇒ OK	346'-K > M_u ⇒ OK

12. $A_{s,min}$ = max | $\frac{3\sqrt{f'_c}\, b\, d}{f_y}$ = $\frac{3\sqrt{4{,}000}(16)(21.44)}{60{,}000}$ = 1.08
$\frac{200 b d}{f_y}$ = $\frac{200(16)(21.44)}{60{,}000}$ = 1.14 ⇒ 1.14 in²

13. Spacing Requirement: Meets spacing requirement of 1", 1 d_b ⇒ OK!

14. $V_u = \frac{w_u \ell_n}{2}$ 3.97 (29.25) / 2 = 58 K

15. $V_c = 2\sqrt{f'_c}\, b_w d$ $2\sqrt{4000}(16)(21.44)$ = 43.39 K

16. $V_s = 2 A_s f_y\, d/s$ 2(.2)(60)$\left(\frac{21.44}{12}\right)$ = 42.88 K

17. $\phi V_n = \phi (V_c + V_s)$ = 0.75 (43.39 + 42.88) = 64.7 > 58 K ⇒ OK!

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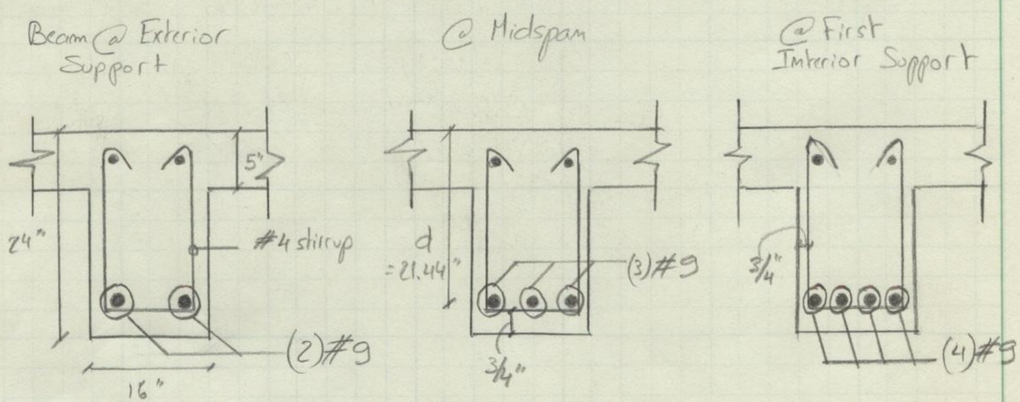
Deflection: (Assume Simply Supported)

$$\Delta_{LL} = \frac{5 w_L \ell^4 (12)^3}{384\ EI} = \frac{5(1.5)(30.75)^4 \times 1000}{384\ 57{,}000\sqrt{4{,}000}\ \frac{16(24)^3}{12}} = 0.454''$$

$$\Delta_{LL\ max} = \frac{\ell}{480} = \frac{30.75 \times 2}{480} = 0.769'' \longrightarrow OK!$$

$$\Delta_{TL} = \frac{5(2.807)(30.75)^4(1728)(1000)}{384\ 57{,}000\sqrt{4{,}000}\ \left(\frac{16(24)^3}{12}\right)} = 0.849''$$

$$\Delta_{TL\ max} = \frac{\ell}{280} = 1.32'' \longrightarrow OK!$$



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Beam Design: B2:

$b = 20"$ $\quad h \approx \frac{\ell}{12}$ to $\frac{\ell}{18} \approx 30.75"$ to $20.5"$

select $h = 24$ in (between two values)

$w_{\text{beam web}} = \left(\frac{(24-4)(20)}{144}\right) \times 150 = 417\ \text{lb/ft} = .417\ \text{K/ft}$

$w(\text{slab and SDL}) = (70\,\text{psf} \times 12) = 840\ \text{lb/ft} = .840\ \text{K/ft}$

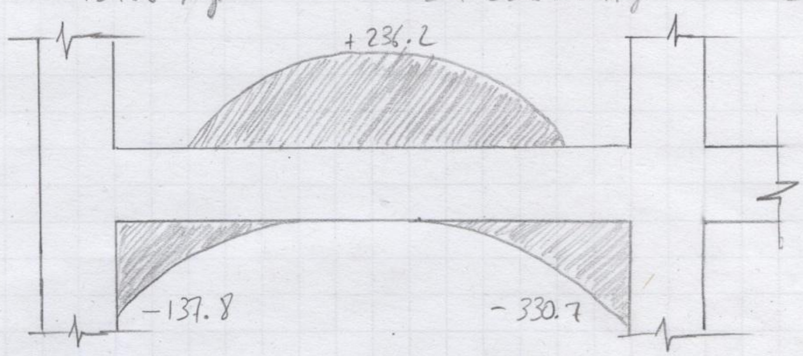
$w_u = \left|\begin{array}{l} 1.2\,w_D + 1.6\,w_L = 1.2(.840 + .417) + 1.6(1.5) = 3.91\ \text{K/ft} \\ 1.4(1.257) = 1.76\ \text{K/ft} \end{array}\right.$

unreduced for labratory > 100psf

First ext: $M_u = -\frac{w_u \ell_n^2}{24} = \frac{-3.91\left(30.75 - \frac{20}{12}\right)^2}{24}$; $M_u = -137.8\ \text{K-ft}$

Interior: $+\frac{w_u \ell_n^2}{14} = \frac{+3.91(29.08)^2}{14} = +236.2\ \text{K-ft}$

First int: $\frac{-w_u \ell_n^2}{10} = \frac{-3.91(29.08)^2}{10} = -330.7\ \text{K-ft}$



$b_{\text{eff}} = \left|\begin{array}{l} 1/4 \text{ span length} = 1/4(30.75)(12) = 92.25" \\ b_w + 16\,h_f = 20 + 16(4) = 84" \\ b_w + 2(1/2 \text{ clear distance}) = 20 + 2\left(1/2\left(7' - \frac{20}{12}\right)\right) = 84" \end{array}\right.$ min

$\Rightarrow b_{\text{eff}} = 84"$

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Assume $f'_c = 4{,}000$ use #4 stirrups

Description:	First exterior	Midspan	First Interior
1. M_u (k-ft)	-137.8	$+236.2$	-330.7
2. $A_s = \frac{M_u}{4 \cdot d}$ (in²)	$-137.8/4(21) = 1.64$	$236.2/4(21) = 2.81$	$330.7/4(21) = 3.94$
3. A_s (chosen)	2#9 = 2.0 in²	3#9 = 3.0 in²	4#9 = 4.0 in²
4. $\rho = \frac{A_s}{b \cdot d} < 0.025$	0.0048	0.0071	0.0095
5. d	$24'' - 1.5 - .5 - \frac{1.128}{2} = 21.44$	21.44	21.44
6. b_{eff}	84"	84"	84"
7. $M_{u,TBM} = \phi\, 0.85 f'_c b_{eff} h_f (d - \frac{h_f}{2})$	$[0.9(.85)(4)(84)(4)(21.44 - \frac{4}{2})]/12 = 1666$ '-K	$> M_u$ Treat as rectangular beam	
8. $a = \frac{A_s f_y}{0.85 f'_c b}$	1.765	2.65	3.53
9. $c = a/\beta_1 (=.85)$	2.08	3.117	4.15
10. $\varepsilon_s = \frac{(d-c)}{c}\varepsilon_u$	$\left(\frac{21.44 - 2.08}{2.08}\right).003 = 0.028$	0.0176	0.0125
9. if $\varepsilon_s > 0.005$ the $\phi = 0.9$	$\Rightarrow \phi = 0.9$	$\Rightarrow 0.9$	$\Rightarrow 0.9$
10. $\phi M_n = [\phi A_s f_y (d - \frac{a}{2})]/12$	185 '-K $> M_u \Rightarrow$ OK	271.5 '-K $> M_u \Rightarrow$ OK	354.15 '-K $> M_u \Rightarrow$ OK!

11. $A_{s,min} = \frac{3\sqrt{f'_c}\, bd}{f_y}$, max $\frac{200\, bd}{f_y}$

$\frac{3\sqrt{4{,}000}(20)(21.44)}{60{,}000} = 1.356$ in²

$\frac{200(20)(21.44)}{60{,}000} = 1.43$ in²

$A_{s,min} = 1.43$ in²

12. Spacing requirement — Meets spacing requirements $\Rightarrow$ OK!

13. $V_u = w_u l_n / 2$ — $[3.91(30.75 - (20/12))]/2 = V_u = 56.86^K$

14. $V_c = 2\sqrt{f'_c}\, b \cdot d$ — $2\sqrt{4{,}000}(20)(21.44) = 54.24^K$

15. $V_s = 2(A_s) f_y \left(\frac{d}{12}\right)$ — $2(.2)(60)\left(\frac{21.44}{12}\right)$ #4 stirrup $V_s = 42.88^K$

16. $\phi V_n = \phi(V_c + V_s)$ — $\phi V_n = 0.75(54.24 + 42.88) = 72.84$

$\phi V_n > V_u \Rightarrow$ OK!

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Deflection: (Assume simply supported)

$$\Delta_{LL} = \frac{5 w_L \ell^4 (12)^3}{384 E I} = \frac{5(1.5)(30.75)^4(1728) \times 1000}{384(57{,}000\sqrt{4{,}000})\left(\frac{20(24)^3}{12}\right)} = 0.363''$$

$\Delta_{LL} \Rightarrow$ OK!

$$\Delta_{LL\,max} = \frac{\ell}{480} = \frac{30.75 \times 12}{480} = 0.769$$

$$\Delta_{TL} = \frac{5(1.257 + 1.5)(30.75)^4(1728) \times 1000}{384(57{,}000\sqrt{4{,}000})\left(\frac{20(24)^3}{12}\right)} = 0.668''$$

$$\Delta_{TL\,max} = \frac{\ell}{280} = \frac{30.75 \times 12}{280} = 1.319''$$

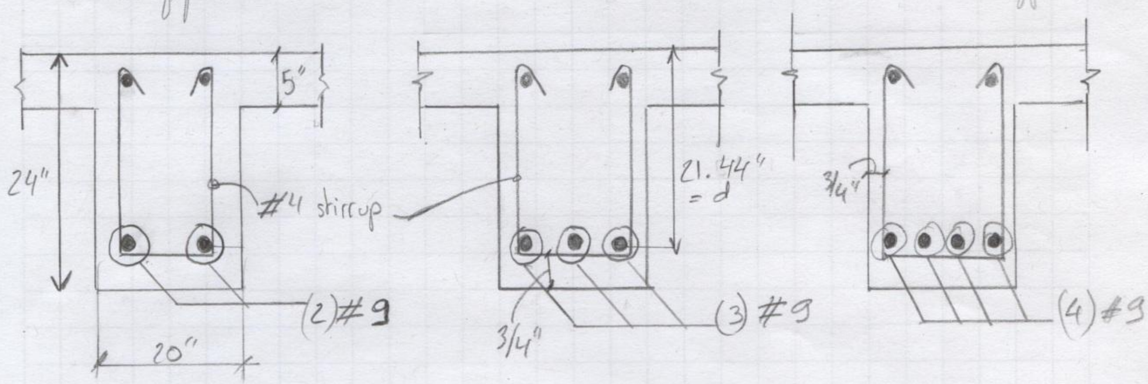
$\Rightarrow \Delta_{TL}$ OK!

Long term deflection:

$$w_{TL} = 3(1.257 + .25(1.5)) = 4.896$$

$$\Delta_{TL} = \frac{5(4.896)(30.75)^4(1728) \times 1000}{384(57{,}000\sqrt{4{,}000})\left(\frac{20(24)^3}{12}\right)} = 1.186'' < \Delta_{TL\,max} \Rightarrow \text{OK!}$$

Beam @ Exterior Support   @ Midspan   @ First Interior support



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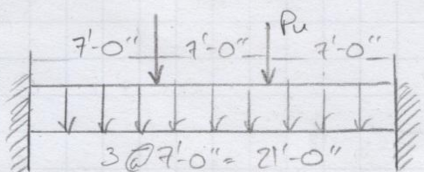
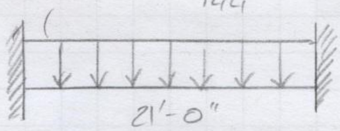
Girder Design:

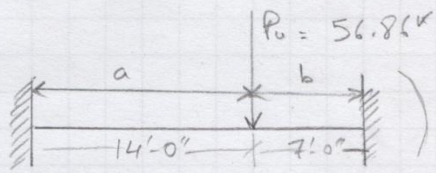
Assume $b = 20''$ to match columns and beams
and $h = 24''$ to match beams for constructibility, cost, uniformity



$P_u = 56.86^K$ (from V_u previous)

$w_{self\text{-}weight} = \frac{(24-4)(20)}{144} \times 150 = 417\ lb/ft$

= 

+ 2()

$M^- = \frac{w\ell^2}{12} = \frac{.417(21')^2}{12} = 15.32\ ft\text{-}K$

$M^- = \frac{P a^2 b}{\ell^2}$ when $a = 7$: $= 88.45'\text{-}K$; when $a = 14$: $176.9'\text{-}K$

$M^+_{@x} = \frac{w}{12}(6\ell x - \ell^2 - 6x^2)$

$= \frac{.417}{12}\left(6(21)(14') - (21)^2 - 6(14)^2\right)$

$M^+_{@14'} = 5.12\ ft\text{-}K$

$M^+_{@load} = \frac{2Pa^2b^2}{\ell^3}$ $\left[M^+_{@x}\right]$ $16.85K\text{-}ft$

$V = \frac{Pa^2}{\ell^3}(a+3b)$ 14.74^K 42.12^K

$\left[M^+ @ x = 7\right] = R_1 x - \frac{Pab^2}{\ell^2}$

$14.74(7) - \frac{56.86(14)(7)^2}{(21)^2}$

$M^+_{@x} = 14.74$

$V = \frac{w\ell}{2} = \frac{.417(21)}{2} = 4.38^K$

$V_{max} = 4.38^K + 14.74 + 42.12 = 61.24^K$

$M^+_{max} = 5.12 + 16.85 + 14.74 = 36.71'\text{-}K$

$M^-_{max} = 15.32 + 176.9 + 88.45 = 280.67\ '\text{-}K$

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Description:	@ Midspan	@ Supports
1. M_u (K-ft)	36.71'-K	280.67'-K
2. $A_s = \frac{M_u}{4d}$	$\frac{36.71}{4(21)} = 0.44 in^2$	3.34 in^2
3. A_s, chosen	(1) #9 = 1.0 in^2	(4)#9 4 in^2
4. $\rho = \frac{A_s}{b.d}$	$\frac{1.0}{20(21)} = 0.0024$	0.0095

5. b_{eff} = min | 1/4 (21')(12) = 63" ⇒ controls
20 + 16(4) = 84"
20 + 2(1/2[21(12)-20)] = 252"

6. $M_{u,TBM} = [\phi(0.85)(f'_c)(b_{eff})(h_f)(d - \frac{h_f}{2})]/12 = \frac{0.9(.85)(4)(63)(4)(4-\frac{d}{2})}{12}$

7. $d = 24 - 1.5" - \underbrace{0.5"}_{\#4 \text{ stirrup}} - \frac{1.128}{2} = 21.44$ ⇒ $M_{u,TBM}$ = 1249'-K > M_u

TREAT AS RECTANGULAR BEAM

8. $a = \frac{A_s f_y}{0.85 f'_c b}$	0.88	3.53
9. $c = a/\beta_1$	1.04	4.15
10. $\varepsilon_s = \left(\frac{d-c}{c}\right).003$	$\left(\frac{21.44-1.04}{1.04}\right).003 = 0.059$ > .005 ⇒ $\phi = 0.9$	0.0125 > 0.005 ⇒ $\phi = 0.9$

11. $A_{s,min}$ = from previous since same dimensions $A_{s,min}$ = 1.43 in^2
12. Spacing Requirements is the same

13. From previous $\phi V_n = \phi(V_c + V_s) = 72.84$

$V_u = V_{max} = 61.24^k$

⇒ $\phi V_n > V_u$ ⇒ OK!

