Albuquerque, NM April 4, 2012

Structural Final Report

Courtesy: Antione Predock

Faculty Advisor: Dr. Richard Behr

NICOLE TRUJILLO

UNM SA+P

UNIVERSITY OF NEW MEXICO SCHOOL OF ARCHITECTURE + PLANNING

Nicole Trujillo — Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2012/nbt5004/index.html

GENERAL INFORMATION

Function:	Architecture School
Size:	108,000 GSF
Height:	65 Feet
Construction:	Nov 2005 - Sept 2007
Construction Cost:	$22 Million
Delivery:	Design-bid-build

ARCHITECTURE

- Curtain-wall system located at the west side of the building
- Green roof located at the south-east side of the building
- Shading devices used on the south end of the building
- Breezeway located at the center of the building

CONSTRUCTION

- 6,756 cubic yards of concrete
- 965,824 pounds of rebar
- 28,000 sq ft of glass

STRUCTURE

- 4 main trusses (spanning 96') at 45 tons each
- Geopier Foundations System consisting of 418 piers to stabilize soils
- Cantilevered concrete wall on the SW side spans 38' and SE side spans 25'
- Concrete walls were poured using specialized 'Agilia' concrete product

PROJECT TEAM

Owner:	University of New Mexico
Design Architect:	Antione Predock
Executive Architect:	Jon Anderson
Structural Engineer:	Chavez-Grieves
MEP Engineer:	Bridges & Paxton
Civil Engineer:	Jeff Mortensen & Assoc.
General Contractor:	Jaynes
Mechanical Contractor:	Yearout Mechanical
Electrical Contractor:	McDade-Woodcock

Sketch Courtesy: Antione Predock

MEP SYSTEMS

- Mechanical:
  -Two AHU's one at 40,000- 53,000 CFM
  -Thirteen Fan Coil Units ranginging from 400-2390 CFM
  -Direct Digital Control (DDC)
- Lighting/Electrical
  -12.47 kV main switchgear
  -Main power is 280Y/120V 3 phase, 4 wire
  -Packaged Engine Generator
  -Uses Fluorescent, HID, and Incandescent lighting
- Fire Protection
  - Automatic Sprinklers are provided throughout the building
  - Fire pumps located in the lower level mechanical room

Table of Contents

List of Figures and Tables

Acknowledgements

I am very grateful to the following individuals for their support and assistance:

The University of New Mexico
- Robert Doran

UNM School of Architecture and Planning
- Roger Schluntz
- Lisa Stewart

Jon Anderson Architects
- James Lucero

Kenneth Guthrie, Specification Writer
- Kenneth Guthrie

Chavez-Grieves Consulting Engineers
- Chris Romero

The Pennsylvania State University
- Professor M. Kevin Parfitt
- Professor Robert Holland
- Dr. Richard Behr
- Dr. Thomas Boothby
- Dr. Jelena Srebric

Thank you to all of my family and friends, especially my fiancé and my parents.

Executive Summary

George Pearl Hall is the School of Architecture and Planning at the University of New Mexico and is located in Albuquerque, New Mexico. Antoine Predock was the design architect for the building, creating a Spanish-Pueblo style architecture school.

The building is approximately 108,000 square feet and the height is 71.33 feet. The design and construction of the project lasted seven years, from 2000 until 2007. The programmatic addition of the Fine Arts Library, as well as the fluctuating budget led to the lengthy construction time. The architect intended to create a building that would teach students about making architecture. Therefore, the structure and HVAC equipment is exposed throughout the building.

Pearl Hall has received numerous construction merits and design awards. The tectonic structure that is both aesthetic but can also be challenging in terms of structural design.

This report focuses on the structural system in Pearl Hall. Yet, two breadth studies were performed to evaluate the mechanical system and architectural features.

The structural system in Pearl Hall is composed of concrete slab on deck and uses steel beams, girders, and columns as the framing system. The typical interior bay is 30 feet by 32 feet. Special reinforced concrete shear walls function as the lateral force resisting system for Pearl Hall. According to ASCE 7-05, Pearl Hall is located in Seismic Design Category D. The building is designed for seismic forces and drift as the controlling lateral load case.

The design goal is to provide possible cost savings of an alternative lateral force resisting system. The proposed redesigns are: a modified special reinforced shear wall system, special concentric braced frames, and a special moment frame system. The cost was decreased 3.5 times by using the moment frames instead of the existing shear walls.

The architecture breadth study looks at the cost impact of enclosing the breezeway in Pearl Hall by adding architectural glazing. This would increase more functional space for Pearl Hall to use as classrooms and faculty offices. It was determined that the material cost for the redesign would be $2032.

The mechanical breadth study focused on the performance issue in regards to occupant thermal comfort on the critique bridge on level 2. The results of the study showed evidence using more insulating glazing, VNE 1-30 Glazing that it will provide the most energy cost savings for Pearl Hall. VNE 1-30 glazing provides 9.73% decrease in consumption than the current VRE 3-54 glazing.

The goal of this thesis was to investigate more cost effective lateral force resisting system for Pearl Hall. In addition, it was a personal goal to learn ETABS and investigate design requirement for high seismic regions. In addition, it was to design a usable enclosure for the breezeway and investigate a solution for the heat loss on the critique bridge.

Based on the results discussed, these goals are clearly met.

1. Building Introduction

George Pearl Hall contains the School of Architecture and Planning at the University of New Mexico located in Albuquerque, New Mexico. George Pearl Hall is situated along old Route 66 at the edge of the University of New Mexico campus (Figure 1&2). At 108,000 gross square feet, Pearl Hall functions as a classroom, office, studio, and a library. It reaches a height of 71.83 feet with three levels, a mezzanine and a basement. In November 2005, the contractor received "notice to proceed," and construction was completed in September 2007. As design-bid-build, the project was rewarded to the lowest bidder. The design architect was Antoine Predock who worked with the executive architect Jon Anderson. Jon Anderson architects produced the design drawings. Due to the extremely tight budget, the design team used value-engineering to lower costs and produce a more efficient design.

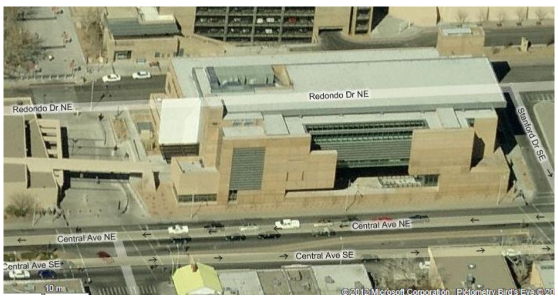


Figure 1. *Bird's Eye Southwest view of Pearl Hall. (Credit: Bing Maps)*

Figure 2. *Southwest view of Pearl Hall from Old Rt. 66. (Courtesy: Patrick Coulie, Photographer)*

Antoine Predock's George Pearl Hall has elements of the traditional Spanish-Pueblo style in buildings across the UNM campus. Yet, it had been called "tectonically expressive and formally complex." The building in plan holds to the rectangular site. Yet, the interaction between the architectural concrete walls, structural steel ceiling beams and glazing systems demonstrates the complex relationship between plan and section. Pearl Hall houses the School of Architecture for the University, The Perish Memorial Fine Arts Library and numerous classrooms, faculty offices and a first floor patio and breezeway. Predock intended to create a building where students could be educated through the architecture by seeing structural supports such as wide flange beams as well as the conduits and duct work. The studio spaces are hung from four giant, 96-foot long steel trusses, which also support the library occupying the top floor (Figure 3).

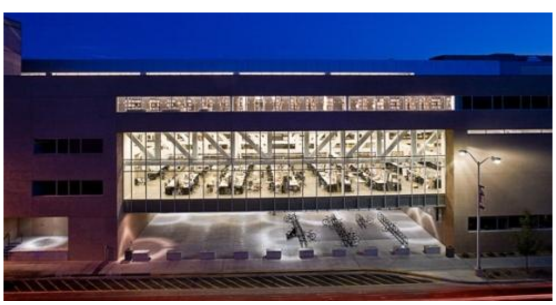


Figure 3. *South View at Night (Courtesy: Kirk Gittings, Photographer)*

Figure 4. *Rendering of Southwest View (Courtesy: Jon Anderson Architects)*

The facade is comprised of large cantilevered concrete and glass sections on the south side. The north, east and west walls are framed with steel studs and glass windows. A massive plenum wall of cast-in-place concrete is cantilevered from the west and east corners (Figure 5). That wall splits open to the center to reveal a recessed curtain wall of steel, aluminum, and glass with deep louvers shading the interior. Albuquerque's climate was factored into the construction, so that the massive southern wall and the concrete floors throughout help to stabilize temperature shifts. The southern wall also serves as a plenum chamber for HVAC air circulation which is part of the mechanical system.

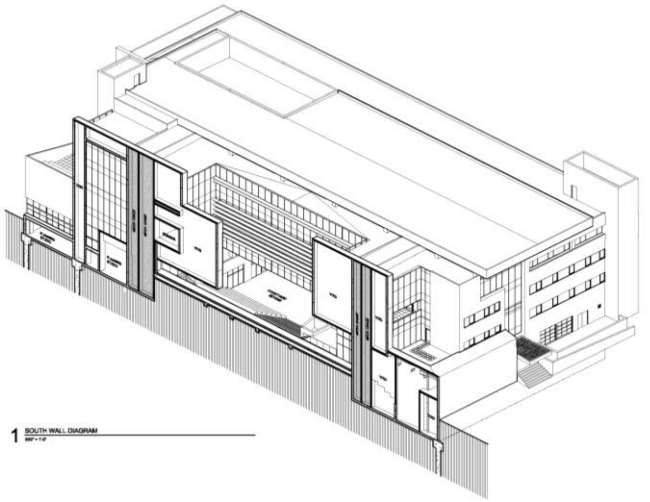


Figure 5. *South Wall Diagram*
(Courtesy: Jon Anderson Architects)

George Pearl Hall applies sustainable design standards to the building. Deep louvers control direct sun to minimize heat gain and glare (Figure 4). In addition, light shelves reflect sun onto the interior ceiling providing indirect light. Low-e Solarban 60 glazing is used in combination with fritted glass on the east and west elevations to control heat gain. A setback for overhanging studios and the critique bridge are established by the winter solstice altitude angle to maximize winter sun and minimize summer sun. Also, the roof drains are directed to storage tanks providing irrigation water for the green roof planting beds (Figure 7).

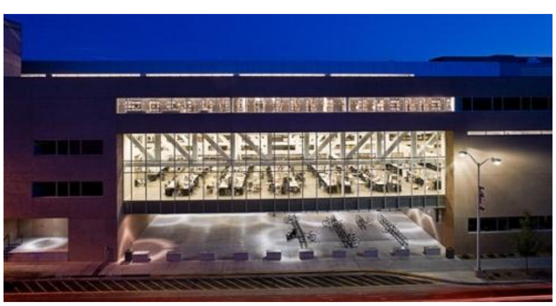


Figure 6. *South View at Night*
(Courtesy: Kirk Gittings, Photographer)

Figure 7. *Green Roof*

2. Structural Overview

George Pearl Hall consists of three levels, a basement, and a mezzanine. The building footprint is approximately 35,000 square feet in plan dimension. The structural engineer for the project was Chavez-Grieves Consulting Engineers. Chavez-Grieves utilized RAM to design the gravity and lateral members for Pearl Hall (Figure 8).

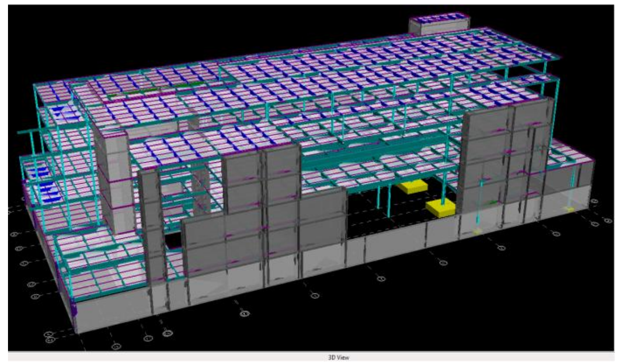


Figure 8. *RAM Structural Model of Pearl Hall. (Courtesy: Chavez-Grieves Consulting Engineers)*

Pearl Hall uses primarily steel frame construction with concrete shear walls. The south side walls are comprised of large cantilevered concrete and glass sections. The floors system utilized in Pearl Hall is composite deck supported by composite steel beams and steel girders. In order to provide a column-free 96 foot breezeway at ground level, four wide flange steel trusses span 96 feet. The trusses were used because the Fine Arts library is located on Level 4. Therefore, the larger gravity load on the floor required the use of the four, 96 foot trusses to help distribute the load to the foundation. The foundation of the building consists of a Geopier system, which are aggregate piers. Originally, a pier foundation was recommended for Pearl Hall. Through the Value-Engineering process, the foundation system was changed to use the Geopier system in lieu of the piers. The lateral stability of the steel frame is dependent upon the concrete shear walls.

2.1 Foundations

Terracon performed eight soil test borings which were drilled from May 1toMay 2, 2003. The pediment soils at and around the site consist of alluvium, which range from poorly sorted mudflow materials to well-sorted stream gravel. Pediment soils occur at the base of a mountain. The results determined that the underlying soils at the site consist mainly of silty sand at a boring termination depth of approximately 31.5 feet. Also, a lean clay with sand layer was encountered at a depth of 61.5 feet from three boring tests. The laboratory tests concluded that the near surface soils exhibited moderately high compressibility for the loads. Drilled piers or augered cast-in-place piles were recommended for Pearl Hall. Yet, due to

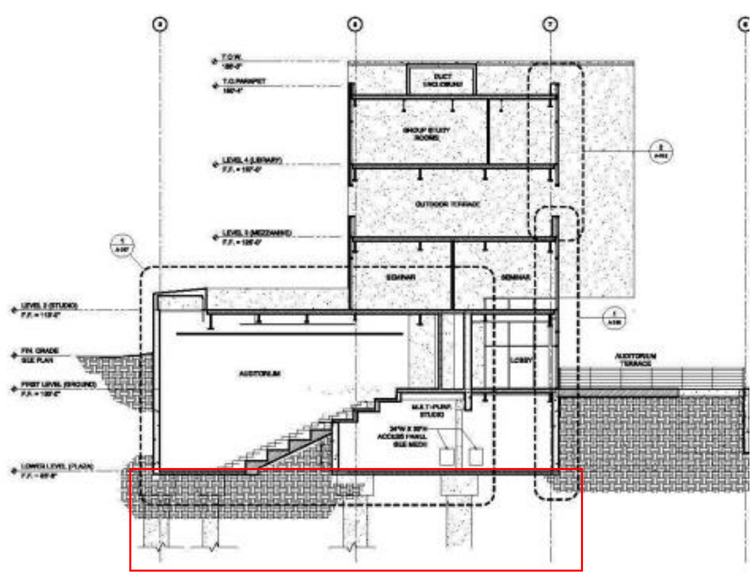


Figure 9. *Section through Lower Level at Auditorium (Courtesy: Jon Anderson Architects)*

budget, an alternative system was used.

Geopiers are short aggregate piers composed of highly densified graded aggregate, which is placed in thin lifts within a drilled or excavated cavity (Figure 9). The system prestresses the soil vertically at the bottom of the cavity, and horizontally during construction of the thin lifts. This results in a very stiff soil/aggregate layer that can support loads with settlements of one inch or less and a reduced differential settlement. The aggregate piers (Geopier) were designed and installed to provide an equivalent soil bearing pressure of 8500 psf at the building footings (Figure 10).

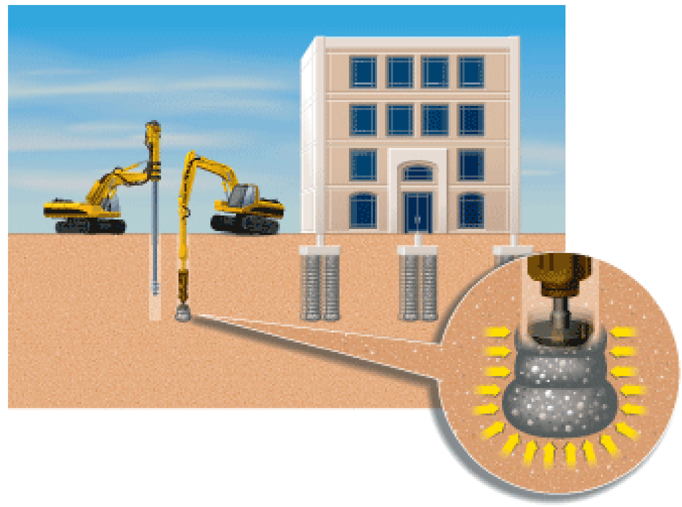


Figure 10. *The Geopier® System (Credit: geopier.com)*

Groundwater levels were indicated to be below the maximum depth explored at the time of the boring tests. Therefore, the 14 foot basement can be situated on the site. Since perched groundwater may occur at times due to the relatively impermeable layers, a drainage system was constructed around the perimeter of the basement foundation walls and footings. It is sloped to a sump and pump system. The floor slab is a 5 inch concrete slab reinforced with #4 @ 18 inch on center each way. The building is located on a pediment surface on the east side of the Albuquerque-Belen basin. The fluctuation of groundwater was the cause for the use of a groundwater monitoring plan. As a result, it might be necessary to investigate the lateral soil pressure of the basement wall.

2.2 Floor Systems

Pearl Hall uses floors that are made of composite floor deck on with a typical floor thickness of 6.5 inches. The deck is supported by w-shaped steel beams. Then, they transfer the load to the girders which carry the moment to the columns. The roof deck is comprised of a 2.5 inch concrete pad with type B, 18 gage galvanized metal deck over mechanical space and a 5 inch normal weight concrete with type C, 20 gage galvanized formdeck.

The beams at each floor were designed to support the gravity load of the curtain wall system. The glass curtain wall system is supported laterally at all floors and roof level.

2.3 Framing System

Pearl Hall utilizes shear wall systems. It is a steel structure with reinforced concrete shear walls for lateral resistance. It uses typical bays of 32 foot by 30 foot bays (Figure 11). The floors are supported by a configuration of beams and girders, which frame into the columns.

The stairways are framed into the shear walls. The west stairs are cantilevered off a 24 inch thick reinforced concrete wall.

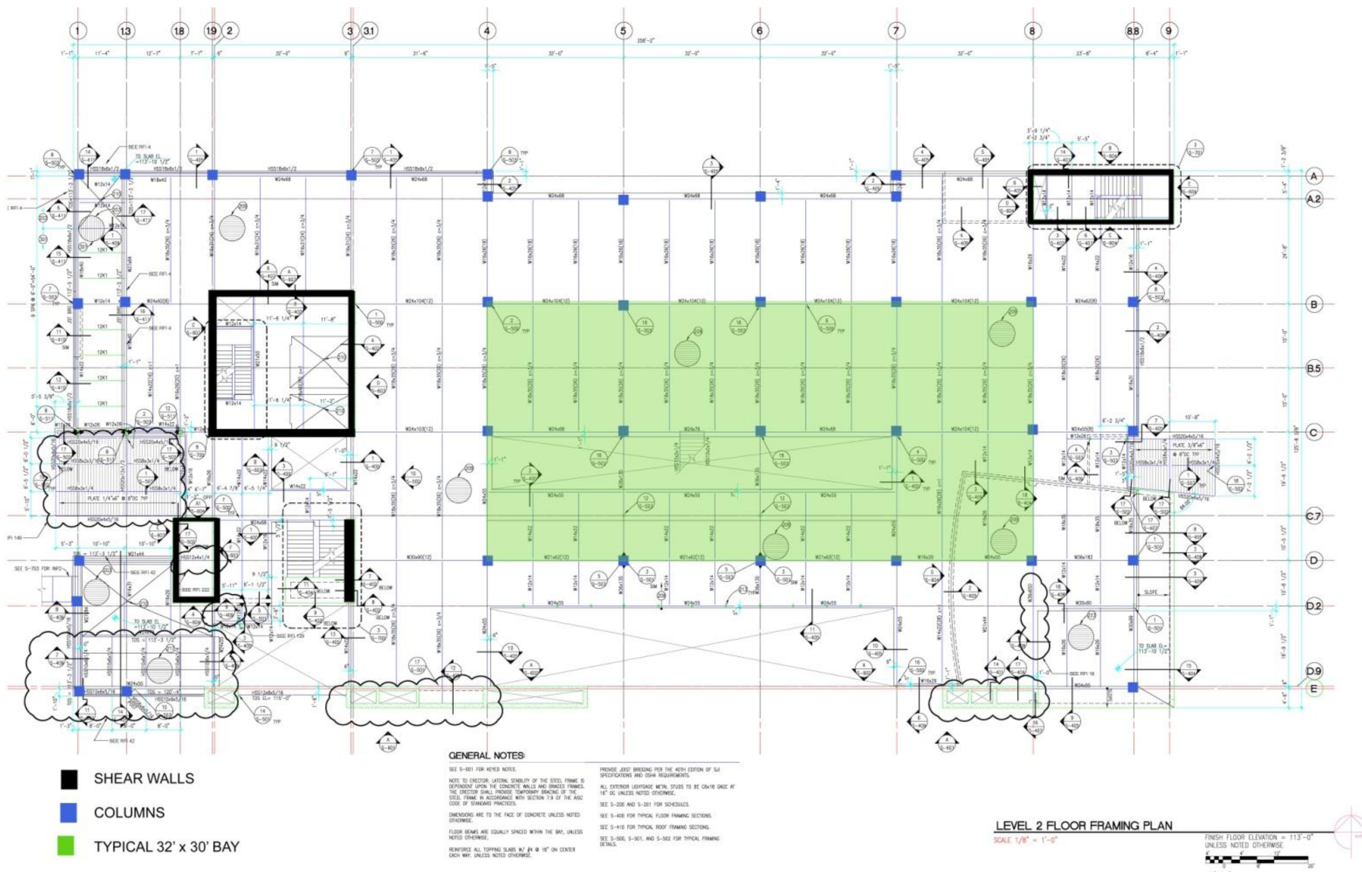


Figure 11. *Framing Plan-Level 2 Plan showing shear walls and column layout. Modified by N. Trujillo. (Courtesy of Jon Anderson Architect)*

2.4 Lateral System

Pearl Hall uses shear walls as the main lateral force resisting system (Figure 12). As the lateral loads are dissipated through the reinforced concrete shear walls, which range from 12 inch to 24 inch thick to transfer the load from the above grade stories. Story forces are carried through the beams into the columns. Then, the loads move into to the grade slab. Below grade, the structure uses shear walls around the stair cores and south wall to carry lateral loads to the foundation.

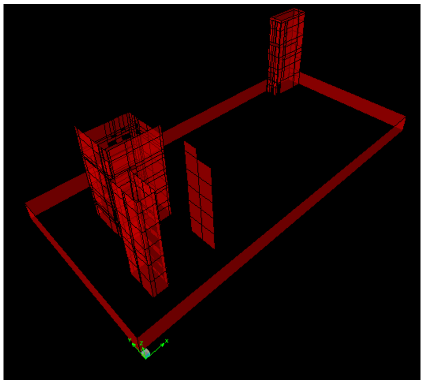


Figure 12. *ETABS model of shear walls.*

2.5 Roof Systems

There are two roof levels on Pearl Hall (Figure 13). Low Roof is at 71 ft-4 in. and High Roof at 81ft -6in. In addition, Pearl Hall has a green roof located at the Southeast corner of the building

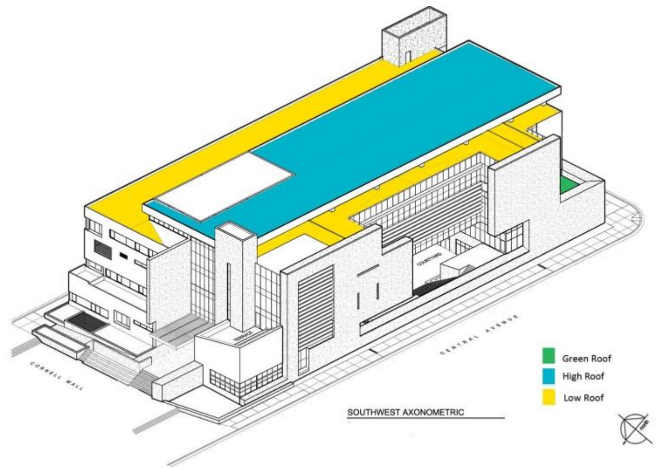


Figure 13. *Southwest axonometric indicating roof levels. Modified by N. Trujillo (Courtesy of Jon Anderson Architect)*

2.6 Design Codes

Table 1. *Design Codes*

Code Used for Design	Codes Used for Report
International Building Code, 2003 Edition	IBC 2006
ASCE Standard Minimum Design Loads for Buildings and Other Structures: SEI/ASCE 7-02	ASCE 7-05
AISC Manual of Steel Construction – Allowable Stress Design, 9th Edition (1989)	AISC Manual of Steel Construction –LRFD, 13th Edition (2006) AISC 360-05
SJI Standard Specification for Steel Joists and Joist Girders, 2002 Edition	SJI Standard Specification for Steel Joists and Joist Girders, 2005 Edition
ACI Building Code Requirements for Structural Concrete, ACI 318-02	ACI 318-08

Building Code Analysis (Sheet G-100 Jon Anderson Architects)
Assembley A-3
Lower Level (Auditorium)
Level 1 (Auditorium & Gallery)
Level 4 (Library, Staff Area)
Business B
Lower Level (Classrooms, Studios, Offices, Storage, Mechanical Space)
Level 1 (Offices, Storage & Seminars)
Level 2 (Offices, Studios, Classrooms, Storage & Seminars)
Level 3 (Offices, Studio, Classrooms, Mechanical Spaces)

Figure 14. *Building Occupancy (Obtained from the Design Documents)*

2.7 Material Summary

Materials		
Cast in Place Concrete (Normal Weight Concrete)		
	F'c = 4000 psi @ 28 days	all interior and exterior concrete (ie footings, predestals, tie beams, grade beams, retailing wall, exterior slabs, equipment pads, etc.)
	F'c = 3000 psi @ 28 days	all interior slabs on grade
	F'c = 3000 psi @ 28 days	all concrete fill over metal deck
Precast/prestressed concrete		
	F'c = 5000psi @ 28 days, min	
	F'c = 3500 psi min @ transfer of prestress	Prestressing tendons shall conform to ASTM A416, FPU = 270 KSI
Reinforcing Steel		
	All, ASTM A615 Grade 60	
	Stirrups, ties and indicated field-bent bars, ASTM A615 Grade 40	
Glass Curtain Wall System		
	Type VE-85 Viracon	Two (1/4" glass) with One (1/2" air space)
		2- Glass (1⁄4" plate) = 2(3.3) psf = 6.6 psf
		Therefore, total curtain wall: 10 psf
Structural Steel		
	Wide Flange Shapes:	ASTM A992, Grade 50
		Fy = 50 ksi, Fu = 65 ksi
	Channels, Angles,	ASTM A36
	Flat bars, and plates	Fy = 36 ksi, Fu = 58 ksi
	Rectangular and square	ASTM A500, Grade B
	structural tubing	Fy = 46 ksi, Fu = 58 ksi
	Round Structural Tubing	ASTM A500, Grade B
		Fy = 42 ksi, Fu = 58 ksi
	Structural Pipe	ASTM A53, Type E or S, Grade B
		Fy = 35 ksi, Fu = 60 ksi

Figure 15. *Materials (Obtained from the Design Documents)*

Foundation
5" Concrete slab reinforced with #4 @ 18" O.C. each way over 4" aggregate base course over compacted subgrade finish floor elevation
Floor drain, sloped slab to drain.
4" Concrete housekeeping pad reinforced with #4 @ 18" O.C. each way, 2-#4 each way.
2' square sump pump
Floor Slab
5" maximum topping slab reinforced with #4 @ 1" on center each way over waterproofing membrane, over insulation over 3.5" normal weight concrete reinforced with #4 @ 12" on center each way over 3" VLI, 18 gage, galvanized composite deck (total slab thickness = 6.5").
Pavers over pedestals over 5" maximum topping slab reinforced with #4 @ 18" on center wach way over waterproofing membrane over insulation over 3.5" normal weight concrete reinforced with #4 @ 12" on center each way over 3" VLI, 18 gage galvanized composite deck (total slab thickness = 6.5").
Galvanized steel grating wih 1"x3/16" bearing bars at 1-3/16" on center and cross bars at 4" on center.
HSS 10x8x.25 glazing supports @ 8' on center
3.5" normal weight concrete reinforced with #4 @ 12" on center each way over 3" VLI, 18 gage, galvanized composite deck (total slab thickness = 6.5 inches.
Precast concrete pavers over waterproofing membrane over insulation over 3.5" normal weight concrete reinforced with #4 @ 12" on center each way over 3VLI, 18 gage, galvanized composite deck (total slab thickness = 6.5")
4" cocnrete topping slab reinforced with #4 @ a8" on center each way over waterproofing membrane over insulation over 3.5" normal weight concrete reinforced with reinforced with #4 @ 12" on center each way over 3VLI, 18 gage, galvanized composite deck (total slab thickness = 6.5")
5" concrete topping slab reinforced with #4 @ 18" on center over 4" aggregate base course. Finish floor.
3.5" normal weight concrete reinforced with #4 @ 12" on center each way over 1.5" VLI, 20 gage, galvanized composite deck (total slab thickness = 5 inches.
4" concrete slab reinforced with 6x6xW1.4xW1.4 welded wire fabric in flat sheets over 12 gage pan.
4" normal weight concrete, reinforced with 4x4xW2.9xW2.9 welded wire fabric in flat sheets over 1"C, 20 gage galvanized metal deck (total slab thickness = 5")
Roof Floor
1.5" Type B, 18 Gage, galvinized metal deck with nestable sidelaps.
2.5" Concrete pad reincorced with #4 @ 12" oc each way over 1.5" type B metal deck. Total slab thickness is 6 inches.
5" maximum normal weight concrete reinforced with #4 @ 12" oc center each way over 1", type C, 20 gage galvinized formdeck (total slab thickness = 6 inches)

Figure 16. *Foundation, Floor Slab, and Roof Slab materials (Obtained from the Design Documents)*

3. Structural Depth

3.1 Existing Structural System

As it was briefly introduced before, Pearl Hall has an existing lateral system composed of shear walls as seen in Figure 17. This includes three shear wall cores and the lower level stem walls. The shear walls are made of 4 ksi concrete and varying thickness from 12-24 inches.

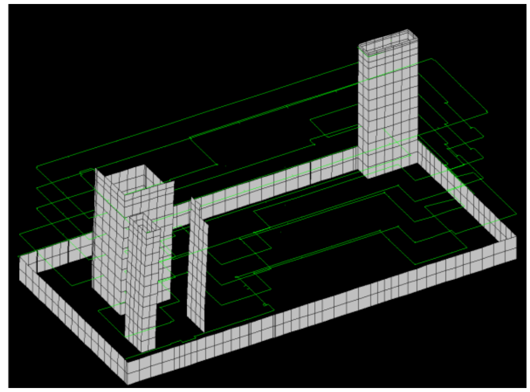


Figure 17. *RAM Model showing lateral system and diaphragms (Courtesy of Chavez-Grieves)*

Pearl Hall was evaluated for gravity loads as well as lateral loads. It was determined that Pearl Hall met strength requirements for gravity beams and columns. In addition, floor and roof decks were adequate for strength requirements as well. Please refer to calculations in Appendix A.

In order to evaluate lateral loads, an ETABS model was created to evaluate strength and serviceability requirements for the existing lateral force resisting system in Pearl Hall. The model was developed by the use of design drawings from Jon Anderson Architects (Appendix J).

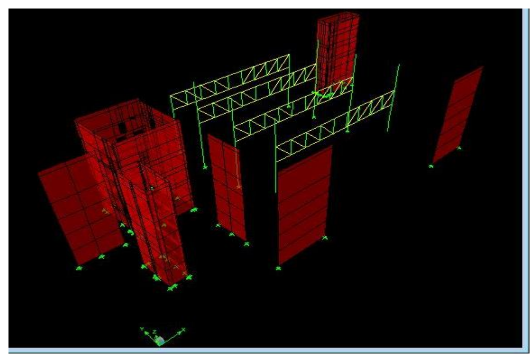


Figure 18. *Initial ETABS Model Showing lateral system*

In ETABS, the shear walls in Pearl Hall were modeled with openings (Figure 18). Initially the model was built using the (4) 96 foot trusses as a part of the lateral system, as was mentioned in the proposal. Yet, with discussion from the structural engineer on record, it was determined that the trusses were designed for gravity loading only (Figure 19).

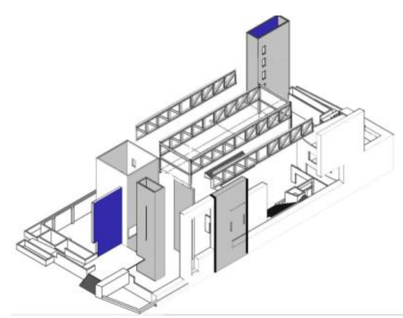


Figure 19. *Southwest axonometric highlighting shear wall locations in grey, possible shear wall locations in blue, and trusses. Modified by N. Trujillo. (Courtesy of Jon Anderson Architect)*

In addition, the shear walls were analyzed in ETABS to determine if serviceability requirements were met. Drift values were output from ETABS and considered very small. The values were on the magnitude of 0.001 in. This raised concern on the accuracy of the model. Therefore, it was desirable to verify the load assumptions from the structural engineer on record, Chaves-Grieves Consulting Engineers. The firm provided me with a RAM Model which they used to design Pearl Hall. It was determined that the center of mass and building period were within 10% of the ETABS model. It addition, the drift values were very small. Please refer to Figure 20.

Modal Period		
ETABS	0.52	s
RAM	0.56	s
% Difference	**-6%**	

Centers of Mass & Centers of Rigidity ETABS				Centers of Mass & Centers of Rigidity RAM				% Difference from ETABS			
XCM (ft)	YCM (ft)	XCR (ft)	YCR (ft)	XCM (ft)	YCM (ft)	XCR (ft)	YCR (ft)	XCM (ft)	YCM (ft)	XCR (ft)	YCR (ft)
238.81	118.501	205.485	117.748	240.29	114.92	202.95	110.3	**-1%**	**3%**	**1%**	**6%**
148.22	61.13	54.05	74.47	123.25	55.08	44.64	65.31	**17%**	**10%**	**17%**	**12%**
119.95	77.07	69.61	95.40	115.39	68.86	68.38	93.12	**4%**	**11%**	**2%**	**2%**
133.65	71.61	72.74	96.06	131.76	67.64	70.01	93.84	**1%**	**6%**	**4%**	**2%**
129.09	62.42	75.66	96.73	124.14	59.84	72.77	94.90	**4%**	**4%**	**4%**	**2%**
128.22	68.93	79.94	97.21	138.68	57.47	77.65	96.97	**-8%**	**17%**	**3%**	**0%**
119.25	72.31	97.77	69.68	122.29	69.54	90.39	110.56	**-3%**	**4%**	**8%**	**-59%**

Figure 20. *Comparison of the Center of Mass and Rigidity as well as Modal Period of ETABS versus RAM model*

There were many minor differences between the analysis of the structural engineer on record and the one performed for this investigation. Notable was the different codes used, as noted previously. The structural engineer on record used ASCE 7-02 for seismic design, while the code used for analysis in this report was ASCE 7-05. In ASCE 7-05 the modal spectrum response accelerations differed: from Ss = 0.620g and S1 = 0.185g in ASCE 7-02 to Ss = 0.520g and S1 = 0.15g. The differences can be attributed to different assumptions in the structural model. Therefore, the ETABS model was considered to be consistent with the RAM model from the structural engineers on record.

3.1.1 ETABS Modeling of Existing System

ETABS was chosen as the computer modeling software for this thesis. The ETABS model was used to check lateral drifts, deflections, and periods of vibration of the existing lateral system (Figure 21). The ETABS output of shear, axial, and moment values were used during the design check and reinforcement design of the shear walls. In addition, PCAColumn was used to check the design reinforcement in the shear walls by the Axial vs. Moment interaction diagrams.

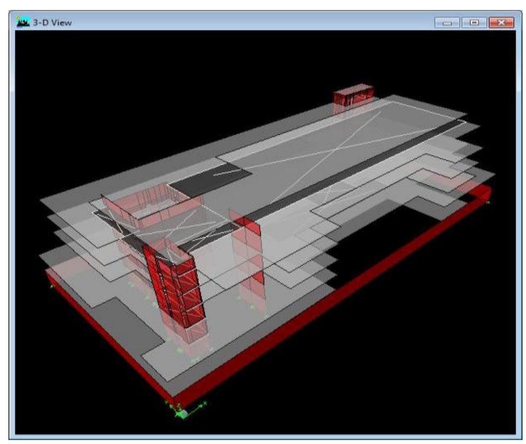


Figure 21. *ETABS Model Showing lateral system and diaphragms*

Due to the complex geometry of Pearl Hall, real and accidental torsional effects must be considered for the design forces (Figure 22). Therefore, the computer model was necessary in order to check and propose redesigns for the lateral system. Since, Pearl Hall is a structure with irregular plans and soft stories, these irregularities will be considered because a realistic three-dimensional computer model is created. According to ASCE 7-05 Sect. 12.7.3 concrete elements should consider effects of cracked sections. ACI 318-08 permits the use of 50% stiffness values based on gross section. Therefore, the walls are models using area elements setting f_{22} = 0.5.

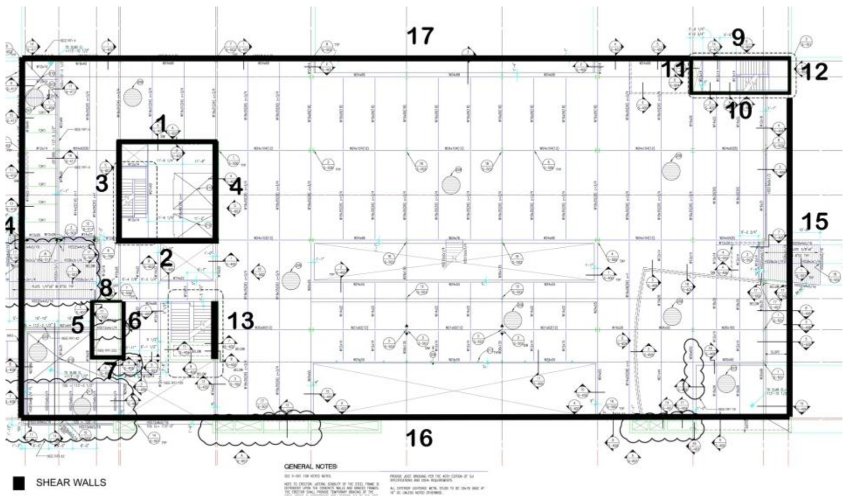


Figure 22. *Base Floor Plan indicating Shear Wall Numbers. Modified by N. Trujillo. (Courtesy of Jon Anderson Architect)*

3.1.2 Center of Rigidity

The centers of rigidity of each shear wall were calculated as rectangular wall areas. The ETABS model was created with openings indicated in the design drawings. Therefore, the difference in center of rigidity calculations can be attributed to the difference in areas.

	Centers of Mass & Centers of Rigidity					
	Hand Calculations*		ETABS		% Difference from ETABS	
Story	XCR (ft)	YCR (ft)	XCR (ft)	YCR (ft)	XCR	YCR
STAIR 3	240.00	117.84	205.549	117.752	-17%	0%
HGH ROOF	61.62	73.32	54.05	74.43	-14%	2%
LOW ROOF	57.17	71.40	69.64	95.37	18%	25%
STORY4	57.41	71.47	72.79	96.02	21%	26%
STORY3	57.41	71.47	75.75	96.66	24%	26%
STORY2	56.74	71.27	80.08	97.05	29%	27%
STORY1	65.28	68.70	97.87	69.64	33%	1%
* Assume that the general area of wall is rectangular yet has openings						

Figure 23. *Comparison of the Center of Mass and Rigidity of ETABS versus Hand Calculations*

High Roof Shear Wall Data*												
Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	Δ (in)**	I =Ri/∑Ri
2	12	34.00	90	7.00	1.00	7.00	34.00	238.00	68924.57	1.51E-05	13.42%
3	12	33.00	0	7.00	33.00	231.00	1.00	7.00	66669.25	1.56E-05	12.98%
5	12	23.17	0	7.00	23.17	162.19	1.00	7.00	44138.86	2.35E-05	8.60%
6	12	23.17	0	7.00	23.17	162.19	1.00	7.00	44138.86	2.35E-05	8.60%
7	12	10.33	90	7.00	1.00	7.00	10.33	72.31	13537.42	7.58E-05	2.64%
8	12	10.33	90	7.00	1.00	7.00	10.33	72.31	13537.42	7.58E-05	2.64%
9	12	32.00	90	7.00	1.00	7.00	32.00	224.00	64408.42	1.61E-05	12.54%
10	12	32.00	90	7.00	1.00	7.00	32.00	224.00	64408.42	1.61E-05	12.54%
11	18	12.33	0	7.00	12.33	86.31	1.50	10.50	27421.50	3.75E-05	5.34%
12	18	12.33	0	7.00	12.33	86.31	1.50	10.50	27421.50	3.75E-05	5.34%
13	24	21.17	0	7.00	21.17	148.19	2.00	14.00	78900.71	1.31E-05	15.37%
* Assume that the general area of wall is rectangular								∑Ri =	513506.93		100.00%
** Using a 1k load applied at the top of each LFRS system											

Figure 24. *High Roof Level Relative Stiffness Calculations for Center of Rigidity Calculation*

3.1.3 Gravity Loads

The dead and live loads used for the analysis for Pearl Hall were calculated in accordance with ASCE 7-05 and specified loads on the drawings. The reason for such a large dead load on Level 4 is due to the Fine Arts Library (Figure 25). Figure 26 compares the design live loads to ASCE 7-05. The dead load calculations can be found in Appendix A. It was desirable to compare the values for the calculated dead and live loads to those from the RAM Model. Figure 26 shows the difference in the dead loads. Therefore, it was decided to use the RAM Model dead and live loads in order to design a more accurate lateral system (Figure 27).

Figure 25. *Perish Memorial Fine Arts Library*

Live Load	Design Loads		ASCE 7-05 Live Loads		Notes
Classrooms	80	PSF	40	PSF	
Offices	50	PSF	50+20	PSF	Office load + Partition Load
First Floor Cooridors	100	PSF	100	PSF	
Cooridors above First Floor	80	PSF	80	PSF	
Mechanical Room - Maintenance*	40	PSF	N/A		* Equipment Weight Included in Dead Load
Stair and Exit - Ways**	100	PSF	100	PSF	** Minimum Concentrated Load in Dead Load = 300lbs
Library Stacks Areas	150	PSF	150	PSF	
File System Areas	300	PSF	300	PSF	
Roof (Ordinary, flat)	20	PSF	20	PSF	
Roof (Roof Garden)	Not Specified		100	PSF	
Assembly (auditorium, fixed seats)	Note Specified		60	PSF	

Figure 26. *Design Live Loads*

Difference in Dead Load from Calculated to RAM Model					
Level	Area (SF)	Calculated Floor Weight (kip)	RAM Model, Floor Weight (kip)	% Difference	Floor Weight Used for ETABS and Seismic Calcs (k)
Stair 3	380	37	44	17%	44
High Roof	12,071	1,021	662	-54%	661.5
Low Roof	13,748	2,544	1,352	-88%	1352
Level 4	24,275	2,638	4,581	42%	4580.8
Level 3	13,392	1,681	2,185	23%	2185.2
Level 2	25,867	3,057	3,958	23%	3958.4
Level 1	23,434	2,744	5,140	47%	5140.1

Figure 27. *Difference in Dead Load Hand Calculations from RAM Model*

Live Loads		
Level	Area (SF)	Live Load (kips)
Stair 3	64	6
High Roof	12,071	264
Low Roof	13,201	272
Level 4	24,626	4,301
Level 3	14,638	533
Level 2	28,407	1,000
Level 1	25,541	960

Figure 28. *Live Loads on Pearl Hall from RAM Model.*

3.1.4 Wind Loads

Wind loads were analyzed using the analytical procedure of ASCE 7-05 §6.5. Primary loads were calculated in the North-South, and East-West directions using Method 2- Analytical Procedure. Figure 29 lists the assumptions that were used to determine gust effect factors, wind pressures, and story shears. The following tables show calculated story forces for wind acting in the North-South direction and the East-West direction. Please refer to Appendix C for more information regarding wind analysis.

Wind Load Design Criteria	
Basic Wind Speed	90 MPH
Wind Importance Factor	IW = 1.15
Building Category	III
Exposure	C
Internal Pressure Coefficient , GC_{pi}	GCPI = 0.18
Apply Directionality Factor	Kd = 0.85
Topography Factor	Kzt = 1.00
Mean Roof Height (ft): Top Story Height + Parapet =	71.83
Fundamental Frequency, n1 = 75/H = 1.044 >1	Structure is Rigid

Figure 29. *Wind Load Design Criteria*

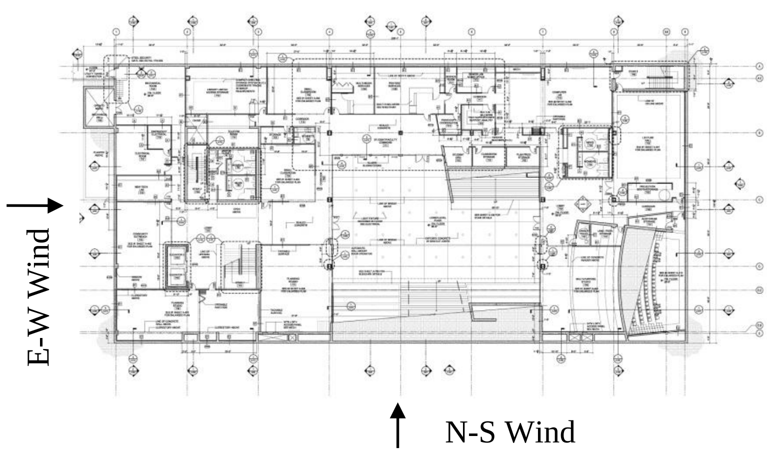


Figure 30. *Wind Directions on Pearl Hall.*
(Courtesy of Jon Anderson Architect)

According to ASCE 7-05, all wind load cases were considered. Each wind case will provide an image of the wind forces and the tabulation of results.

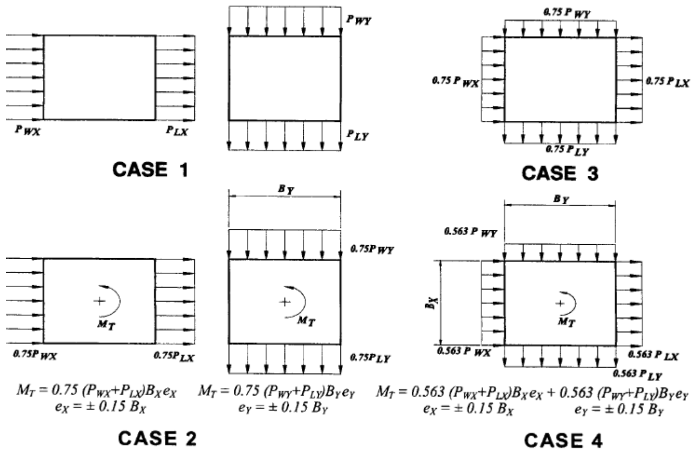


Figure 31. *ASCE 7-05 Figure 6-9 Design Wind Load Cases*

Floor	WIND 1X Story Force (k)	WIND 2X Story Force (k)	WIND 3X Story Force (k)	WIND 4X Story Force (k)
Stair 3	0.3	0.2	0.2	0.2
High Roof	7.0	5.2	5.2	3.9
Low Roof	26.0	19.5	19.5	14.6
Level 4	32.5	24.4	24.4	18.3
Level 3	30.4	22.8	22.8	17.1
Level 2	29.5	22.1	22.1	16.6
Level 1	28.3	21.2	21.2	16.0
Base	13.6	10.2	10.2	7.7
Floor	**WIND 1Y Story Force (k)**	**WIND 2Y Story Force (k)**	**WIND 3Y Story Force (k)**	**WIND 4Y Story Force (k)**
Stair 3	0.9	0.7	0.7	0.5
High Roof	22.9	17.1	17.1	13.4
Low Roof	60.4	45.3	45.3	47.4
Level 4	75.4	56.6	56.6	89.8
Level 3	71.9	53.9	53.9	130.3
Level 2	71.4	53.6	53.6	170.6
Level 1	70.3	52.7	52.7	210.1
Base	407.1	305.3	305.3	229.2

Figure 32. *Calculated Wind Loads for Cases 1 to 4*

Floor	WIND2XPE Mz (k-ft)	WIND2XNE Mz (k-ft)	WIND2YPE Mz (k-ft)	WIND2YNE Mz (k-ft)	WIND4XPYCW Mz (k-ft)	WIND4XPYCCW Mz (k-ft)	WIND4XNYCW Mz (k-ft)	WIND4XNYCCW Mz (k-ft)
Stair 3	4.5	-4.5	103.7	-103.7	81.3	74.5	-81.3	-74.5
High Roof	3922.0	-3922.0	138527.8	-138527.8	106932.4	101044.1	-106932.4	-101044.1
Low Roof	42053.8	-42053.8	379517.7	-379517.7	316459.7	253322.9	-316459.7	-253322.9
Level 4	52727.5	-52727.5	507987.4	-507987.4	420910.0	341748.4	-420910.0	-341748.4
Level 3	49235.2	-49235.2	484375.2	-484375.2	400563.6	326645.1	-400563.6	-326645.1
Level 2	47725.4	-47725.4	526589.7	-526589.7	431119.3	-431119.3	-431119.3	-359467.5
Level 1	45896.5	-45896.5	518552.8	-518552.8	423713.3	-423713.3	-423713.3	-354807.3
Base	22081.4	-22081.4	249481.9	-249481.9	203853.5	-203853.5	-203853.5	-170702.0

Figure 33. *Wind Cases 1 to 4 Torsional Moments*

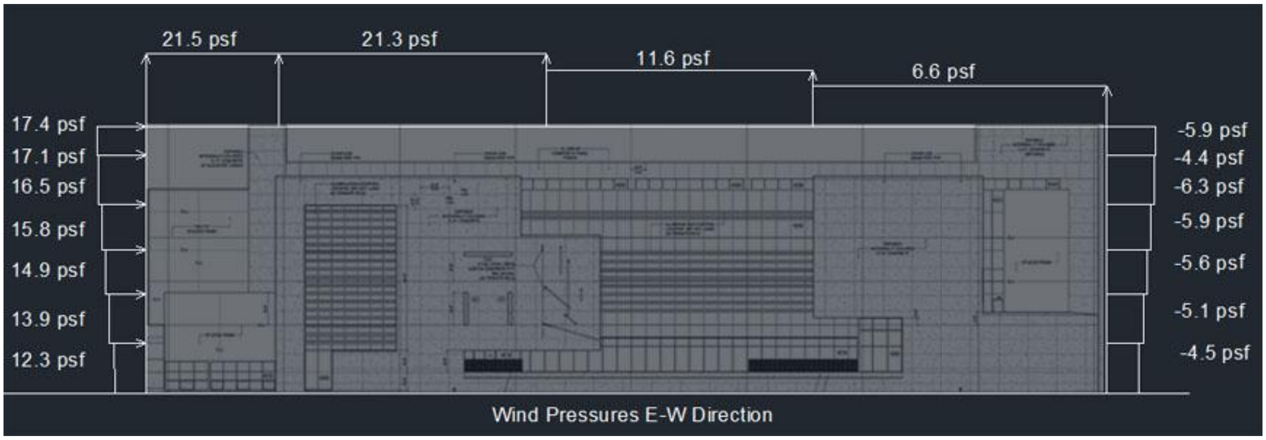


Figure 34 . *Total Base Shear from Windward Pressures in E-W Direction for Wind Case 1*

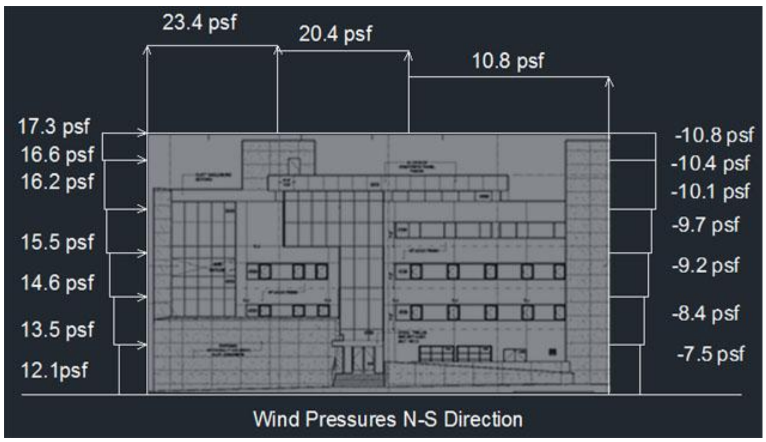


Figure 35. *Total Base Shear from Windward Pressures in E-W Direction for Wind Case 1*

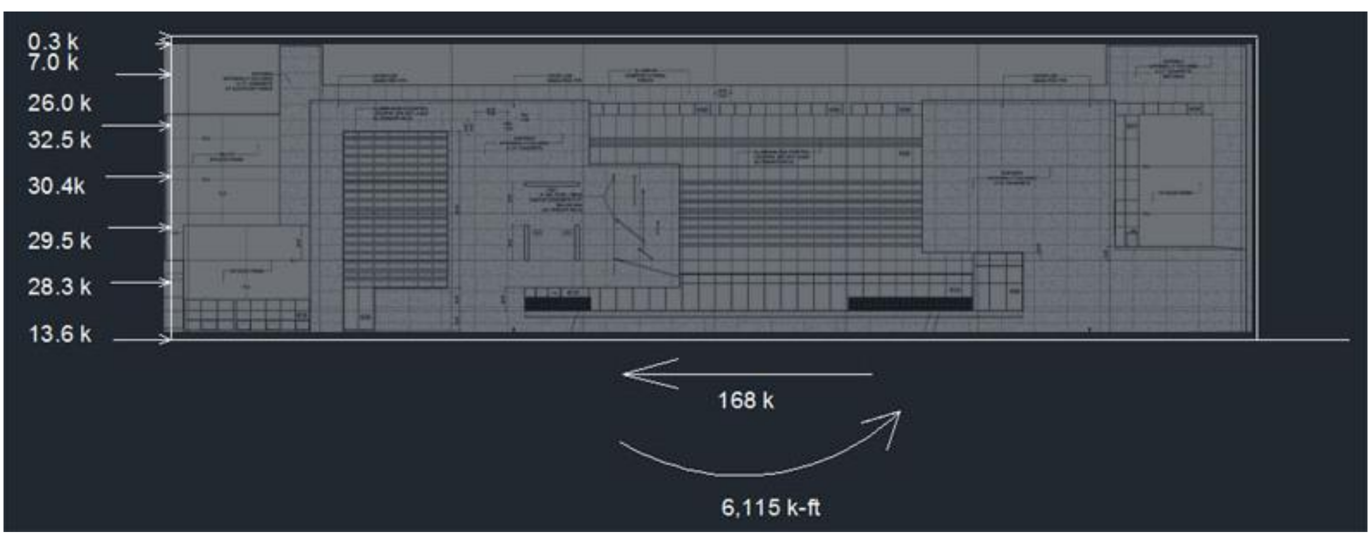


Figure 36 . *Wind Forces in E-W Direction*

Figure 37 . *Wind Forces in N-S Direction*

WIND 1X Level	% Difference of ETABS from Hand Calculations Hand Calculated Story Force (k)	ETABS Fx (k)	% Difference
Stair 3	0.3	0.3	-14.70%
High Roof	7.0	5.4	23.13%
Low Roof	26.0	21.3	18.06%
Level 4	32.5	31.0	4.66%
Level 3	30.4	29.3	3.69%
Level 2	29.5	28.9	1.97%
Level 1	28.3	28.2	0.60%
< 10%, therefore can use ETABS Calculated Wind Forces			

WIND 1Y Level	% Difference of ETABS from Hand Calculations Hand Calculated Story Force (k)	ETABS Fx (k)	% Difference
Stair 3	0.9	0.9	5.60%
High Roof	22.9	22.5	1.80%
Low Roof	60.4	60.5	-0.21%
Level 4	75.4	75.2	0.25%
Level 3	71.9	71.8	0.24%
Level 2	71.4	73.2	-2.43%
Level 1	70.3	74.2	-5.47%
< 10%, therefore can use ETABS Calculated Wind Forces			

Figure 38. *Comparison of Wind Loads from ETABS output versus Hand Calculations*

3.1.5 Seismic Loads

Pearl Hall Seismic loads were determined using ASCE 7-05 Equivalent Lateral Force Method.

Occupancy Category	III
Importance Factor (I)	1.25
Seismic Design Category	D

The following values describe the site's response to earthquake ground motion.

Mapped Spectral Response Accelerations	S_s=0.564 S_1=0.170

The site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters were determined according to ASCE 7-05 § 11.4.3.

Site Class	D

Site Class Factors	F_a=1.349 F_v=2.120
$S_{MS}=F_a(S_s)$	0.761
$S_{M1} = F_v(S_1)$	0.360

The following design spectral acceleration parameters were determined by ASCE 7-05 §11.4.4.

$S_{DS} = 2/3(S_{MS})$	0.507
$S_{D1} = 2/3(S_{M1})$	0.240

Table 2. *Modal Period for Existing Special Reinforced Shear Walls*

$T_a = C_t(h_n^x)$	0.493 s
$T_a = \frac{0.0019}{\sqrt{C_w}} h_n$	$T_{a,X}$ = 0.420 s $T_{a,Y}$ = 0.430 s
T (ETABS Calculated)	T_X = 0.295 s T_Y = 0.5243 s
$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]}$	$C_{w,X}$ = 0.11 $C_{w,Y}$ = 0.10
$C_s = \frac{S_{DS}}{(R/I)}$	0.106
$C_s = \frac{S_{D1}}{T(R/I)}, T \leq T_L$	$C_{s,X}$ = 0.106 $C_{s,Y}$ = 0.096
$C_s \geq 0.01$	OK

The main lateral force resisting system is special reinforced concrete shear walls. The base shear value was determined in accordance with Chapter 12 of ASCE 7-05. The following design values and limitations were used for the existing design. Please refer to Appendix D for detailed calculations.

Table 3. *Seismic Design Criteria for Existing Special Reinforced Shear Walls*

Special Reinforced Concrete Shear Walls	
Response Modification Factor (R)	6
Deflection Amplification Factor (C_d)	5
System Overstrength Factor (Ω_0)	2.5
Building Height Limitation	160 ft
$S_{M1} = F_v(S_1)$	0.360
Diaphragm Type	Concrete filled metal deck
Diaphragm Flexibility	Rigid
$V = C_s * W$	X: 1764 kip Y: 1594 kip

Figure 39 . *Seismic Forces in E-W Direction*

Figure 40 . *Seismic Forces in N-S Direction*

% Difference of ETABS from Hand Calculations			
Level	Hand Calculated Fx (k) = V*C_{vx}	ETABS Fx (k)	% Difference
Stair 3	6.7	6.7	0.04%
High Roof	154.8	154.7	0.04%
Low Roof	228.6	228.5	0.04%
Level 4	665.0	664.7	0.04%
Level 3	237.2	237.1	0.04%
Level 2	274.4	274.2	0.04%
Level 1	197.3	197.2	0.04%
Base Shear	1,764.0	1,763.2	0.04%
< 10%, therefore can use ETABS Calculated Seismic Forces			

% Difference of ETABS from Hand Calculations			
Level	Hand Calculated Fy (k) = V*C_{vx}	ETABS Fy (k)	% Difference
Stair 3	6.1	6.1	-0.02%
High Roof	140.8	140.76	0.02%
Low Roof	207.7	207.63	0.02%
Level 4	602.5	602.34	0.02%
Level 3	214.2	214.15	0.02%
Level 2	246.6	246.56	0.02%
Level 1	175.9	175.84	0.02%
Base Shear	1593.7	1593.37	0.02%
< 10%, therefore can use ETABS Calculated Seismic Forces			

Figure 41. *Comparison of Seismic Forces in N-S and E-W Directions*

3.1.6 Torsion Effects

Inherent Torsion

ASCE 7-05 §12.8.4.1, specifies that rigid diaphragms must consider inherent torsional moment at each level. The seismic loads are applied at the center of mass, while rigid diaphragms resist the force at the center of rigidity. If the center of mass and the center of rigidity do not align, then there will be a torsional moment around the center of rigidity. Torsion effects may have a significant impact on the controlling load case used for structural design.

Inherent Torsion in the N-S Direction with Exisiting Shear Walls					
Story	COM	COR	Eccentricity	Story Force (k)	Torsion(k-ft)
Stair 3	118.06	117.75	-0.31	8.52	-3
High Roof	61.36	74.47	13.11	123.23	1,615
Low Roof	76.78	95.40	18.62	225.73	4,202
Level 4	71.30	96.06	24.76	609.77	15,099
Level 3	61.25	96.73	35.49	219.54	7,791
Level 2	68.47	97.21	28.74	269.97	7,759
Level 1	72.03	69.68	-2.34	174.17	-408
				Total	36,055

Inherent Torsion in the E-W Direction with Exisiting Shear Walls					
Story	COM	COR	Eccentricity	Story Force (k)	Torsion(ft-k)
Stair 3	240.13	205.49	-34.64	9.68	-335
High Roof	151.41	54.05	-97.37	140.16	-13,647
Low Roof	121.09	69.61	-51.48	257.75	-13,270
Level 4	134.71	72.74	-61.97	701.91	-43,499
Level 3	130.33	75.66	-54.67	255.27	-13,956
Level 2	129.42	79.94	-49.49	318.29	-15,751
Level 1	120.01	97.77	-22.24	210.55	-4,682
				Total	-105,140

Figure 42. *Inherent Torsion in N-S and E-W Directions*

Accidental Torsion

ASCE 7-05 §12.8.4.2, specifies that rigid diaphragms must also consider accidental torsional moment for seismic loading. The displacement of the center of mass away from its actual location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces is causes accidental torsion. First the amplification factor needed to be calculated, then the accidental torsion (Figure 43 and 44).

Special Reinf. Shear WallAmplification Factor, Ao in the E-W Direction							
Story	δx	δxpe	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.17	0.14	0.17	0.31	2.4	1.9	Irregular, 1a
HGH ROOF	0.23	0.22	0.23	0.45	2.7	2.0	Irregular, 1a
LOW ROOF	0.22	0.21	0.22	0.42	2.6	1.9	Irregular, 1a
STORY4	0.17	0.16	0.17	0.34	2.6	2.0	Irregular, 1a
STORY3	0.12	0.11	0.12	0.23	2.6	2.0	Irregular, 1a
STORY2	0.07	0.07	0.07	0.13	2.6	1.9	Irregular, 1a
STORY1	0.01	0.01	0.01	0.02	2.7	2.0	Irregular, 1a
Special Reinf. Shear Wall,Amplification Factor, Ao in the N-S Direction							
Story	δy	δype	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.63	0.69	0.63	1.31	3.1	2.1	Irregular, 1a
HGH ROOF	0.72	0.80	0.72	1.52	3.1	2.1	Irregular, 1a
LOW ROOF	0.13	0.16	0.13	0.29	3.2	2.2	Irregular, 1a
STORY4	0.39	0.43	0.39	0.82	3.1	2.1	Irregular, 1a
STORY3	0.25	0.28	0.25	0.53	3.1	2.1	Irregular, 1a
STORY2	0.04	0.05	0.04	0.09	3.3	2.2	Irregular, 1a
STORY1	0.02	0.02	0.02	0.03	3.0	2.1	Irregular, 1a

Figure 43. *Amplification Factor in N-S and E-W Directions*

Accidental Torsion in the E-W Direction with Exisiting Shear Walls					
Story	Bx (ft)	%5 By (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	12.00	0.60	2.10	9.68	12
High Roof	70.71	3.54	2.63	140.22	368
Low Roof	120.00	6.00	2.20	257.86	568
Level 4	120.00	6.00	2.61	702.21	1,836
Level 3	120.00	6.00	2.62	255.38	669
Level 2	120.00	6.00	2.60	318.42	829
Level 1	120.00	6.00	2.72	210.64	574
				Total	4,856

Accidental Torsion in the N-S Direction with Exisiting Shear Walls					
Story	By(ft)	%5 By (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	32.00	1.60	3.10	8.52	42
High Roof	232.08	11.60	3.11	123.23	383
Low Roof	236.34	11.82	3.12	225.73	705
Level 4	244.67	12.23	3.12	609.77	1,902
Level 3	244.67	12.23	3.12	219.54	685
Level 2	256.00	12.80	3.12	269.97	843
Level 1	256.00	12.80	3.02	174.17	526
				Total	5,087

Figure 44. *Accidental Torsion in N-S and E-W Directions*

3.1.7 Structural Irregularities

ASCE 7-05 §12.3 specifies limitations for diaphragm flexibilities and also determines the structural irregularities the building for the horizontal and the vertical planes of the building.

Horizontal structural irregularities were determined according to ASCE 7-05 §12.3.2.1. The descriptions of the horizontal irregularities are listed in ASCE 7-05 Table 12.3-1. Table 4 discusses each irregularity type for Pearl Hall. Since, the building does not have horizontal irregularity type 5, then the design of the seismic forces are permitted to be applied independently in each of the two orthogonal directions

Table 4. *Horizontal Structural Irregularities*

Horizontal Structural Irregularities			
Type	**Irregularity**	**Comment**	**Status**
1a	Torsional	See Appendix D. **Design forces for lateral force connections to be increased 25% in Design Categories D.**	**Not Good**
2	Reentrant Corner	This irregularity does exist. See Appendix C.	**Not Good**
3	Diaphragm Discontinuity	Irregularity does exist. See Appendix D. **Design forces for lateral force connections to be increased 25% in Design Categories D.**	**Not Good**
4	Out of plane Offsets	No vertical element out of plane offsets exists.	Good
5	Non Parallel System	All lateral force resisting systems are parallel to the orthogonal axes.	Good

Vertical structural irregularities determined according to ASCE 7-05 §12.3.2.2. The descriptions of the vertical irregularities are listed in ASCE 7-05 Table 12.3-2. Table 5 discusses each irregularity type for Pearl Hall.

Table 5. *Vertical Structural Irregularities*

Vertical Structural Irregularities			
Type	**Irregularity**	**Comment**	**Status**
1a	Stiffness-Soft Story	See Appendix D.	**Not Good** (Level 4 to 1)
2	Weight (Mass)	The library on Level 4 causes more than 1.5 story weight of Level 3.	Not Good
3	Vertical Geometric	Each shear wall is rectangular in elevation.	Good
4	In-Plane Discontinuity of Vertical Lateral Force Resisting Element	Each shear wall is continuous.	Good
5a,b	Discontinuity in Lateral Strength	14 out of 16 shear walls have no to small openings.	Good

According to ASCE 7-05 §12.3.3.4, the seismic forces need to be increased due to irregularities for Seismic Design Categories. Since Pearl Hall has a horizontal structural irregularity of Type 1a, the design forces determined from Section 12.8.1 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. In addition, modal response spectrum analysis is required.

3.1.8 Modal Response Spectrum Analysis

Pearl Hall is located in Seismic Design Category D. It is a Category III structure and it is less than 160 ft high. Since Pearl Hall has Vertical Irregularity 1a and Horizontal Irregularity1a, ASCE 7-05 specifies that modal response spectrum analysis is required for obtaining design forces.