Instead of (1) #9 use (2)#6 @ Midspan

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Deflection:

$$\Delta_{max} = \frac{PL^3}{28EI} + \frac{5w_L l^4 (12)^3}{384EI}$$

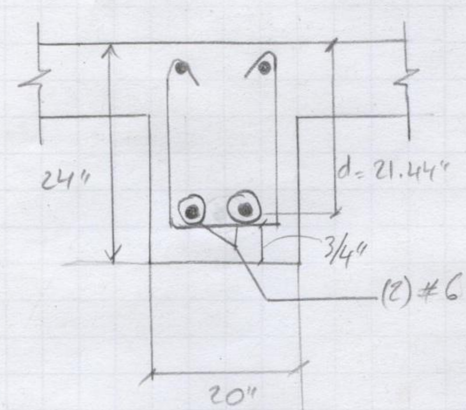
$$LL: \frac{23.06\,(21)^3\,1728}{28(57{,}000\sqrt{4{,}000})\left(\frac{20(24)^3}{12}\right)} + 0 = 0.391'' < \frac{l}{480} = 0.525''$$

$\Rightarrow$ OK!

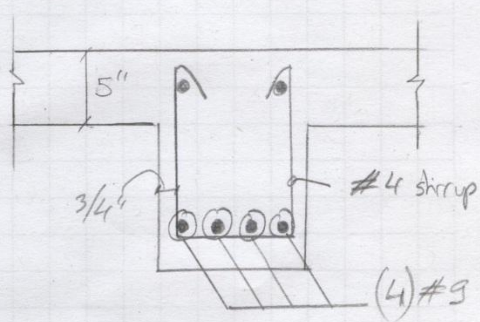
$$TL: \frac{(23.06 + 19.33)(21)^3(1728)}{28\,(57{,}000\sqrt{4{,}000}\,\left(\frac{20(24)^3}{12}\right)} + \frac{5(417)(21)^4(1728)}{384(57{,}000)\sqrt{4000}\left(\frac{20(24)^3}{12}\right)}$$

$$\Delta_{TL} = 0.718'' + 0.022 = 0.74'' < \frac{l}{240} = 1.05''$$

Girder @ Midspan



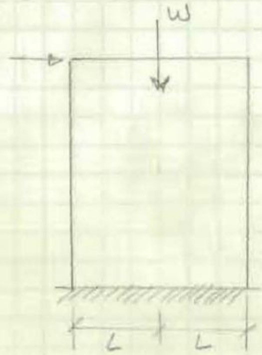
Girder @ Supports



Overturning Moment

Final Report | Overturning Moment | 1/1

Controlling Base Overturning Moment:



$M_{Resisting} = \frac{1}{2} \left| \frac{B}{L} \times W \right._{min}$

$= \frac{1}{2} \left| \frac{195}{145} \right. = 72.5\,W$

Controlling load combination:

$(0.9 - 0.2\,S_{DS})D + \rho Q_E + 1.6H$

§ 12.4.2.3 (ASCE 7-05)

① $M_{resisting} = (0.9 - 0.2\,S_{DS})\,D$

② $M_{overturning} = \rho\,Q_E$

① $M_{resisting} = (0.9 - 0.2(1.091))(20000\text{ kips})(72.5)$

$= 988{,}610\text{ ft-k}$

$M_{overturning} = (1.3)(169{,}437) = 220{,}268\text{ k-ft}$

$\frac{2}{3} M_R > M_o$

$\frac{2}{3}(988{,}610) > 220{,}268\text{ k-ft}$

$659{,}073 > 220{,}268\text{ k-ft} \rightarrow \text{OK!}$

Appendix E: Fixed Base Iteration/Modal Response

Iterations

Original design								
Period (sec)	total deflection in Y (inch)	total deflection in X (inch)	Max drift in Y (inch) between 5-6	Max drift in X(inch) between 3-4	S5= 2%= 3.48"	S3= 1%= 1.74"	S5= 2%= 3.48"	S3= 1%= 1.74"
1.841	37.74368	26.559545	5.500085	3.91366	NG	NG	NG	NG

Layout A- Added moment frames on (C,G,I 9-6)(K,9-6)(3,B-F) without base isolation										
Wall size	Beam size	Period (sec)	total deflection in Y (inch)	total deflection in X (inch)	Max drift in Y (inch) between 5-6	Max drift in X (inch) between 3-4	Y -direction		X-direction	
							S5= 2%= 3.48"	S3= 1%= 1.74"	S5= 2%= 3.48"	S3= 1%= 1.74"
12"	20x24	1.737	32.21	26.29	4.686	3.875	NG	NG	NG	NG
	20x28	1.633	28.23	22.97	4.098	3.374	NG	NG	OK	NG
	20x32	1.553	25.25	20.65	3.660	3.022	NG	NG	OK	NG
	20x36	1.489	22.97	18.93	3.324	2.762	OK	NG	OK	NG
16"	20x24	1.622	27.98	23.37	4.072	3.441	NG	NG	OK	NG
	20x28	1.533	24.84	20.65	3.603	3.034	NG	NG	OK	NG
	20x32	1.463	22.42	18.71	3.249	2.742	OK	NG	OK	NG
	20x36	1.406	20.53	17.25	2.971	2.522	OK	NG	OK	NG

Layout B - Added moment frames on (C,E,G,H,I, 9-6)(K,J,9-6)(2,3,4,B-F) without base isolation										
Wall size	Beam size	Period (sec)	total deflection in Y (inch)	total deflection in X (inch)	Max drift in Y (inch) between 5-6	Max drift in X (inch) between 3-4	Y -direction		X-direction	
							S5= 2%= 3.48"	S3= 1%= 1.74"	S5= 2%= 3.48"	S3= 1%= 1.74"
12"	20x24	1.687	29.42	26.09	4.271	3.844	NG	NG	NG	NG
	20x28	1.581	25.58	22.77	3.703	3.343	NG	NG	OK	NG
	20x32	1.501	22.78	20.45	3.290	2.992	OK	NG	OK	NG
	20x36	1.439	20.68	18.73	2.980	2.733	OK	NG	OK	NG
16"	20x24	1.582	25.82	23.20	3.744	3.416	NG	NG	OK	NG
	20x28	1.491	22.73	20.48	3.290	3.008	OK	NG	OK	NG
	20x32	1.421	20.42	18.54	2.950	2.716	OK	NG	OK	NG
	20x36	1.366	18.65	17.08	2.690	2.496	OK	NG	OK	NG
20"	20x36	1.307	17.05	15.77	2.460	2.305	OK	NG	OK	NG
24"	20x36	1.258	15.75	14.67	2.272	2.145	OK	NG	OK	NG
	20x42	1.204	14.22	13.40	2.049	1.955	OK	NG	OK	NG
	24x42	1.184	14.22	13.40	2.049	1.955	OK	NG	OK	NG
28"	20x42	1.165	13.291	12.602	1.916	1.839	OK	NG	OK	NG
32"	24x42	1.113	12.808	12.301	1.847	1.794	OK	NG	OK	NG
	24x48	1.077	11.847	11.473	1.708	1.670	OK	OK	OK	OK

Model Response

S_s=	164%	1.636
S_1=	65%	0.646

F_a=	1.0
F_v=	1.5

S_{ms}=	$Fa.S_s$=	1.6
S_{m1}=	$Fv.S_1$=	0.969

Category=	II	SDS = D
S_{DS}=	D	
S_{D1}=	D	

S_{DS}=	2/3 S_{MS}	1.091
S_{D1}=	2/3 S_{M1}	0.646

C_T=	0.02	h=	107	R=	6.5
T_L=	8	x=	0.75	I=	1

C_u=	1.4	T_a=	$C_T.h_n^x$ =	0.67	$T<T_L$
		T=	$C_u.T_a$ =	0.93	

Cs=	$S_{DS}/(R/I)$	0.16779487	≤	Cs=	$S_{D1}/(T.(R/I))$	0.1067	Cs=	0.1067
			>	Cs=	$0.5S_1/(R/I)$	0.049692308		

$$C_{vx}=\frac{w_x.h_x^K}{\sum w_i h_i^K}$$

$$F_x= C_{vx}.V$$

Seismic Forces														
Level	Story weight, w_x	Story height (ft), h_x	$w_x.h_x^K$	C_{vx}	Story force (k)	Story Shear (k)	Overturning moment (k-ft)	Bx (ft)	5% Bx	Ax	By (ft)	5% By	Ax	M_{zx} (k-ft)
8	1648	121.5	564183.7326	0.16183	383.2	383.2	46559	145	7.25	1.0	191	9.55	1.0	3660
7	3133	101.5	861718.7654	0.24718	585.3	968.5	59407	145	7.25	1.0	191	9.55	1.0	5589
6	2944	87	671221.7942	0.19254	455.9	1424.4	39663	145	7.25	1.0	191	9.55	1.0	4354
5	2893	72.5	528510.0985	0.1516	359.0	1783.4	26025	145	7.25	1.0	191	9.55	1.0	3428
4	2893	58	402933.6984	0.11558	273.7	2057.0	15873	145	7.25	1.0	191	9.55	1.0	2614
3	2893	43.5	284012.5809	0.08147	192.9	2249.9	8391	145	7.25	1.0	191	9.55	1.0	1842
2	2895	29	173634.6248	0.04981	118	2367.9	3420	145	7.25	1.0	191	9.55	1.0	1126
1	2895	14.5	74747.71396	0.02144	51	199338.2	736	146	7.3	2.0	192	9.6	2.0	975
Σ	22194	$\sum w_i h_i^K$=	3486215.295			Base Shear =	**2368 kip**			Σ M_{zy} =			Σ M_{zx} =	**22613 k-ft**
						Total Overturning Moment =	**199338 kip**							

1. For periods less than T_0, the design spectral response acceleration, S_a, shall be taken as given by Eq. 11.4-5:

$$S_a = S_{DS}\left(0.4+0.6\frac{T}{T_0}\right) \tag{11.4-5}$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken equal to S_{DS}.

3. For periods greater than T_S, and less than or equal to T_L, the design spectral response acceleration, S_a, shall be taken as given by Eq. 11.4-6:

$$S_a = \frac{S_{D1}}{T} \tag{11.4-6}$$

4. For periods greater than T_L, S_a shall be taken as given by Eq. 11.4-7:

$$S_a = \frac{S_{D1}T_L}{T^2} \tag{11.4-7}$$

where

S_{DS} = the design spectral response acceleration parameter at short periods

S_{D1} = the design spectral response acceleration parameter at 1-s period

T = the fundamental period of the structure, s

$T_0 = 0.2\frac{S_{D1}}{S_{DS}}$

$T_S = \frac{S_{D1}}{S_{DS}}$ and

Modal Information								
Mode	Period	UX%	UY%	Sa	$S_a/(R/I)$	Cm,i	$(Cm,i*UX\%)^2$	$(Cm,i*UY\%)^2$
1	1.844882	0.0848	60.7839	0.053870446	0.167795	0.05387045	2.08686E-09	0.001072206
2	1.497176	49.564	3.6154	0.066381384	0.167795	0.06638138	0.001082494	5.75977E-06
3	1.150446	23.3993	8.9318	0.086387901	0.167795	0.0863879	0.000408612	5.95366E-05
4	0.404201	2.0679	10.9419	0.245879192	0.167795	0.16779487	1.20397E-05	0.000337088
5	0.37798	10.5813	5.326	0.262936175	0.167795	0.16779487	0.000315236	7.98656E-05
6	0.252525	6.6079	2.0383	0.39356347	0.167795	0.16779487	0.000122937	1.16975E-05
							$C_{m,x}$=SQRT($\sum(C_{m,i}$*UX%$)^2$)=	0.044060424
							$C_{m,y}$=SQRT($\sum(C_{m,i}$*UY%$)^2$)=	0.039574653
							0.85Cs=	0.090686413

$$C_{m,i} = min \begin{matrix} \dfrac{S_{D1}}{T_i \dfrac{R}{I}} \\ \dfrac{S_{DS}}{\dfrac{R}{I}} \end{matrix}$$

$$V_m = W(\Sigma(C_{m,i}M\%_i)^2)^{1/2} \geq 0.85V_{ELF}$$

Modal Information									
Mode	Period	UX%	UY%	Envelope	Sa	$S_a/(R/I)$	Cm,i	$(Cm,i*UX\%)^2$	$(Cm,i*UY\%)^2$
1	1.491	0.0848	60.7839	C	0.433266	0.066656	0.066656	3.19503E-09	0.001641573
2	1.062	49.564	3.6154	C	0.608286	0.093583	0.093583	0.002151404	1.14473E-05
3	0.701156	23.3993	8.9318	C	0.921336	0.141744	0.141744	0.001100056	0.000160283
4	0.3204	2.0679	10.9419	B	1.091	0.167795	0.167795	1.20397E-05	0.000337088
5	0.3013	10.5813	5.326	B	1.091	0.167795	0.167795	0.000315236	7.98656E-05
6	0.1561	6.6079	2.0383	B	1.091	0.167795	0.167795	0.000122937	1.16975E-05
								$C_{m,x}$=SQRT($\sum(C_{m,i}$*UX%$)^2$)=	0.060841401
								$C_{m,y}$=SQRT($\sum(C_{m,i}$*UY%$)^2$)=	0.047349274
								0.85Cs=	0.090686413