ASCE 7-05 §12.9 requires an analysis of the number of modes, modal response parameters, combines response parameters, scaling design values of combined response, horizontal shear distribution, p-delta effects, and soil structure interaction reduction. Table 6 describes the additional analysis for design.

Table 6. *Modal Response Spectrum Analysis for Existing Special Reinforced Shear Walls*

Number of Modes	15 modes
Modal Response Parameters	The value for each force related design parameter of interest, including story drifts, support forces, and individual member forces for each mode of response shall be computed using the properties of each mode and the response spectra defined in either Section 11.4.5 or 21.2 divided by the quantity RI . The value for displacement and drift quantities shall be multiplied by the quantity Cd/I .

Combined Response Parameters	SRSS
Scaling Design Values of Combined Response	Scaled Member Force = 0.85*(Vbase/Vt)*Member Force
Horizontal Shear Distribution	The distribution of horizontal shear shall be in accordance with the requirements of Section 12.8.4 except that amplification of torsion per Section 12.8.4.3 is not required where accidental torsional effects are included in the dynamic analysis model.
P-Delta Effects	The P-delta effects shall be determined in accordance with Section 12.8.7. The base shear used to determine the story shears and the story drifts shall be determined in accordance with Section 12.8.6

In order to specify the response spectrum scale, the scale factor shall be g*I/R, where g is acceleration due to gravity (use 386.4 in/sec^2) for models in kips-inch units. After analysis is performed, review the Response Spectrum Base Reaction for seismic in the x and y directions. If reported dynamic base shear is more than 85% of the static base shear then no further action is required. However, when dynamic base shear is less than 85% of static base shear then readjust the scale factor to match the response spectrum base shear equal to 85% of static base shear (Figure 45). So, the new scale factor = (g*I/R) * 0.85*static base shear /response spectrum base shear. Then, use this readjusted scale factor in response spectrum case and rerun the analysis. Then, create a load case for 1.2Dead + 1.0 Live + 1.0 Modal.

Modal Response Spectrum Analysis - SF							
Shear Walls	**SF**	**V**	**Vt**	**SF**	**Vt**	**0.85*V**	**Vt > 0.85*Vt**
x	6.7083	1702.2	69.9	138.9	1447.0	1446.9	ok
y	6.7083	1539.0	52.6	166.737	1308.3	1308.1	ok

Figure 45. *Modal Spectrum Response Scale Factor*

3.1.9 Load Combinations

The load combinations considered when generating the model of the lateral system in ETABS are listed below. All of these combinations are based on LRFD design method.

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

3.1.10 Controlling Load Case

Using load combinations as well as torsional effects from lateral loads, it was concluded that seismic loading controls the structural design of Pearl Hall. This was expected since the base shear in the North-South direction for seismic loads was approximately 1631 kips as opposed to a base shear of 407 kips for wind in the North-South direction. Figure 46 visualizes the magnitude that the factored seismic loads compare to wind. The controlling LRFD load combination for this structure is 1.2 (Dead) + 1.0 (Seismic) + 1.0 (Live).

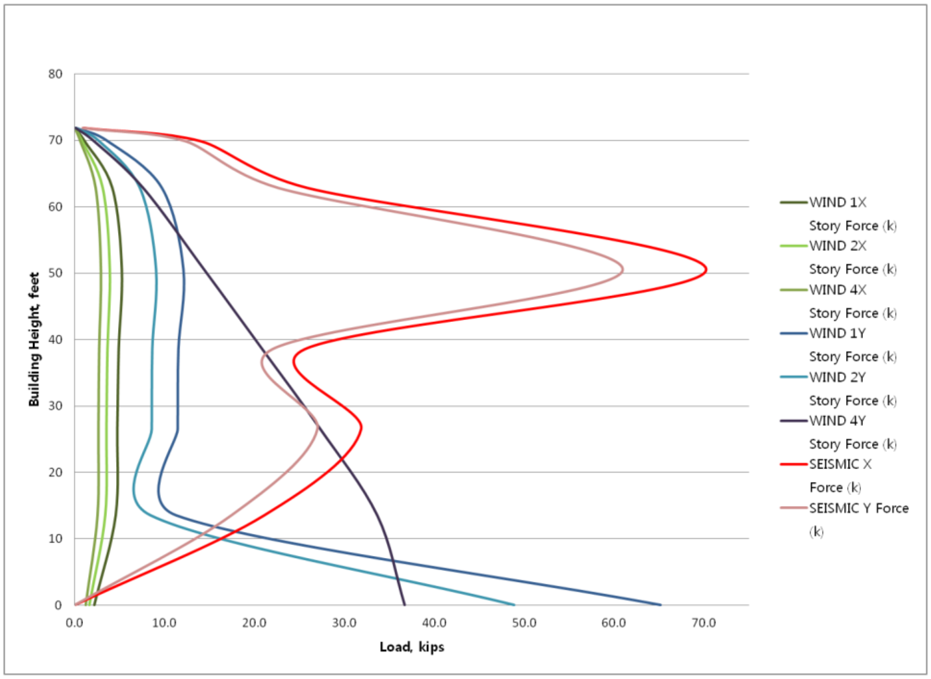


Figure 46. *Factored Wind and Seismic Loads*

It was determined that seismic forces would control over wind loads in Pearl Hall. By hand calculations, the overturning moment and shear forces of the building were significantly higher for seismic loads than wind loads. Earthquake effects, in comparison to wind loads, are generally considered to be primarily a strength issue, rather than a serviceability issue. The existing system is adequate to support the lateral loads.

Exisiting Shear Walls - Determination of Controlling Load				
LEVEL	**Load**	**Wall 13*, Vmax (k)**	**UX (in)**	**UY (in)**
LEVEL 2	SEISMICX	-3.1	0.0592	0.0148
LEVEL 2	SEISMICXNE	-21.4	**0.0607**	0.0176
LEVEL 2	SEISMICXPE	-14.9	0.0577	0.012
LEVEL 2	SEISMICY	188.4	0.0142	0.0829
LEVEL 2	SEISMICYNE	**194.7**	0.0114	0.0778
LEVEL 2	SEISMICYPE	183.0	0.0169	**0.0881**
LEVEL 2	WIND1X	-1.6	0.0045	0.0013
LEVEL 2	WIND1Y	40.2	0.0029	0.0173
LEVEL 2	WIND2XNE	-1.8	0.0037	0.0015
LEVEL 2	WIND2XPE	-0.8	0.0032	0.0006
LEVEL 2	WIND2YNE	32.7	0.0009	0.0106
LEVEL 2	WIND2YPE	27.4	0.0035	0.0154
LEVEL 2	WIND3XNY	-31.4	0.0012	-0.012
LEVEL 2	WIND3XPY	28.9	0.0056	0.0139
LEVEL 2	WIND4XNYCCW	**-68.0**	0.0013	-0.0181
LEVEL 2	WIND4XNYCW	-56.3	-0.0037	-0.0273
LEVEL 2	WIND4XPYCCW	66.0	0.0039	0.0197
LEVEL 2	WIND4XPYCW	66.0	0.0039	0.0197
*Section Cut at Level 2, Left Side, 1				

Figure 47. *Determination of Controlling Load Case*

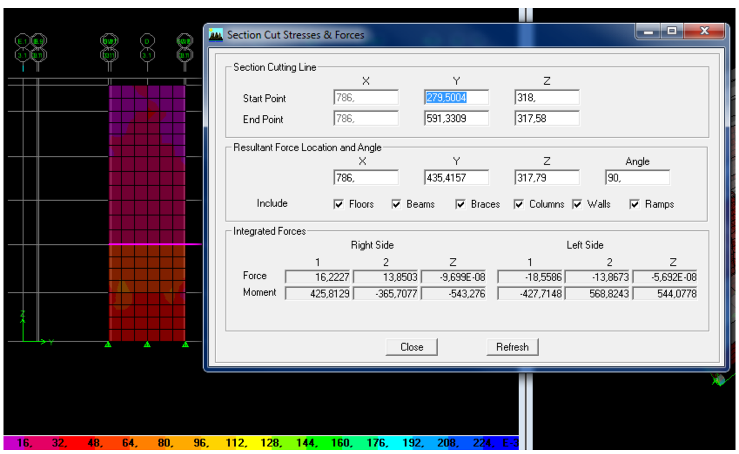


Figure 48. *ETABS Output for Maximum Shear in Wall 13*

Since Pearl Hall is controlled by seismic forces, a comparison was preformed of the hand calculated story forces and shears to that output from ETABS.

Seismic Deisgn E-W Direction (X)						
	Story Loads			Story Shears		
Level	**Hand Calculated Fx (k) = V*C_{vx}**	**ETABS Fx (k)**	**% Difference**	**Hand Calculated V(k)**	**ETABS Vx (k)**	**% Difference**
Stair 3	6.7	6.7	0.04%	6.7	6.7	0.00%
High Roof	154.8	154.7	0.04%	161.5	161.4	0.04%
Low Roof	228.6	228.5	0.04%	390.1	390.0	0.04%
Level 4	665.0	664.7	0.04%	1,055.1	1,054.7	0.04%
Level 3	237.2	237.1	0.04%	1,292.3	1,291.8	0.04%
Level 2	274.4	274.2	0.04%	1,566.7	1,566.1	0.04%
Level 1	197.3	197.2	0.04%	1,764.0	1,763.2	0.04%
Base Shear (k)	1764	1763	0.04%			
Overturning Moment (ft-k)	78,304	78,304	0.00%			
Level	**Hand Calculated Fy (k) = V*C_{vy}**	**ETABS Fy (k)**	**% Difference**	**Hand Calculated V(k)**	**ETABS Vy(k)**	**% Difference**
Stair 3	6.1	6.1	-0.02%	6.1	6.1	-0.02%
High Roof	140.8	140.8	0.02%	146.9	146.9	0.02%
Low Roof	207.7	207.6	0.02%	354.6	354.5	0.02%
Level 4	602.5	602.3	0.02%	957.1	956.8	0.02%
Level 3	214.2	214.2	0.02%	1,171.2	1,171.0	0.02%
Level 2	246.6	246.6	0.02%	1,417.9	1,417.5	0.02%
Level 1	175.9	175.8	0.02%	1,593.7	1,593.4	0.02%
Base Shear (k)	1594	1594	0.00%			
Overturning Moment (ft-k)	70,900	70,900	0.00%			

Figure 49 . E-W *and N-S Directions Calculated Seismic Forces and Shear*

3.1.11 Serviceability

Drift is a serviceability requirement that is addressed in ASCE 7-05. Seismic drift is limitations are based on the occupancy category and normally would be limited to an allowable story drift of 0.015*height. Story drifts for seismic loading were determined in ETABS and compared to drift limitations in Figure 50. Due to irregularity, the amplified drift must be compared with the allowable drift value.

$$\delta x = \delta xe * Cd/I \text{ (Amplified Drift)}$$

Special Reinforced Shear Wall - Seismic Drift X Direction						
Story	**Story Height (ft)**	**ETABS Displacement (in)**	**ETABS δxe (in)**	**δx (in) = δxe*Cd/I**	**Δallowable(in) = 0.015hx**	
Stair 3	71.83	0.162	0.045	0.179	0.36	ok
High Roof	69.83	0.206	0.032	0.129	1.26	ok
Low Roof	62.83	0.174	0.031	0.124	2.22	ok
Level 4	50.5	0.143	0.041	0.164	2.16	ok
Level 3	38.5	0.102	0.048	0.190	2.16	ok
Level 2	26.5	0.055	0.047	0.186	2.34	ok
Level 1	13.5	0.008	0.008	0.032	2.43	ok

Special Reinforced Shear Wall - Seismic Drift Y Direction						
Story	**Story Height (ft)**	**ETABS Displacement (in)**	**ETABS δxe (in)**	**δx (in) = δxe*Cd/I**	**Δallowable(in) = 0.015hx**	
Stair 3	71.83	0.594	0.146	0.585	0.36	not ok
High Roof	69.83	0.447	0.155	0.620	1.26	ok
Low Roof	62.83	0.292	0.054	0.217	2.22	ok
Level 4	50.5	0.238	0.085	0.341	2.16	ok
Level 3	38.5	0.153	0.074	0.295	2.16	ok
Level 2	26.5	0.079	0.066	0.264	2.34	ok
Level 1	13.5	0.013	0.013	0.052	2.43	ok

Figure 50. *Actual Seismic Drift and Amplified Drift vs. Code Limitations*

3.1.12 Existing Design Check Summary

The following table provides a summary of the analysis of the existing lateral system.

Table 7. *Summary of the analysis of the existing lateral system.*

Check	**Comment**	**Status**
Modal Period ASCE 7-05 Approximate period = 0.493 s RAM model period = 0.5566 s ETABS model period = 0.5556s	The ETABS model period is higher than the approximate period it can be concluded that the structure is not overdesigned.	OK
Torsion **Inherent and accidental torsion**	Torsion Inherent and accidental torsion were both taken into account in the ETABS Model	NOT OK
Redundancy	Structure is assigned to SDC D, therefore value for ρ is allowed to be taken as 1 per ASCE 7-05 § 12.3.4.1	OK
Member Spot Checks	Member sizes meet strength requirements. Refer to Appendix A for detailed calculations.	OK
Story Drift	Drift requirements are met in both orthogonal directions OK	OK

3.2 Existing Lateral System Problem Statement

3.2.1 Problem Statement

The existing design of Pearl Hall has an adequate structural design. The metal deck over open steel joist and steel beams supported by steel girder beams provided the most economical gravity system. In addition, the four wide flange steel trusses span 96 feet, in order to provide a column free breezeway at the ground level. Since Pearl Hall is located in Seismic Design Category D, the seismic loads controlled for the lateral system design. In addition, modal response spectrum analysis had to be performed due to torsional and stiffness-soft story irregularities. Therefore, the seismic base shear forces had to be scaled for 85% of the ratio of seismic base shear over seismic base shear for modal analysis.

The lateral system provided adequate strength for the seismic loads in both orthogonal directions. Serviceability requirements were met for allowable drift requirements. Yet, drift values had a small magnitude indicating a very rigid structure. The allowable drift was approximately 10 times larger than the actual amplified drift. Therefore, it was desirable to redesign the lateral force resisting system to be more economical and meet strength and serviceability requirements.

It was discussed with the structural engineer on record about the alternative lateral force resisting system redesign. According to the structural engineer on record, Pearl Hall was originally designed to have braced frames as the lateral system. Due, to design criteria from the architect, it was decided that concrete shear walls were more aesthetic and suitable for the design.

3.2.2 Problem Solution

Two solutions have been proposed for comparison. First, a seismic analysis will be performed using the existing reinforced concrete shears walls, to be identified as lateral system #1. Some existing walls will be used and some will be eliminated, in order to decrease material costs. Secondly, a seismic analysis using a braced frame system identified as lateral system #2. Since the building, uses steel trusses, beams, girders, columns, and metal decking it would be interesting to evaluate the performance and economics of a complete steel building.

Due to budget fluctuations and programmatic changes, the there was a tight budget for the construction of Pearl Hall. Therefore, a cost analysis of each system would be completed to find the most cost effective system.

The goal of this structural depth is to design a lateral system that would impose the least cost for Pearl Hall. Yet, due to the structural irregularities of Pearl Hall, all designs must use modal analysis. Therefore, they will all be designed to meet strength and serviceability requirements for the scaled response spectrum.

3.3 Lateral Force Resisting Redesign System #1

3.3.1 ETABS Modeling

The ETABS model was used to design the modified shear wall lateral system (Figure 63). The ETABS output of shear, axial, and moment values were used during the design check and reinforcement design of the shear walls. In addition, PCAColumn was used to check the design reinforcement in the shear walls by the Axial vs. Moment interaction diagrams.

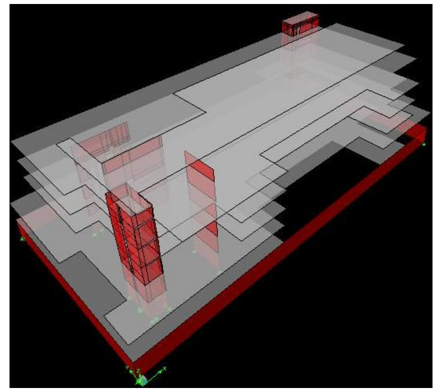


Figure 63. *ETABS Modified Special Reinforced Shear Wall Design*

According to ASCE 7-05 Sect. 12.7.3 concrete elements should consider effects of cracked sections. ACI 318-08 permits the use of 50% stiffness values based on gross section. Therefore, the walls are models using area elements setting $f_{22} = 0.5$. Due to the structural irregularities of Pearl Hall, all designs must use modal analysis. Therefore, they will all be designed to meet strength and serviceability requirements for the scaled response spectrum.

The modified design eliminated walls 3 and 4 from the previous design (Figure 53).

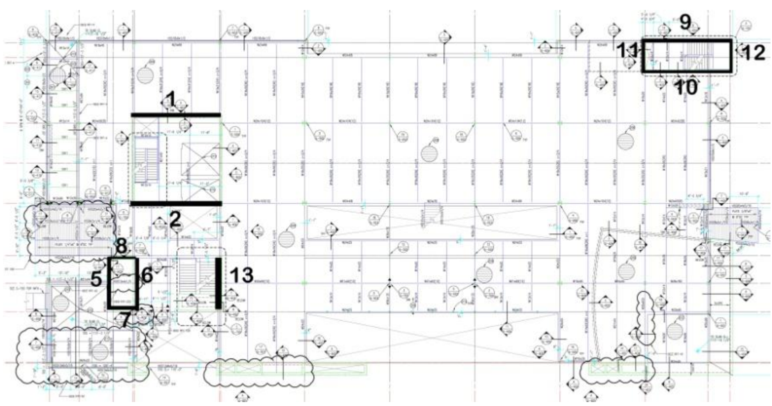


Figure 53. *Modified Special Reinforced Shear Wall Layout. Modified by N. Trujillo. (Courtesy of Jon Anderson Architect)*

3.3.2 Center of Rigidity

Centers of Mass & Centers of Rigidity						
	Hand Calculations*		ETABS		% Difference from ETABS	
Story	XCR (ft)	YCR (ft)	XCR (ft)	YCR (ft)	XCR	YCR
STAIR 3	240.00	117.84	203.8	117.8	-18%	0%
HGH ROOF	93.23	91.23	75.7	81.3	-23%	-12%
LOW ROOF	83.76	94.62	100.6	101.1	17%	6%
STORY4	84.25	94.53	101.1	100.5	17%	6%
STORY3	84.25	91.30	100.5	99.1	16%	8%
STORY2	82.80	94.79	96.7	96.9	14%	2%
STORY1	111.33	66.66	105.7	69.7	-5%	4%
* Assume that the general area of wall is rectangular						

Figure 54. *Comparison of the Center of Mass and Rigidity of ETABS versus Hand Calculations*

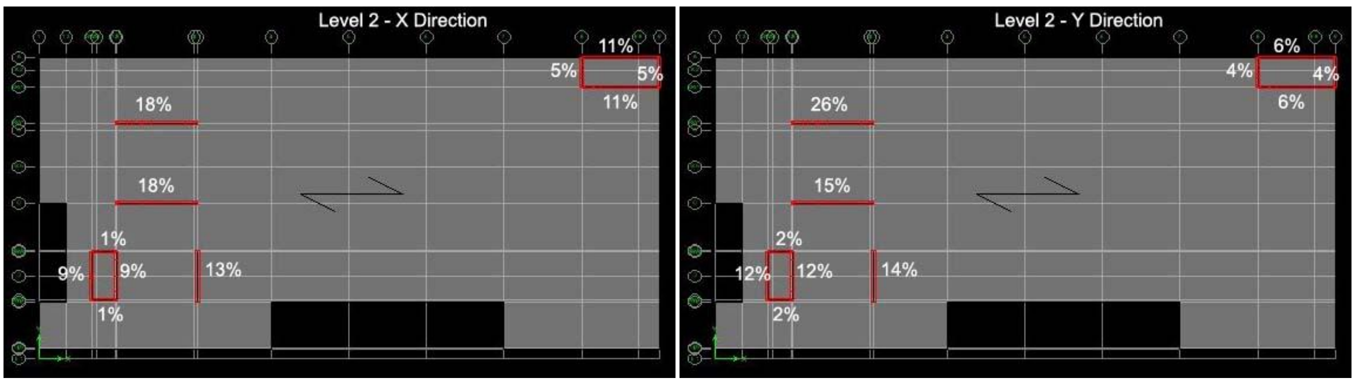


Figure 55. *Relative Stiffness of Walls on Level 2 in both the X and Y Directions.*

3.3.3 Seismic Loads

Pearl Hall Seismic loads were initially determined using ASCE 7-05 Equivalent Lateral Force Method. The site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters were determined according to ASCE 7-05 § 11.4.3. The design spectral acceleration parameters were determined by ASCE 7-05 §11.4.4. The main lateral force resisting system proposed for redesign is special reinforced concrete shear walls. The base shear value was determined in accordance with Chapter 12 of ASCE 7-05. The following design values and limitations were used for the existing design. Please refer to Appendix D for detailed calculations.

Table 8. *Seismic Design Criteria for Modified Special Reinforced Shear Walls*

Occupancy Category	III
Importance Factor (I)	1.25
Seismic Design Category	D
Site Class	D
Site Class Factors	F_a=1.349 F_v=2.120
$S_{MS}=F_a(S_s)$	0.761
$S_{M1} = F_v(S_1)$	0.360
$S_{DS} = 2/3(S_{MS})$	0.507
$S_{D1} = 2/3(S_{M1})$	0.240
C_t	0.020
C_s	X: 0.106 Y: 0.106
Response Modification Factor (R)	6 (Special Reinforced Concrete Shear Walls)
Deflection Amplification Factor (C_d)	5
System Overstrength Factor (Ω_0)	2.5
Building Height Limitation	160 ft
Diaphragm Type	Concrete filled metal deck
Diaphragm Flexibility	Rigid
Vx= C_{s*}W	1792 kip
Vy= C_{s*}W	1792 kip

% Difference of ETABS from Hand Calculations			
Level	Hand Calculated Fx (k) = V*C_{vx}	ETABS Fx (k)	% Difference
Stair 3	6.9	6.7	2.89%
High Roof	157.6	153.1	2.89%
Low Roof	228.5	221.9	2.89%
Level 4	679.6	659.9	2.89%
Level 3	240.1	233.2	2.89%
Level 2	278.8	270.7	2.89%
Level 1	200.8	195.0	2.89%
Base Shear	1,792.3	1,740.5	2.89%
< 10%, therefore can use ETABS Calculated Seismic Forces			

% Difference of ETABS from Hand Calculations			
Level	Hand Calculated Fx (k) = V*C_{vx}	ETABS Fy (k)	% Difference
Stair 3	6.9	6.7	2.89%
High Roof	157.6	153.1	2.89%
Low Roof	228.5	221.9	2.89%
Level 4	679.6	659.9	2.89%
Level 3	240.1	233.2	2.89%
Level 2	278.8	270.7	2.89%
Level 1	200.8	195.0	2.89%
Base Shear	1792.3	1740.53	2.89%
< 10%, therefore can use ETABS Calculated Seismic Forces			

Figure 56. *Comparison of Seismic Forces in N-S and E-W Directions*

3.3.4 Modal Response Spectrum Analysis

In order to specify the response spectrum scale, the scale factor shall be g*I/R, where g is acceleration due to gravity (use 386.4 in/sec2 for models in kips-inch units. After analysis is performed, review the Response Spectrum Base Reaction for seismic in the x and y directions. If reported dynamic base shear is more than 85% of the static base shear then no further action is required. However, when dynamic base shear is less than 85% of static base shear then readjust the scale factor to match the response spectrum base shear equal to 85% of static base shear (Figure 57). So, the new scale factor = (g*I/R) * 0.85*static base shear /response spectrum base shear. Then, use this readjusted scale factor in response spectrum case and rerun the analysis. Then, create a load case for 1.2Dead + 1.0 Live + 1.0 Modal.

Modal Response Spectrum Analysis - SF							
Shear Walls	SF	V	Vt	SF	Vt	0.85*V	Vt > 0.85*Vt
x	6.7083	1740.5	795.8	12.4708	1740.1	1479.5	ok
y	6.7083	1683.1	762.6	12.5846	1683.1	1430.6	ok

Figure 57. *Modal Spectrum Response Scale Factor*

3.3.5 Torsion Effects

Torsion creates additional shear in walls. Therefore, many frames will be controlled by shear versus flexure.

Inherent Torsion in the N-S Direction with Shear Wall Design #2					
Story	COM	COR	Eccentricity	Story Force (k)	Torsion(k-ft)
Stair 3	118.5	117.82	-0.68	6.92	-5
High Roof	60.8	81.25	20.50	157.60	3,231
Low Roof	74.7	101.11	26.36	228.52	6,024
Level 4	70.8	100.51	29.67	679.56	20,162
Level 3	61.2	99.11	37.88	240.15	9,098
Level 2	68.0	96.88	28.91	278.79	8,059
Level 1	71.3	69.71	-1.55	200.78	-310
				Total	46,259

Inherent Torsion in the E-W Direction with Exisiting Shear Walls					
Story	COM	COR	Eccentricity	Story Force (k)	Torsion(ft-k)
Stair 3	238.8	203.82	-35.00	6.92	-242
High Roof	147.2	75.70	-71.49	157.60	-11,267
Low Roof	118.0	100.62	-17.38	228.52	-3,971
Level 4	132.6	101.11	-31.51	679.56	-21,414
Level 3	126.1	100.48	-25.61	240.15	-6,149
Level 2	126.8	96.75	-30.04	278.79	-8,375
Level 1	118.1	105.66	-12.45	200.78	-2,500
				Total	-53,918

Figure 58. *Inherent Torsion in N-S and E-W Directions*

Accidental Torsion in the N-S Direction with Shear Wall Design #2					
Story	By(ft)	%5 By (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	32.00	1.60	1.00	6.92	11
High Roof	232.08	11.60	1.00	157.60	158
Low Roof	236.34	11.82	1.00	228.52	229
Level 4	244.67	12.23	1.00	679.56	680
Level 3	244.67	12.23	1.00	240.15	240
Level 2	256.00	12.80	1.00	278.79	279
Level 1	256.00	12.80	1.00	200.78	201
				Total	1,796

Accidental Torsion in the E-W Direction with Exisiting Shear Walls					
Story	Bx (ft)	%5 Bx (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	12.00	0.60	2.79	6.92	12
High Roof	70.71	3.54	2.64	157.60	415
Low Roof	120.00	6.00	2.62	228.52	600
Level 4	120.00	6.00	2.63	679.56	1,784
Level 3	120.00	6.00	2.63	240.15	631
Level 2	120.00	6.00	2.61	278.79	729
Level 1	120.00	6.00	2.71	200.78	544
				Total	4,715

Figure 59. *Accidental Torsion in N-S and E-W Directions*

3.3.6 Serviceability

Drift is a serviceability requirement that is addressed in ASCE 7-05. Seismic drift is limitations are based on the occupancy category and normally would be limited to an allowable story drift of 0.015*height. Story drifts for seismic loading were determined in ETABS and compared to drift limitations in Figure 50. Due to irregularity, the amplified drift must be compared with the allowable drift value.

$$\delta x = \delta xe * Cd/I \text{ (Amplified Drift)}$$

Modified Special Reinforced Shear Wall - Seismic Drift X Direction						
Story	Story Height (ft)	ETABS Displacement (in)	ETABS δxe (in)	δx (in) = δxe*Cd/I	Δallowable(in) = 0.015hx	
Stair 3	71.83	0.161	0.065	0.260	0.28	ok
High Roof	69.83	0.226	0.047	0.186	0.97	ok
Low Roof	62.83	0.179	0.042	0.170	1.71	ok
Level 4	50.5	0.137	0.045	0.179	1.66	ok
Level 3	38.5	0.092	0.049	0.198	1.66	ok
Level 2	26.5	0.043	0.039	0.156	1.80	ok
Level 1	13.5	0.004	0.004	0.016	1.87	ok

Modified Special Reinforced Shear Wall - Seismic Drift Y Direction						
Story	Story Height (ft)	ETABS Displacement (in)	ETABS δxe (in)	δx (in) = δxe*Cd/I	Δallowable(in) = 0.015hx	
Stair 3	71.83	0.545	0.042	0.170	0.28	ok
High Roof	69.83	0.503	0.123	0.491	0.97	ok
Low Roof	62.83	0.380	0.094	0.377	1.71	ok
Level 4	50.5	0.286	0.108	0.432	1.66	ok
Level 3	38.5	0.177	0.094	0.376	1.66	ok
Level 2	26.5	0.083	0.074	0.294	1.80	ok
Level 1	13.5	0.010	0.010	0.039	1.87	ok

Figure 60. *Actual Seismic Drift and Amplified Drift vs. Code Limitations*

3.3.7 Strength Check Modified Shear Wall Layout

Seismic Loads control for Pearl Hall. Therefore all lateral force resisting systems redesigns were designed for seismic loads and scaled for modal response spectrum analysis. In addition, since Pearl Hall is located in Seismic Design Category D, Special Reinforced Shear Walls are required. Using FEMA 451, the modified shear walls were designed using of existing reinforcement (Figure 61). It was determined that the Walls 1, 2, and 5 would need to increase the thickness of the walls in order to provide enough shear resistance. The walls were previously 12 in. thick and were increased to 18 in. thick. Please see Appendix E for all detailed calculations. Pearl Hall meets all serviceability criteria for this design.

INPUT DATA & DESIGN SUMMARY **Wall 5 b**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	240.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	4320	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			546.4	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	463.0	k
FACTORED MOMENT LOAD	M_u	=	17087.8	ft-k
FACTORED SHEAR LOAD	V_u	=	405.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	405.0	kip
Vn = Vc + Vs	d	=	192.0	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2185.8	kip
	ΦVn	=	1639.3	kip
ACI 318-08 § 21.9.4	Vn	≤	36720.0	kip
$V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	437.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	273.2	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	163.9	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	102.8	kip	
S = (Av*fy*d)/Vs	Ag	=	4320	in^2	
	0.0025*Ag	=	10.8	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 80	S	=	8.00	in	USE
S ≤ 3t = 54	Av	=	4320.00		
S ≤ 18" Governs	# bars required	=	14		
ρt = Av/(S*t)	ρt	=	0.0750	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.09584	>0.0025	OK
Max. Spacing S ≤ L/3 = 80	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" Governs					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	7		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.9250	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	14	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	7	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	172.80	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	21.98	in^2
	a	=	21.54	in
jd = d - (a/2)	jd	=	181.23	in
	As	=	20.95	in^2

	A/bar	=	1.56	in^2
	# bars required	=	13	
TRY #11				
Check Capacity:	a	=	20.54	in
C=T 0.85*f'c*a*b =As*fy	c	=	24.17	in
c = a/0.85	εt	=	0.03	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 24.17 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 65.9 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0750 > $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.0958 > $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 1639.3 kips > V_u = 405.0 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 205,054 kip-ft > M_u = 17,088 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	14	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	16	#11	@	6	" O.C.

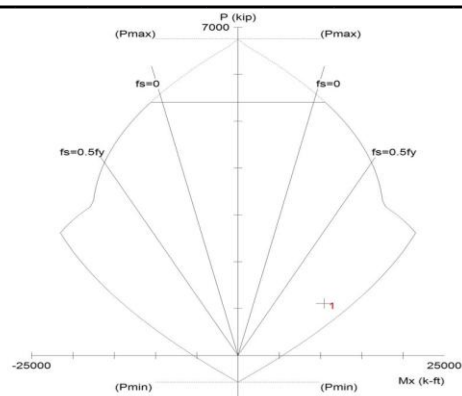


Figure 61. *Design of Reinforcement for Wall 5a.*

3.4 Lateral Force Resisting Redesign System #2

3.4.1 Introduction

Pearl Hall is primarily a steel building. Therefore, it was desired to design the building with either braced frames or moment frames. It was decided that inverted V braces would be used to the redesign of system #2.

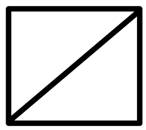
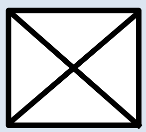
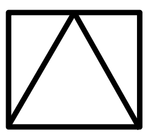
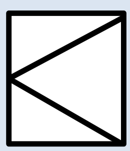
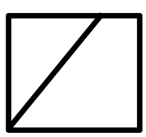
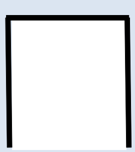
Lateral Bracing System	Advantages	Disadvantages
Diagonal Brace 	~Larger Unbraced Length for Brace ~Best placed against wall	~Larger members/sections required ~More obstruction of circulation within building
X-Brace 	~Small members/sections required ~Braced at all four corners ~Best placed against wall	~More connections required, which add cost for material and labor ~More obstruction of circulation within building
Inverted V Brace 	~Small members/sections required ~Less obstruction of circulation within building	~More design requirements; shear transfer at midpoint of beam
K- Brace 	~Small members/sections required ~Best placed against wall	~AISC 341-05 does not allow use for special seismic design ~More obstruction of circulation within building
Ecentric Brace 	~Less obstruction of circulation within building	~Larger members/sections required ~More design requirements; eccentric force effects
Moment Frame 	~Provide the most flexible floorplan	~More expensive because of connections and larger member sizes

Figure 62. *Lateral Bracing Comparisons*

3.4.2 ETABS Modeling of Special Concentric Braced Frame

Since the braced frames could impede current circulation through Pearl Hall. The frames were placed in the same location as the shear walls. The ETABS model was created by releasing moments in all beams and braces. In addition, the seismic design took into consideration P-delta effects as well as modal analysis. The lateral bracing of beams and special seismic compact section criteria had to be met according to AISC 340-05.

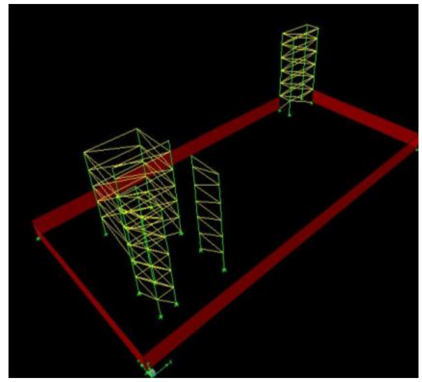


Figure 63. *ETABS SCBFDesign*

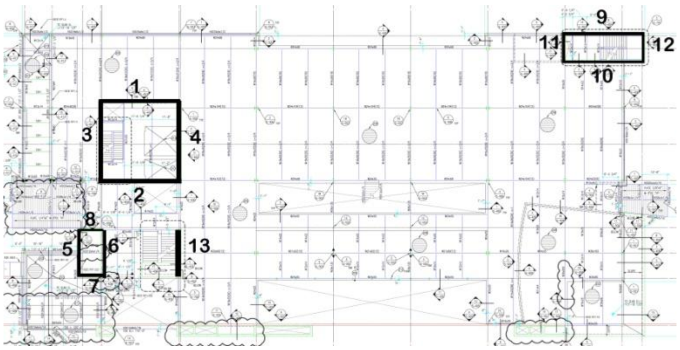


Figure 64 . *SCBF Layout. Modified by N. Trujillo. (Courtesy of Jon Anderson Architect)*

3.4.3 Center of Rigidity

Special Brace Frame Design				
	COM		COR	
Level	X	Y	X	Y
Stair 3	240.1	118.1	220.5	120.7
High Roof	151.4	61.4	50.7	65.1
Low Roof	121.1	76.8	70.1	97.8
Level 4	134.7	71.4	74.0	97.7
Level 3	130.4	61.3	78.0	97.2
Level 2	129.4	68.5	77.0	94.8
Level 1	120.0	72.0	96.8	67.4

Figure 65. *ETABS Output for Center of Mass and Center of Rigidity for SCBF*

Relative stiffness was computed using STAAD.Pro for each SCBF. Each frame was input in STAAD.Pro and was assigned the W-shape from the design. Then, a one kip load was applied at the uppermost story level (Figure 66). The deflection was measured. Since, stiffness is load divided by deflection, the relative stiffness of each frame was calculated by the inverse of the deflection. Figure 67 shows the results of these calculations.

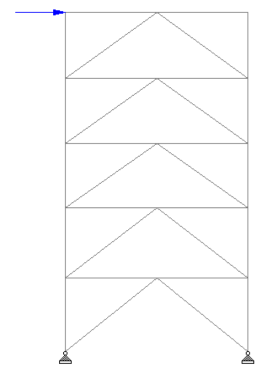


Figure 66. *STAAD.Pro Frame*

In order to spot check individual frame story force values obtained from ETABS, the hand calculated seismic loads per story were used. To obtain the direct force on each story, the distribution factor of the frame would be multiplied by the total story force. Then, the torsional force on each frame can be calculated using the following equation:

$$\text{Torsional force} = \text{Fi} = \text{My}(\text{ki} * \text{xi})/\text{Ip} + \text{Mx}(\text{ki} * \text{yi})/\text{Ip}$$

My = torsional moment in the y-direction, Mx = torsional moment in the x-direction, ki = frame stiffness
xi = distance of frame from x-axis, yi = distance of frame from y-axis, Ip = Ix + Iy

The total forces for these frames then can be calculated by adding the direct force and the torsional force. These forces then can be multiplied by a factor of 1.0 because this is the LRFD load factor for seismic loading.

		Stiffness (k/in)							Distribution Factors						
	Frame	Stair 3	High Roof	Low Roof	Level 4	Level 3	Level 2	Level 1	Stair 3	High Roof	Low Roof	Level 4	Level 3	Level 2	Level 1
X- Direction	3	-	22.22	24.39	29.41	38.46	22.22	22.22	-	0.12	0.11	0.11	0.11	0.05	0.03
	4	-	-	22.73	28.57	37.04	55.56	111.11	-	-	0.11	0.11	0.11	0.12	0.13
	5	-	29.41	32.26	38.46	50.00	71.43	142.86	-	0.17	0.15	0.15	0.15	0.16	0.16
	6	-	29.41	32.26	38.46	50.00	71.43	142.86	-	0.17	0.15	0.15	0.15	0.16	0.16
	11	34.48	35.71	37.04	45.45	58.82	83.33	166.67	0.50	0.20	0.17	0.17	0.17	0.18	0.19
	12	34.48	35.71	37.04	45.45	58.82	83.33	166.67	0.50	0.20	0.17	0.17	0.17	0.18	0.19
	13	-	25.64	28.57	35.71	45.45	66.67	125.00	-	0.14	0.13	0.14	0.13	0.15	0.14
									1.00	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
Y- Direction	1	-	-	22.08	27.77	36.68	53.56	106.50	-	-	0.13	0.13	0.14	0.14	0.14
	2	-	21.74	23.81	29.41	38.46	55.56	111.11	-	0.17	0.14	0.14	0.14	0.14	0.14
	7	-	33.33	37.04	45.45	58.82	83.33	166.67	-	0.25	0.22	0.22	0.22	0.22	0.22
	8	-	33.33	37.04	45.45	58.82	83.33	166.67	-	0.25	0.22	0.22	0.22	0.22	0.22
	9	20.00	21.28	24.39	30.30	38.46	55.56	111.11	0.50	0.16	0.14	0.15	0.14	0.14	0.14
	10	20.00	21.28	24.39	30.30	38.46	55.56	111.11	0.50	0.16	0.14	0.15	0.14	0.14	0.14
									1.00	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Figure 67. *Relative Stiffness and Distribution Factors for SCBF*

3.4.4 Seismic Loads

Pearl Hall Seismic loads were initially determined using ASCE 7-05 Equivalent Lateral Force Method. The site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters were determined according to ASCE 7-05 § 11.4.3. The design spectral acceleration parameters were determined by ASCE 7-05 §11.4.4. The main lateral force resisting system proposed for redesign is special concentric braced frames. The base shear value was determined in accordance with

Chapter 12 of ASCE 7-05. The following design values and limitations were used for the existing design. Please refer to Appendix D for detailed calculations.

Table 9. *Seismic Design Criteria for Special Concentric Brace Frames*

Occupancy Category	III
Importance Factor (I)	1.25
Seismic Design Category	D
Site Class	D
Site Class Factors	F_a=1.349 F_v=2.120
$S_{MS}=F_a(S_s)$	0.761
$S_{M1} = F_v(S_1)$	0.360
$S_{DS} = 2/3(S_{MS})$	0.506
$S_{D1} = 2/3(S_{M1})$	0.240
C_t	0.020
C_s	X: 0.069 Y: 0.069
Response Modification Factor (R)	6 (Special Steel Concentric Brace Frames)
Deflection Amplification Factor (C_d)	5
System Overstrength Factor (Ω_0)	2
Building Height Limitation	160 ft
Diaphragm Type	Concrete filled metal deck
Diaphragm Flexibility	Rigid
Vx= C_{s*}W	853 kip
Vy= C_{s*}W	853 kip

% Difference of ETABS from Hand Calculations			
Level	Hand Calculated Fx (k) = V*C_{vx}	ETABS Fx (k)	% Difference
Stair 3	2.1	2.1	-0.71%
High Roof	72.4	73.1	-0.88%
Low Roof	84.1	79.6	5.32%
Level 4	382.5	381.5	0.24%
Level 3	107.7	108.7	-0.88%
Level 2	136.4	137.6	-0.89%
Level 1	68.1	68.9	-1.15%
Base Shear	853.2	851.4	0.21%
< 10%, therefore can use ETABS Calculated Seismic Forces			

% Difference of ETABS from Hand Calculations			
Level	Hand Calculated Fx (k) = V*C_{vx}	ETABS Fy (k)	% Difference
Stair 3	2.1	2.1	-0.71%
High Roof	72.4	73.1	-0.88%
Low Roof	84.1	79.6	5.32%
Level 4	382.5	381.5	0.24%
Level 3	107.7	108.7	-0.88%
Level 2	136.4	137.6	-0.89%
Level 1	68.1	68.9	-1.15%
Base Shear	853.2	851.41	0.21%
< 10%, therefore can use ETABS Calculated Seismic Forces			

Figure 68. *Comparison of Seismic Forces in N-S and E-W Directions*

3.4.5 Modal Response Spectrum Analysis

In order to specify the response spectrum scale, the scale factor shall be g*I/R, where g is acceleration due to gravity (use 386.4 in/sec2 for models in kips-inch units. After analysis is performed, review the Response Spectrum Base Reaction for seismic in the x and y directions. If reported dynamic base shear is more than 85% of the static base shear then no further action is required. However, when dynamic base shear is less than 85% of static base shear then readjust the scale factor to match the response spectrum base shear equal to 85% of static base shear (Figure 69). So, the new scale factor = (g*I/R) * 0.85*static base shear /response spectrum base shear. Then, use this readjusted scale factor in response spectrum case and rerun the analysis. Then, create a load case for 1.2Dead + 1.0 Live + 1.0 Modal.

Modal Response Spectrum Analysis - SF							
SCBF	SF	V	Vt	SF	Vt	0.85*V	Vt > 0.85*Vt
x	6.7083	1296.4	608.1	12.1564	1103.7	1102.0	ok
y	6.7083	1297.3	474.8	15.5787	1102.8	1102.7	ok

Figure 69. *Modal Spectrum Response Scale Factor*

3.4.6 Torsion Effects

Torsion creates additional shear in walls. Therefore, many frames will be controlled by shear versus flexure.

Inherent Torsion in the N-S Direction with Special Concentric Braced Frames					
Story	COM	COR	Eccentricity	Story Force (k)	Torsion(k-ft)
Stair 3	118.06	120.67	2.61	2.08	5
High Roof	61.36	65.10	3.75	42.02	157
Low Roof	76.78	97.81	21.03	45.02	947
Level 4	71.36	97.73	26.36	208.37	5,493
Level 3	61.25	97.16	35.91	56.84	2,041
Level 2	68.47	94.81	26.34	355.57	9,365
Level 1	72.03	67.39	-4.65	28.39	-132
				Total	17,877

Inherent Torsion in the E-W Direction with Special Concentric Braced Frames					
Story	COM	COR	Eccentricity	Story Force (k)	Torsion(ft-k)
Stair 3	240.13	220.48	-19.65	2.08	-41
High Roof	151.42	50.74	-100.68	72.41	-7,290
Low Roof	121.13	70.14	-50.99	84.08	-4,288
Level 4	134.69	73.98	-60.71	382.46	-23,220
Level 3	130.38	77.95	-52.42	107.74	-5,648
Level 2	129.44	76.96	-52.48	136.38	-7,158
Level 1	120.03	96.75	-23.28	68.07	-1,584
				Total	-49,228

Accidental Torsion in the N-S Direction with Special Concentric Braced Frames					
Story	By(ft)	%5 By (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	32.00	1.60	3.14	2.08	10
High Roof	232.08	11.60	3.12	42.02	131
Low Roof	236.34	11.82	3.14	45.02	141
Level 4	244.67	12.23	3.14	208.37	654
Level 3	244.67	12.23	3.14	56.84	178
Level 2	256.00	12.80	3.15	355.57	1,119
Level 1	256.00	12.80	2.89	28.39	82
				Total	2,316

Accidental Torsion in the E-W Direction with Special Concentric Braced Frames					
Story	Bx (ft)	%5 Bx (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	12.00	0.60	2.81	2.08	4
High Roof	70.71	3.54	2.63	72.41	190
Low Roof	120.00	6.00	2.63	84.08	221
Level 4	120.00	6.00	2.63	382.46	1,007
Level 3	120.00	6.00	2.64	107.74	284
Level 2	120.00	6.00	2.62	136.38	358
Level 1	120.00	6.00	2.74	68.07	186
				Total	2,250

Figure 70. *Inherent and Accidental Torsion in the N-S and E-W Directions for SCBF*

3.4.7 Serviceability

Drift is a serviceability requirement that is addressed in ASCE 7-05. Seismic drift is limitations are based on the occupancy category and normally would be limited to an allowable story drift of 0.015*height. Story drifts for seismic loading were determined in ETABS and compared to drift limitations in Figure 50. Due to irregularity, the amplified drift must be compared with the allowable drift value. Pearl Hall meets all serviceability criteria for this design.

$$\delta x = \delta xe * Cd/I \text{ (Amplified Drift)}$$

SCBF - Seismic Drift X Direction						
Story	Story Height (ft)	ETABS Displacement (in)	ETABS δxe (in)	δx (in) = δxe*Cd/I	Δallowable(in) = 0.015hx	
Stair 3	71.83	0.274	0.105	0.421	0.36	not ok
High Roof	69.83	0.379	0.056	0.223	1.26	ok
Low Roof	62.83	0.323	0.040	0.162	2.22	ok
Level 4	50.5	0.283	0.089	0.354	2.16	ok
Level 3	38.5	0.194	0.080	0.319	2.16	ok
Level 2	26.5	0.115	0.111	0.445	2.34	ok
Level 1	13.5	0.003	0.003	0.013	2.43	ok
SCBF - Seismic Drift Y Direction						
Story	Story Height (ft)	ETABS Displacement (in)	ETABS δxe (in)	δx (in) = δxe*Cd/I	Δallowable(in) = 0.015hx	
Stair 3	71.83	0.824	0.006	0.015	0.36	ok
High Roof	69.83	0.830	0.406	1.055	1.26	ok
Low Roof	62.83	0.424	0.037	0.095	2.22	ok
Level 4	50.5	0.388	0.113	0.294	2.16	ok
Level 3	38.5	0.275	0.110	0.285	2.16	ok
Level 2	26.5	0.165	0.161	0.418	2.34	ok
Level 1	13.5	0.005	0.005	0.012	2.43	ok

Figure 71. *Actual Seismic Drift and Amplified Drift vs. Code Limitations*

3.4.8 Strength Check

The design of the special concentric braced frames failed (Please refer to Appendix G). There were 24 failed brace members out of 156 total bracing members, which is 15.4% braces failed. Then, 24 out of 84 columns failed, which is 28.5% failed columns. The beams failed from column-beam moment ratios as well as strength ratios. Therefore, more braces would need to be added to the design. Since the architect wanted an open breezeway, frames cannot be added at the center of the building. Also, the braces would not provide the best aesthetic design option for Pearl Hall. As a result, it was desired to change the design to special moment frames.

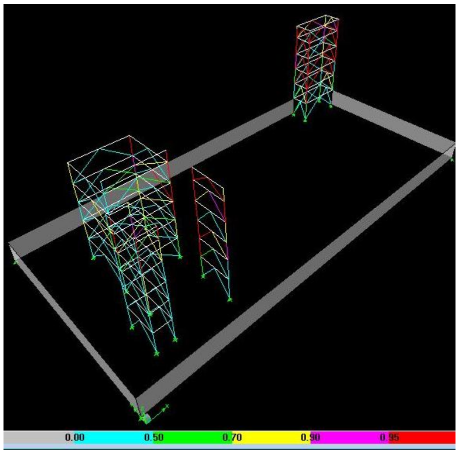


Figure 72. *ETABS Model indicating the of the SCBF Design*

3.4.9 ETABS Modeling of Special Moment Resisting Frames

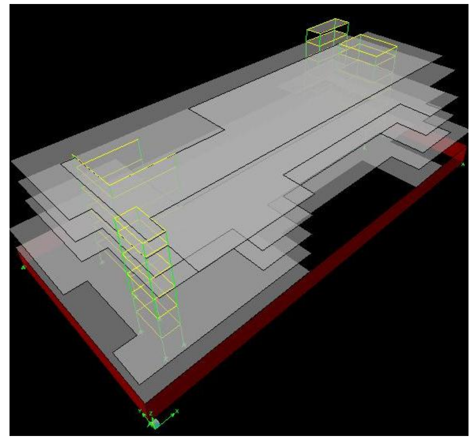


Figure 73. *ETABS SMFDesign*

The strong column-weak beam design is required for special moment frames. For a system with weak columns, a mechanism is created when the columns of only one story reach their flexural capacities. This is because there is less dissipation of seismic energy prior to the collapse. Yet, for a system with strong columns and weak beams, a mechanism is created when all the beams on all stories give way. Therefore, there is much more seismic energy dissipated prior to collapse. As a result, ETABS has an option to design reduced beam sections (RBS) for the required special moment resisting frames.

In order to create an efficient design, it was desired to change the layout of frames from the previous two designs. Since moment frames are very flexible in terms of architecture layout, 4 frames were added to the east side of Pearl Hall and 2 frames were eliminated from the west in aims of reducing the eccentricity of center of rigidity from the center of mass (See Figure 74).

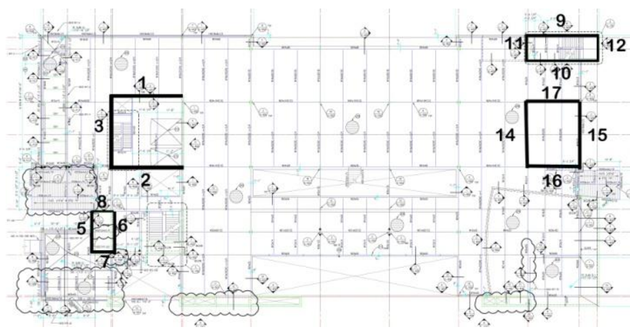


Figure 74. *SMF Layout. Modified by N. Trujillo.*
(Courtesy of Jon Anderson Architect)

3.4.10 Center of Rigidity

Since the center of mass and the center of rigidity do not align, torsion will control many member designs (Figure 75). A hand calculation was performed to determine the stiffness of the frame elements. Figure 76 shows that Wall 7 takes 18% of the diaphragm shear on level 2.

Special Moment Frame Design				
	COM		COR	
Level	X	Y	X	Y
Stair 3	240.1	121.4	232.2	113.1
High Roof	148.9	62.4	139.2	70.7
Low Roof	122.4	77.9	145.5	71.1
Level 4	135.0	72.0	152.8	69.7
Level 3	133.9	64.6	157.5	67.2
Level 2	130.5	68.4	161.2	64.5
Level 1	119.1	73.4	96.0	64.1

Figure 75. *ETABS Output for Center of Mass and Center of Rigidity for SMF*

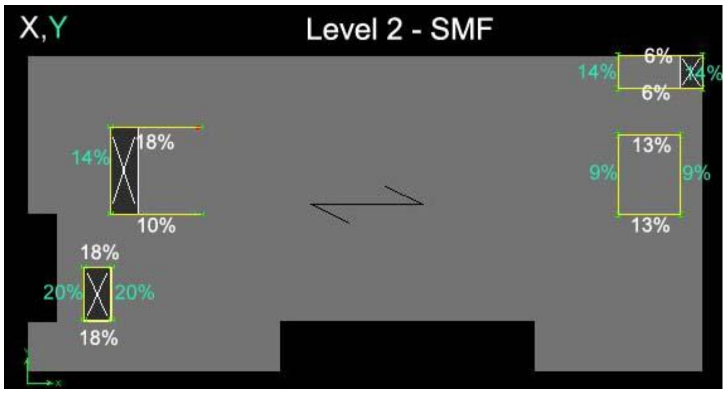


Figure 76. *Hand Calculated Relative Stiffness for SMF at Level 2 in both X and Y directions*

Relative stiffness was computed using STAAD.Pro for each SMF. Each frame was input in STAAD.Pro and was assigned the W-shape from the design. Then, a one kip load was applied at the uppermost story level (Figure 77). The deflection was measured. Since, stiffness is load divided by deflection, the relative stiffness of each frame was calculated by the inverse of the deflection. Figure 78 shows the results of these calculations.

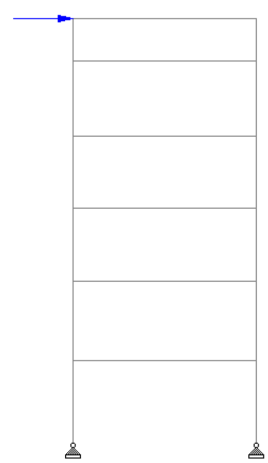


Figure 77 . *STAAD.Pro SMF Frame*

In order to spot check individual frame story force values obtained from ETABS, the hand calculated seismic loads per story were used. To obtain the direct force on each story, the distribution factor of the frame would be multiplied by the total story force. Then, the torsional force on each frame can be calculated using the following equation:

$$\text{Torsional force} = \text{Fi} = \text{My}(\text{ki} * \text{xi})/\text{Ip} + \text{Mx}(\text{ki} * \text{yi})/\text{Ip}$$

My = torsional moment in the y-direction, Mx = torsional moment in the x-direction, ki = frame stiffness
xi = distance of frame from x-axis, yi = distance of frame from y-axis, Ip = Ix + Iy

The total forces for these frames then can be calculated by adding the direct force and the torsional force. These forces then can be multiplied by a factor of 1.0 because this is the LRFD load factor for seismic loading.

		Stiffness (k/in)							Distribution Factors						
	Frame	Stair 3	High Roof	Low Roof	Level 4	Level 3	Level 2	Level 1	Stair 3	High Roof	Low Roof	Level 4	Level 3	Level 2	Level 1
X- Direction	3	-	45.45	47.62	55.56	71.43	45.45	45.45	-	0.15	0.15	0.15	0.15	0.08	0.05
	5	-	50.00	52.63	62.50	76.92	100.00	166.67	-	0.16	0.16	0.16	0.16	0.18	0.18
	6	-	50.00	52.63	62.50	76.92	100.00	166.67	-	0.16	0.16	0.16	0.16	0.18	0.18
	11	33.33	33.33	35.71	41.67	52.63	71.43	125.00	0.50	0.11	0.11	0.11	0.11	0.13	0.14
	12	33.33	33.33	35.71	41.67	52.63	71.43	125.00	0.50	0.11	0.11	0.11	0.11	0.13	0.14
	16	-	47.62	50.00	58.82	71.43	90.91	142.86	-	0.15	0.15	0.15	0.15	0.16	0.16
	17	-	47.62	50.00	58.82	71.43	90.91	142.86	-	0.15	0.15	0.15	0.15	0.16	0.16
									1.00	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
Y- Direction	1	-	62.50	66.67	76.92	90.91	125.00	200.00	-	0.24	0.24	0.24	0.22	0.27	0.27
	2	-	45.45	47.62	55.56	71.43	0.73	1.26	-	0.18	0.17	0.17	0.18	0.00	0.00
	7	-	55.56	58.82	71.43	90.91	125.00	200.00	-	0.21	0.21	0.22	0.22	0.27	0.27
	8	-	55.56	58.82	71.43	90.91	125.00	200.00	-	0.21	0.21	0.22	0.22	0.27	0.27
	9	19.61	20.00	21.28	24.39	30.30	41.67	71.43	0.50	0.08	0.08	0.08	0.07	0.09	0.10
	10	19.61	20.00	21.28	24.39	30.30	41.67	71.43	0.50	0.08	0.08	0.08	0.07	0.09	0.10
	14	-	0.05	0.05	0.04	0.03	0.02	0.01	-	0.00	0.00	0.00	0.00	0.00	0.00
	15	-	0.05	0.05	0.04	0.03	0.02	0.01	-	0.00	0.00	0.00	0.00	0.00	0.00
									1.00	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Figure 78. *Relative Stiffness and Distribution Factors for SMF*

3.4.11 Seismic Loads

Pearl Hall Seismic loads were initially determined using ASCE 7-05 Equivalent Lateral Force Method. The site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters were determined according to ASCE 7-05 § 11.4.3. The design spectral acceleration parameters were determined by ASCE 7-05 §11.4.4. The main lateral force resisting system proposed for redesign is special moment frames. The base shear value was determined in accordance with Chapter 12 of ASCE 7-05. The following design values and limitations were used for the existing design. Please refer to Appendix D for detailed calculations. These loads are much smaller in magnitude than shear walls and braced frames.

Table 10. *Seismic Design Criteria for Special Steel Moment Frames*

Occupancy Category	III
Importance Factor (I)	1.25
Seismic Design Category	D
Site Class	D
Site Class Factors	F_a=1.349 F_v=2.120
$S_{MS}=F_a(S_s)$	0.761
$S_{M1} = F_v(S_1)$	0.360
$S_{DS} = 2/3(S_{MS})$	0.507
$S_{D1} = 2/3(S_{M1})$	0.240
C_t	0.028
C_s	X: 0.048

	Y: 0.037
Response Modification Factor (R)	8 (Special Steel Moment Frames)
Deflection Amplification Factor (C_d)	5.5
System Overstrength Factor (Ω_0)	3
Building Height Limitation	Not Limited
Diaphragm Type	Concrete filled metal deck
Diaphragm Flexibility	Rigid
$V_x = C_s * W$	584 kip
$V_y = C_s * W$	454 kip

% Difference of ETABS from Hand Calculations			
Level	Hand Calculated Fx (k) = V*C_{vx}	ETABS Fx (k)	% Difference
Stair 3	1.5	1.5	0.89%
High Roof	51.1	50.6	0.85%
Low Roof	55.4	55.0	0.72%
Level 4	262.7	261.5	0.45%
Level 3	73.9	73.8	0.10%
Level 2	98.6	99.0	-0.38%
Level 1	41.5	42.0	-1.24%
Base Shear	584.6	583.4	0.21%
< 10%, therefore can use ETABS Calculated Seismic Forces			

% Difference of ETABS from Hand Calculations			
Level	Hand Calculated Fy (k) = V*C_{vx}	ETABS Fy (k)	% Difference
Stair 3	1.2	1.2	0.63%
High Roof	42.0	41.67	0.82%
Low Roof	45.0	44.71	0.68%
Level 4	208.4	207.49	0.42%
Level 3	56.8	56.8	0.07%
Level 2	72.7	73	-0.41%
Level 1	28.4	28.75	-1.29%
Base Shear	454.5	758.47	-66.87%
< 10%, therefore can use ETABS Calculated Seismic Forces			

Figure 79. *Comparison of Seismic Forces in N-S and E-W Directions*

3.4.12 Modal Response Spectrum Analysis

In order to specify the response spectrum scale, the scale factor shall be g*I/R, where g is acceleration due to gravity (use 386.4 in/sec2 for models in kips-inch units. After analysis is performed, review the Response Spectrum Base Reaction for seismic in the x and y directions. If reported dynamic base shear is more than 85% of the static base shear then no further action is required. However, when dynamic base shear is less than 85% of static base shear then readjust the scale factor to match the response spectrum base shear equal to 85% of static base shear (Figure 80). So, the new scale factor = (g*I/R) * 0.85*static base shear /response spectrum base shear. Then, use this readjusted scale factor in response spectrum case and rerun the analysis. Then, create a load case for 1.2Dead + 1.0 Live + 1.0 Modal.

Modal Response Spectrum Analysis - SF							
SMF	SF	V	Vt	SF	Vt	0.85*V	Vt > 0.85*Vt
x	5.0313	581.3	386.6	6.42989	495.0	494.1	ok
y	5.0313	545.7	367.3	6.35366	464.0	463.9	ok

Figure 80. *Modal Spectrum Response Scale Factor*

3.4.13 Torsion Effects

Torsion creates additional shear in walls. Therefore, many frames will be controlled by shear versus flexure.

Inherent Torsion in the N-S Direction with Special Moment Frames					
Story	COM	COR	Eccentricity	Story Force (k)	Torsion(k-ft)
Stair 3	121.40	113.09	-8.31	1.21	-10
High Roof	62.36	70.74	8.38	42.02	352
Low Roof	77.88	71.08	-6.80	45.02	-306
Level 4	71.99	69.65	-2.34	208.37	-487
Level 3	64.58	67.24	2.66	56.84	151
Level 2	68.39	64.49	-3.89	72.70	-283
Level 1	73.42	64.12	-9.30	28.39	-264
				Total	-847

Inherent Torsion in the E-W Direction with Special Moment Frames					
Story	COM	COR	Eccentricity	Story Force (k)	Torsion(ft-k)
Stair 3	240.07	232.21	-7.86	1.46	-11
High Roof	148.91	139.18	-9.73	51.07	-497
Low Roof	122.39	145.51	23.13	55.37	1,280
Level 4	134.99	152.77	17.77	262.69	4,669
Level 3	133.90	157.48	23.58	73.89	1,742
Level 2	130.46	161.18	30.72	98.57	3,028
Level 1	119.10	96.02	-23.08	41.53	-958
				Total	9,253

Accidental Torsion in the N-S Direction with Special Moment Frames					
Story	By(ft)	%5 By (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	32.00	1.60	3.15	1.21	6
High Roof	232.08	11.60	3.15	42.02	132
Low Roof	236.34	11.82	3.16	45.02	142
Level 4	244.67	12.23	3.15	208.37	656
Level 3	244.67	12.23	3.15	56.84	179
Level 2	256.00	12.80	3.15	72.70	229
Level 1	256.00	12.80	2.94	28.39	84
				Total	1,429

Accidental Torsion in the E-W Direction with Special Moment Frames					
Story	Bx (ft)	%5 Bx (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	12.00	0.60	2.80	1.46	2
High Roof	70.71	3.54	2.63	51.07	135
Low Roof	120.00	6.00	2.69	55.37	149
Level 4	120.00	6.00	2.63	262.69	692
Level 3	120.00	6.00	2.63	73.89	195
Level 2	120.00	6.00	2.60	98.57	256
Level 1	120.00	6.00	2.74	41.53	114
				Total	1,543

Figure 81. *Inherent and Accidental Torsion in the N-S and E-W Directions for SMF*

3.4.14 Serviceability

Drift is a serviceability requirement that is addressed in ASCE 7-05. Seismic drift is limitations are based on the occupancy category and normally would be limited to an allowable story drift of 0.015*height. Story drifts for seismic loading were determined in ETABS and compared to drift limitations in Figure 50. Due to irregularity, the amplified drift must be compared with the allowable drift value.

$$\delta x = \delta xe * Cd/I \text{ (Amplified Drift)}$$

SMF - Seismic Drift X Direction

Story	Story Height (ft)	ETABS Displacement (in)	ETABS δxe (in)	δx (in) = δxe*Cd/I	Δallowable(in)/ρ = 0.015hx/1.3	
Stair 3	71.83	0.037	0.002	0.007	0.28	ok
High Roof	69.83	0.038	0.003	0.011	0.97	ok
Low Roof	62.83	0.036	0.005	0.022	1.71	ok
Level 4	50.5	0.031	0.008	0.036	1.66	ok
Level 3	38.5	0.023	0.011	0.050	1.66	ok
Level 2	26.5	0.011	0.011	0.049	1.80	ok
Level 1	13.5	0.000	0.000	0.001	1.87	ok

SMF - Seismic Drift Y Direction

Story	Story Height (ft)	ETABS Displacement (in)	ETABS δxe (in)	δx (in) = δxe*Cd/I	Δallowable(in)/ρ = 0.015hx/1.3	
Stair 3	71.83	0.032	0.085	0.374	0.28	not ok
High Roof	69.83	-0.053	0.005	0.021	0.97	ok
Low Roof	62.83	-0.048	0.012	0.051	1.71	ok
Level 4	50.5	-0.060	0.039	0.171	1.66	ok
Level 3	38.5	-0.021	0.027	0.118	1.66	ok
Level 2	26.5	-0.048	0.062	0.274	1.80	ok
Level 1	13.5	0.014	0.014	0.063	1.87	ok

Figure 82. *Actual Seismic Drift and Amplified Drift vs. Code Limitations*

3.4.15 Strength Check

The design of the special concentric braced frames passed. A beam and column were checked for strength from Frame 11 because it carries 14% load in Y direction (Please refer to Appendix G). The moment frames were difficult to design because seismic compact section criteria. Yet, a suitable design was achieved.

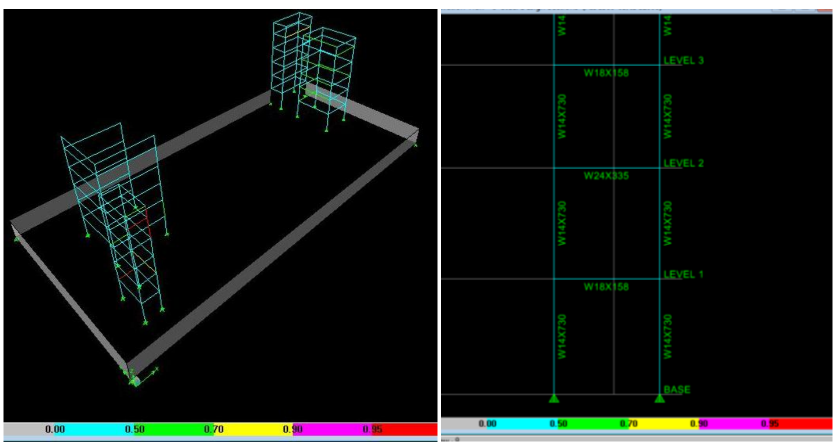


Figure 83. *Comparison of the Center of Mass and Rigidity of ETABS versus Hand Calculations*

3.5 Depth Study Comparison

The existing design was compared to the other redesigns in terms of period of vibrations. The smaller the period the more rigid the building (Figure 84). The special moment frames provide a more flexible building.

Modal Period		
Exisiting Shear Walls	0.5556	s
Shear Walls Design #2	0.6319	s
OCBF	1.4536	s
SMF	1.9963	s

Figure 84. Comparison of Modal Periods

The existing shear wall system, the modified shear walls, and the special moment frames were all compared for their cost. It was determined that the cost for the existing special reinforced shear walls are 3.5 times that of the special moment frames. This may be attributed to the fact that Agilia® concrete was used in Pearl Hall. It is a concrete mix that is a self-consolidating concrete. The Agilia Architectural product is specially designed for heavily reinforced for seismic zone construction and applications with an architectural finish requirement. The concrete mix is expensive, but the architectural look is very aethestic. But the modified shear walls were about 7.6% savings from the existing shear wall design. Please refer to Appendix H for all the cost calculations.