Seismic Forces														
Level	Story weight, w_x	Story height (ft), h_x	$w_x \cdot h_x^k$	C_{vx}	Story force (k)	Story Shear (k)	Overturning moment (k-ft)	Bx (ft)	5% Bx	Ax	By (ft)	5% By	Ax	M_{zx} (k-ft)
8	1648	121.5	564183.7326	0.16183	326	325.7	39575	145	7.25	1.0	191	9.55	1.0	3111
7	3133	101.5	861718.7654	0.24718	497	823.2	50496	145	7.25	1.0	191	9.55	1.0	4751
6	2944	87	671221.7942	0.19254	388	1210.7	33714	145	7.25	1.0	191	9.55	1.0	3701
5	2893	72.5	528510.0985	0.1516	305	1515.8	22121	145	7.25	1.0	191	9.55	1.0	2914
4	2893	58	402933.6984	0.11558	233	1748.5	13492	145	7.25	1.0	191	9.55	1.0	2222
3	2893	43.5	284012.5809	0.08147	164	1912.4	7133	145	7.25	1.0	191	9.55	1.0	1566
2	2895	29	173634.6248	0.04981	100	2012.7	2907	145	7.25	1.0	191	9.55	1.0	957
1	2895	14.5	74747.71396	0.02144	43	169437.4	626	146	7.3	1.0	192	9.6	1.0	414
Σ	22194	$\Sigma w_i h_i^k$=	3486215.295			Base Shear =	2013 kip			ΣM_{zy} =			ΣM_{zx}=	19635 k-ft
						Total Overturning Moment =	169437 kip-ft							

Appendix F: Base Isolation Design

CA S-3 with base isolation - X Direction - Summary for Normalizing Response Accelerations					
Earthquake Name / Recording Station	Year	M	PGA (g)	PGV (cm/sec)	Scale factor from PEER
Imperial Valley-06/ El Centro #6	1979	6.5	0.4417	111.8402	1.2484
Imperial Valley-06/ El Centro #7	1979	6.5	0.4624	108.7935	1.3587
Northridge-01 / Rinaldi Receiving Station	1994	6.7	0.8698	167.051	1.0038
Northridge-01 / Sylmar - Olive View	1994	6.7	0.7326	122.7694	1.1408
Chi Chi, Taiwan / TCU065	1999	7.6	0.8225	127.8078	0.8084
Duzce, Turkey / Duzce	1999	7.1	0.5193	79.455	1.042
Irpinia, Italy-01 / Sturno	1980	6.9	0.3056	45.4864	1.7069

CA S-3 with base isolation - Y Direction - Summary for Normalizing Response Accelerations					
Earthquake Name / Recording Station	Year	M	PGA (g)	PGV (cm/sec)	scale factor from PEER
Imperial Valley-06/ Bonds Corner	1979	6.5	0.7639	44.2457	1.3953
Imperial Valley-06/ Chihuahua	1979	6.5	0.2843	30.4074	2.6337
Northridge-01 / LA - Sepulveda VA	1994	6.7	0.7312	69.979	1.179
Northridge-01 / Northridge - Saticoy	1994	6.7	0.4133	53.1713	1.498
Chi Chi, Taiwan / TCU067	1999	7.6	0.5583	91.7142	1.1668
Cape Mendocino / Cape Mendocino	1992	7	1.4314	118.3109	1.1184
Nahanni, Canada / Site 1	1989	6.9	1.178	43.826	1.7579

S_S =	1.636	
S_1 =	0.646	
S_{M1} =	0.49	
S_{D1} =	0.646	
R =	6.5	
W =	20,000	kips
b =	145	ft
Variation =	10%	(Variation in stiffness from the mean stiffness values of the isolators is considered

d =	195	ft
e =	20.8	ft (with 5% accidental torsion)
g =	386.4	in./sec^2
$T_{str.}$ =	1.491	
T_D =	7.455	sec.
T_M =	8.6	sec.
Damping =	15%	

Effective Period of Design Displacement:		
T_D =	$2\pi\sqrt{\frac{W}{k_{D,MIN}g}}$	
$k_{D,MIN}$ =	36.8	k/in.

Effective Period at Maximum Displacement:		
T_M =	$2\pi\sqrt{\frac{W}{k_{M,MIN}g}}$	
$k_{M,MIN}$ =	27.8	k/in.

Design Effective Damping in the System:	Maximum Effective Damping in the System:
$\beta_D = \frac{1}{2\pi}\left[\frac{total\ area\ of\ hysteresis\ loop}{K_{D,MAX}\ D^2}\right]$	$\beta_M = \frac{1}{2\pi}\left[\frac{total\ area\ of\ hysteresis\ loop}{K_{M,MAX}\ D^2}\right]$
$k_{D,MAX}$ = 44.9 k/in.	$k_{D,MAX}$ = 34.0 k/in.

B_D =	1.35	(Table 17.5-1 Damping Coefficient)
B_M =	1.35	*Assumed same level of damping assigned to both directions

Design Displacement:	Maximum Displacement:
$D_D = \frac{gS_{D1}T_D}{4\pi^2 B_D}$	$D_M = \frac{gS_{M1}T_M}{4\pi^2 B_M}$
D_D = 34.92 in.	D_M = 30.46 in.

Total Displacement:		
$D_{TD} = D_D\left[1 + y\frac{12e}{b^2+d^2}\right]$	49.3	in.
$D_{TM} = D_M\left[1 + y\frac{12e}{b^2+d^2}\right]$	43.0	in.

Minimum Lateral Forces: (Isolation System and Structural Elements below the Isolation System)

$V_b = k_{D,MAX}D_D$ 1569 kips

Structure Elements Above the Isolation System:

$V_S = \frac{k_{D,MAX}D_D}{R_I}$ 784.5 kips

R_I = (3/8)R = 2.438 $1.0 \le R_I \le 2.0$ ∴ 2.0

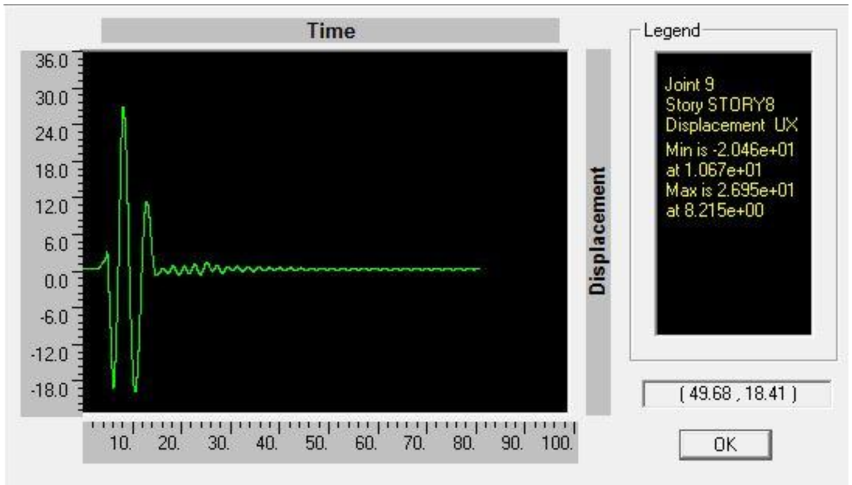


Figure 67 - **Max displacement for Chi Chi TCU065 in the X-direction.**

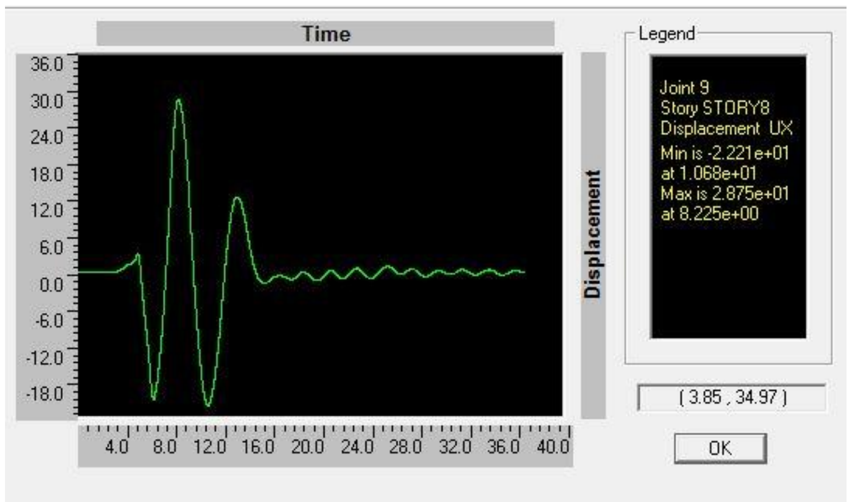


Figure 68 - **Max Displacement for El-Centro in the X-direction.**

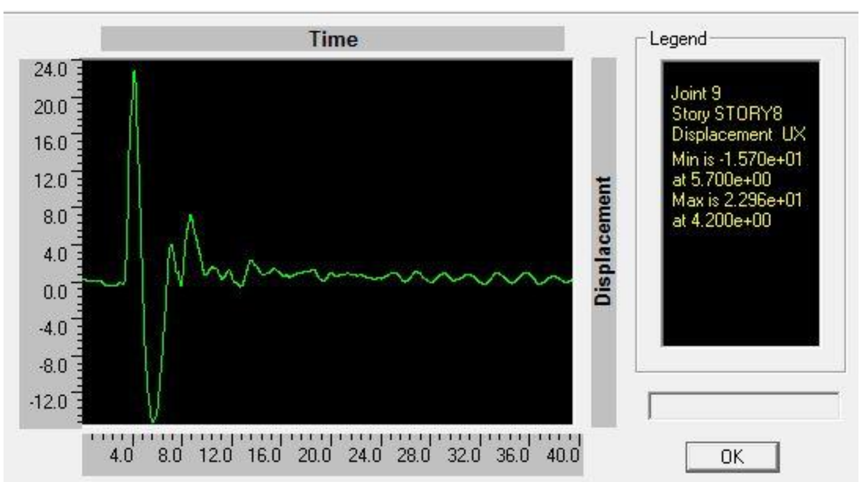


Figure 69 - **Max Displacement for Olive View in the X-direction.**

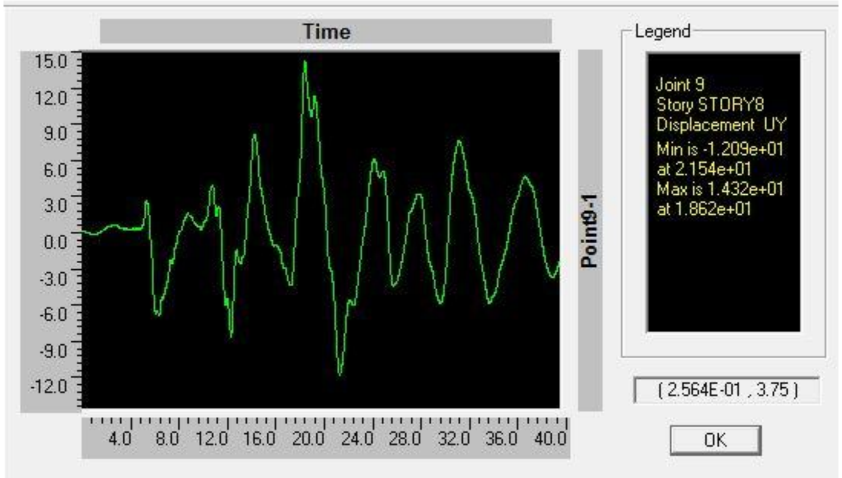


Figure 70 - **Max Displacement for Chihuahua in the Y-direction.**

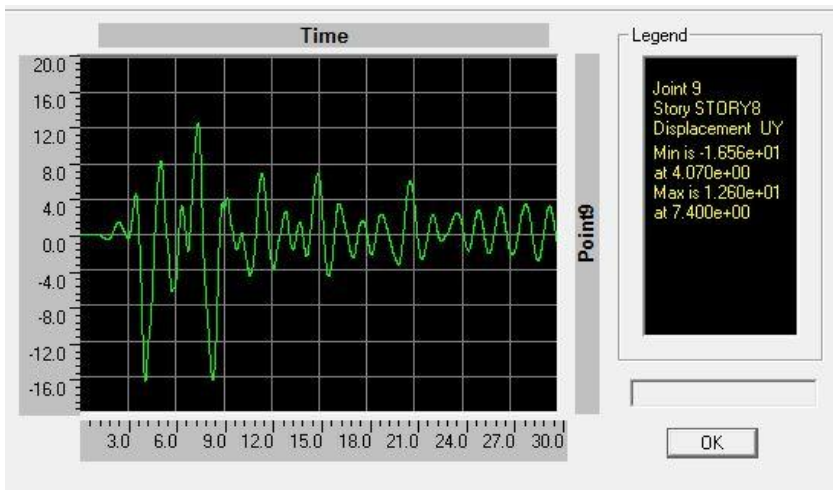


Figure 71 - **Max Displacement for Northridge St. in the Y-direction.**

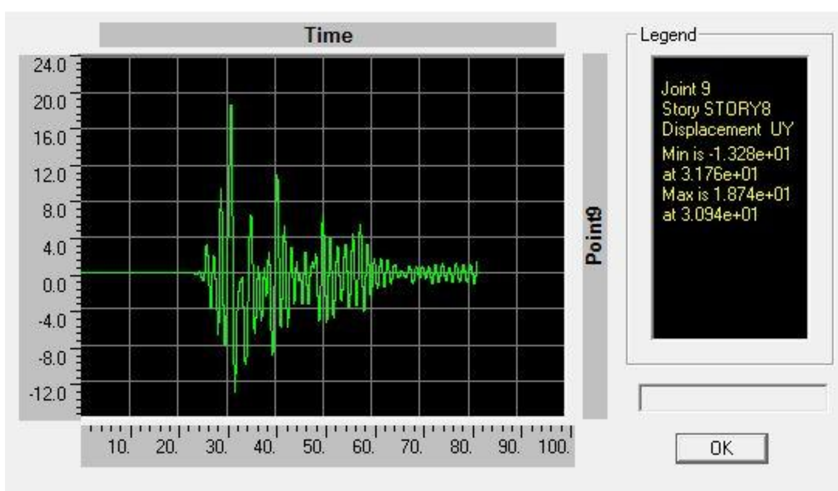


Figure 72 - **Max Displacement for ChiChi,TCU067 in the Y-direction.**

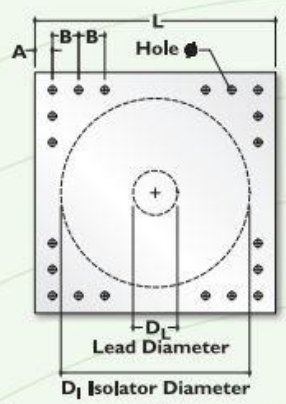
DYNAMIC ISOLATION SYSTEMS

Section 3: Engineering

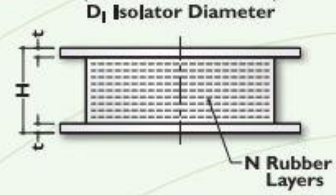
Isolator Engineering Properties

Isolator Properties: U.S. Units

DEVICE SIZE				MOUNTING PLATE DIMENSIONS					
Isolator Diameter, D_I (in)	Isolator Height, H (in)	Number of Rubber Layers, N	Lead Diameter D_L (in)	L (in)	t (in)	Hole Qty.	Hole Ø (in)	A (in)	B (in)
12.0	5-11	4-14	0-4	14	1	4	1 1/16	2	-
14.0	6-12	5-16	0-4	16	1	4	1 1/16	2	-
16.0	7-13	6-20	0-5	18	1	4	1 1/16	2	-
18.0	7-14	6-20	0-5	20	1	4	1 1/16	2	-
20.5	8-15	8-24	0-7	22.5	1	8	1 1/16	2	2
22.5	8-15	8-24	0-7	24.5	1	8	1 1/16	2	2
25.5	8-15	8-24	0-8	27.5	1.25	8	1 1/16	2	2
27.5	8-17	8-30	0-8	29.5	1.25	8	1 5/16	2.5	3
29.5	9-18	8-30	0-9	31.5	1.25	8	1 5/16	2.5	3
31.5	9-20	8-33	0-9	33.5	1.25	8	1 5/16	2.5	3
33.5	9-21	8-35	0-10	35.5	1.5	12	1 5/16	2.5	3.75
35.5	10-22	9-37	0-10	37.5	1.5	12	1 5/16	2.5	3.75
37.5	10-23	10-40	0-11	39.5	1.5	12	1 5/16	2.5	3.75
39.5	11-25	11-40	0-11	41.5	1.5	12	1 9/16	3	4.5
41.5	12-26	12-45	0-12	43.5	1.75	12	1 9/16	3	4.5
45.5	13-30	14-45	0-13	47.5	1.75	12	1 9/16	3	4.5
49.5	14-30	16-45	0-14	52.5	1.75	16	1 9/16	3	4.5
53.5	16-30	18-45	0-15	56.5	2	16	1 9/16	3	4.5
57.1	17-30	20-45	0-16	60	2	20	1 9/16	3	4.5
61.0	18-30	22-45	0-16	64	2	20	1 9/16	3	4.5



Isolator Diameter, D_I (in)	DESIGN PROPERTIES			Maximum Displacement, D_{max} (in)	Axial Load Capacity, P_{max} (kips)
	Yielded Stiffness, K_d (k/in)	Characteristic Strength, Q_d (kips)	Compression Stiffness, K_v (k/in)		
12.0	1-5	0-15	>250	6	100
14.0	1-7	0-15	>500	6	150
16.0	2-9	0-25	>500	8	200
18.0	2-11	0-25	>500	10	250
20.5	2-13	0-40	>1,000	12	300
22.5	3-16	0-40	>3,000	14	400
25.5	3-20	0-50	>4,000	16	600
27.5	3-24	0-50	>4,500	18	700
29.5	4-27	0-60	>5,000	18	800
31.5	4-30	0-60	>6,000	20	900
33.5	4-35	0-80	>7,000	22	1,100
35.5	4-35	0-80	>8,000	22	1,300
37.5	4-35	0-110	>10,000	24	1,500
39.5	5-36	0-110	>11,000	26	1,700
41.5	5-36	0-130	>12,000	28	1,900
45.5	6-37	0-150	>16,000	30	3,100
49.5	7-38	0-170	>21,000	32	4,600
53.5	8-40	0-200	>29,000	34	6,200
57.1	9-41	0-230	>30,000	36	7,500
61.0	10-42	0-230	>37,000	36	9,000



(1) The axial load capacities correspond to maxiumum displacements based on design limits of 250% rubber shear strain or 2/3 the isolator diameter. An isolator's actual displacement and load capacity are dependent on the rubber modulus and number of rubber layers.

(2) Rubber Shear Moduli (G) are available from 55 psi to 100 psi.

(3) For analytical bilinear modeling of the Elastic Stiffness use K_e=10*K_d.

Table 1								
Direction of Earthquake	Earthquake	Station	Scale Factor	Magnitude	Peak time in X (sec)	Peak time in Y (sec)	Max Displacement (inch)	
							X	Y
X-Direction	Imperial Valley	El centro 7	525	6.5	5.48	11.27	16.38	1.88
	Northridge-01	Sylmar - Olive View	441	6.7	4.82	14.60	21.22	1.76
	Chi Chi, Taiwan	TCU065	312	7.6	5.42	12.37	9.20	1.50
Y-Direction	Imperial Valley	Chihuahua	1018	6.5	32.41	14.91	1.51	9.23
	Northridge-01	Northridge - Saticoy	579	6.7	7.31	4.07	1.22	16.56
	Chi Chi, Taiwan	TCU067	451	7.6	44.27	30.94	1.57	18.74

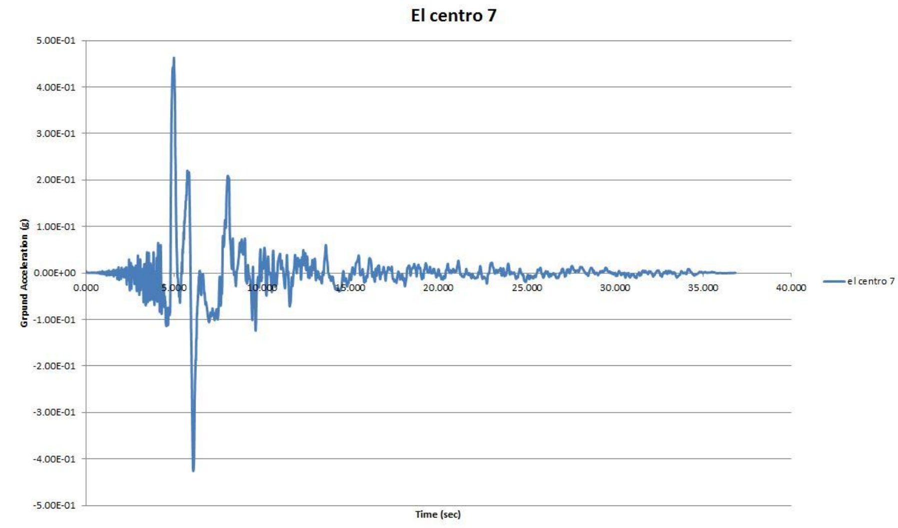
Direction of Earthquake	Earthquake	Station	Scale Factor	Magnitude	Max interstory drift		Max interstory drift location	S5= 2%= 3.48"		S3= 1%= 1.74"	
					X	Y		X	Y	X	Y
X-Direction	Imperial Valley	El centro 7	525	6.5	1.458	0.334	Story 1-Story 2	OK	OK	OK	OK
	Northridge-01	Sylmar - Olive View	441	6.7	1.729	0.789	Story 1-Story 2	OK	OK	OK	OK
	Chi Chi, Taiwan	TCU065	312	7.6	1.032	0.277	Story 1-Story 2	OK	OK	OK	OK
Y-Direction	Imperial Valley	Chihuahua	1018	6.5	0.164	0.734	Story 1-Story 2	OK	OK	OK	OK
	Northridge-01	Northridge - Saticoy	579	6.7	0.161	1.321	Story 1-Story 2	OK	OK	OK	OK
	Chi Chi, Taiwan	TCU067	451	7.6	0.177	1.493	Story 1-Story 2	OK	OK	OK	OK

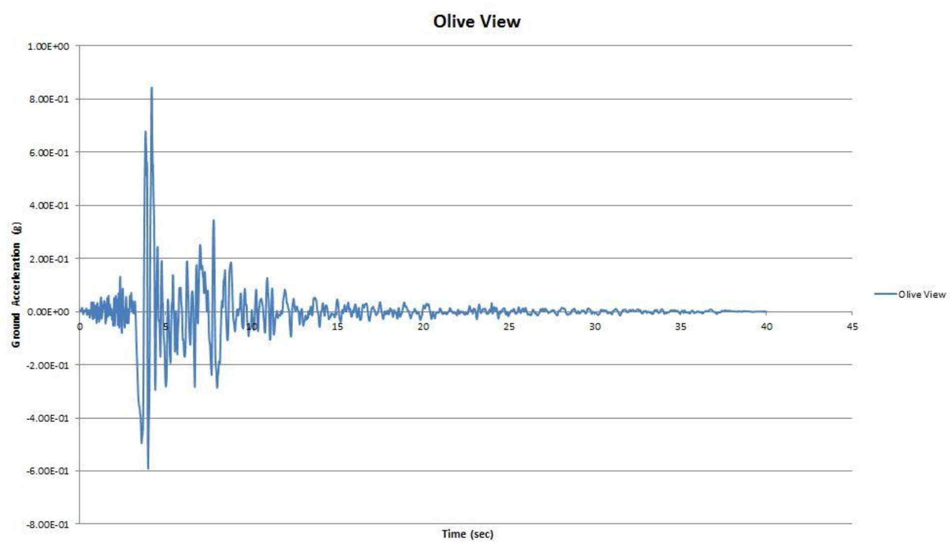
Direction of Earthquake	Earthquake	Station	Scale Factor	Magnitude	base displacement (inch)	Max for Isolator	base δ < max
X-Direction	Imperial Valley	El centro 7	525	6.5	11.05	24	OK
	Northridge-01	Sylmar - Olive View	441	6.7	14.44	24	OK
	Chi Chi, Taiwan	TCU065	312	7.6	6.14	24	OK
Y-Direction	Imperial Valley	Chihuahua	1018	6.5	6.36	24	OK
	Northridge-01	Northridge - Saticoy	579	6.7	11.39	24	OK
	Chi Chi, Taiwan	TCU067	451	7.6	12.92	24	OK

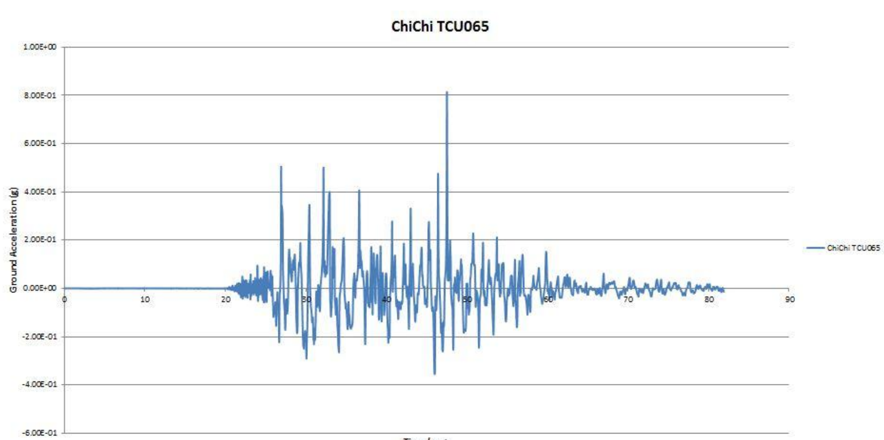
Isolator Dimensions	
DI (in)	37.5
H (in)	23
N	40
DL (in)	11
L (in)	39.5
t (in)	1.5
Hole Qty	12
Hole D (in)	1 5/16
A (in)	2.5
B (in)	3.75

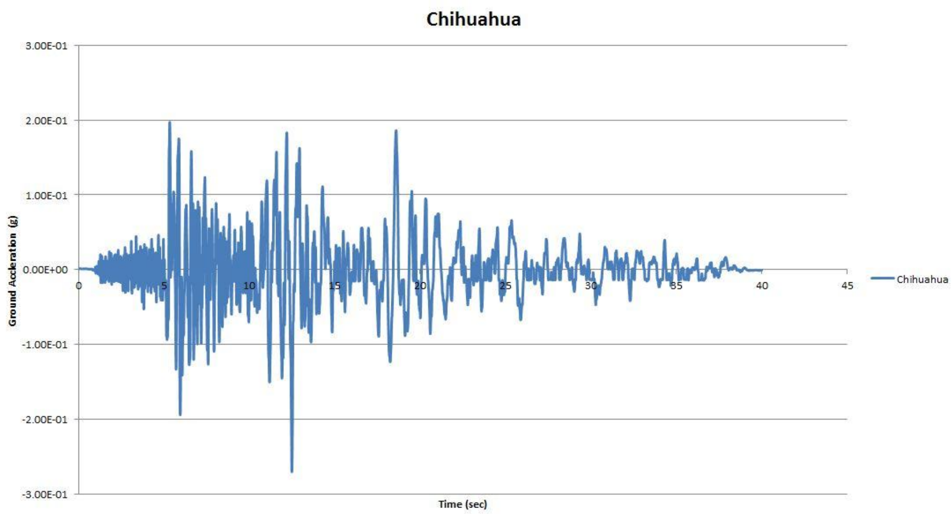
Isolator Properties	
Linear Properties	
Effective Stiffness	4
Effective Damping	0.15
Nonlinear Properties	
Stiffness	40
Yield Strength	110
Post Yield Stiffness Ratio	0.2

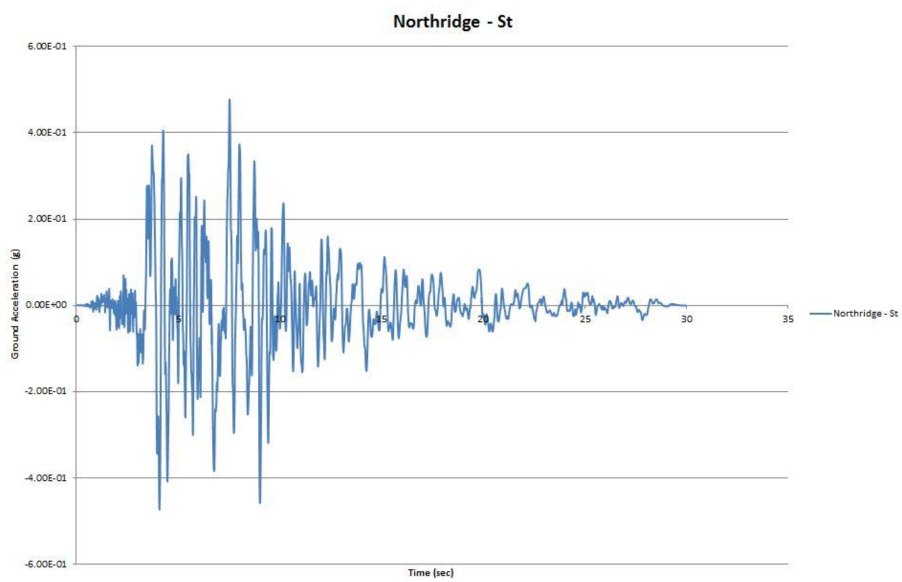
Appendix G: Earthquake Scaling for Time History with nonlinear isolator properties