RS Mea 2007 Cost Comparisons				
Lateral System	Total Cost	Location Factor*	Cost 2007**	Cost 2011
Exisiting Special Reinforced Shear Walls	$ 5,874,944	$ 5,187,576	$ 5,874,944	$ 6,535,566
Modified Special Reinforced Shear Walls	$5,413,136	$ 4,779,799	$ 5,413,136	$ 6,021,829
Special Moment Frames	$ 1,696,717	$ 1,498,201	$ 1,696,717	$ 1,887,509
* Location: Albuquerque, NM (88.3 Location Factor)				
** Project Completion Date: 2007 ; Compare to today (166.3/185)				

Figure 85. *Seismic Design Criteria for Existing Special Reinforced Shear Walls*

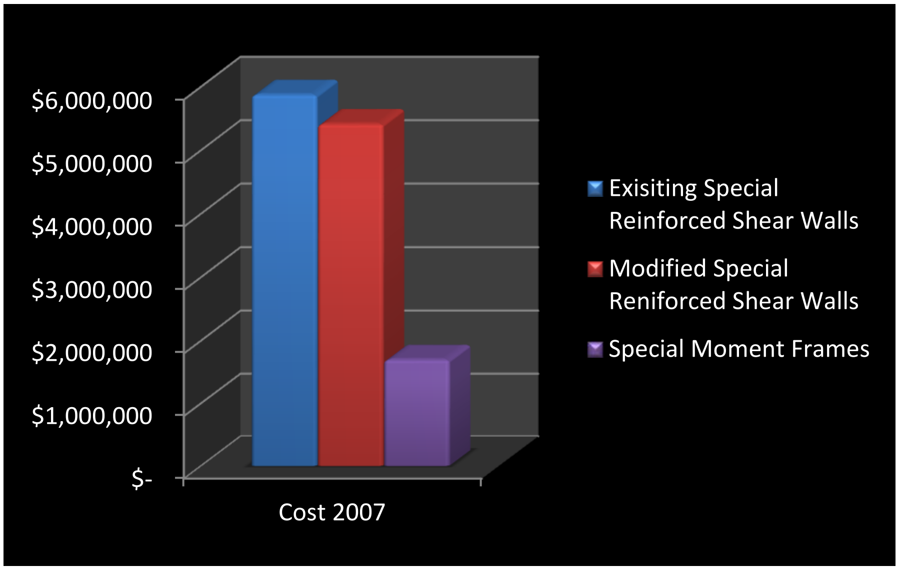


Figure 86. *Seismic Design Criteria for Existing Special Reinforced Shear Walls*

4. Breadth Study-Architectural

4.1 Thesis Problem Statement

In order to potentially improve the design of Pearl Hall, user feedback was obtained from an architecture student in the School of Architecture and Planning at the University of New Mexico. Since the students use Pearl Hall on a daily basis, it was important to see how they feel about the building.

Figure 87. *Looking out Pearl Hall toward the open breezeway on the lower level.*

The breezeway on the lower level (See Figure 87) is rarely used. Classes use it for outside presentations occasionally, and it is used for a dance once a year. Most of the school year it is too cold to use the space, so students rarely gather there. Therefore, the breezeway can be closed in order to provide a pleasant space for students and faculty to use. An enclosure will be designed for the breezeway to maintain aesthetics, yet make the space functional.

4.2 Design

In order to evaluate the best possible design, two designs were created for each of the north and south glazing. In Figure 87 it shows that the South glazing faces Central Ave, which the North Glazing faces the UNM campus. The design options aimed at making a more usable space, while keeping the intent of the architect. Antoine Predock intended to have a column free breezeway so that the breezeway would provide space available for lectures and functions by the School of Architecture and Planning (Figure 88).

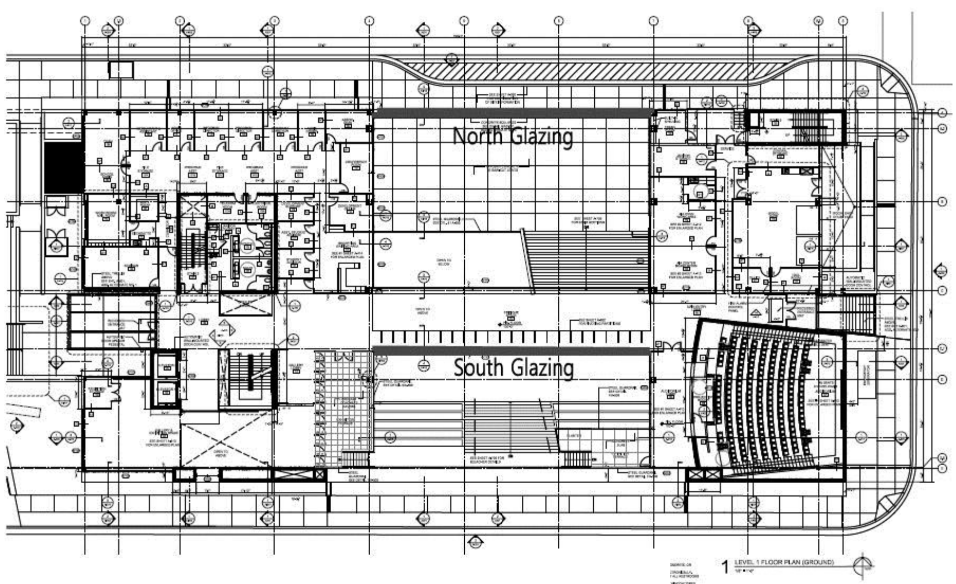


Figure 88. *North Glazing Design South Glazing Design on First Level Floor Plan. Modified by Nicole Trujillo. (Courtesy: Jon Anderson Architects)*

The first design aimed to extend a curtain wall to the outmost perimeter of the building in order to add as much additional usable space as possible. Yet, it was determined that the additional glazing would change the architectural look and design of the building which is undesirable.

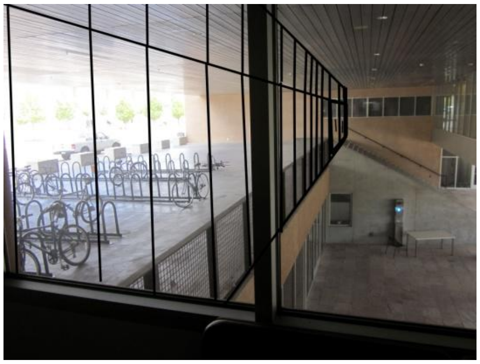


Figure 89. *North Enclosure Design #1.*

Figure 90. *South Enclosure Design #1.*

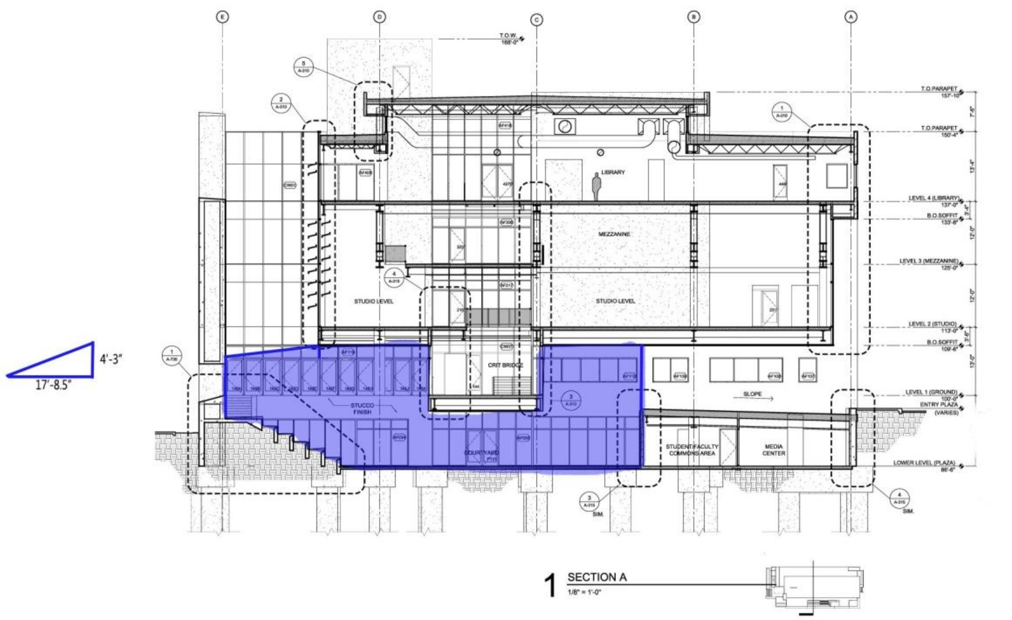


Figure 91. Design #1 - *Building Section facing East. Modified by Nicole Trujillo. (Courtesy: Jon Anderson Architects)*

The second design is the preferred design because it adds additional vertical glazing to enclose the breezeway. Yet, it will still have an open appearance, because the glazing it hidden under the Crit-Bridge (See Figure 93).

Figure 92. *North Enclosure Design #2.*

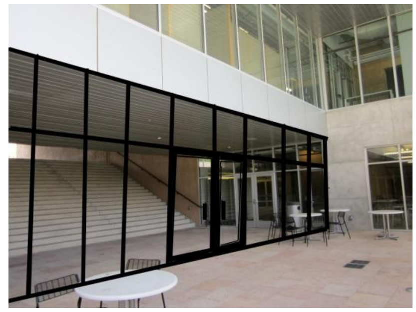


Figure 93. *South Enclosure Design #2*

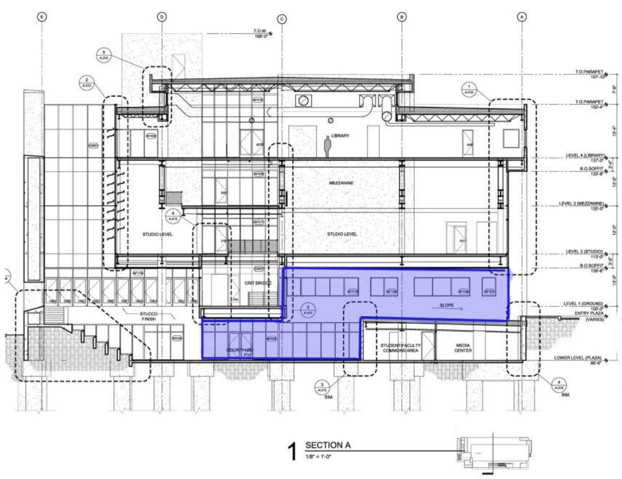


Figure 94. Design #2 - *Building Section facing East. Modified by Nicole Trujillo. (Courtesy: Jon Anderson Architects)*

4.3 Cost

Design #2 was preferred as the best design option. The cost of the additional glazing would be approximately $2032 for materials, plus additional labor costs as shown in Table 11. Yet, there would be benefits for the additional glazing. The former dean suggested that the enclosure on the north façade would enable an expansion of faculty office and an additional classroom. In addition, there would still be a column free breezeway as the architect intended. Yet, it would provide more functional space for the students and faculty at Pearl Hall.

Table 11. *Breezeway Enclosure Cost*

Additional Glazing Area	2544 SF
Cost/SF Viracon 3-54 Glazing	$8/SF
Cost of Glazing	$2032

5. Breadth Study-Mechanical

5.1 Thesis Problem Statement

Pearl Hall has been experiencing a performance issue in regards to occupant thermal comfort. An architecture student disclosed some information in regards to the current thermal comfort in Pearl Hall. The architecture student revealed that in one of their courses, a professor at Pearl Hall found that the bridge on level 1 was 36˚F on December 1, 2011. The studio on level 2 was 56˚F, and the offices on level 3 were 72˚F, and ceiling on level 4 was 81˚F.

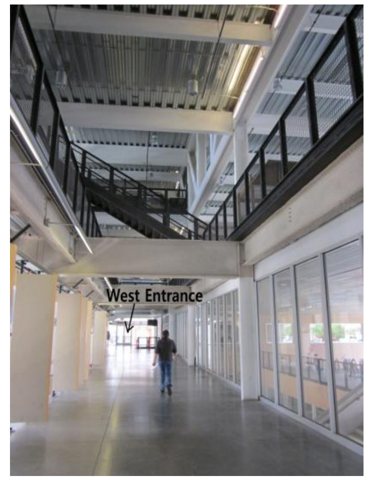


Figure 95 . *Crit-Bridge (looking toward the West Entrance).*

Therefore, solutions were investigated to properly heat and insulate the space. The proposed design was to change the glazing to more insulating glass units, IGUs.

5.2 Mechanical Systems Background

The specifications specified that high pressure steam is the primary source for heating hot water generation. The pressure steam is distributed throughout the piping network at the ground level. The heating hot water then is generated in the mechanical room in the basement of Pearl Hall. The system consists of two steam-to-hot water shell and tube heat exchangers and two circulating pumps. Heating hot water is distributed to all air-handling units and fan coils. The heating coils are provided with two-way control valves. The hot water circulating pumps are provided with variable frequency drives to allow variable volume water flow as the heating load increases and decreases.

Chilled water serves Pearl Hall and is connected to the campus piping system. The chilled water enters the building at the basement level. Then, this chilled water is distributed to all air handling units and fan coil units throughout the building. The cooling coils for air handling units and fan coils are provided with two-way control valves.

The main air-handling units are located in the mechanical equipment room on Level 3 and the ground level. A dedicated outside air unit, located in the Level 3 Fan room, provides outside air to Level 4 where radiant heating and cooling maintain comfort control. All other floors are served by air VAV systems. The single duct VAV units deliver cold air to terminal valves that incorporate hot water heating coils.

5.3 Design

The glazing is to be redesigned in Pearl Hall. The existing glazing in Pearl Hall is Viracon VRE 3-54. The two other options to be designed are VNE-30 and VRE-1-63. These glazing materials were chosen because they have a higher U value than the existing glazing.

Glazing	
Type	**U-Value**
VRE 3-54	0.25
VNE 1-30	0.18
VRE 1-63	0.13

Figure 96. *Glazing U Values*

TRACE 700 Input		
Room Names:	Critique Bridge-112	
	Critique Space-211,212,213	
	Studio-304,321	
Room Construction:	Slab – 6" HW Concrete	
	Roof – 6" HW Concrete	
Critique Bridge-112	North Wall – 100% Glazing	
	South Wall – 100% Glazing	
Critique Space-211,212,213	South Wall – 100% Glazing	
Studio-304,321	South Wall – 100% Glazing	
Critique Bridge-112	15 SF/Occupant	
	1876 SF	
	13 ft Height	
	VAV Min = 790 cfm	
Critique Space-211,212,213	100 SF/Occupant	
	2600 SF	
	12 ft Height	
	VAV Min = 1980 cfm	
Studio-304,321	100SF/Occupant	
	544 SF	
	12 ft Height	
	VAV Min = 600 cfm	
Design:	Cooling db = 75F	
	Heating Db = 70F	
	Rel.Humidity = 50%	
Thermostat:	Cooling driftpoint = 81F	
	Heating drfitpoint = 64F	
Internal Loads:	Classroom – 20sf/person	
	People – College	
	Sensible = 255 Btu/hr	
	Latent = 225 Btu/hr	
	Lights:	
	Recessed fluorescent, not vented, 80% load to space	
	Heat gain 1 W/sf	
Airflows:	**Main Supply:**	
Critique Bridge-112	Cooling: 9160 cfm	Heating: 9160 cfm
Critique Space-211,212,213	Cooling: 9430 cfm	Heating: 9430 cfm
Studio-304,321	Cooling: 5620cfm	Heating: 5620 cfm
	Apply ASHRAE Std.62.1-2004/2007	
	Classrooms (9plus)	
Plants:	Cooling Plant: Water Cooled Chiller	
	Heating Plant: Boiler	

Figure 97. *Trace 700 Input*

Pearl Hall - Existing Viracon 3-54 Glazing		Area	COOLING					HEATING		
Room	Type	ft²	% OA	cfm/ft²	cfm/ton	ft²/ton	Btu/hr·ft²	% OA	cfm/ft²	Btu/hr·ft²
Critique Bridge-112	Zone System- Variable Volume Reheat (30% Min Flow Default)	1,876	12.7	4.88	1066.4	218.4	54.95	100	0.42	-59.73
Critique Space 211,212,213	Zone System- Variable Volume Reheat (30% Min Flow Default)	2,600	6.07	3.63	1,356.50	374	32.09	28.89	0.76	-35.48
Studio 3 04,321	Zone System- Variable Volume Reheat (30% Min Flow Default)	544	2.13	10.33	584.2	56.6	212.19	19.95	1.1	-211.3
Pearl Hall - Viracon VRE 1-30 Glazing		**Area**	**COOLING**					**HEATING**		
Room	**Type**	**ft²**	**% OA**	**cfm/ft²**	**cfm/ton**	**ft²/ton**	**Btu/hr·ft²**	**% OA**	**cfm/ft²**	**Btu/hr·ft²**
Critique Bridge-112	Zone System- Variable Volume Reheat (30% Min Flow Default)	1,876	12.7	4.88	1083.7	221.9	54.07	100	0.42	-57.29
Critique Space 211,212,213	Zone System- Variable Volume Reheat (30% Min Flow Default)	2,600	6.07	3.63	1,382.20	381.1	31.49	28.89	0.76	-33.7
Studio 3 04,321	Zone System- Variable Volume Reheat (30% Min Flow Default)	544	2.13	10.33	590.3	57.1	210.03	19.95	1.1	-203.02
Pearl Hall - Viracon VNE 1-63 Glazing		**Area**	**COOLING**					**HEATING**		
Room	**Type**	**ft²**	**% OA**	**cfm/ft²**	**cfm/ton**	**ft²/ton**	**Btu/hr·ft²**	**% OA**	**cfm/ft²**	**Btu/hr·ft²**
Critique Bridge-112	Zone System- Variable Volume Reheat (30% Min Flow Default)	1,876	12.7	4.88	1096.6	224.6	53.44	100	0.42	-55.55
Critique Space 211,212,213	Zone System- Variable Volume Reheat (30% Min Flow Default)	2,600	6.07	3.63	1,401.30	386.4	31.06	28.89	0.76	-32.44
Studio 3 04,321	Zone System- Variable Volume Reheat (30% Min Flow Default)	544	2.13	10.33	594.7	57.6	208.46	19.95	1.1	-197.11

Figure 98. *TRACE 700 Engineering Checks for VRE 3-54, VRE 1-30, and VNE 1-63 glazing*

5.4 Cost

The results of the study showed evidence that VNE 1-30 Glazing is the least expensive and will provide the most energy cost savings for Pearl Hall. VNE 1-30 glazing provides 9.73% decrease in consumption than the current VRE 3-54 glazing. Also, VNE 1-30 is 2.3% cheaper in material cost than VRE 3-54. Please refer to Appendix I for detailed calculations.

According to faculty at Pearl Hall, the proposed solution is to use large fans to blow the hot air from Level 3 to Level 1. In addition, there are plans to add vestibules to the East Entrance and the West Entrance.

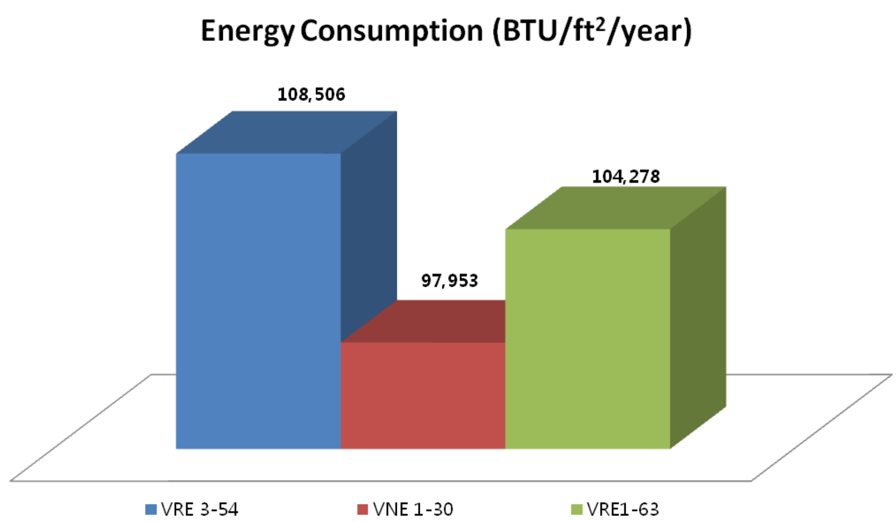


Figure 99. *TRACE 700 Energy Consumption per year for VRE 3-54, VRE 1-30, and VNE 1-63 glazing*

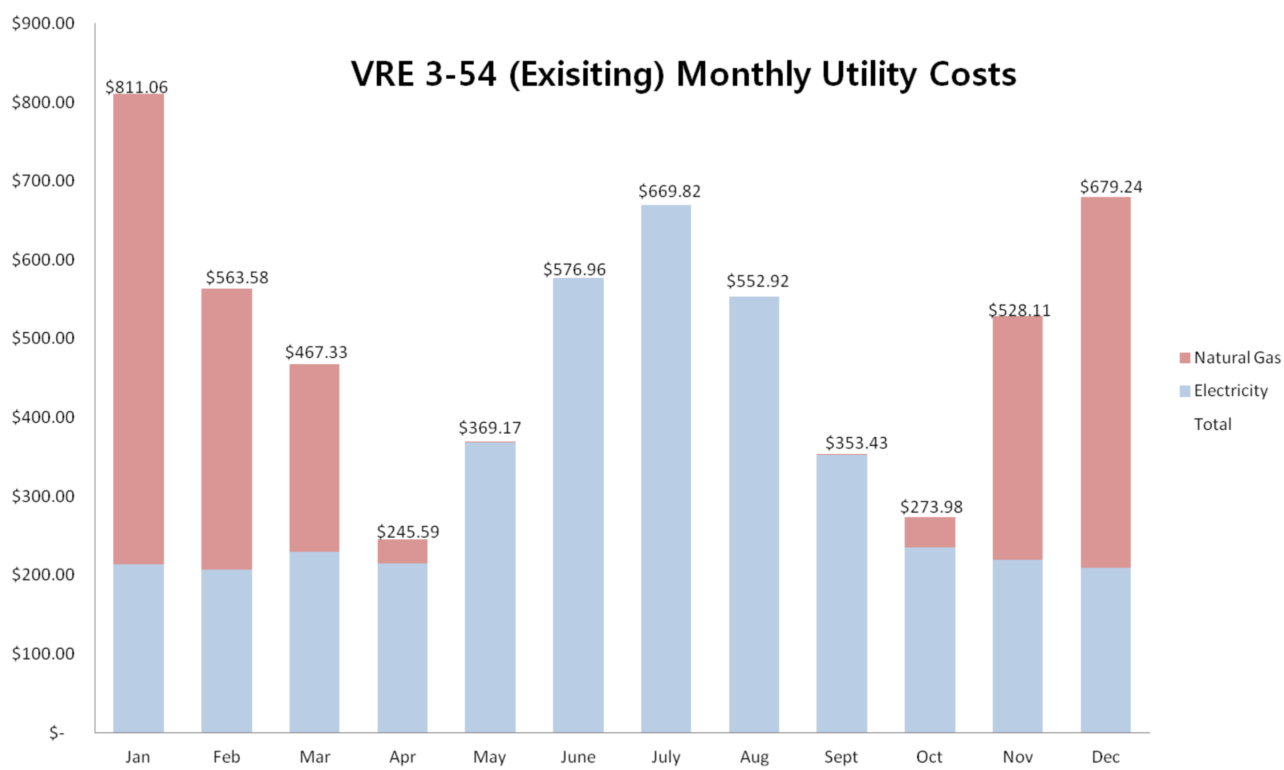


Figure 100. *VRE 3-54 Monthly Utility Costs*

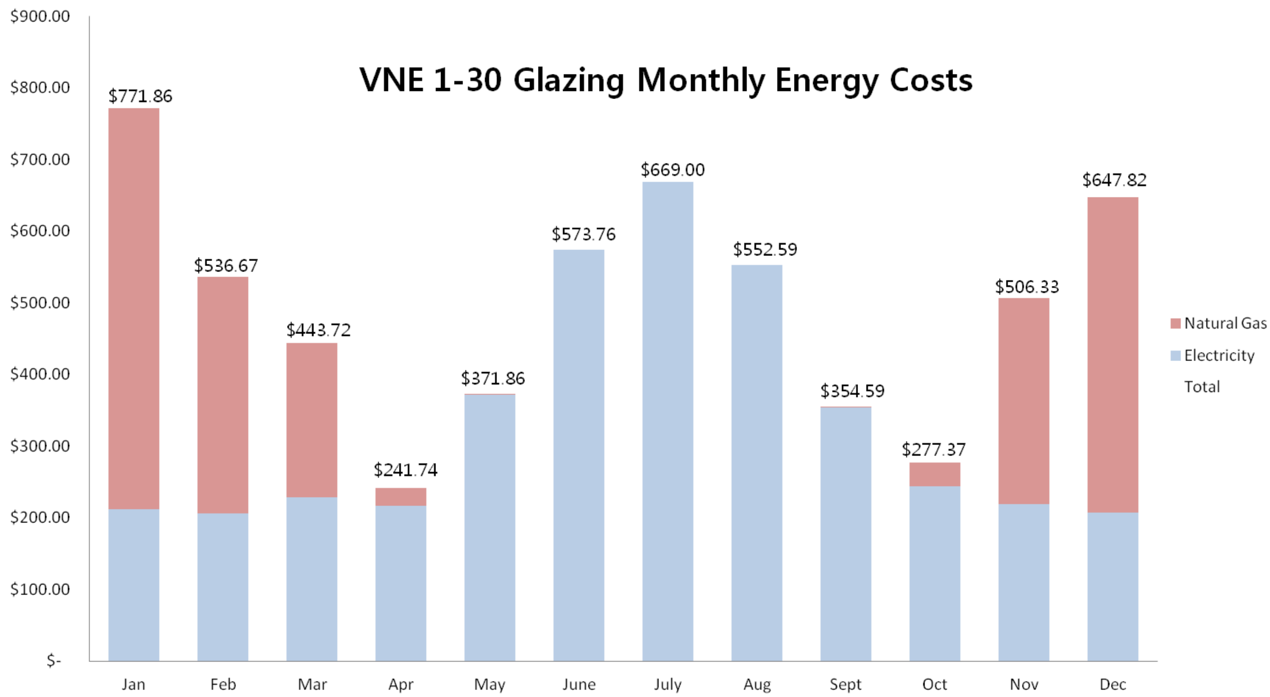


Figure 101. *VNE 1-30 Monthly Utility Costs*

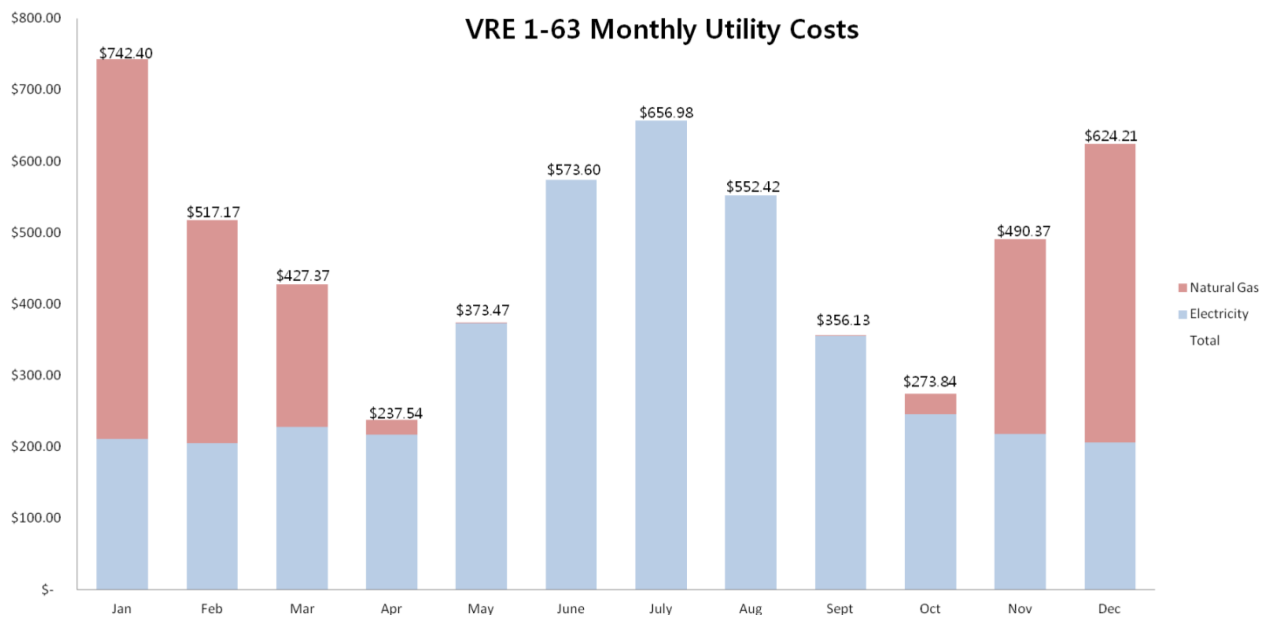


Figure 102. *VRE 1-63 Monthly Utility Costs*

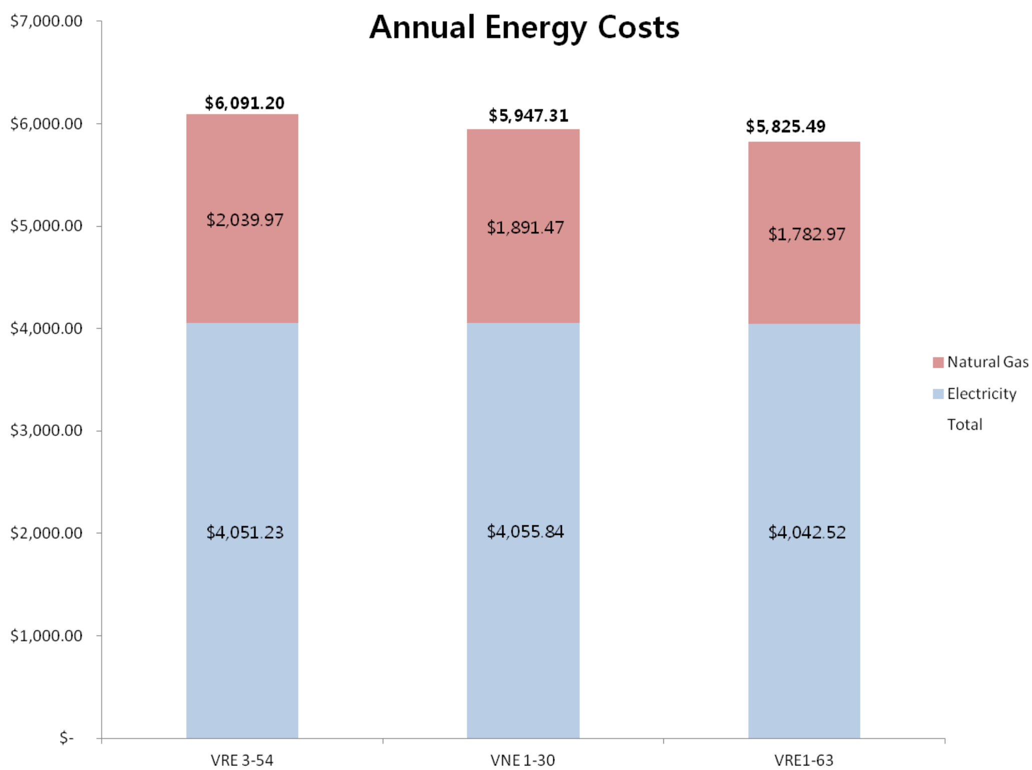


Figure 103. Annual Energy *Costs*

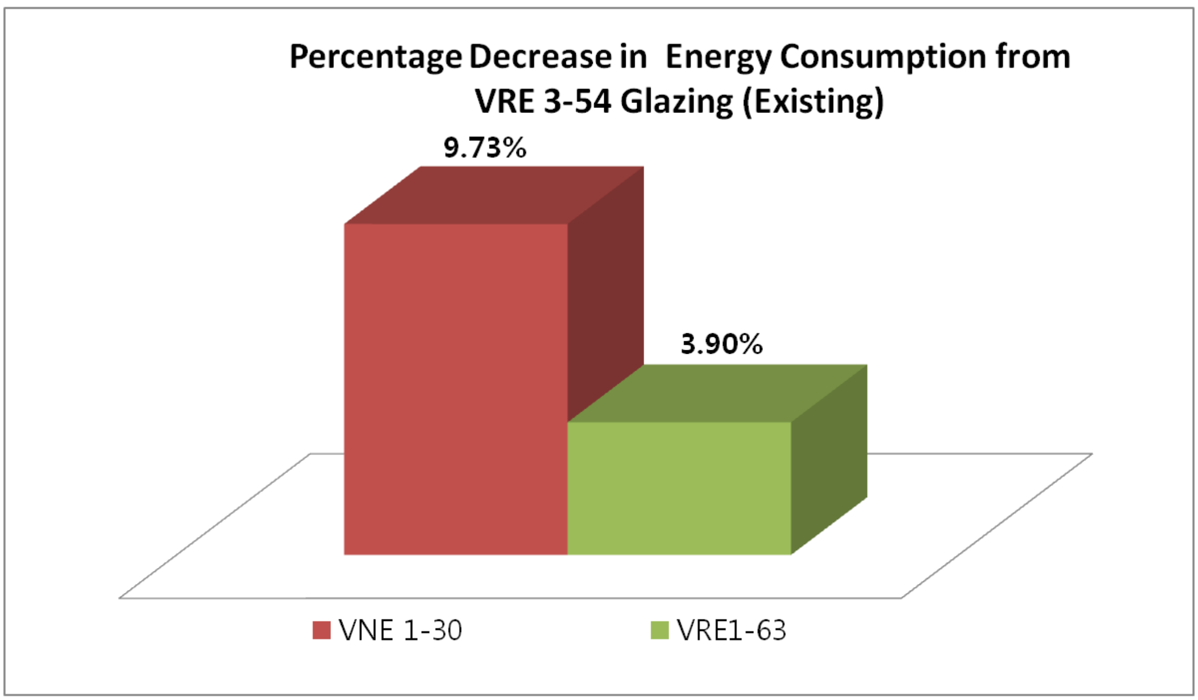


Figure 104. *Percentage Decrease in Energy Consumption from Existing Glazing*

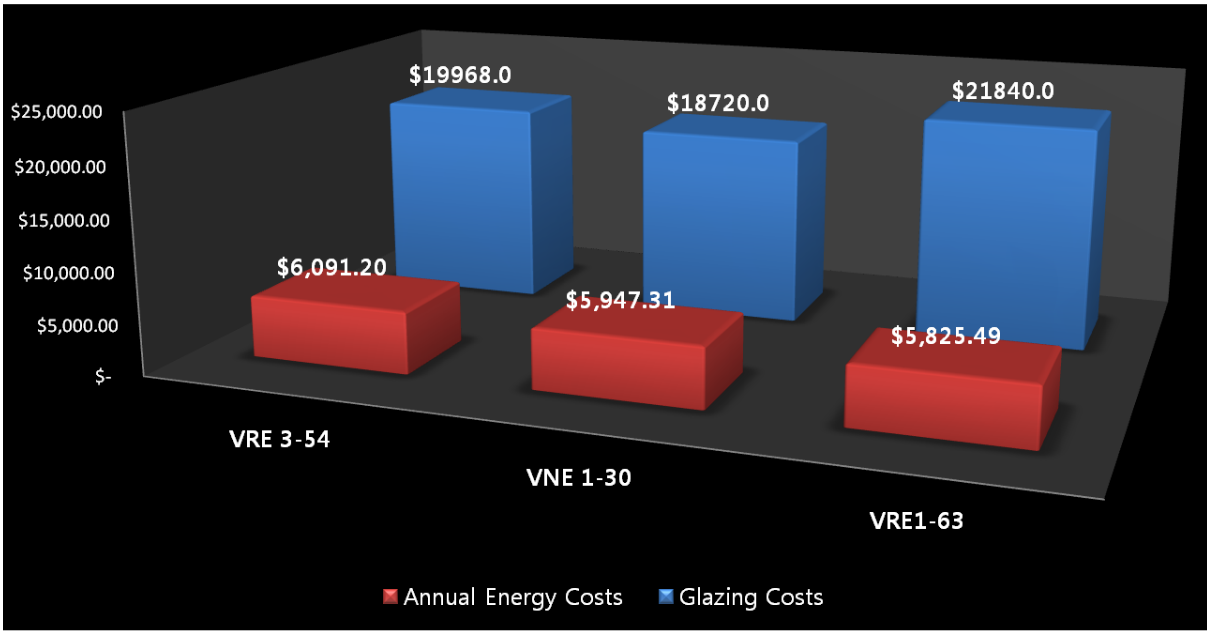


Figure 105. *Annual Energy and Glazing Costs for for VRE 3-54, VRE 1-30, and VNE 1-63 glazing*

6. Conclusion

The main focus of this final thesis report is to optimize the foundation and lateral systems for Pearl Hall. Since Pearl Hall is located in Albuquerque, New Mexico, it is located Seismic Design Category D. Therefore, seismic loads controlled for strength and serviceability. Due to structural irregularities, modal response spectrum analysis was required for design forces.