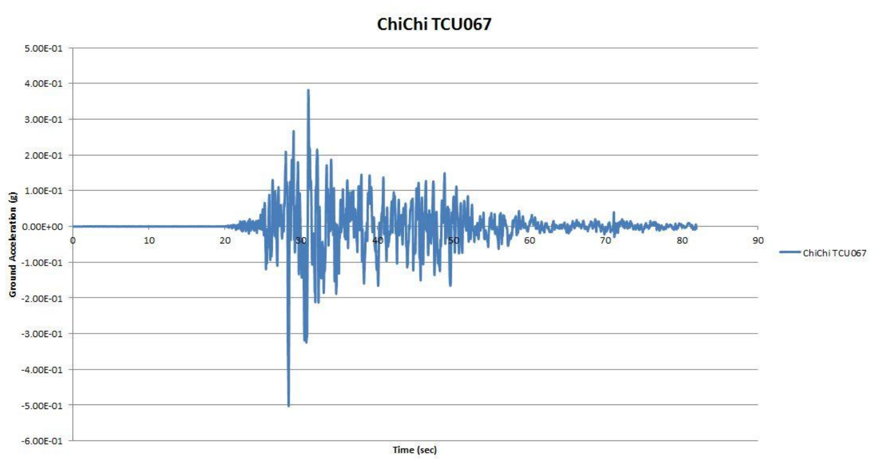


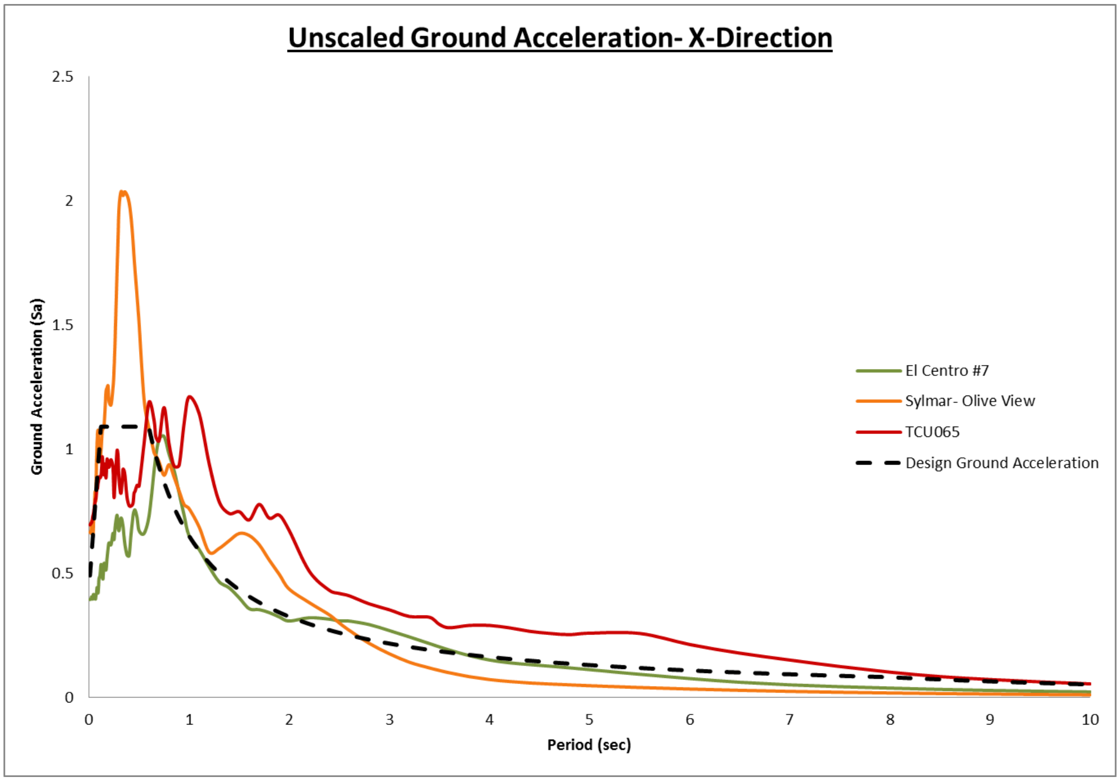


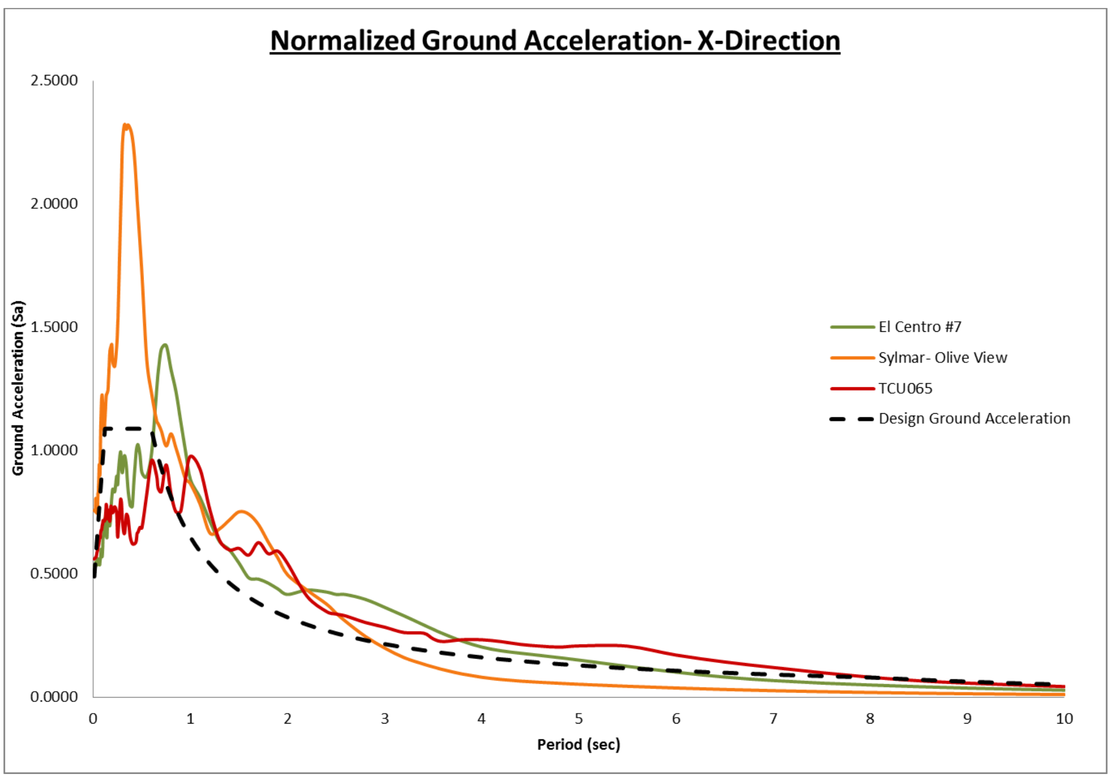


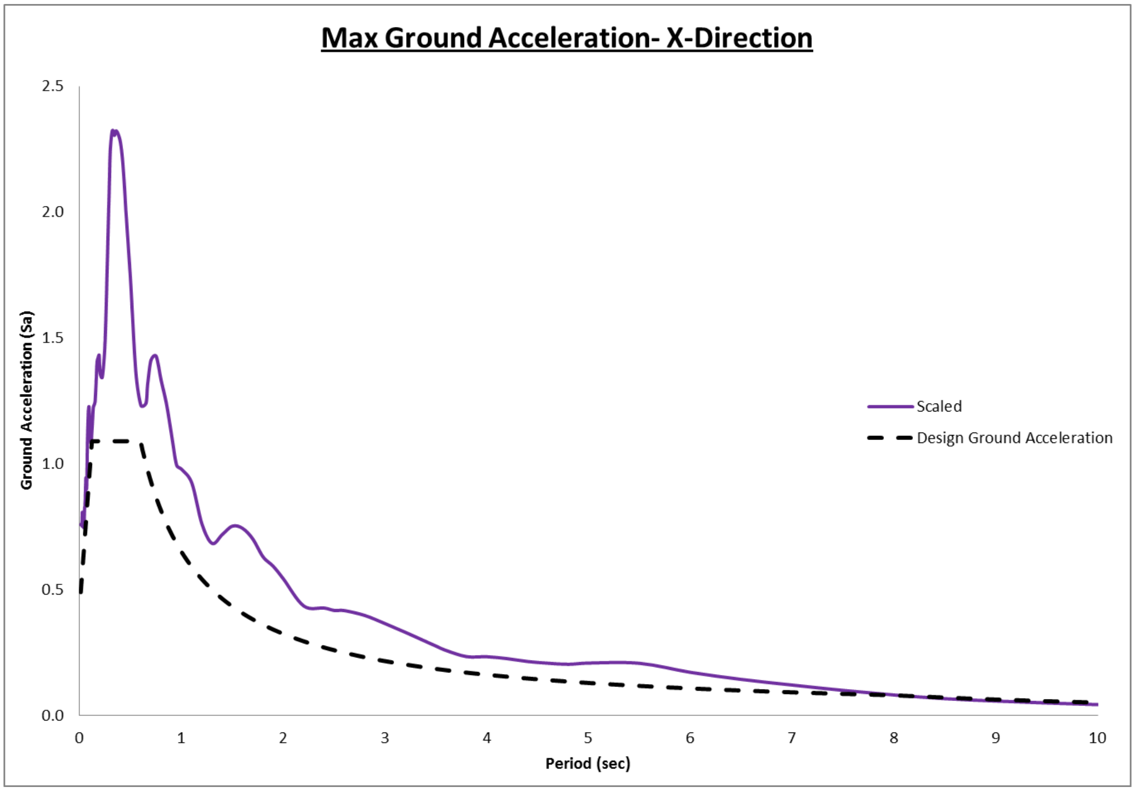

Chihuahua
Ground Acceleration (g)
Time (sec)
Chihuahua

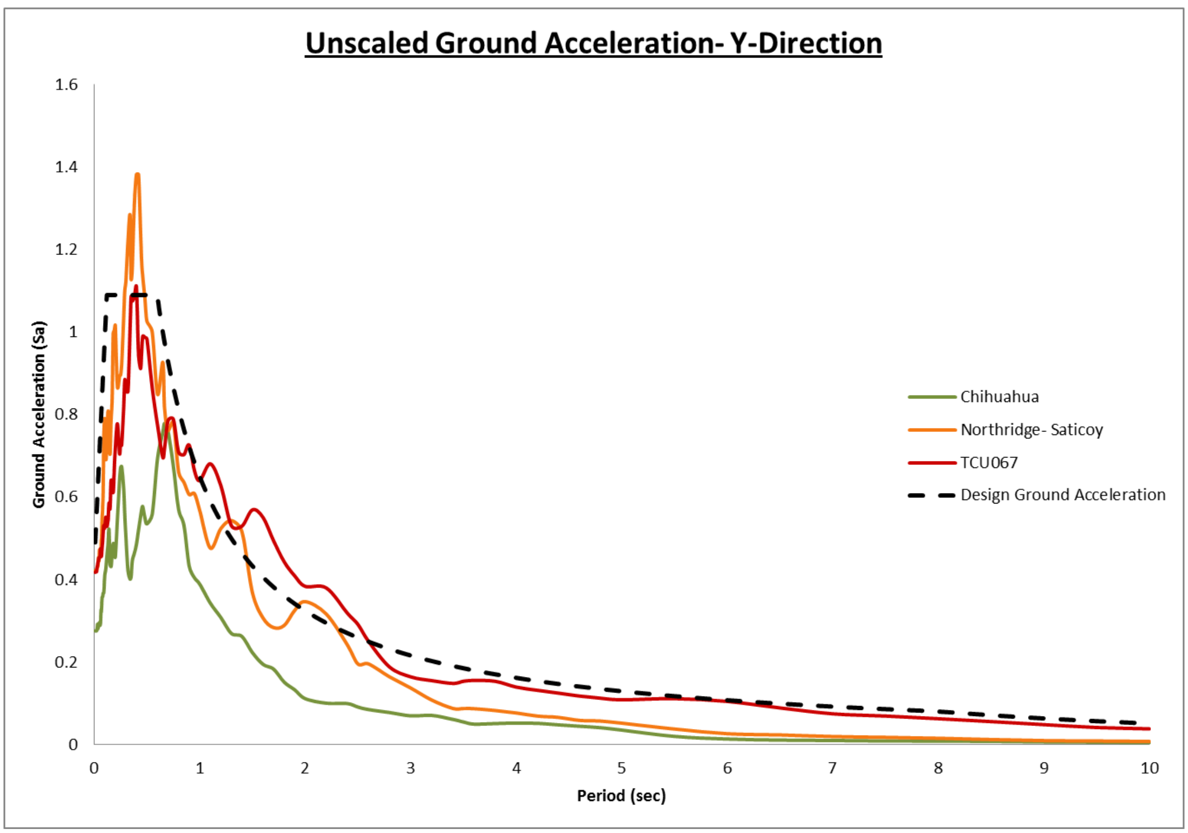

Northridge - St
Ground Acceleration (g)
Time (sec)
Northridge - St

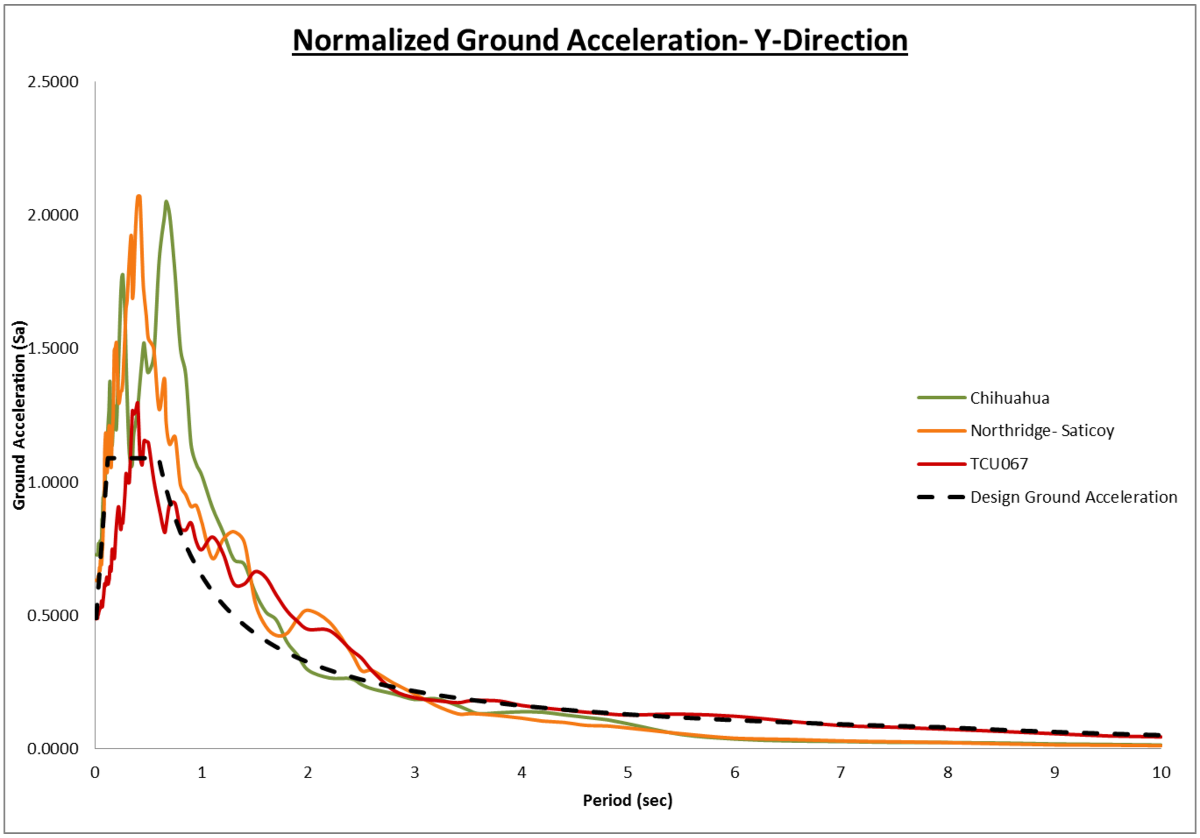

ChiChi TCU067
Ground Acceleration (g)
Time (sec)
ChiChi TCU067

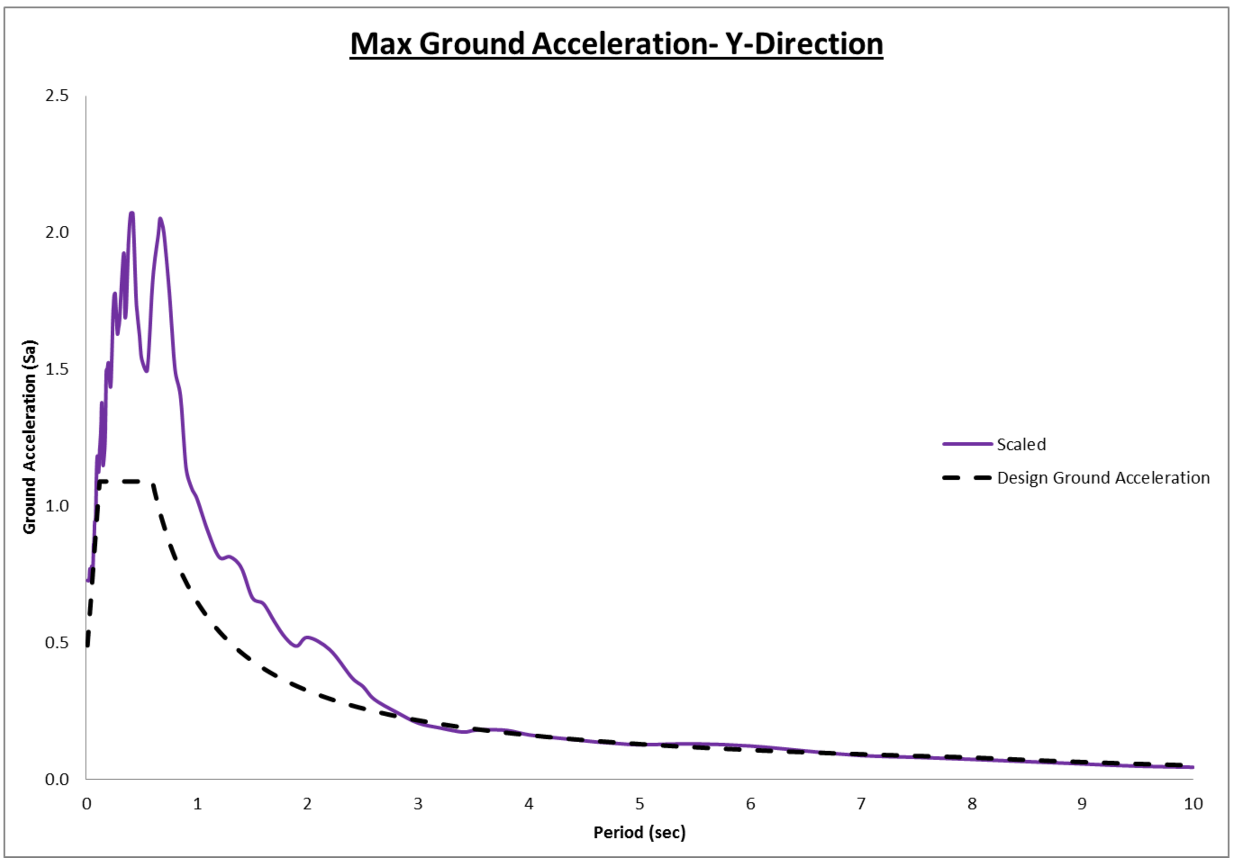

Unscaled Ground Acceleration- X-Direction
Ground Acceleration (Sa)
Period (sec)
El Centro #7
Sylmar- Olive View
TCU065
Design Ground Acceleration

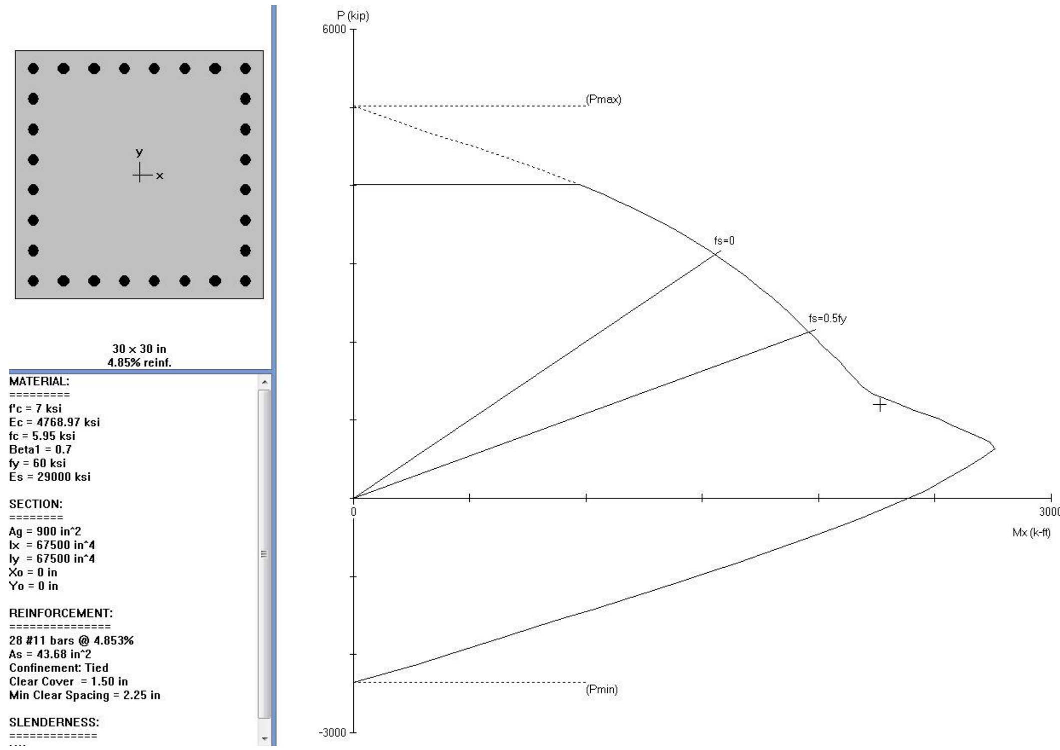

Normalized Ground Acceleration- X-Direction
Ground Acceleration (Sa)
Period (sec)
El Centro #7
Sylmar- Olive View
TCU065
Design Ground Acceleration


Max Ground Acceleration- X-Direction
Ground Acceleration (Sa)
Period (sec)
Scaled
Design Ground Acceleration
2.5
2.0
1.5
1.0
0.5
0.0
0 1 2 3 4 5 6 7 8 9 10


Unscaled Ground Acceleration- Y-Direction
Ground Acceleration (Sa)
Period (sec)
Chihuahua
Northridge- Saticoy
TCU067
Design Ground Acceleration
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0 1 2 3 4 5 6 7 8 9 10


Normalized Ground Acceleration- Y-Direction
2.5000
2.0000
1.5000
1.0000
0.5000
0.0000
Ground Acceleration (Sa)
0
1
2
3
4
5
6
7
8
9
10
Period (sec)
Chihuahua
Northridge- Saticoy
TCU067
Design Ground Acceleration


Max Ground Acceleration- Y-Direction
2.5
2.0
1.5
1.0
0.5
0.0
Ground Acceleration (Sa)
0
1
2
3
4
5
6
7
8
9
10
Period (sec)
Scaled
Design Ground Acceleration



Appendix H: Construction Management Breadth

Area take offs

Concrete	Slab	0.42	ft	x	13,240	sq.ft				=	5517	ft3	204	cu. yards
	Beam B1	31	ft	x	2.5555	sq.ft	x	26	Beam B1	=	2060	ft3	76	cu. yards
	Beam B2	31	ft	x	4.6667	sq.ft	x	35	Beam B2	=	5063	ft3	188	cu. yards
	Girders	21	ft	x	4.6667	sq.ft	x	26	Girders	=	2548	ft3	94	cu. yards
	Columns	14.5	ft	x	4	sq.ft	x	43	Columns	=	2494	ft3	92	cu. yards
	Walls	14.5	ft	x	192.185	sq.ft				=	2787	ft3	103	cu. yards
											20468	ft3	758	cu. yards

Walls	length (ft)				Thickness of 12"	
1	25			=	33.25	sq.ft
2	16.25	x	2	=	43.225	sq.ft
3	8			=	10.64	sq.ft
4	11.25	x	2	=	29.925	sq.ft
5	9.75	x	2	=	25.935	sq.ft
6	16			=	21.28	sq.ft
7	14			=	18.62	sq.ft
8	7			=	9.31	sq.ft
					192.185	sq.ft

Formwork	Slab	21.00	ft	x	31	ft				x	20	bays	=	13020	sq.ft
	Beam B1	31	ft	x	5.50	ft of formwork				x	35	Beam B1	=	5968	sq.ft
	Beam B2	31	ft	x	5.50	ft of formwork				x	26	Beam B2	=	4433	sq.ft
	Girders	21	ft	x	5.50	ft of formwork				x	26	Girders	=	3003	sq.ft
	Columns	14.5	ft	x	2.00	width (ft)	x	4	faces	x	43	columns	=	4988	sq.ft
	Walls	14.5	ft	x	20	length (ft)	x	2	faces	x	11	walls	=	6804	sq.ft
			+	x	1.33	Thikness (ft)	x	2	faces						

Typical floor 4 through 8

Slab:	No. 4 @ 12" for top and bottom	1.34	lb/ft	x	31.0	ft	x	21	bars	=	870	lb per bay
	No.4 @ 18" for tansverse steel	0.67	lb/ft	x	21.0	ft	x	21	bars	=	295	lb per bay
											1164	
B1:	Exterior L= 31'											
	2 #9 @exterior support for l/3	6.80	lb/ft	x	10.3	=	70					
	4 #9 @interior support for l/3	13.60	lb/ft	x	10.3	=	141					
	3 #9 @Midspan for l/2	10.20	lb/ft	x	15.5	=	158					
							369	lb per beam				
	Interior L=31'			x								
	3#9 @supports for l/3	10.20	lb/ft	x	10.3	=	105					
	3#9 @Midspan for l/2	10.20	lb/ft	x	15.5	=	158					
							264	lb per beam				
B2:	same as B1											
G1:	L=21'											
	4#9 @supports for l/3	13.60	lb/ft	x	21.0	=	286					
	2#6 @Midspan for l/2	3.00	lb/ft	x	21.0	=	63					
							349	lb per beam				

Results per floor

Slab	1164	lb per bay	x	20	bays	=	23286			11.64	tons	
B1 ext	369	lb per beam	x	25	beams	=	9223			4.61	tons	5.93
B1 int	264	lb per beam	x	10	beams	=	2635			1.32	tons	
B2 ext	369	lb per beam	x	16	beams	=	5902			2.95	tons	4.27
B2 int	264	lb per beam	x	10	beams	=	2635			1.32	tons	
G1	349	lb per beam	x	25	beams	=	8717			4.36	tons	
							52,398	lbs	or	26.20	tons	

Column	8	12 #9	40.8	lb/ft	x	14.5	ft	=	592	lbs
	7	12 #9	40.8	lb/ft	x	14.5	ft	=	592	lbs
	6	12 #9	40.8	lb/ft	x	14.5	ft	=	592	lbs
	5	16 #9	54.4	lb/ft	x	14.5	ft	=	789	lbs
	4	12 #11	63.8	lb/ft	x	14.5	ft	=	924	lbs
	3	12 #11	63.8	lb/ft	x	14.5	ft	=	924	lbs
	2	20 #11	106.3	lb/ft	x	14.5	ft	=	1541	lbs
	1	16 #11	85.0	lb/ft	x	14.5	ft	=	1233	lbs
	Base	28 #11	148.8	lb/ft	x	14.5	ft	=	2157	lbs

1/2 bottom floor + 1/2 upper floor

8	296	+	0	=	296	lbs	0.15	tons
7	296	+	295.8	=	592	lbs	0.30	tons
6	296	+	295.8	=	592	lbs	0.30	tons
5	394	+	295.8	=	690	lbs	0.35	tons
4	462	+	394.4	=	857	lbs	0.43	tons
3	462	+	462.231	=	924	lbs	0.46	tons
2	770	+	462.231	=	1233	lbs	0.62	tons
1	616	+	770.385	=	1387	lbs	0.69	tons

Walls per floor

	length (ft)	# of bars												
1	25	25	x	2	curtains	x	1.502	lb/ft	x	14.5	ft	=	1089.0	
2	16.25	16	x	2	curtains	x	1.502	lb/ft	x	14.5	ft	=	696.9	
3	8	8	x	2	curtains	x	1.502	lb/ft	x	14.5	ft	=	348.5	
4	11.25	11	x	2	curtains	x	1.502	lb/ft	x	14.5	ft	=	479.1	
5	9.75	10	x	2	curtains	x	1.502	lb/ft	x	14.5	ft	=	435.6	
6	16	16	x	2	curtains	x	1.502	lb/ft	x	14.5	ft	=	696.9	
7	14	14	x	2	curtains	x	1.502	lb/ft	x	14.5	ft	=	609.8	
8	7	7	x	2	curtains	x	1.502	lb/ft	x	14.5	ft	=	304.9	
													4660.7	lbs
												or	2.33	tons

Weight of Reinforcements in tons				
Story	Floor+beams	Columns	Walls	Total
8	26.20	0.15	2.33	28.68
7	26.20	0.30	2.33	28.83
6	26.20	0.30	2.33	28.83
5	26.20	0.35	2.33	28.87
4	26.20	0.43	2.33	28.96
3	26.20	0.46	2.33	28.99
2	28.82	0.62	2.56	32.00
1	30.13	0.69	2.68	33.50

Assume a 10% increase for additional reinforcement (yellow)
Assume a 15% increase for additional reinforcement (orange)

Sample Cost Calculations

Concrete	Slab	Material	$202.00	per cu.yds	x	204	cu. yds	=	$41,208	
		Labor	$41.74	per cu.yds	x	204	cu. yds	=	$8,515	
	Beams	Material	$202.00	per cu.yds	x	198	cu. yds	=	$39,996	
		Labor	$35.89	per cu.yds	x	198	cu. yds	=	$7,106	
	Girders	Material	$202.00	per cu.yds	x	65	cu. yds	=	$13,130	
		Labor	$35.89	per cu.yds	x	65	cu. yds	=	$2,333	
	Columns	Material	$202.00	per cu.yds	x	78	cu. yds	=	$15,756	
		Labor	$23.09	per cu.yds	x	78	cu. yds	=	$1,801	
	Walls	Material	$202.00	per cu.yds	x	78	cu. yds	=	$15,756	
		Labor	$29.34	per cu.yds	x	78	cu. yds	=	$2,289	
									∑=	$147,890

Formwork	Slab	Material	$2.92	per sq.ft	x	13020	sq. ft	=	$38,018	
		Labor	$4.12	per sq.ft	x	13020	sq. ft	=	$53,642	
	Beams	Material	$0.66	per sq.ft	x	10039	sq. ft	=	$6,626	
		Labor	$5.20	per sq.ft	x	10039	sq. ft	=	$52,203	
	Girders	Material	$0.66	per sq.ft	x	3003	sq. ft	=	$1,982	
		Labor	$5.20	per sq.ft	x	3003	sq. ft	=	$15,616	
	Columns	Material	$0.60	per sq.ft	x	4572	sq. ft	=	$2,743	
		Labor	$5.35	per sq.ft	x	4572	sq. ft	=	$24,460	
	Walls	Material	$0.60	per sq.ft	x	6804	sq. ft	=	$4,082	
		Labor	$5.20	per sq.ft	x	6804	sq. ft	=	$35,381	
									∑=	$234,754

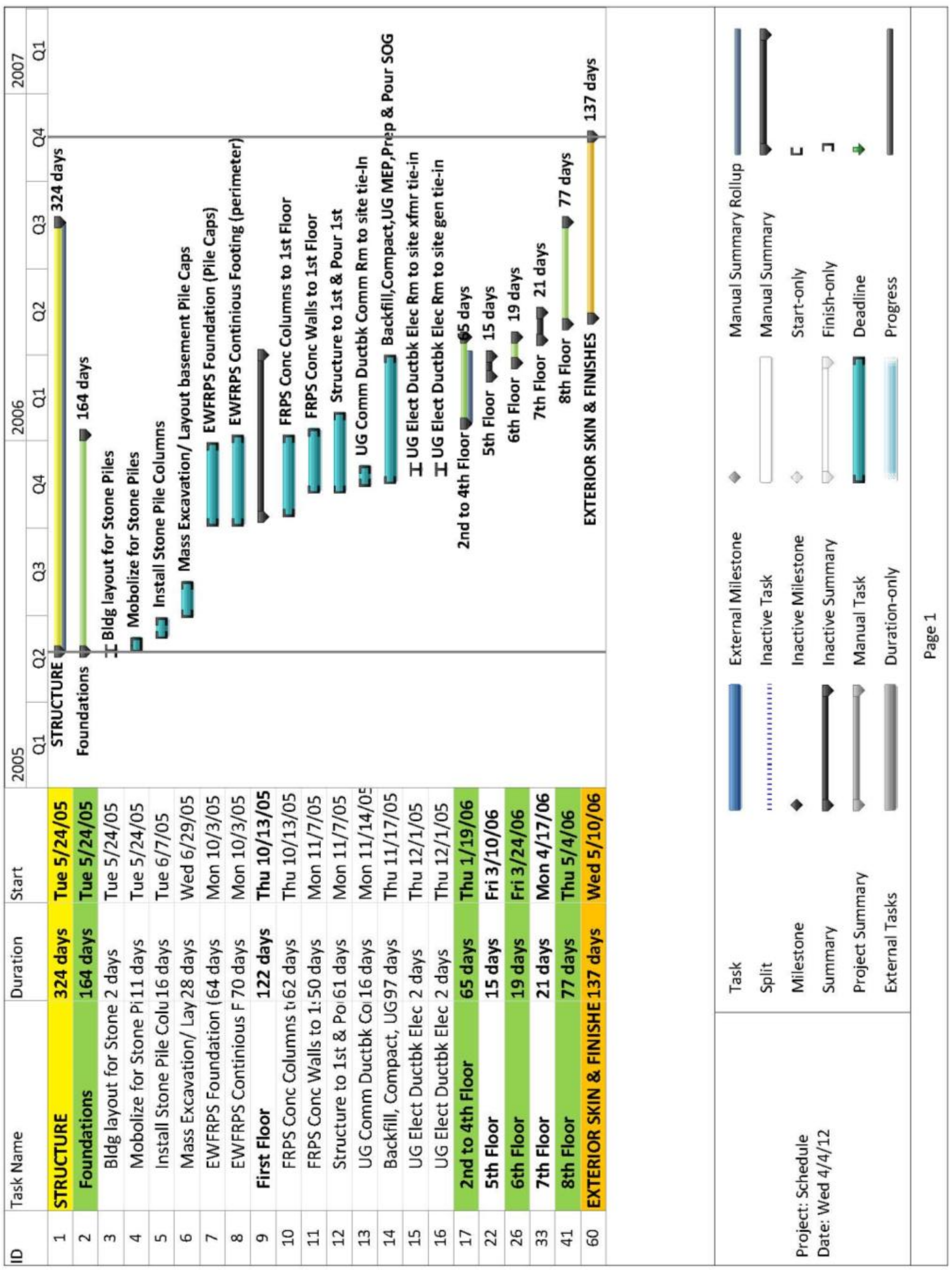
Steel	slab	Material	$1,050.00	per tons	x	13.386	tons	=	$14,055	
		Labor	$540.00	per tons	x	13.386	tons	=	$7,228	
	beams + g	Material	$980.00	per tons	x	11.73	tons	=	$11,495	
		Labor	$980.00	per tons	x	11.73	tons	=	$11,495	
	columns	Material	$980.00	per tons	x	0.69	tons	=	$676	
		Labor	$685.00	per tons	x	0.69	tons	=	$473	
	walls	Material	$930.00	per tons	x	2.68	tons	=	$2,492	
		Labor	$525.00	per tons	x	2.68	tons	=	$1,407	
									∑=	$49,323

Isolator Costs:

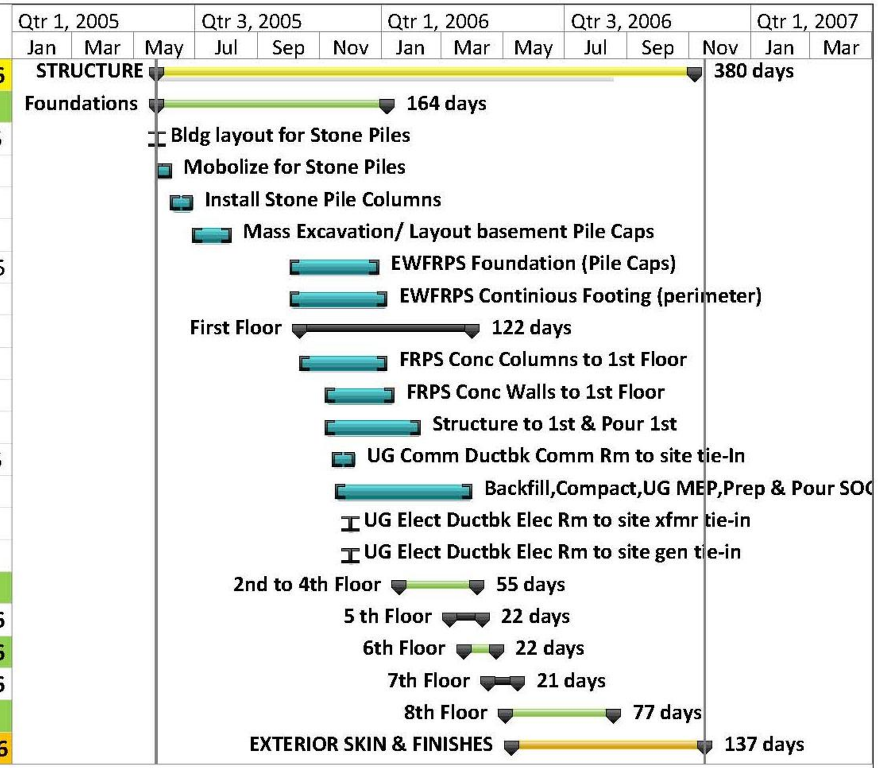
Crane type: portable hydrolic, floor type, 4,000lb capacity				
4150 per day	x	6	=	11 days

Base isolator cost	Material	14,245 per unit	x	66	=	$940,170
	Equipment	4150 per day	x	11	=	$45,650
					∑=	$985,820

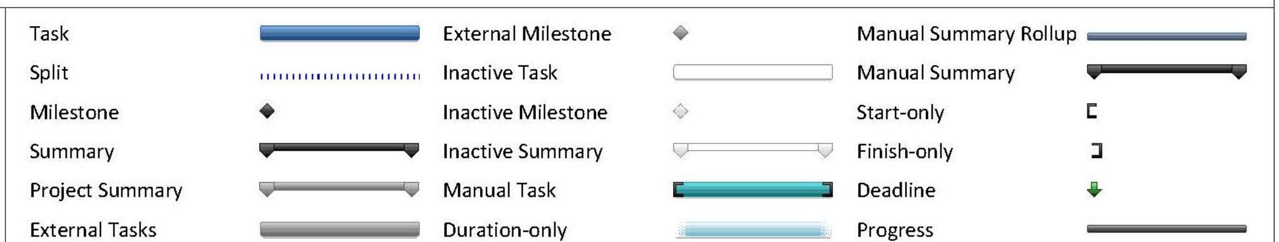
Schedule



ID	Task Name	Duration	Start	Finish
1	**STRUCTURE**	**380 days**	**Tue 5/24/05**	**Mon 11/6/06**
2	**Foundations**	**164 days**	**Tue 5/24/05**	**Fri 1/6/06**
3	Bldg layout for Stone	2 days	Tue 5/24/05	Wed 5/25/05
4	Mobolize for Stone Pi	11 days	Tue 5/24/05	Tue 6/7/05
5	Install Stone Pile Colu	16 days	Tue 6/7/05	Tue 6/28/05
6	Mass Excavation/ Lay	28 days	Wed 6/29/05	Fri 8/5/05
7	EWFRPS Foundation (	64 days	Mon 10/3/05	Thu 12/29/05
8	EWFRPS Continious F	70 days	Mon 10/3/05	Fri 1/6/06
9	**First Floor**	**122 days**	**Thu 10/13/05**	**Fri 3/31/06**
10	FRPS Conc Columns t	62 days	Thu 10/13/05	Fri 1/6/06
11	FRPS Conc Walls to 1:	50 days	Mon 11/7/05	Fri 1/13/06
12	Structure to 1st & Po	68 days	Mon 11/7/05	Wed 2/8/06
13	UG Comm Ductbk Co	16 days	Mon 11/14/0	Mon 12/5/05
14	Backfill, Compact, UG	97 days	Thu 11/17/05	Fri 3/31/06
15	UG Elect Ductbk Elec	2 days	Thu 12/1/05	Fri 12/2/05
16	UG Elect Ductbk Elec	2 days	Thu 12/1/05	Fri 12/2/05
17	**2nd to 4th Floor**	**55 days**	**Thu 1/19/06**	**Wed 4/5/06**
22	**5 th Floor**	**22 days**	**Fri 3/10/06**	**Mon 4/10/06**
26	**6th Floor**	**22 days**	**Fri 3/24/06**	**Mon 4/24/06**
33	**7th Floor**	**21 days**	**Mon 4/17/06**	**Mon 5/15/06**
41	**8th Floor**	**77 days**	**Thu 5/4/06**	**Fri 8/18/06**
60	**EXTERIOR SKIN & FINISHE**	**137 days**	**Wed 5/10/06**	**Thu 11/16/06**