It was desired to redesign the lateral system and compare the designs in terms of the most cost effective. There were two proposed redesigns: the modified shear walls and the special moment frames. These systems were compared for their cost. It was determined that the cost for the existing special reinforced shear walls are 3.5 times that of the special moment frames. But the modified shear walls were about 7.6% savings from the existing shear wall design.

The architectural breadth study focuses on designing an enclosure for the breeze way on the lower level. The cost of the additional glazing would be approximately $2032 for materials. Yet, there would be benefits for the additional glazing. The former dean suggested that the enclosure on the north façade would enable an expansion of faculty office and an additional classroom.

The mechanical breadth study focuses on the fact that Pearl Hall has been experiencing a performance issue in regards to occupant thermal comfort. The results of the study showed evidence using more insulating glazing, VNE 1-30 Glazing that it will provide the most energy cost savings for Pearl Hall. VNE 1-30 glazing provides 9.73% decrease in consumption than the current VRE 3-54 glazing. Also, VNE 1-30 is 2.3% cheaper in material cost than VRE 3-54.

The goals of this thesis were to create an efficient lateral system for Pearl Hall. Based on the results discussed, these goals are clearly met. From a feasibility standpoint, each proposed study impacts the structure in a positive manner. It is the recommendation of the author to implement all changes proposed within this thesis report.

All calculations were done in accordance with applicable design codes. Please refer to the appendices for further review of detailed calculations and design drawings.

References

2005 Seismic Provisions, ANSI/AISC 341-05 and the 2005 AISC Moment Frame Prequalified Connection Standard ANSI/AISC 358-05 on CD-ROM. Chicago, IL: American Institute of Steel Construction, 2006.

AISC Steel Construction Manual. [Chicago, Ill.]: American Institute of Steel Construction, 2005.

American Concrete Institute (2008) Building Code Requirements for Structural Concrete (ACI 318-08), ACI, Farmington Hills, MI.

American Society of Civil Engineers. ASCE 7-05: Minimum Design Loads for Buildings and Other Structures. ASCE: Reston, VA, 2006.

Dagostino, Frank R., and Leslie Feigenbaum. *Estimating in Building Construction: Appendix*. Upper Saddle River, NJ: Prentice Hall, 2003.

Hamburger, Ronald O. Seismic Design of Steel Special Moment Frames: A Guide for Practing Engineers. [Gaithersburg, Md.]: U.S. Dept. of Commerce, Building and Fire Research Laboratory, National Institute of Standards and Technology, 2009.

Johnson, Timothy E. *Low-E Glazing Design Guide*. Boston: Butterworth Architecture, 1991.

McCormac, Jack C., and Stephen F. Csernak. *Structural Steel Design*. Boston: Prentice Hall, 2012.

Mead, Christopher Curtis., and Antoine Predock. *Roadcut: The Architecture of Antoine Predock*. Albuquerque: University of New Mexico, 2011.

Moehle, Jack P., John D. Hooper, and Christopher D. Lubke. Seismic Design of Reinforced Concrete Special Moment Frames: A Guide for Practing Engineers. [Gaithersburg, Md.]: U.S. Dept. of Commerce, National Institute of Standards and Technology, 2008

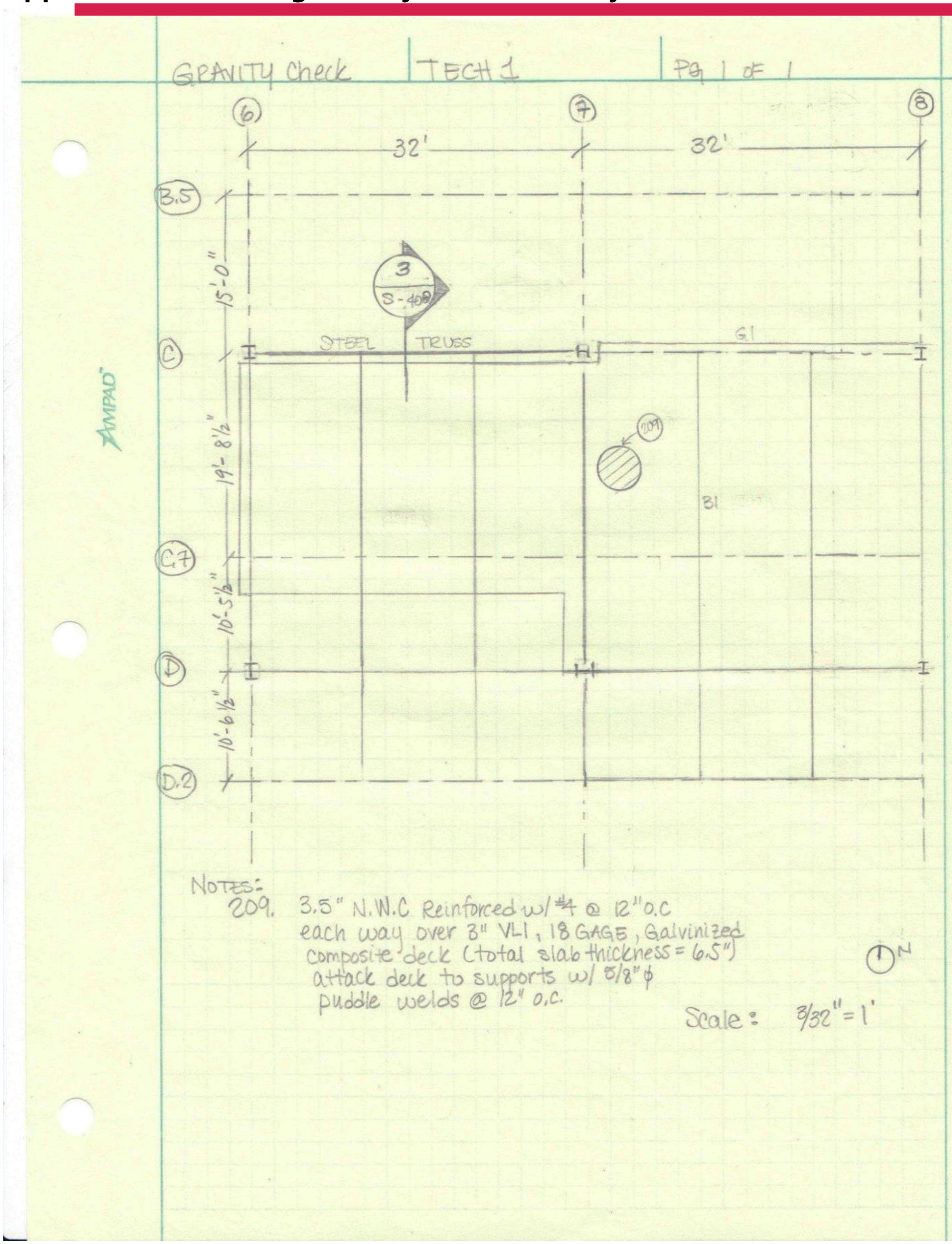
"New Mexico Best of 2008 University of New Mexico School of Architecture and Planning." Southwest Contractor Dec. 2008: 102-24.

RSMeans Construction Publishers and Consultants. Building Construction Cost Data 2007 66th Annual Edition. Reed Construction Data, Inc.: Kingston, MA, 2007.

Seismic Design Criteria for Steel Moment-frame Structures: FEMA 350, 351, 352 and 353 Policy Guide: FEMA 354 ; State of the Art Reports: FEMA 355A, B, C, D, E and F. [Washington, D.C.]: Federal Emergency Management Agency, 2000.

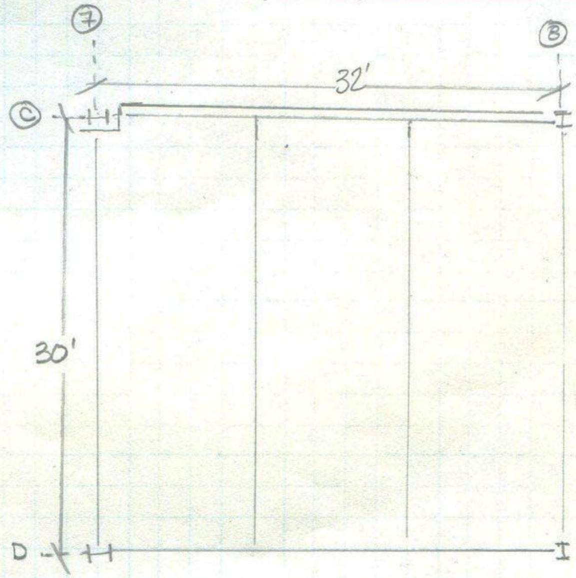
Waier, Phillip R., and Christopher Babbitt. *Building Construction Cost Data, 2007*. Kingston, MA: R.S. Means, 2006.

Appendix A – Existing Gravity and Lateral System Checks



GRAVITY Check - SLAB | TECH 1 | PG 1 OF 1

Floor LOAD



Dead: Slab + deck

Live load: LL = 80 psf (corridors)
LL = 80 psf (classroom)
160 psf

Use: SDI No. 31
Unshored
3 span 10'-6" span
Composite deck
3 VLI t = 3.5
Normal weight concrete

① Maximum clear span

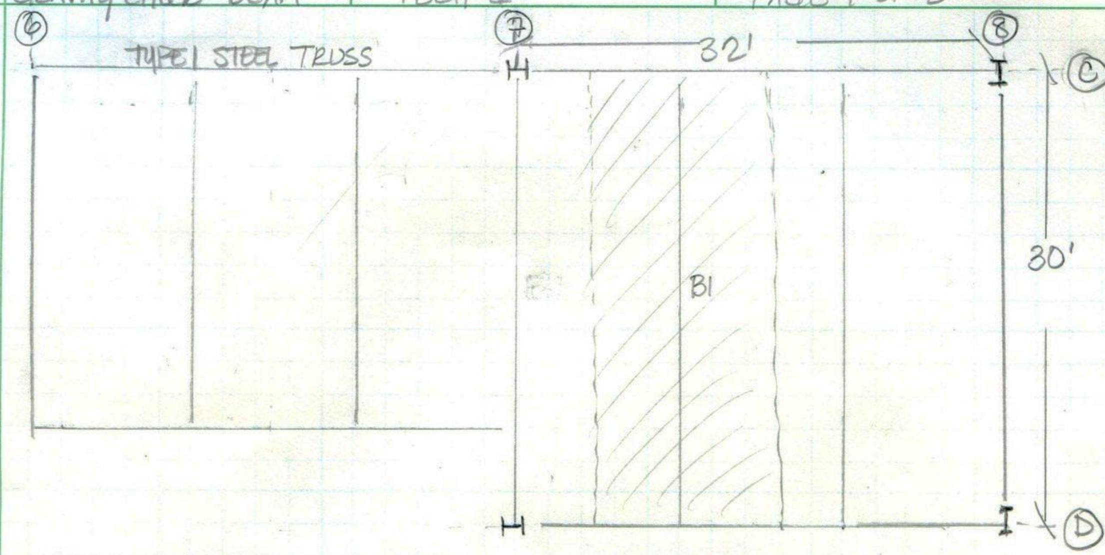
6.5" total thickness

Max span = 13'-10" > 10'-6" span ✓ ok

② Reinforced concrete allowable loads
in Superimposed live load 229 psf ≥ 180 psf ✓ ok
req'd = 229 psf for 18 gage
Slab information
6.5 depth → Use 6x6 - W2.1 x W2.1

Use 3 VLI w/ 3.5" concrete
reinforced with WWF 6x6 - W2.1 x W2.1

GRAVITY CHECK - BEAM | TECH 1 | PAGE 1 OF 2



ASTM A992 GRADE 50 WIDE FLANGE BEAM

3 span deck for unshored construction
Composite deck
max spacing = 12'-0" > 10'-8" ✓

Classroom, Corridor loading LL = 100 psf
1. self wt = 10 psf
2. Superimposed dead wt = 10 psf
3. slab NWC. t = 3.5"
3VLI d = 6.5"
slab wt = 63 psf

$L = \max \left| \begin{matrix} 80 \\ 70+50 \end{matrix} \right. = 80$ psf

$A_T = 30'(10'\text{-}8") = 320$

$K_L = 2$ (ASCE 7-05, Table 4-2)

$L = 100 \times \max \left| \begin{matrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{2(320)}} = 0.842 \end{matrix} \right. = 0.842(80) = 67.36$ ft^2 ∴ reduction ✓

$2(320) > 400 ft^2$ ✓

LOADING:

DL = slab + deck + supt + bm = 63 + 10 + 10 = 83 psf

$W_u = 1.2(83) + 1.6(67) = 206.8$ psf

$w_u = 207$ psf $(10'\text{-}8") = 2205$ plf $= 2.21$ klf

$V_u = \frac{w_u l}{2} = \frac{2.21\ klf(30')}{2} = 33.15$ kip

$M_u = \frac{w_u l^2}{8} = \frac{2.21\ klf(30')^2}{8} = 248.6$ k-ft

GRAVITY CHECK-BEAM | TECH I | PAGE 2 OF 2

- beams spaced @ 10'-8" O.C. w/ steel weight allowance (0p
- beam simply supported with deck on flange ∴ $L_b = 0$

→ Using AISC Steel Manual, 13th edition

- (Z_x table 3-2) Using LRFD

$M_u \leq \phi_b M_{px}$

W18x40 $\phi_b M_{px} = 294$ kip-ft $> M_u = 248.6$ k-ft

$V_u \leq \phi_v V_{nx}$

W18x40 $\phi_v V_{nx} = 169$ k $> V_u = 33.15$ k

- $\Delta_{LL} \leq L/360$ for W18x40 $I_x = 612 \text{ in}^4$ $w_{LL} = 0.0842 \text{ksf}(10'\text{-}8") = 0.898$k

$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_x} = \frac{5(0.714)(30')^4(1728)}{384(29000)(612)} = 0.923"$

$L/360 = \frac{30(12)}{360} = 1" > \Delta_{LL}$ ∴ ok ✓

- $\Delta_{TL} \leq L/240$ for W18x40 $w_{TL} = (0.0842 + 0.083)(10'\text{-}8") = 1.783$ klf

$\Delta_{TL} = \frac{5 w_{TL} L^4}{384 E I_x} = \frac{5(1.59)(30')^4(1728)}{384(29000)(612)} = 1.642"$

$L/240 = \frac{30(12)}{240} = 1.5" < \Delta_{TL}$ ∴ DOES NOT WORK

$w_{bm} = 4.4\left(\frac{31}{10'\text{-}8"}\right)$

DID NOT WORK: TRY W21x44, $I_x = 843 \text{ in}^4$

$\phi_b M_{px} = 358$ kft $> M_u$

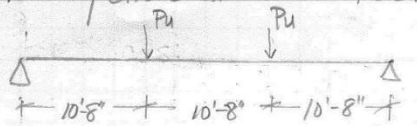
$\phi V V_{nx} = 215^k > V_u$

$\Delta_{LL} = \frac{5(0.714)(30)^4(1728)}{384(29000)(843)} = 0.532" < L/360 = 1"$ ∴ ok ✓

$\Delta_{TL} = \frac{5(1.59)(30)^4(1728)}{384(29000)(612)} = 1.18" < L/240 = 1.5"$ ∴ ok ✓

There may be the discrepancy because they used such a higher classroom load than ASCE 7-05. Therefore, they may have used judgement to use the W18x40 instead.

GRAVITY CHECK-GIRDER | TECH 1 | PAGE 1 OF 2



$\therefore L_b = 10'\text{-}8''$

$A_T = 30(32) = 960\ SF$

$K_{LL} = 2$

$A = K_{LL} A_T = 2(960) = 1920\ SF > 400\ SF$ ✓

$L = L_0\left(0.25 + \frac{15}{\sqrt{A_I}}\right) = L_0\left(0.25 + \frac{15}{\sqrt{1920}}\right)$

$= L_0(0.5925)$

LL = 100 psf corridors/classroom

LL on girder $= 0.5925\,(100psf)(30')/2$ *

$= 0.89\ klf/2 =$

DL on girder = slab + deck + sup + girder

$= (63/2) + 10 + 10$

$= 51.5\ psf\ (30')$

$= 1.545\ klf$

DL from W18 $= 0.04(30) = 1.2k$

$P_u = [1.2(1.55) + 1.6(0.89)](10'\text{-}8'')$

$+ 1.2(1.2)$

$= 36.47^k$

$V_u = P_u = 36.47\ k$

$M_u = \frac{P_u L}{3} = \frac{36.5(32')}{3} = 583.5^k ft$

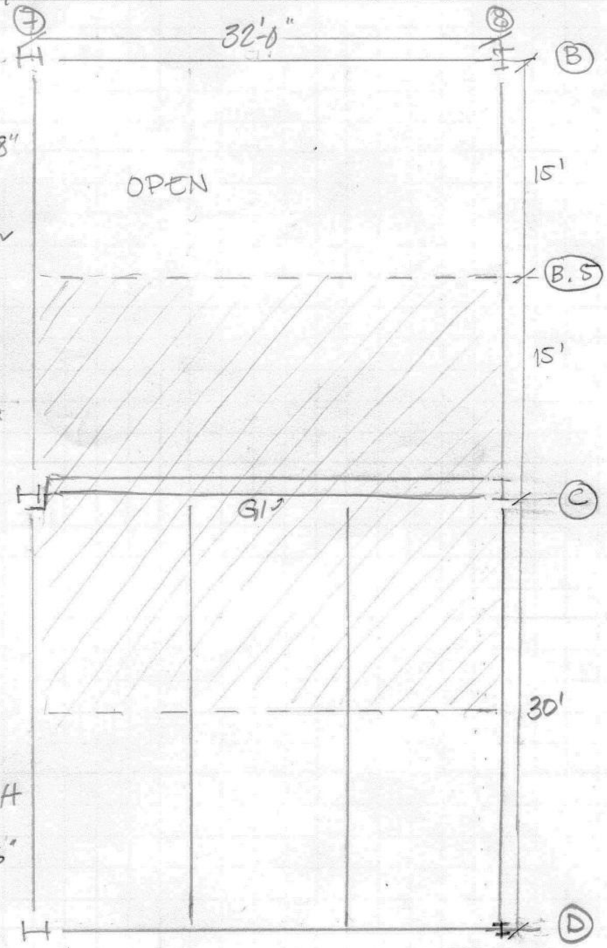
$\Delta_{LL} = \frac{P_u L^3}{28EI} \le \frac{L}{360} = \frac{32(12)}{360} = 1.066''$

$P_{u\,LL} = 0.89(10.66) = 9.49^k$

$\Delta_{TL} = \frac{P_u L^3}{28EI} \le \frac{L}{240} = \frac{32(12)}{240} = 1.6''$

$P_{u\,TL} = (1.55 + .89)(10.66) + 1.2$

$= 27.2\ k$



NOTE: Live load and slab + deck weight divided by 2 because they only load girder from C-D.

Using Z tables (Table 3-2 AISC Steel Manual) LRFD

$M_u \le \phi_b M_{px}$

W24x68 $\quad \phi_b M_{px} = 664\ kip\text{-}ft > M_u = 583.5^k ft$ ✓ ok

$V_u \le \phi_v V_{nx}$

W24x68 $\quad \phi_v V_{nx} = 295\ k > V_u = 36.47\ k$ ✓ ok

$I_x = 1830\ in^4$

$\Delta_{LL} = \frac{9.49(32^3)(1728)}{28(29000)(1830)} = 0.362'' < L/360 = 1.066''$ ✓ ok

$\Delta_{TL} = \frac{27.2(32^3)(1728)}{28(29000)(1830)} = 1.037'' < L/240 = 1.6''$ ✓ ok

OR CAN USE W21x68 but W24x68 provides a deeper connection to the truss?

The design used a W24x68.

GRAVITY CHECK - GIRDER | TECH 1 | PAGE 2 OF 2

Flange local buckling

$b_f/2t_f = 7.66$ (AISC Table 1-1)

$\lambda_p = 0.38\sqrt{29000/50}$ (Table B4.1)

$= 9.15$

$\lambda_p > b_f/2t_f$ Yes, flange compact

Web local buckling

$h/t_w = 52.0$ (AISC Table 1-1)

$\lambda_p = 3.76\sqrt{E/F_y}$ (Table B4.1)

$= 3.76\sqrt{29000/50}$

$= 90.6$

$\lambda_p > h/t_w$ Yes, web compact

Shear capacity

$V_u = W_u L/2 = 36.5k$

$d = 23.7\,in$

$t_w = 0.45\,in$

$A_w = d \cdot t_w = 10.67\,in^2$ $\phi_{vn} = 1.0$

$\phi V_n = \phi_{vn}(0.6\,F_{yw})A_w = 320.1k$

$\phi V_n > V_u$ ok ✓

$\boxed{\text{Use } W24x68}$

Appendix B – Building Weight Calculations

				Building Weight				
Level	**Area (SF)**	**Beams (kip)**	**Columns (kip)**	**Floor (kip)**	**Superimposed (kip)**	**Walls (kip)**	**Total Floor Weight (kip)**	**Weight/Area (psf)**
Stair 3	380	0	0	26	11	0	37	98
High Roof	12,071	43	11	821	145	0	1,021	85
Low Roof	13,748	65	25	1,551	905	4	2,551	186
Level 4	24,275	153	24	1,551	905	4	2,638	109
Level 3	13,392	169	24	922	561	4	1,681	125
Level 2	25,867	203	33	1,790	1,028	5	3,057	118
Level 1	23,434	154	25	1,609	951	5	2,744	117
						Total Weight (kip)	**13,729**	

Stair 3							
Approx. Area	=	380	SF				
Ht.	=	2	ft	Total Weight	=	37.06	k

Walls			
Height	=	0	ft
SW1, Length	=	0	ft
SW2, Length	=	0	ft
SW3, Length	=	0	ft
SW4, Length	=	0	ft
SW5, Length	=	0	ft
SW6, Length	=	0	ft
SW7, Length	=	0	ft
SW8, Length	=	0	ft
SW9, Length	=	32	ft
SW10, Length	=	32	ft
SW11, Length	=	12.33	ft
SW12, Length	=	12.33	ft
SW13, Length	=	0	ft
Unit Wt.	=	145	pcf
Weight	**=**	**0.00**	**k**

Superimposed			
Partitions	=	0	psf
Misc.	=	10	psf
Finishes	=	0	psf
Roof	=	20	psf
Weight	**=**	**10.8798**	**k**

Floor			
3VLI Deck	=	68	psf
Weight	**=**	**25.83864**	**k**

Beams		
Length (ft)	Joist	Weight (lb)
11.5	10K1	57.5
11.5	10K1	57.5
11.5	10K1	57.5
11.5	10K1	57.5
11.5	10K1	57.5
11.5	10K1	57.5
	Total Weight (k) =	**0.345**

High Roof							
Approx. Area	=	12,071	SF				
Ht.	=	7	ft	Total Weight	=	1021.31	k

Walls			
Height	=	2	ft
SW1, Length	=	0	ft
SW2, Length	=	0	ft
SW3, Length	=	0	ft
SW4, Length	=	0	ft
SW5, Length	=	0	ft
SW6, Length	=	0	ft
SW7, Length	=	0	ft
SW8, Length	=	0	ft
SW9, Length	=	32	ft
SW10, Length	=	32	ft
SW11, Length	=	12.33	ft
SW12, Length	=	12.33	ft
SW13, Length	=	0	ft
Unit Wt.	=	145	pcf
Weight	**=**	**0.20**	**k**

Superimposed			
Mechanical	=	0	psf
Misc.	=	10	psf
Finishes	=	0	psf
Roof	=	20	psf
Weight	**=**	**145.492023**	**k**

Floor			
3VLI Deck	=	68	psf
Weight	**=**	**820.8418**	**k**

Columns		
Height	Shape	Weight
7	HSS6X6X1/8	68.95
7	W10X33	231
7	W10X33	231
7	W10X33	231
7	W10X33	231
7	W10X33	231
7	W10X39	273
7	W10X45	315
7	W12X136	952
7	W12X40	280
7	W12X40	280
7	W12X53	371
7	W12X79	553
7	W12X87	609
7	W14X145	1015
7	W14X159	1113
7	W14X159	1113
7	W14X211	1477
7	W14X211	1477
7	W14X43	301
	Total Weight (k) =	**11.35295**

Beams		
Length (ft)	Beam Size	Weight
9.33	W10X12	111.96
9.33	W10X12	111.96
20.54	W12X14	287.56
7.58	W12X14	106.12
7.58	W12X14	106.12
7.58	W12X14	106.12
30	W12X16	480
30	W12X16	480
30	W14X22	660
30	W14X22	660
30	W14X22	660
30	W14X22	660
23.67	W14X34	804.78
23.67	W14X34	804.78
23.67	W14X43	1017.81
30	W16X26	780
19.54	W16X26	508.04
30	W16X36	1080
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
31.5	W21X50	1575
32	W21X50	1600
32	W21X50	1600
32	W21X50	1600
32	W21X50	1600
20.04	24K7	202.404
30	24K7	303
30	24K7	303
30	24K7	303
30	24K7	303
29.5	24K7	297.95
29.5	24K7	297.95
29.5	24K7	297.95
1080	24K7	10908
	Total Weight (k) =	43.416504

Note: Low Roof, Level 4, Level 3, Level 2, and Level 1 detailed building weight calculations are available upon request.

Difference in Dead Load from Calculated to RAM Model					
Level	**Area (SF)**	**Calculated Floor Weight (kip)**	**RAM Model, Floor Weight (kip)**	**% Difference**	**Floor Weight Used for ETABS and Seismic Calcs (k)**
Stair 3	380	37	14.2	-161%	14
High Roof	12,071	1,021	511	-100%	511.27
Low Roof	13,748	2,544	668	-281%	667.55
Level 4	24,275	2,638	3,870	32%	3869.78
Level 3	13,392	1,681	1,473	-14%	1473.26
Level 2	25,867	3,057	2,823	-8%	2823.35
Level 1	23,434	2,744	2,979	8%	2979.48

Appendix C – Wind Load Calculations

ASCE 7-05 Chapter 6 Method 2

Wind Load Design Criteria	
Basic Wind Speed	90 MPH
Wind Importance Factor	IW = 1.15
Building Category	III
Exposure	C
Internal Pressure Coefficient , GCpi	GCPI = 0.18
Apply Directionality Factor	Kd = 0.85
Topography Factor	Kzt = 1.00
Mean Roof Height (ft): Top Story Height + Parapet = 71.83	
Fundamental Frequency, n1 = 75/H = 1.044 >1	Structure is Rigid

Main Wind Force Resisting System – Method 2 | All Heights

Figure 6-6 (con't) | External Pressure Coefficients, C_p | Walls & Roofs

Enclosed, Partially Enclosed Buildings

Wall Pressure Coefficients, C_p			
Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
200 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

External Pressure Coefficients (Cp)				
Wall Pressure Coefficients (C_p)				
Surface	L/B (X)	L/B (Y)	Cp (X)	Cp (Y)
Windward Wall	All Values	All Values	0.8	0.8
Side Wall	All Values	All Values	-0.7	-0.7
Leeward Wall				
Stair 3	2.60	0.39	-0.270	-0.5
High Roof	3.87	0.26	-0.207	-0.5
Low Roof	1.97	0.51	-0.306	-0.5
Level 4	2.04	0.49	-0.298	-0.5
Level 3	2.04	0.49	-0.298	-0.5
Level 2	2.13	0.47	-0.293	-0.5
Level 1	2.13	0.47	-0.293	-0.5
Base	2.13	0.47	-0.293	-0.5
Roof Pressure Coefficients (C_p)				
h/L	X:	0.281	Y:	0.599
			Cp (X)	Cp (Y)
Roof - 0 to h/2			-0.900	-0.979
Roof - h/2 to h			-0.900	-0.861
Roof - h to 2h			-0.500	-0.539
Roof - > 2h			-0.300	-0.379

Importance Factor, I (Wind Loads)

Table 6-1

Category	Non-Hurricane Prone Regions and Hurricane Prone Regions with V = 85-100 mph and Alaska	Hurricane Prone Regions with V > 100 mph
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

Terrain Exposure Constants

Table 6-2

Exposure	α	z_g (ft)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (ft)	$\bar{\epsilon}$	z_{min} (ft)*
B	7.0	1200	1/7	0.84	1/4.0	0.45	0.30	320	1/3.0	30
C	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15
D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7

*z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of 0.6*h* or z_{min}. For buildings with h ≤ z_{min}, $\bar{z}$ shall be taken as z_{min}.

$$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right) \tag{6-4}$$

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

In SI: $I_{\bar{z}} = c\left(\frac{10}{\bar{z}}\right)^{1/6}$

where $I_{\bar{z}}$ = the intensity of turbulence at height $\bar{z}$ where $\bar{z}$ = the equivalent height of the structure defined as $0.6h$, but not less than z_{min} for all building heights h. z_{min} and c are listed for each exposure in Table 6-2; g_Q and g_v shall be taken as 3.4. The background response Q is given by

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

where B, h are defined in Section 6.3; and $L_{\bar{z}}$ = the integral length scale of turbulence at the equivalent height given by

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

Gust Effect Factor (Gf)		
Variable	N-S Wind (Y)	E-W Wind (X)
I (Table 6-1)	1.15	1.15
c (Table 6-2)	0.2	0.2
g_Q	3.4	3.4
g_v	3.4	3.4
Z_{mean}	43.10	43.10
$I_{z, mean}$	0.191	0.191
$L_{z, mean}$	527.43	527.42
	Q	
Stair 3	0.963	0.970
High Roof	0.902	0.954
Low Roof	0.901	0.935
Level 4	0.899	0.935
Level 3	0.899	0.935
Level 2	0.896	0.935
Level 1	0.896	0.935
Base	0.896	0.935
	G	
Stair 3	0.907	0.910
High Roof	0.877	0.903
Low Roof	0.877	0.893
Level 4	0.876	0.893
Level 3	0.876	0.893
Level 2	0.874	0.893
Level 1	0.874	0.893
Base	0.874	0.893

	E-W Wind (X)		N-S Wind (Y)	
PEARL HALL	**B (ft)**	**L (ft)**	**B (ft)**	**L (ft)**
Stair 3	12.33	32	32	12.33
High Roof	60	232.08	232.08	60
Low Roof	120	236.34	236.34	120
Level 4	120	244.67	244.67	120
Level 3	120	244.67	244.67	120
Level 2	120	256	256	120
Level 1	120	256	256	120
Base	120	256	256	120