Project: Schedule
Date: Wed 4/4/12



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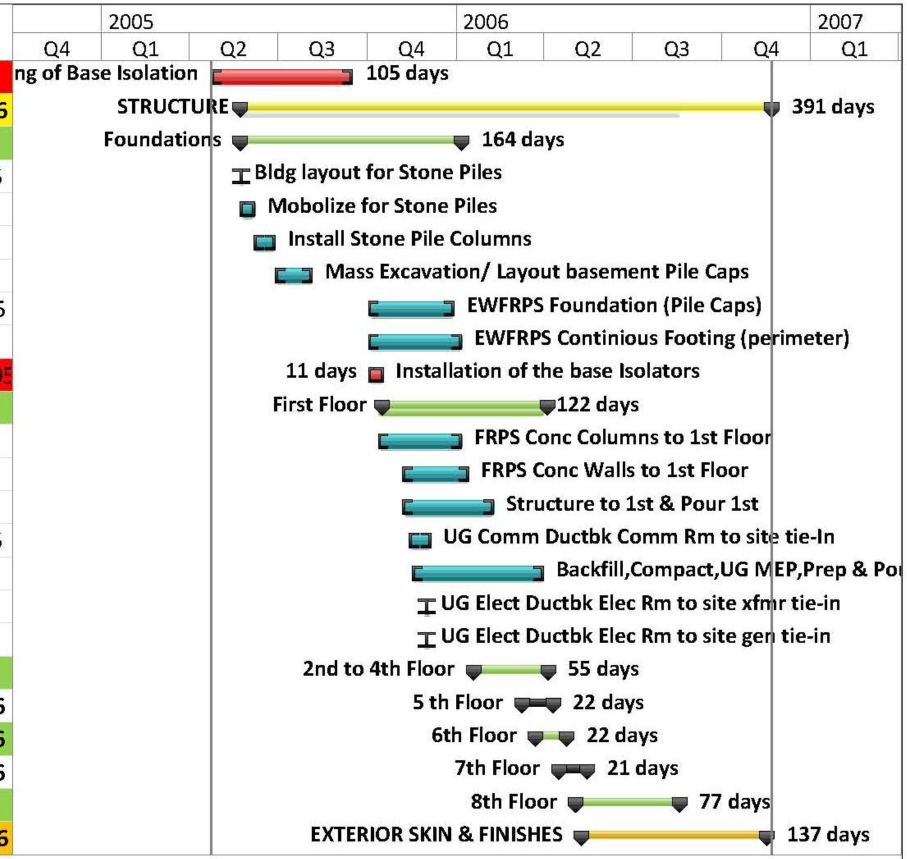
ID	Task Name	Duration	Start	Finish
1	Manufacturing and Shippir	105 days	Sun 4/24/05	Thu 9/15/05
2	STRUCTURE	391 days	Tue 5/24/05	Tue 11/21/06
3	Foundations	164 days	Tue 5/24/05	Fri 1/6/06
4	Bldg layout for Stone	2 days	Tue 5/24/05	Wed 5/25/05
5	Mobolize for Stone Pi	11 days	Tue 5/24/05	Tue 6/7/05
6	Install Stone Pile Colu	16 days	Tue 6/7/05	Tue 6/28/05
7	Mass Excavation/ Lay	28 days	Wed 6/29/05	Fri 8/5/05
8	EWFRPS Foundation (	64 days	Mon 10/3/05	Thu 12/29/05
9	EWFRPS Continious F	70 days	Mon 10/3/05	Fri 1/6/06
10	Base Isolation	11 days	Mon 10/3/05	Mon 10/17/05
11	First Floor	122 days	Mon 10/17/0!	Tue 4/4/06
12	FRPS Conc Columns t	62 days	Thu 10/13/05	Fri 1/6/06
13	FRPS Conc Walls to 1	50 days	Mon 11/7/05	Fri 1/13/06
14	Structure to 1st & Po	68 days	Mon 11/7/05	Wed 2/8/06
15	UG Comm Ductbk Co	16 days	Mon 11/14/05	Mon 12/5/05
16	Backfill, Compact, UG	97 days	Thu 11/17/05	Fri 3/31/06
17	UG Elect Ductbk Elec	2 days	Thu 12/1/05	Fri 12/2/05
18	UG Elect Ductbk Elec	2 days	Thu 12/1/05	Fri 12/2/05
19	2nd to 4th Floor	55 days	Thu 1/19/06	Wed 4/5/06
24	5 th Floor	22 days	Fri 3/10/06	Mon 4/10/06
28	6th Floor	22 days	Fri 3/24/06	Mon 4/24/06
35	7th Floor	21 days	Mon 4/17/06	Mon 5/15/06
43	8th Floor	77 days	Thu 5/4/06	Fri 8/18/06
62	EXTERIOR SKIN & FINISHE	137 days	Wed 5/10/06	Thu 11/16/06



Project: Schedule
Date: Wed 4/4/12

Task | Split | Milestone | Summary | Project Summary | External Tasks
External Milestone | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only
Manual Summary Rollup | Manual Summary | Start-only | Finish-only | Deadline | Progress

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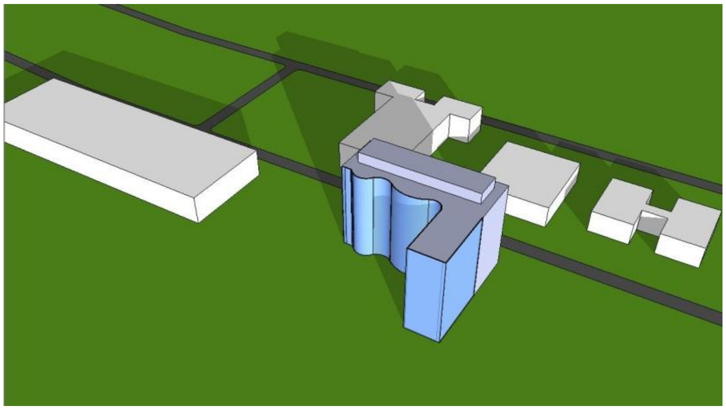
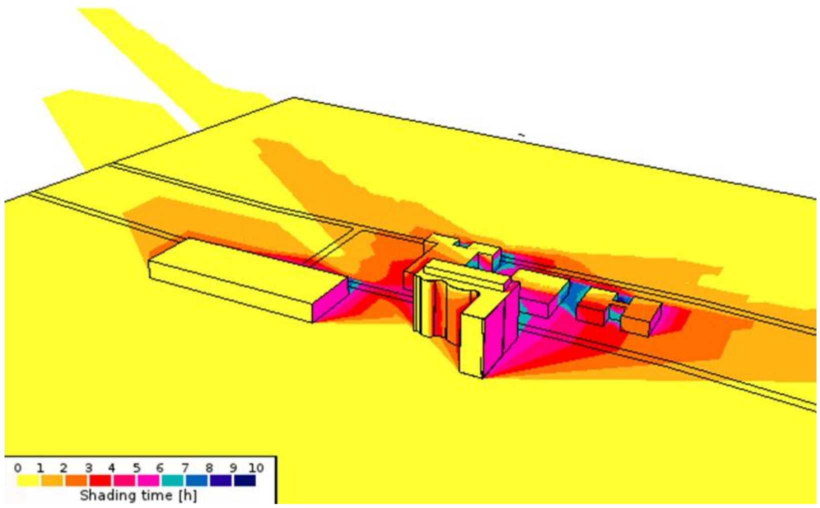
Appendix I: Sustainability Breadth/ BIPV curtain wall

Sample Sketchup model made



Sample picture from ShadowAnalysis



PV Areas: Actual building

Elevation	# of Spandrel	# of 30% Silkscreen	Area of Typ. Panel (ft^2)	Area of Spandrel	Area of 30% Silkscreen	total area	actual PV area of 30%	total PV area
East	794	532	7.5625	6005	4023	10028	1207	7212
North	240	62	7.5625	1815	469	2284	141	1956
West	224	112	7.5625	1694	847	2541	254	1948
South	486	160	7.5625	3675	1210	4885	363	4038
			∑	13189	6549	19738	1965	15154

New

Elevation	# of Spandrel	# of 30% Silkscreen	Area of Typ. Panel (ft^2)	Area of Spandrel	Area of 30% Silkscreen	total area	actual PV area of 30%	total PV area
South	794	532	7.5625	6005	4023	10028	1207	7212
East	240	62	7.5625	1815	469	2284	141	1956
North	224	112	7.5625	1694	847	2541	254	1948
West	486	160	7.5625	3675	1210	4885	363	4038
			∑	13189	6549	19738	1965	15154

	sf	$/sf	Sell
Curtain Wall West and South Wall:	13,206	$ 78.00	$ 1,030,068

POSSIBLY DEPRECIATE ONLY OVER 39 YEARS AT $25,641 PER YEAR
TAX SAVINGS WOULD BE 10,985 PER YEAR FOR A NET COST OF 14,656

BUYING A BiPV WALL USING COST SEGREGATION

		$/sf
PV Design	$ 198,090	$ 15.00
Electrical Design	$ 198,090	$ 15.00
Curtain Wall Design	$ 198,090	$ 15.00
Curtain Wall Aluminum	$ 264,120	$ 20.00
Vision Glass ** not part of the tax deductible portion **	$ 39,618	$ 3.00
Thin Film at Spandrel	$ 726,330	$ 55.00
Inverters & Monitoring	$ 158,472	$ 12.00
Wiring	$ 198,090	$ 15.00
Fabrication	$ 264,120	$ 20.00
Installation	$ 264,120	$ 20.00

	sf	$/sf			ADD for BiPV	Deduct for Tax Credit and MACRS	Savings after 5 years
BiPV Curtain Wall:	13,206	$ 187.00	$ 2,469,522	52.8 kW			
Total:	13,206	$ 190.00	**$ 2,509,140**		1,479,072	1,689,647	210,575

Federal Investment Tax Credit 30% of total BiPV until 2017:		$ (740,857)	30%	740,857
MACRS Depreciation Value:		$ 2,469,522		
Depreciation Schedule Per Year:	yr 1	$ 493,904		167,927
	yr 2	$ 493,904		167,927
	yr 3	$ 493,904		167,927
	yr 4	$ 493,904		167,927
	yr 5	$ 493,904		167,927
State Depreciation: (10 Year Straight Line)		$ 246,952.20	10%	21,831

TAX SAVINGS

YEAR 1	YEAR 2	YEAR 3	YEAR 4	YEAR 5	YEAR 6	YEAR 7	YEAR 8	YEAR 9	YEAR 10	$ 1,479,072
908,784	167,927	167,927	167,927	167,927						
21,831	21,831	21,831	21,831	21,831	21,831	21,831	21,831	21,831	21,831	
930,615	189,758	189,758	189,758	189,758	21,831	21,831	21,831	21,831	21,831	$ 1,798,800
63%	13%	13%	13%	13%						

NET OUT OF POCKET COSTS $ (319,728)

95% Return in 36 months

Actual 13% posetive return on investment per year for two years

Break Even Point

PV performance	13.47	kWh/SF/YR
No Pv SD EUI	5.53	(KWh/SF/YR)
Floor Plate	12934	SF
Story height	14.5	ft

	South Façade PV Coverage	PV GEN (kWh/YR)	West Façade PV Coverage	PV GEN (kWh/YR)	East Façade PV Coverage	PV GEN (kWh/YR)	TOTAL PV GEN (kWh/YR)	Consumption (kWh/YR)	Net Consumption
Façade Length	140		80		80				
PV (KWh/SF/YR)	11.43		11.33		11.53				
80% performance	9.144		9.064		9.224				
Story 8	46%	8,539	46%	4,837	46%	4,922	18,297	71,525	53,228
7	46%	8,539	46%	4,837	46%	4,922	18,297	71,525	53,228
6	46%	8,539	46%	4,837	46%	4,922	18,297	71,525	53,228
5	46%	8,539	46%	4,837	46%	4,922	18,297	71,525	53,228
4	46%	8,539	46%	4,837	46%	4,922	18,297	71,525	53,228
3	46%	8,539	46%	4,837	46%	4,922	18,297	71,525	53,228
2	46%	8,539	46%	4,837	46%	4,922	18,297	71,525	53,228
1	46%	8,539	46%	4,837	46%	4,922	18,297	71,525	53,228
	∑	68,309	∑	38,692	∑	39,375	146,377	572,200	425,823

Total PV Offset (%) = 25.58%

x

0.1297 $/kWh

Total Savings= $18,985

Total BIPV curtain wall	Sell exisiting Panels	Addition for BiPV	Deduct for Tax Credit and MACRS	Local Utility Savings	Savings after 5 years
$2,469,522	$1,030,068	$1,479,072	$1,689,647	$94,925	$305,500