6.5.10 Velocity Pressure. Velocity pressure, q_z, evaluated at height z shall be calculated by the following equation:

$$q_z = 0.00256K_zK_{zt}K_dV^2I \text{ (lb/ft}^2) \tag{6-15}$$

[In SI: $q_z = 0.613K_zK_{zt}K_dV^2I$ (N/m²); V in m/s]

Velocity Pressure Coefficients (Kz) and Velocity			
Level	Elevation (ft)	Kz	q_z(psf)
Stair 3	71.83	1.177	23.86
High Roof	69.83	1.169	23.70
Low Roof	62.83	1.141	23.13
Level 4	50.50	1.092	22.13
Level 3	38.50	1.031	20.90
Level 2	26.50	0.952	19.30
Level 1	13.50	0.850	17.23
Base	0.00	0.850	17.23

Wind Pressures E-W Direction (X)							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressures (psf)		Net Pressures (psf)	
				(+)(GCpi)	(-)(GCpi)	(+)(GCpi)	(-)(GCpi)
Windward Walls	Stair 3	71.83	17.4	4.30	-4.30	21.7	13.1
	High Roof	69.83	17.1	4.27	-4.27	21.4	12.8
	Low Roof	62.83	16.5	4.16	-4.16	20.7	12.4
	Level 4	50.50	15.8	3.98	-3.98	19.8	11.8
	Level 3	38.50	14.9	3.76	-3.76	18.7	11.2
	Level 2	26.50	13.8	3.47	-3.47	17.3	10.3
	Level 1	13.50	12.3	3.10	-3.10	15.4	9.2
	Base	0.00	12.3	3.10	-3.10	15.4	9.2
Leeward Walls	Stair 3	71.83	-5.9	4.30	-4.30	-1.6	-10.2
	High Roof	69.83	-4.4	4.27	-4.27	-0.2	-8.7
	Low Roof	62.83	-6.3	4.16	-4.16	-2.2	-10.5
	Level 4	50.50	-5.9	3.98	-3.98	-1.9	-9.9
	Level 3	38.50	-5.6	3.76	-3.76	-1.8	-9.3
	Level 2	26.50	-5.1	3.47	-3.47	-1.6	-8.5
	Level 1	13.50	-4.5	3.10	-3.10	-1.4	-7.6
	Base	0.00	-4.5	3.10	-3.10	-1.4	-7.6
Side Walls	All	All	-0.7	4.30	-4.30	3.6	-5.0
Roof - 0 to h/2		0 to 35.92	-0.9	4.30	-4.30	3.4	-5.2
Roof - h/2 to h		35.92 to 71.83	-0.9	4.30	-4.30	3.4	-5.2
Roof - h to 2h		71.83 to 143.66	-0.5	4.30	-4.30	3.8	-4.8
Roof - > 2h		>143.66	-0.3	4.30	-4.30	4.0	-4.6

Wind Pressures N-S Direction (Y)							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressures (psf)		Net Pressures (psf)	
				(+)(GCpi)	(-)(GCpi)	(+)(GCpi)	(-)(GCpi)
Windward Walls	Stair 3	71.83	17.3	4.30	-4.30	21.6	13.0
	High Roof	69.83	16.6	4.27	-4.27	20.9	12.4
	Low Roof	62.83	16.2	4.16	-4.16	20.4	12.1
	Level 4	50.50	15.5	3.98	-3.98	19.5	11.5
	Level 3	38.50	14.6	3.76	-3.76	18.4	10.9
	Level 2	26.50	13.5	3.47	-3.47	17.0	10.0
	Level 1	13.50	12.1	3.10	-3.10	15.2	8.9
	Base	0.00	12.1	3.10	-3.10	15.2	8.9
Leeward Walls	Stair 3	71.83	-10.8	4.30	-4.30	-6.5	-15.1
	High Roof	69.83	-10.4	4.27	-4.27	-6.1	-14.7
	Low Roof	62.83	-10.1	4.16	-4.16	-6.0	-14.3
	Level 4	50.50	-9.7	3.98	-3.98	-5.7	-13.7
	Level 3	38.50	-9.2	3.76	-3.76	-5.4	-12.9
	Level 2	26.50	-8.4	3.47	-3.47	-5.0	-11.9
	Level 1	13.50	-7.5	3.10	-3.10	-4.4	-10.6
	Base	0.00	-7.5	3.10	-3.10	-4.4	-10.6
Side Walls	All	All	-0.7	4.30	-4.30	3.6	-5.0
Roof - 0 to h/2		0 to 35.92	-1.0	4.30	-4.30	3.3	-5.3
Roof - h/2 to h		35.92 to 71.83	-0.9	4.30	-4.30	3.4	-5.2
Roof - h to 2h		71.83 to 143.66	-0.5	4.30	-4.30	3.8	-4.8
Roof - > 2h		>143.66	-0.4	4.30	-4.30	3.9	-4.7

Windward Forces in E-W Direction (Cases 1-IV)

Wind Forces E-W Direction (WIND 1X)										
Floor	B_x (ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
Stair 3	12	2	71.83	1.0	12.0	-	0.0	0.3	0.3	20.0
High Roof	70.7083	7	69.83	3.5	247.5	1.0	70.7	7.0	7.3	507.0
Low Roof	120	12.333	62.83	6.2	740.0	3.5	420.0	26.0	33.2	2138.1
Level 4	120	12	50.50	6.0	720.0	6.2	740.0	32.5	65.8	3781.7
Level 3	120	12	38.50	6.0	720.0	6.0	720.0	30.4	96.2	4951.8
Level 2	120	13	26.50	6.5	780.0	6.0	720.0	29.5	125.6	5732.5
Level 1	120	13.5	13.50	6.8	810.0	6.5	780.0	28.3	153.9	6115.0
Base	120	-	0.00	-	0.0	6.8	810.0	13.6	167.6	6115.0
									Total Base Shear (k) =	168
									Total Overturning Moment (k-ft) =	6115

Wind Forces E-W Direction (WIND 2X)												
Floor	B_x (ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	M_T (+e_x)	M_T (-e_x)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)					
Stair 3	12	2	71.83	1.0	12.0	-	0.0	0.2	0.2	15.0	4.5	-4.5
High Roof	70.7083	7	69.83	3.5	247.5	1.0	70.7	5.2	5.4	380.2	3922.0	-3922.0
Low Roof	120	12.333	62.83	6.2	740.0	3.5	420.0	19.5	24.9	1603.6	42053.8	-42053.8
Level 4	120	12	50.50	6.0	720.0	6.2	740.0	24.4	49.3	2836.3	52727.5	-52727.5
Level 3	120	12	38.50	6.0	720.0	6.0	720.0	22.8	72.1	3713.9	49235.2	-49235.2
Level 2	120	13	26.50	6.5	780.0	6.0	720.0	22.1	94.2	4299.4	47725.4	-47725.4
Level 1	120	13.5	13.50	6.8	810.0	6.5	780.0	21.2	115.5	4586.3	45896.5	-45896.5
Base	120	-	0.00	-	0.0	6.8	810.0	10.2	125.7	4586.3	22081.4	-22081.4
									Total Base Shear (k) =	126		
									Total Overturning Moment (k-ft) =	4586		

Wind Forces E-W Direction (WIND 3X)										
Floor	B_x (ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
Stair 3	12	2	71.83	1.0	12.0	-	0.0	0.2	0.2	15.0
High Roof	70.7083	7	69.83	3.5	247.5	1.0	70.7	5.2	5.4	380.2
Low Roof	120	12.333	62.83	6.2	740.0	3.5	420.0	19.5	24.9	1603.6
Level 4	120	12	50.50	6.0	720.0	6.2	740.0	24.4	49.3	2836.3
Level 3	120	12	38.50	6.0	720.0	6.0	720.0	22.8	72.1	3713.9
Level 2	120	13	26.50	6.5	780.0	6.0	720.0	22.1	94.2	4299.4
Level 1	120	13.5	13.50	6.8	810.0	6.5	780.0	21.2	115.5	4586.3
Base	120	-	0.00	-	0.0	6.8	810.0	10.2	125.7	4586.3
									Total Base Shear (k) =	126
									Total Overturning Moment (k-ft) =	4586

Wind Forces E-W Direction (WIND 4X)												
Floor	B_x (ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	$M_{T\ (-e_x, -e_y)}$	$M_{T\ (+e_x, -e_y)}$
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)					
Stair 3	12	2	71.83	1.0	12.0	-	0.0	0.2	0.2	11.3	-81.3	-74.5
High Roof	70.7083	7	69.83	3.5	247.5	1.0	70.7	3.9	4.1	285.4	-106932.4	-101044.1
Low Roof	120	12.333	62.83	6.2	740.0	3.5	420.0	14.6	18.7	1203.7	-316459.7	-253322.9
Level 4	120	12	50.50	6.0	720.0	6.2	740.0	18.3	37.0	2129.1	-420910.0	-341748.4
Level 3	120	12	38.50	6.0	720.0	6.0	720.0	17.1	54.1	2787.9	-400563.6	-326645.1
Level 2	120	13	26.50	6.5	780.0	6.0	720.0	16.6	70.7	3227.4	-431119.3	-359467.5
Level 1	120	13.5	13.50	6.8	810.0	6.5	780.0	16.0	86.7	3442.7	-423713.3	-354807.3
Base	120	-	0.00	-	0.0	6.8	810.0	7.7	94.3	3442.7	-203853.5	-170702.0
									Total Base Shear (k) =	94		
									Total Overturning Moment (k-ft) =	3443		

Windward Forces in N-S Direction (Cases 1-IV)

Wind Forces N-S Direction (WIND 1Y)										
Floor	B_y(ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
Stair 3	32	2	71.83	1.0	32.0	-	0.0	0.9	0.9	64.7
High Roof	232.08	7	69.83	3.5	812.3	1.0	32.0	22.9	23.8	1661.2
Low Roof	236.34	12.333	62.83	6.2	1457.4	3.5	812.3	60.4	84.2	5456.0
Level 4	244.67	12	50.50	6.0	1468.0	6.2	1457.4	75.4	159.6	9265.2
Level 3	244.67	12	38.50	6.0	1468.0	6.0	1468.0	71.9	231.5	12034.2
Level 2	256	13	26.50	6.5	1664.0	6.0	1468.0	71.4	302.9	13926.9
Level 1	256	13.5	13.50	6.8	1728.0	6.5	1664.0	70.3	373.3	14876.4
Base	256	-	0.00	-	0.0	6.8	1728.0	33.8	407.1	14876.4
									Total Base Shear (k) =	407
									Total Overturning Moment (k-ft) =	14876

Wind Forces N-S Direction (WIND2Y)												
Floor	B_Y(ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	M_T (+e_y)	M_T (-e_y)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)					
Stair 3	32	2	71.83	1.0	32.0	-	0.0	0.7	0.7	48.5	103.7	-103.7
High Roof	232.08	7	69.83	3.5	812.3	1.0	32.0	17.1	17.8	1245.9	138527.8	-138527.8
Low Roof	236.34	12.333	62.83	6.2	1457.4	3.5	812.3	45.3	63.1	4092.0	379517.7	-379517.7
Level 4	244.67	12	50.50	6.0	1468.0	6.2	1457.4	56.6	119.7	6948.9	507987.4	-507987.4
Level 3	244.67	12	38.50	6.0	1468.0	6.0	1468.0	53.9	173.6	9025.7	484375.2	-484375.2
Level 2	256	13	26.50	6.5	1664.0	6.0	1468.0	53.6	227.2	10445.2	526589.7	-526589.7
Level 1	256	13.5	13.50	6.8	1728.0	6.5	1664.0	52.7	279.9	11157.3	518552.8	-518552.8
Base	256	-	0.00	-	0.0	6.8	1728.0	25.4	305.3	11157.3	249481.9	-249481.9
									Total Base Shear (k) =	305		
									Total Overturning Moment (k-ft) =	11157		

Wind Forces N-S Direction (WIND3Y)										
Floor	B_Y(ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
Stair 3	32	2	71.83	1.0	32.0	-	0.0	0.7	0.7	48.5
High Roof	232.08	7	69.83	3.5	812.3	1.0	32.0	17.1	17.8	1245.9
Low Roof	236.34	12.333	62.83	6.2	1457.4	3.5	812.3	45.3	63.1	4092.0
Level 4	244.67	12	50.50	6.0	1468.0	6.2	1457.4	56.6	119.7	6948.9
Level 3	244.67	12	38.50	6.0	1468.0	6.0	1468.0	53.9	173.6	9025.7
Level 2	256	13	26.50	6.5	1664.0	6.0	1468.0	53.6	227.2	10445.2
Level 1	256	13.5	13.50	6.8	1728.0	6.5	1664.0	52.7	279.9	11157.3
Base	256	-	0.00	-	0.0	6.8	1728.0	25.4	305.3	11157.3
									Total Base Shear (k) =	305
									Total Overturning Moment (k-ft) =	11157

Wind Forces N-S Direction (WIND4Y)												
Floor	B_Y(ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	$M_{T\ (+}e_x,+e_y)$	$M_{T\ (-}\ e_x,+e_y)$
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)					
Stair 3	32	2	71.83	1.0	32.0	-	0.0	0.5	0.5	36.4	81.3	74.5
High Roof	232.08	7	69.83	3.5	812.3	1.0	32.0	12.9	13.4	935.2	106932.4	101044.1
Low Roof	236.34	12.333	62.83	6.2	1457.4	3.5	812.3	34.0	47.4	3071.7	316459.7	253322.9
Level 4	244.67	12	50.50	6.0	1468.0	6.2	1457.4	42.5	89.8	5216.3	420910.0	341748.4
Level 3	244.67	12	38.50	6.0	1468.0	6.0	1468.0	40.5	130.3	6775.3	400563.6	326645.1
Level 2	256	13	26.50	6.5	1664.0	6.0	1468.0	40.2	170.6	7840.9	431119.3	-431119.3
Level 1	256	13.5	13.50	6.8	1728.0	6.5	1664.0	39.6	210.1	8375.4	423713.3	-423713.3
Base	256	-	0.00	-	0.0	6.8	1728.0	19.1	229.2	8375.4	203853.5	-203853.5
									Total Base Shear (k) =	229		
									Total Overturning Moment (k-ft) =	8375		

Approximate Fundamental Frequency. To estimate the dynamic response of structures, knowledge of the fundamental frequency (lowest natural frequency) of the structure is essential. This value would also assist in determining if the dynamic response estimates are necessary. Most computer codes used in the analysis of structures would provide estimates of the natural frequencies of the structure being analyzed. However, for the preliminary design stages some empirical relationships for building period T_a ($T_a = 1/n_1$) are available in the earthquake chapters of ASCE 7. However, it is noteworthy that these expressions are based on recommendations for earthquake design with inherent bias toward higher estimates of fundamental frequencies [Refs. C6-48, C6-49]. For wind design applications these values may be unconservative because an estimated frequency higher than the actual frequency would yield lower values of the gust effect factor and concomitantly a lower design wind pressure. However, [Refs. C6-48, C6-49] also cite lower bound estimates of frequency that are more suited for use in wind applications. These expressions are

For steel Moment-Resisting-Frames MRFs

$$n_1 = 22.2/H^{0.8} \tag{C6-14}$$

For concrete MRFs: $n_1 = 43.5/H^{0.9}$ (C6-15)

For concrete shearwall systems:

$$n_1 = 385(C_w)^{0.5}/H \tag{C6-16}$$

where

$$c_w = \frac{100}{A_B}\sum_{i=1}^{n}\left(\frac{H}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

n_1 = building natural frequency (Hz)
H = building height (ft)
n = number of shear walls in the building effective in resisting lateral forces in the direction under consideration
A_B = base area of the structure (ft^2)
A_i = area of shear wall (ft^2)
D_i = length of shear wall "i" (ft)
h_i = height of shear wall "i" (ft)

Observation from wind tunnel testing of buildings where frequency is calculated using analysis software reveals the following expression for frequency, applicable to all buildings in steel or concrete:

$n_1 = 100/H$ (ft) average value (C6-17)

$n_1 = 75/H$ (ft) lower bound value (C6-18)

Appendix D – Seismic Load Calculations

ASCE 7-05 Equivalent Lateral Force Method (Special Reinforced Concrete Shear Walls)

Seismic Load Design Criteria	ASCE 7-05		
Building Height (h), ft	71.830		
Occupancy Category	III		*Table 1-1*
Ss	0.564	g	*§11.4.1, Fig. 22-1*
S1	0.170	g	*§11.4.1, Fig.22-2*
Importance Factor	1.250		*Table 11.5-1*
Soil Site Class	D		*§11.4.2*
Seismic Design Category	D		*Table 11.6-1*
Fa	1.349		*Table 11.4-1*
Fv	2.120		*Table 11.4-2*
SMS	0.761	g	*EQ. 11.4-1*
SM1	0.360	g	*EQ. 11.4-2*
SDS	0.507	g	*EQ. 11.4-3*
SD1	0.240	g	*EQ. 11.4-4*

Period Determination				
C_t		0.020		*Table 12.8-2*
x		0.750		*Table 12.8-2*
TL		6.000	sec	*Fig. 22-15*
Cu		1.46		*Table 12.8-1*
$T_a = C_t h_n^x$		0.493	sec	*EQ. 12.8-7*
$C_w = \frac{100}{A_B}\sum_{i=1}^{x}\left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$				*EQ. 12.8-10*
	X	0.11		
	Y	0.10		
$T_a = \frac{0.0019}{\sqrt{C_w}} h_n$	X	0.420	sec	*EQ. 12.8-9*
	Y	0.430	sec	
Tx		0.295	sec	*ETABS*
Ty		0.5243	sec	ETABS

Calculation of Seismic Response Coefficient Special Reinforced Concrete Shear Walls				
R (Special reinforced concrete shear walls)		6.000		*Table 12.2-1*
$C_s = \frac{S_{DS}}{(R/I)}$		0.106		*EQ. 12.8-2*
$C_s = \frac{S_{D1}}{T(R/I)}, T \le T_L$	X	0.119		*EQ. 12.8-3*
	Y	0.116		
$C_s \geq 0.01$		ok		*EQ. 12.8-5*
$C_s =$	X	0.106		*ETABS*
	Y	0.096		ETABS
k	X	1.00		*§12.8.3*
k	Y	1.01		*§12.8.3*
Base shear, V = C_sW		1764.0	kip	*EQ. 12.8-1*
Base shear, V = C_sW		1593.7	kip	*EQ. 12.8-1*

Calculation of Cw

for the calculation of the approximate fundamental period, Ta for concrete shear wall structures

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Direction	hi	Di	Ai	CW/(100/AB)
1	12	34.00	90	62.83	X	62.83	34.00	34.00	11.59
2	12	34.00	90	69.83	X	69.83	34.00	34.00	7.99
3	12	33.00	0	69.83	Y	69.83	33.00	33.00	7.40
4	12	33.00	0	62.83	Y	62.83	33.00	33.00	10.76
5	12	23.17	0	69.83	Y	69.83	23.17	23.17	2.87
6	12	23.17	0	69.83	Y	69.83	23.17	23.17	2.87
7	12	10.33	90	68.83	X	68.83	10.33	10.33	0.30
8	12	10.33	90	69.83	X	69.83	10.33	10.33	0.28
9	12	32.00	90	71.83	X	71.83	32.00	32.00	6.18
10	12	32.00	90	71.83	X	71.83	32.00	32.00	6.18
11	18	12.33	0	71.83	Y	71.83	12.33	18.50	0.63
12	18	12.33	0	71.83	Y	71.83	12.33	18.50	0.63
13	24	23.17	0	69.83	Y	69.83	23.17	46.34	5.74
AB =	30720	$C_w = \frac{100}{A_B}\sum_{i=1}^{x}\left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$						Shear Walls 1-13	
hn=	71.83							CW - X =	0.106
								CW - Y =	0.101

Exisiting Special Reinforced Shear Walls - Seismic Forces (E-W Dirction, X)							
Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	31.0	71.83	2226.3	0.00	6.7	7	481
High Roof	736.3	69.83	51414.1	0.09	154.8	161	11291
Low Roof	1208.6	62.83	75936.6	0.13	228.6	390	25655
Level 4	4373.9	50.50	220881.7	0.38	665.0	1055	59238
Level 3	2046.4	38.50	78787.6	0.13	237.2	1292	68371
Level 2	3438.7	26.50	91125.2	0.16	274.4	1567	75641
Level 1	4853.5	13.50	65522.8	0.11	197.3	1764	78304
Base	-	0.00	0.0	0.00	0.0	1764	78304
		$\Sigma w_x h_x^k$	585894.3	1.0	Total Building Weight, k =		16,688
k = 1.000						Base Shear,k =	1,764
T = 0.295						Total Moment,k-ft =	78,304

Exisiting Special Reinforced Shear Walls - Seismic Forces (N-S Dirction, Y)							
Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	31.0	71.83	2345.0	0.00	6.1	6	438
High Roof	736.3	69.83	54136.2	0.09	140.8	147	10270
Low Roof	1208.6	62.83	79854.4	0.13	207.7	355	23318
Level 4	4373.9	50.50	231661.9	0.38	602.5	957	53743
Level 3	2046.4	38.50	82360.9	0.13	214.2	1171	61990
Level 2	3438.7	26.50	94826.8	0.15	246.6	1418	68525
Level 1	4853.5	13.50	67628.0	0.11	175.9	1594	70900
Base	-	0.00	0.0	0.00	0.0	1594	70900
		$\Sigma w_x h_x^k$	612813.1	1.0	Total Building Weight, k =		16,688
k = 1.012						Base Shear,k =	1,594
T = 0.524						Total Moment,k-ft =	70,900

% Difference of ETABS from Hand Calculations			
Level	**Hand Calculated Fx (k) = V*C_{vx}**	**ETABS Fx (k)**	**% Difference**
Stair 3	6.7	6.7	0.04%
High Roof	154.8	154.7	0.04%
Low Roof	228.6	228.5	0.04%
Level 4	665.0	664.7	0.04%
Level 3	237.2	237.1	0.04%
Level 2	274.4	274.2	0.04%
Level 1	197.3	197.2	0.04%
Base Shear	1,764.0	1,763.2	0.04%
< 10%, therefore can use ETABS Calculated Seismic Forces			

% Difference of ETABS from Hand Calculations			
Level	**Hand Calculated Fy (k) = V*C_{vx}**	**ETABS Fy (k)**	**% Difference**
Stair 3	6.1	6.1	-0.02%
High Roof	140.8	140.76	0.02%
Low Roof	207.7	207.63	0.02%
Level 4	602.5	602.34	0.02%
Level 3	214.2	214.15	0.02%
Level 2	246.6	246.56	0.02%
Level 1	175.9	175.84	0.02%
Base Shear	1593.7	1593.37	0.02%
< 10%, therefore can use ETABS Calculated Seismic Forces			

ASCE 7-05 Equivalent Lateral Force Method (Special Reinforced Concrete Shear Walls)

Seismic Load Design Criteria			ASCE 7-05
Building Height (h), ft	71.830		
Occupancy Category	III		*Table 1-1*
S_S	0.564	g	*§11.4.1, Fig. 22-1*
S_1	0.170	g	*§11.4.1, Fig.22-2*
Importance Factor	1.250		*Table 11.5-1*
Soil Site Class	D		*§11.4.2*
Seismic Design Category	D		*Table 11.6-1*
F_a	1.349		*Table 11.4-1*
F_v	2.120		*Table 11.4-2*
S_{MS}	0.761	g	*EQ. 11.4-1*
S_{M1}	0.360	g	*EQ. 11.4-2*
S_{DS}	0.507	g	*EQ. 11.4-3*
S_{D1}	0.240	g	*EQ. 11.4-4*

Calculation of Seismic Response Coefficient Special Reinforced Concrete Shear Walls (Design #2)				
R (Special reinforced concrete shear walls)		**6.000**		***Table 12.2-1***
Tx		0.211	s	*ETABS*
Ty		0.4105	s	ETABS
$C_s = \frac{S_{DS}}{(R/I)}$		**0.106**		***EQ. 12.8-2***
$C_s = \frac{S_{D1}}{T(R/I)}, T \le T_L$	X	0.237		*EQ. 12.8-3*
	Y	0.122		
$C_s \geq 0.01$		**ok**		***EQ. 12.8-5***
$C_s =$	X	0.106		*ETABS*
	Y	0.106		*ETABS*
k	**X**	**1.00**		***§12.8.3***
k	**Y**	**0.96**		***§12.8.3***
Base shear, $V = C_s * W$		1792.3	kip	*EQ. 12.8-1*
Base shear, $V = C_s * W$		1792.3	kip	*EQ. 12.8-1*

Modified Special Reinforced Shear Walls - Seismic Forces (E-W Dirction, X)

Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V^*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	32.0	71.83	2298.2	0.00	6.9	7	497
High Roof	749.6	69.83	52342.2	0.09	157.6	165	11502
Low Roof	1207.9	62.83	75895.3	0.13	228.5	393	25860
Level 4	4469.1	50.50	225692.0	0.38	679.6	1073	60178
Level 3	2071.6	38.50	79756.4	0.13	240.1	1313	69423
Level 2	3494.0	26.50	92591.4	0.16	278.8	1592	76811
Level 1	4939.4	13.50	66682.0	0.11	200.8	1792	79522
Base	-	0.00	0.0	0.00	0.0	1792	79522
		$\Sigma w_x h_x^k$	595257.5	1.0	Total Building Weight, k =		16,964
k = 1.000						Base Shear,k =	1,792
T = 0.211						Total Moment,k-ft =	79,522

Modified Special Reinforced Shear Walls - Seismic Forces (N-S Dirction, Y)

Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V^*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	32.0	71.83	2298.2	0.00	6.9	7	497
High Roof	749.6	69.83	52342.2	0.09	157.6	165	11502
Low Roof	1207.9	62.83	75895.3	0.13	228.5	393	25860
Level 4	4469.1	50.50	225692.0	0.38	679.6	1073	60178
Level 3	2071.6	38.50	79756.4	0.13	240.1	1313	69423
Level 2	3494.0	26.50	92591.4	0.16	278.8	1592	76811
Level 1	4939.4	13.50	66682.0	0.11	200.8	1792	79522
Base	-	0.00	0.0	0.00	0.0	1792	79522
		$\Sigma w_x h_x^k$	595257.5	1.0	Total Building Weight, k =		16,964
k = 1.000						Base Shear,k =	1,792
T = 0.411						Total Moment,k-ft =	79,522

ASCE 7-05 Equivalent Lateral Force Method (Ordinary Steel Concentric Brace Frames)

Seismic Load Design Criteria			ASCE 7-05
Building Height (h), ft	71.830		
Occupancy Category	III		*Table 1-1*
S_S	0.564	g	*§11.4.1, Fig. 22-1*
S_1	0.170	g	*§11.4.1, Fig.22-2*
Importance Factor	1.250		*Table 11.5-1*
Soil Site Class	D		*§11.4.2*
Seismic Design Category	D		*Table 11.6-1*
F_a	1.349		*Table 11.4-1*
F_v	2.120		*Table 11.4-2*
S_{MS}	0.761	g	*EQ. 11.4-1*
S_{M1}	0.360	g	*EQ. 11.4-2*
S_{DS}	0.507	g	*EQ. 11.4-3*
S_{D1}	0.240	g	*EQ. 11.4-4*

Calculation of Seismic Response Coefficient Special Concentric Brace Frames				
$T_a = C_t h_n^x$		0.493	sec	*EQ. 12.8-7*
Tx		0.720	s	*ETABS*
Ty		0.720	s	ETABS
C_t		0.020		*Table 12.8-2*
x		0.750		*Table 12.8-2*
R (Special Steel Concentric Brace Frames)		6.000		*Table 12.2-1*
$C_s = \frac{S_{DS}}{(R/I)}$		0.106		*EQ. 12.8-2*
$C_s = \frac{S_{D1}}{T(R/I)}, T \le T_L$	X	0.069		*EQ. 12.8-3*
	Y	0.069		
$C_s \geq 0.01$		ok		*EQ. 12.8-5*
$C_s =$	X	0.069		*ETABS*
	Y	0.069		ETABS
k	X	1.11		*§12.8.3*
k	Y	1.11		*§12.8.3*
Base shear, $V = C_s W$		853.2	kip	*EQ. 12.8-1*
Base shear, $V = C_s W$		853.2	kip	*EQ. 12.8-1*

SCBF - Seismic Forces (E-W Dirction, X)

Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = $V*C_{vx}$	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	14.2	71.83	1633.7	0.00	2.1	2	149
High Roof	511	69.83	57004.1	0.08	72.4	74	5206
Low Roof	668	62.83	66192.5	0.10	84.1	159	10489
Level 4	3,870	50.50	301078.9	0.45	382.5	541	29803
Level 3	1,473	38.50	84812.1	0.13	107.7	649	33951
Level 2	2,823	26.50	107362.4	0.16	136.4	785	37565
Level 1	2,979	13.50	53584.2	0.08	68.1	853	38484
Base	-	0.00	0.0	0.00	0.0	853	38484
		$\Sigma w_x h_x^k$	671667.9	1.0	Total Building Weight, k =		12,339
k = 1.110						Base Shear,k =	853
T = 0.720						Total Moment,k-ft =	38,484

SCBF - Seismic Forces (N-S Dirction, Y)

Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = $V*C_{vx}$	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	14.2	71.83	1633.7	0.00	2.1	2	149
High Roof	511	69.83	57004.1	0.08	72.4	74	5206
Low Roof	668	62.83	66192.5	0.10	84.1	159	10489
Level 4	3,870	50.50	301078.9	0.45	382.5	541	29803
Level 3	1,473	38.50	84812.1	0.13	107.7	649	33951
Level 2	2,823	26.50	107362.4	0.16	136.4	785	37565
Level 1	2,979	13.50	53584.2	0.08	68.1	853	38484
Base	-	0.00	0.0	0.00	0.0	853	38484
		$\Sigma w_x h_x^k$	671667.9	1.0	Total Building Weight, k =		12,339
k = 1.110						Base Shear,k =	853
T = 0.720						Total Moment,k-ft =	38,484

ASCE 7-05 Equivalent Lateral Force Method (Special Steel Moment Frames)

Calculation of Seismic Response Coefficient Special Steel Moment Frames				
$T_a = C_t h_n^x$		0.855	sec	*EQ. 12.8-7*
Tx		0.789	s	*ETABS*
Ty		1.0142	s	ETABS
C_t		0.028		*Table 12.8-2*
x		0.800		*Table 12.8-2*
R (Special Steel Moment Frames)		8.000		*Table 12.2-1*
$C_s = \frac{S_{DS}}{(R/I)}$		0.079		*EQ. 12.8-2*
$C_s = \frac{S_{D1}}{T(R/I)}, T \le T_L$	X	0.048		*EQ. 12.8-3*
	Y	0.037		
$C_s \ge 0.01$		ok		*EQ. 12.8-5*
$C_s =$	X	0.048		*ETABS*
	Y	0.037		*ETABS*
k	X	1.14		*§12.8.3*
k	Y	1.26		*§12.8.3*
Base shear, Vx = C_s*W		584.6	kip	*EQ. 12.8-1*
Base shear, Vy = C_s*W		454.5	kip	*EQ. 12.8-1*

Seismic Load Design Criteria	ASCE 7-05		
Building Height (h), ft	71.830		
Occupancy Category	III		*Table 1-1*
S_S	0.564	g	*§11.4.1, Fig. 22-1*
S_1	0.170	g	*§11.4.1, Fig.22-2*
Importance Factor	1.250		*Table 11.5-1*
Soil Site Class	D		*§11.4.2*
Seismic Design Category	D		*Table 11.6-1*
F_a	1.349		*Table 11.4-1*
F_v	2.120		*Table 11.4-2*
S_{MS}	0.761	g	*EQ. 11.4-1*
S_{M1}	0.360	g	*EQ. 11.4-2*
S_{DS}	0.507	g	*EQ. 11.4-3*
S_{D1}	0.240	g	*EQ. 11.4-4*

SMF - Seismic Forces (E-W Dirction, X)

Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	14	71.83	1909.1	0.00	1.5	1	105
High Roof	517	69.83	66638.0	0.09	51.1	53	3671
Low Roof	633	62.83	72253.2	0.09	55.4	108	7150
Level 4	3854	50.50	342792.7	0.45	262.7	371	20416
Level 3	1479	38.50	96415.2	0.13	73.9	444	23260
Level 2	3025	26.50	128629.9	0.17	98.6	543	25873
Level 1	2757	13.50	54189.2	0.07	41.5	585	26433
Base	-	0.00	0.0	0.00	0.0	585	26433
		$\Sigma w_x h_x^k$	762827.4	1.0	Total Building Weight, k =		12,280
k = 1.144						Base Shear,k =	585
T = 0.789						Total Moment,k-ft =	26,433

SMF - Seismic Forces (N-S Dirction, Y)

Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	14	71.83	3091.9	0.00	1.2	1	87
High Roof	517	69.83	107579.6	0.09	42.0	43	3021
Low Roof	633	62.83	115263.2	0.10	45.0	88	5849
Level 4	3854	50.50	533534.9	0.46	208.4	297	16372
Level 3	1479	38.50	145541.0	0.13	56.8	353	18560
Level 2	3025	26.50	186159.0	0.16	72.7	426	20487
Level 1	2757	13.50	72679.9	0.06	28.4	455	20870
Base	-	0.00	0.0	0.00	0.0	455	20870
		$\Sigma w_x h_x^k$	1163849.5	1.0	Total Building Weight, k =		12,280
k = 1.257						Base Shear,k =	455
T = 1.014						Total Moment,k-ft =	20,870

ASCE 7-05 Chapter 12: Horizontal Irregularities

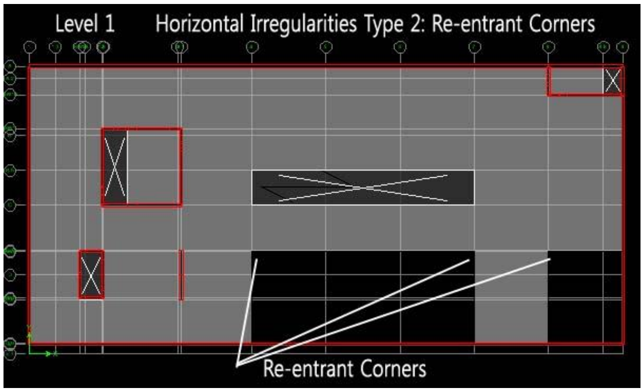
Special Reinf. Shear WallAmplification Factor, Ao in the E-W Direction								
Story	δx	δxpe	δavg	δmax	Ax	Ax (used)	% torsion Δ	Torsion Irreg.
Stair 3	0.17	0.14	0.17	0.31	2.4	2.4	1.9	Irregular, 1a
HGH ROOF	0.23	0.22	0.23	0.45	2.7	2.7	2.0	Irregular, 1a
LOW ROOF	0.22	0.21	0.22	0.42	2.6	2.6	1.9	Irregular, 1a
STORY4	0.17	0.16	0.17	0.34	2.6	2.6	2.0	Irregular, 1a
STORY3	0.12	0.11	0.12	0.23	2.6	2.6	2.0	Irregular, 1a
STORY2	0.07	0.07	0.07	0.13	2.6	2.6	1.9	Irregular, 1a
STORY1	0.01	0.01	0.01	0.02	2.7	2.7	2.0	Irregular, 1a
Special Reinf. Shear Wall,Amplification Factor, Ao in the N-S Direction								
Story	**δy**	**δype**	**δavg**	**δmax**	**Ax**	**Ax (used)**	**% torsion Δ**	**Torsion Irreg.**
Stair 3	0.63	0.69	0.63	1.31	3.1	3.1	2.1	Irregular, 1a
HGH ROOF	0.72	0.80	0.72	1.52	3.1	3.1	2.1	Irregular, 1a
LOW ROOF	0.13	0.16	0.13	0.29	3.2	3.2	2.2	Irregular, 1a
STORY4	0.39	0.43	0.39	0.82	3.1	3.1	2.1	Irregular, 1a
STORY3	0.25	0.28	0.25	0.53	3.1	3.1	2.1	Irregular, 1a
STORY2	0.04	0.05	0.04	0.09	3.3	3.3	2.2	Irregular, 1a
STORY1	0.02	0.02	0.02	0.03	3.0	3.0	2.1	Irregular, 1a
Modified Special Reinf. Shear Wall, Amplification Factor, Ao in the E-W Direction								
Story	**δx**	**δxpe**	**δavg**	**δmax**	**Ax**	**Ax (used)**	**% torsion Δ**	**Torsion Irreg.**
Stair 3	0.17	0.17	0.17	0.34	2.8	2.8	2.0	Irregular, 1a
HGH ROOF	0.26	0.24	0.26	0.50	2.6	2.6	1.9	Irregular, 1a
LOW ROOF	0.24	0.22	0.24	0.46	2.6	2.6	1.9	Irregular, 1a
STORY4	0.17	0.16	0.17	0.34	2.6	2.6	1.9	Irregular, 1a
STORY3	0.11	0.10	0.11	0.21	2.6	2.6	1.9	Irregular, 1a
STORY2	0.06	0.05	0.06	0.11	2.6	2.6	1.9	Irregular, 1a
STORY1	0.00	0.00	0.00	0.01	2.7	2.7	2.0	Irregular, 1a
Modified Special Reinf. Shear Wall, Amplification Factor, Ao in the N-S Direction								
Story	**δy**	**δype**	**δavg**	**δmax**	**Ax**	**Ax (used)**	**% torsion Δ**	**Torsion Irreg.**
Stair 3	0.56	0.61	0.58	0.61	0.8	1.0	1.0	Good
HGH ROOF	0.63	0.69	0.66	0.69	0.8	1.0	1.0	Good
LOW ROOF	0.47	0.52	0.49	0.52	0.8	1.0	1.1	Good
STORY4	0.34	0.37	0.36	0.37	0.8	1.0	1.0	Good
STORY3	0.21	0.24	0.22	0.24	0.8	1.0	1.0	Good
STORY2	0.10	0.11	0.11	0.11	0.8	1.0	1.0	Good
STORY1	0.01	0.01	0.01	0.01	0.7	1.0	1.0	Good

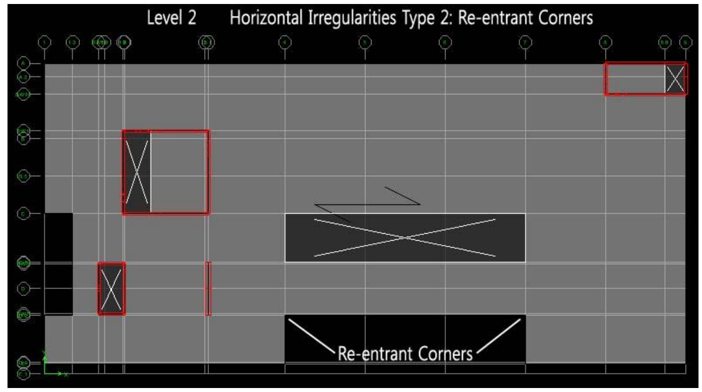
SCBF, Amplification Factor, Ao in the E-W Direction							
Story	δx	δxpe	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.14	0.14	0.14	0.27	2.8	2.0	Irregular, 1a
HGH ROOF	0.20	0.19	0.20	0.39	2.6	1.9	Irregular, 1a
LOW ROOF	0.19	0.18	0.19	0.37	2.6	1.9	Irregular, 1a
STORY4	0.16	0.15	0.16	0.31	2.6	1.9	Irregular, 1a
STORY3	0.11	0.10	0.11	0.21	2.6	1.9	Irregular, 1a
STORY2	0.06	0.06	0.06	0.12	2.6	1.9	Irregular, 1a
STORY1	0.00	0.00	0.00	0.01	2.7	2.0	Irregular, 1a

SCBF, Amplification Factor, Ao in the N-S Direction							
Story	δy	δype	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.48	0.54	0.48	1.03	3.1	2.1	Irregular, 1a
HGH ROOF	0.82	0.92	0.82	1.74	3.1	2.1	Irregular, 1a
LOW ROOF	0.43	0.48	0.43	0.91	3.1	2.1	Irregular, 1a
STORY4	0.32	0.36	0.32	0.69	3.1	2.1	Irregular, 1a
STORY3	0.21	0.24	0.21	0.46	3.1	2.1	Irregular, 1a
STORY2	0.11	0.12	0.11	0.23	3.1	2.1	Irregular, 1a
STORY1	0.00	0.00	0.00	0.00	2.9	2.0	Irregular, 1a

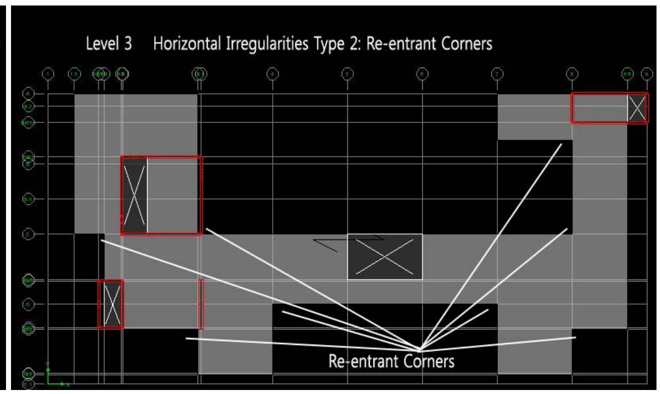
SMF, Amplification Factor, Ao in the E-W Direction							
Story	δx	δxpe	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.28	0.29	0.28	0.57	2.8	2.0	Irregular, 1a
HGH ROOF	0.43	0.41	0.43	0.83	2.6	1.9	Irregular, 1a
LOW ROOF	0.50	0.48	0.50	0.98	2.7	2.0	Irregular, 1a
STORY4	0.34	0.33	0.34	0.67	2.6	1.9	Irregular, 1a
STORY3	0.25	0.23	0.25	0.48	2.6	1.9	Irregular, 1a
STORY2	0.14	0.14	0.14	0.28	2.6	1.9	Irregular, 1a
STORY1	0.00	0.00	0.00	0.01	2.7	2.0	Irregular, 1a

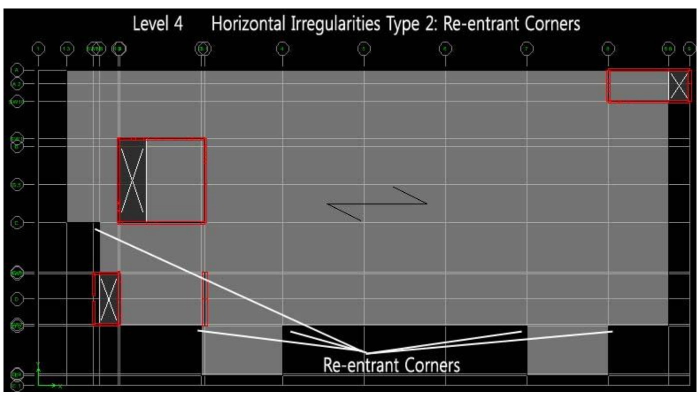
SMF, Amplification Factor, Ao in the N-S Direction							
Story	δy	δype	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.87	0.99	0.87	1.86	3.2	2.1	Irregular, 1a
HGH ROOF	1.41	1.59	1.41	3.01	3.1	2.1	Irregular, 1a
LOW ROOF	0.80	0.90	0.80	1.70	3.2	2.1	Irregular, 1a
STORY4	0.65	0.73	0.65	1.38	3.1	2.1	Irregular, 1a
STORY3	0.48	0.54	0.48	1.02	3.2	2.1	Irregular, 1a
STORY2	0.31	0.35	0.31	0.66	3.2	2.1	Irregular, 1a
STORY1	0.00	0.01	0.00	0.01	2.9	2.1	Irregular, 1a

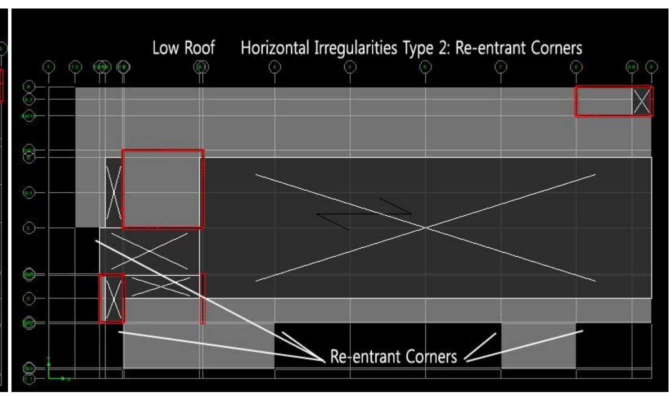

Level 1 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners

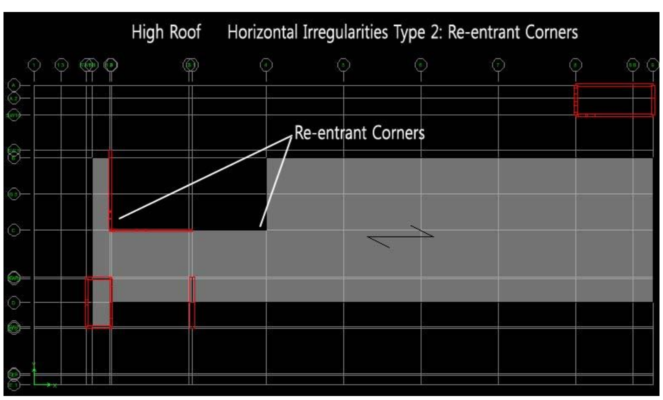

Level 2 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners

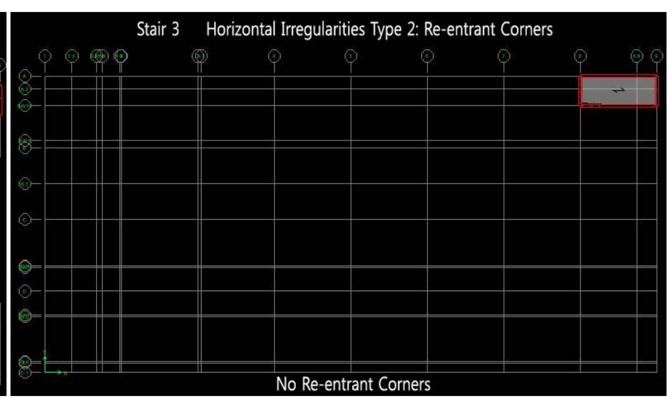

Level 2 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


Level 3 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


Level 4 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


Low Roof Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


High Roof Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


Stair 3 Horizontal Irregularities Type 2: Re-entrant Corners
No Re-entrant Corners

HORIZONTAL STRUCTURAL IRREGULARITIES Type 3: Diaphragm Discontinuity Irregularity				
Story	Total Area (SF)	Area w/o Openings (SF)	% Open	ASCE 7-05 TABLE 12.3-1
Stair 3	380	380	0%	Ok
High Roof	16410.377	12,071	26%	Ok
Low Roof	29360.4	13,748	53%	Not Ok
Level 4	29360.4	24,275	17%	Ok
Level 3	29360.4	13,392	54%	Not Ok
Level 2	30720	25,867	16%	Ok
Level 1	30720	23,434	24%	Ok

ASCE 7-05 Chapter 12: Vertical Irregularities

Special Reinforced Shear Wall - Vertical Irregularity 1a X Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	1.9E-03	1.3E-03	1.5E-03	--	ok
High Roof	3.8E-04	2.7E-04	3.1E-04	--	ok
Low Roof	2.1E-04	1.5E-04	1.7E-04	--	ok
Level 4	2.8E-04	2.0E-04	2.3E-04	6.6E-04	not ok
Level 3	3.3E-04	2.3E-04	2.6E-04	2.3E-04	ok
Level 2	3.0E-04	2.1E-04	2.4E-04	2.2E-04	ok
Level 1	5.0E-05	3.5E-05	4.0E-05	2.4E-04	not ok

Special Reinforced Shear Wall - Vertical Irregularity 1a Y Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	6.1E-03	4.3E-03	4.9E-03	--	ok
High Roof	1.8E-03	1.3E-03	1.5E-03	--	ok
Low Roof	3.7E-04	2.6E-04	2.9E-04	--	ok
Level 4	5.9E-04	4.1E-04	4.7E-04	2.2E-03	not ok
Level 3	5.1E-04	3.6E-04	4.1E-04	7.5E-04	not ok
Level 2	4.2E-04	3.0E-04	3.4E-04	3.9E-04	ok
Level 1	8.1E-05	5.7E-05	6.5E-05	4.1E-04	not ok

Modified Special Reinforced Shear Wall - Vertical Irregularity 1a X Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	2.7E-03	1.9E-03	2.2E-03	--	ok
High Roof	5.5E-04	3.9E-04	4.4E-04	--	ok
Low Roof	2.9E-04	2.0E-04	2.3E-04	--	ok
Level 4	3.1E-04	2.2E-04	2.5E-04	2.8E-03	not ok
Level 3	3.4E-04	2.4E-04	2.7E-04	9.2E-04	not ok
Level 2	2.5E-04	1.7E-04	2.0E-04	7.5E-04	not ok
Level 1	2.5E-05	1.7E-05	2.0E-05	7.2E-04	not ok

Modified Special Reinforced Shear Wall - Vertical Irregularity 1a Y Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	1.8E-03	1.2E-03	1.4E-03	--	ok
High Roof	1.5E-03	1.0E-03	1.2E-03	--	ok
Low Roof	6.4E-04	4.5E-04	5.1E-04	--	ok
Level 4	7.5E-04	5.3E-04	6.0E-04	1.3E-03	not ok
Level 3	6.5E-04	4.6E-04	5.2E-04	9.5E-04	not ok
Level 2	4.7E-04	3.3E-04	3.8E-04	6.8E-04	not ok
Level 1	6.0E-05	4.2E-05	4.8E-05	6.3E-04	not ok

SCBF - Vertical Irregularity 1a X Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next	Soft Story Status
Stair 3	4.4E-03	3.1E-03	3.5E-03	--	ok
High Roof	6.6E-04	4.7E-04	5.3E-04	--	ok
Low Roof	2.7E-04	1.9E-04	2.2E-04	--	ok
Level 4	6.1E-04	4.3E-04	4.9E-04	4.3E-03	not ok
Level 3	5.5E-04	3.9E-04	4.4E-04	1.2E-03	not ok
Level 2	7.1E-04	5.0E-04	5.7E-04	1.2E-03	not ok
Level 1	2.0E-05	1.4E-05	1.6E-05	1.5E-03	not ok

SCBF - Vertical Irregularity 1a Y Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	2.4E-04	1.7E-04	1.9E-04	--	ok
High Roof	4.8E-03	3.4E-03	3.9E-03	--	ok
Low Roof	2.5E-04	1.7E-04	2.0E-04	--	ok
Level 4	7.8E-04	5.5E-04	6.3E-04	1.8E-03	not ok
Level 3	7.6E-04	5.3E-04	6.1E-04	2.0E-03	not ok
Level 2	1.0E-03	7.2E-04	8.2E-04	6.0E-04	ok
Level 1	2.8E-05	1.9E-05	2.2E-05	8.6E-04	not ok

SMF - Vertical Irregularity 1a X Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	7.1E-05	5.0E-05	5.7E-05	--	ok
High Roof	3.0E-05	2.1E-05	2.4E-05	--	ok
Low Roof	3.4E-05	2.4E-05	2.7E-05	--	ok
Level 4	5.6E-05	3.9E-05	4.5E-05	1.1E-04	not ok
Level 3	7.8E-05	5.5E-05	6.3E-05	9.6E-05	not ok
Level 2	7.2E-05	5.0E-05	5.7E-05	1.3E-04	not ok
Level 1	1.2E-06	8.6E-07	9.9E-07	1.7E-04	not ok

SMF - Vertical Irregularity 1a Y Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	3.5E-03	2.5E-03	2.8E-03	--	ok
High Roof	5.6E-05	3.9E-05	4.5E-05	--	ok
Low Roof	7.8E-05	5.4E-05	6.2E-05	--	ok
Level 4	2.7E-04	1.9E-04	2.2E-04	1.2E-03	not ok
Level 3	1.9E-04	1.3E-04	1.5E-04	1.3E-04	ok
Level 2	4.0E-04	2.8E-04	3.2E-04	1.8E-04	ok
Level 1	8.9E-05	6.2E-05	7.1E-05	2.8E-04	not ok

Appendix E – Lateral Force Resisting System Design Checks-Existing System

Level 1 Shear Wall Data*											
Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	Δ (in)**	I =Ri/∑Ri
1	12	34.00	90	13.50	1.00	13.50	34.00	459.00	31043.54	3.33E-05	2.97%
2	12	34.00	90	13.50	1.00	13.50	34.00	459.00	31043.54	3.33E-05	2.97%
3	12	33.00	0	13.50	33.00	445.50	1.00	13.50	29801.04	3.47E-05	2.85%
4	12	33.00	0	13.50	33.00	445.50	1.00	13.50	29801.04	3.47E-05	2.85%
5	12	23.17	0	13.50	23.17	312.80	1.00	13.50	17523.79	5.87E-05	1.67%
6	12	23.17	0	13.50	23.17	312.80	1.00	13.50	17523.79	5.87E-05	1.67%
7	12	10.33	90	13.50	1.00	13.50	10.33	139.46	3408.82	2.97E-04	0.33%
8	12	10.33	90	13.50	1.00	13.50	10.33	139.46	3408.82	2.97E-04	0.33%
9	12	32.00	90	13.50	1.00	13.50	32.00	432.00	28556.02	3.62E-05	2.73%
10	12	32.00	90	13.50	1.00	13.50	32.00	432.00	28556.02	3.62E-05	2.73%
11	18	12.33	0	13.50	12.33	166.46	1.50	20.25	7722.84	1.32E-04	0.74%
12	18	12.33	0	13.50	12.33	166.46	1.50	20.25	7722.84	1.32E-04	0.74%
13	24	21.17	0	13.50	21.17	285.80	2.00	27.00	30112.66	3.41E-05	2.88%
14	12	120.00	0	13.50	120.00	1620.00	1.00	13.50	131256.84	7.94E-06	12.54%
15	12	107.67	0	13.50	107.67	1453.55	1.00	13.50	117279.46	8.88E-06	11.20%
16	12	256.00	90	13.50	1.00	13.50	256.00	3456.00	283841.76	3.67E-06	27.12%
17	12	224.00	90	13.50	1.00	13.50	224.00	3024.00	248069.26	4.20E-06	23.70%
* Assume that the general area of wall is rectangular yet has openings								∑Ri =	1046672.10		100.00%
** Using a 1k load applied at the top of each LFRS system											

Center of Rigidity			
X Direction	k_{ix} (k/ft)	x_i (ft)	$k_{ix}\ x_i$
SW3	357612.53	31.50	11264794.82
SW4	357612.53	65.50	23423620.98
SW5	210285.51	21.67	4556186.11
SW6	210285.51	32.00	6729136.41
SW11	40905.82	224.00	9162903.93
SW12	40905.82	256.00	10471890.20
SW13	342672.27	65.50	22445033.57
SW14	131256.84	0.00	0.00
SW15	117279.46	256.00	30023541.88
	∑ 1808816.31		118077107.89
		x (ft) = $\sum k_{ix}\ x_i / k_{ix}$ =	**65.28**
Y Direction	k_{iy} (k/ft)	y_i (ft)	$k_{iy}\ y_i$
SW1	372522.46	97.00	36134678.63
SW2	372522.46	64.00	23841437.45
SW7	357612.53	24.00	8582700.82
SW8	357612.53	44.00	15734951.50
SW9	210285.51	124.00	26075403.57
SW10	210285.51	111.67	23482583.20
SW16	283841.76	4.00	1135367.03
SW17	248069.26	124.00	30760587.93
	∑ 2412752.03		165747710.13
		y (ft) = $\sum k_{iy}\ y_i / k_{iy}$ =	**68.70**

Existing Special Reinforced Concrete Shear Wall Design Check

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY		**Wall 13**			**X-Direction**						
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi	Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	3718	k	at BASE	
REBAR YIELD STRESS	f_y	=	60	ksi	FACTORED BASE MOMENT LOAD	M_u	=	46338	ft-k		
HEIGHT OF WALL	H	=	376.0	in	FACTORED BASE SHEAR LOAD	V_u	=	330	k		
LENGTH OF SHEAR WALL	L	=	254.0	in							
THICKNESS OF WALL	t	=	24	in	**THE WALL DESIGN IS ADEQUATE.**						
	Acv	=	6096.96	in^2							
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			771.2	Need 1							

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	330.4	kip
Vn = Vc + Vs	d	=	203.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3084.8	kip
	ΦVn	=	2313.6	kip
ACI 318-08 § 21.9.4	Vn	≤	53429.2	kip
V_n ≤ A_{cv} (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			**OK**	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	617.0	kip
Note: If Vu≤Acv*√(f'c) can choose pt, ρl according to Ch.14		=	385.6	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	231.4	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	-176.4	kip	
S = (Av*fy*d)/Vs	Ag	=	6096.96	in^2	
	0.0025*Ag	=	15.2	in^2	
TRY #8	Abar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 84.68	S	=	8.00	in	**USE**
S ≤ 3t = 72					
S ≤ 18" **Governs**	# bars required	=	19		
ρt = Av/(S*t)	ρt	=	0.0794	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.1186	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 84.68	S	=	6	in	
S ≤ 3t = 72					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	11		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.4799	ρl≥ρt	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	19	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	11	#11	@	6	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	182.91	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	56.30	in^2
	a	=	41.40	in
jd = d - (a/2)	jd	=	182.53	in
	As	=	56.41	in^2

TRY #11	A/bar	=	1.56	in^2
	# bars required	=	36	
Check Capacity:	a	=	41.48	in
C=T 0.85*f'c*a*b =As*fy	c	=	48.80	in
c = a/0.85	εt	=	0.01	>0.0025 **OK**
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)		Wall 1		

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0794	>	$(\rho t)_{min.}$ =	0.0025	**OK**
$\rho l_{provd.}$ = 0.1186	>	$(\rho l)_{min.}$ =	0.0025	**OK**

WALL DIST. HORIZ. REINF.	19	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	36	#11	@	6	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

ΦV_n ≤ A_{cv} (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 40072 kips > V_u = 330 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 556,057 kip-ft > M_u = 46,338 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

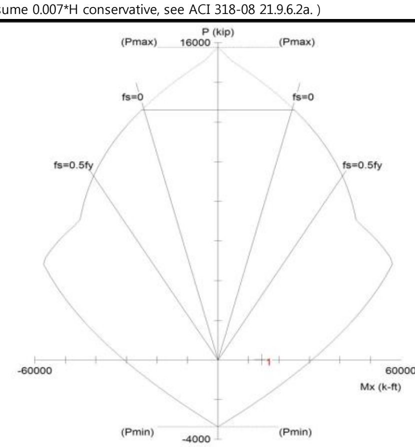
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 60.49 in. **No Boundary Element Needed**

where c = 49 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



Appendix F – Lateral Force Resisting System Design Checks-System #1

Modified Special Reinforced Shear Wall Shear Wall Data* - Level 1

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	Δ(in)**	I =Ri/∑Ri
1	18	34.00	90	13.50	1.50	20.25	34.00	459.00	46565.31	2.22E-05	4.50%
2	18	34.00	90	13.50	1.50	20.25	34.00	459.00	46565.31	2.22E-05	4.50%
5	18	23.17	0	13.50	23.17	312.80	1.50	20.25	26285.69	3.91E-05	2.54%
6	18	23.17	0	13.50	23.17	312.80	1.50	20.25	26285.69	3.91E-05	2.54%
7	12	10.33	90	13.50	1.00	13.50	10.33	139.46	3408.82	2.97E-04	0.33%
8	12	10.33	90	13.50	1.00	13.50	10.33	139.46	3408.82	2.97E-04	0.33%
9	12	32.00	90	13.50	1.00	13.50	32.00	432.00	28556.02	3.62E-05	2.76%
10	12	32.00	90	13.50	1.00	13.50	32.00	432.00	28556.02	3.62E-05	2.76%
11	18	12.33	0	13.50	12.33	166.46	1.50	20.25	7722.84	1.32E-04	0.75%
12	18	12.33	0	13.50	12.33	166.46	1.50	20.25	7722.84	1.32E-04	0.75%
13	24	21.17	0	13.50	21.17	285.80	2.00	27.00	30112.66	3.41E-05	2.91%
14	12	120.00	0	13.50	120.00	1620.00	1.00	13.50	131256.84	7.94E-06	12.67%
15	12	107.67	0	13.50	107.67	1453.55	1.00	13.50	117279.46	8.88E-06	11.32%
16	12	256.00	90	13.50	1.00	13.50	256.00	3456.00	283841.76	3.67E-06	27.41%
17	12	224.00	90	13.50	1.00	13.50	224.00	3024.00	248069.26	4.20E-06	23.95%
								∑Ri =	**1035637.35**		**100.00%**

* Assume that the general area of wall is rectangular yet has openings
** Using a 1k load applied at the top of each LFRS system

Modified Special Reinforced Shear Wall Shear Wall Data* - Level 2

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	Δ(in)**	I =Ri/∑Ri
1	18	34.00	90	13.00	1.50	19.50	34.00	442.00	48996.34	2.11E-05	18.12%
2	18	34.00	90	13.00	1.50	19.50	34.00	442.00	48996.34	2.11E-05	18.12%
5	18	23.17	0	13.00	23.17	301.21	1.50	19.50	27947.89	3.68E-05	10.33%
6	18	23.17	0	13.00	23.17	301.21	1.50	19.50	27947.89	3.68E-05	10.33%
7	12	10.33	90	13.00	1.00	13.00	10.33	134.29	3730.75	2.72E-04	1.38%
8	12	10.33	90	13.00	1.00	13.00	10.33	134.29	3730.75	2.72E-04	1.38%
9	12	32.00	90	13.00	1.00	13.00	32.00	416.00	30088.01	3.44E-05	11.13%
10	12	32.00	90	13.00	1.00	13.00	32.00	416.00	30088.01	3.44E-05	11.13%
11	18	12.33	0	13.00	12.33	160.29	1.50	19.50	8401.54	1.21E-04	3.11%
12	18	12.33	0	13.00	12.33	160.29	1.50	19.50	8401.54	1.21E-04	3.11%
13	24	21.17	0	13.00	21.17	275.21	2.00	26.00	32113.73	3.20E-05	11.87%
								∑Ri =	**270442.77**		**100.00%**

* Assume that the general area of wall in rectangular yet has openings
** Using a 1k load applied at the top of each LFRS system

Modified Special Reinforced Shear Wall Shear Wall Data* - Level 3

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	Δ(in)**	I =Ri/∑Ri
1	18	34.00	90	12.00	1.50	18.00	34.00	408.00	54438.94	1.90E-05	17.85%
2	18	34.00	90	12.00	1.50	18.00	34.00	408.00	54438.94	1.90E-05	17.85%
5	18	23.17	0	12.00	23.17	278.04	1.50	18.00	31703.97	3.25E-05	10.40%
6	18	23.17	0	12.00	23.17	278.04	1.50	18.00	31703.97	3.25E-05	10.40%
7	12	10.33	90	12.00	1.00	12.00	10.33	123.96	4499.25	2.26E-04	1.48%
8	12	10.33	90	12.00	1.00	12.00	10.33	123.96	4499.25	2.26E-04	1.48%
9	12	32.00	90	12.00	1.00	12.00	32.00	384.00	33520.27	3.09E-05	10.99%
10	12	32.00	90	12.00	1.00	12.00	32.00	384.00	33520.27	3.09E-05	10.99%
11	18	12.33	0	12.00	12.33	147.96	1.50	18.00	9999.54	1.02E-04	3.28%
12	18	12.33	0	12.00	12.33	147.96	1.50	18.00	9999.54	1.02E-04	3.28%
13	24	21.17	0	12.00	21.17	254.04	2.00	24.00	36654.01	2.81E-05	12.02%
								∑Ri =	**304977.93**		**100.00%**

* Assume that the general area of wall in rectangular yet has openings
** Using a 1k load applied at the top of each LFRS system

Modified Special Reinforced Shear Wall Shear Wall Data* - Level 4

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	∆(in)**	I =Ri/∑Ri
1	18	34.00	90	12.00	1.50	18.00	34.00	408.00	54438.94	1.90E-05	17.85%
2	18	34.00	90	12.00	1.50	18.00	34.00	408.00	54438.94	1.90E-05	17.85%
5	18	23.17	0	12.00	23.17	278.04	1.50	18.00	31703.97	3.25E-05	10.40%
6	18	23.17	0	12.00	23.17	278.04	1.50	18.00	31703.97	3.25E-05	10.40%
7	12	10.33	90	12.00	1.00	12.00	10.33	123.96	4499.25	2.26E-04	1.48%
8	12	10.33	90	12.00	1.00	12.00	10.33	123.96	4499.25	2.26E-04	1.48%
9	12	32.00	90	12.00	1.00	12.00	32.00	384.00	33520.27	3.09E-05	10.99%
10	12	32.00	90	12.00	1.00	12.00	32.00	384.00	33520.27	3.09E-05	10.99%
11	18	12.33	0	12.00	12.33	147.96	1.50	18.00	9999.54	1.02E-04	3.28%
12	18	12.33	0	12.00	12.33	147.96	1.50	18.00	9999.54	1.02E-04	3.28%
13	24	21.17	0	12.00	21.17	254.04	2.00	24.00	36654.01	2.81E-05	12.02%
* Assume that the general area of wall in rectangular yet has openings								**∑Ri =**	**304977.93**		**100.00%**

** Using a 1k load applied at the top of each LFRS system

Modified Special Reinforced Shear Wall Shear Wall Data* - Low Roof

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	∆(in)**	I =Ri/∑Ri
1	18	34.00	90	12.33	1.50	18.50	34.00	419.32	52532.93	1.97E-05	17.94%
2	18	34.00	90	12.33	1.50	18.50	34.00	419.32	52532.93	1.97E-05	17.94%
5	18	23.17	0	12.33	23.17	285.76	1.50	18.50	30383.76	3.39E-05	10.38%
6	18	23.17	0	12.33	23.17	285.76	1.50	18.50	30383.76	3.39E-05	10.38%
7	12	10.33	90	12.33	1.00	12.33	10.33	127.40	4222.80	2.40E-04	1.44%
8	12	10.33	90	12.33	1.00	12.33	10.33	127.40	4222.80	2.40E-04	1.44%
9	12	32.00	90	12.33	1.00	12.33	32.00	394.66	32317.96	3.20E-05	11.04%
10	12	32.00	90	12.33	1.00	12.33	32.00	394.66	32317.96	3.20E-05	11.04%
11	18	12.33	0	12.33	12.33	152.07	1.50	18.50	9428.09	1.08E-04	3.22%
12	18	12.33	0	12.33	12.33	152.07	1.50	18.50	9428.09	1.08E-04	3.22%
13	24	21.17	0	12.33	21.17	261.09	2.00	24.67	35055.51	2.94E-05	11.97%
* Assume that the general area of wall in rectangular yet has openings								**∑Ri =**	**292826.61**		**100.00%**

** Using a 1k load applied at the top of each LFRS system

Modified Special Reinforced Shear Wall Shear Wall Data* - High Roof

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	∆(in)**	I =Ri/∑Ri
2	18	34.00	90	7.00	1.50	10.50	34.00	238.00	103386.85	1.01E-05	19.68%
5	18	23.17	0	7.00	23.17	162.19	1.50	10.50	66208.30	1.57E-05	12.60%
6	18	23.17	0	7.00	23.17	162.19	1.50	10.50	66208.30	1.57E-05	12.60%
7	12	10.33	90	7.00	1.00	7.00	10.33	72.31	13537.42	7.58E-05	2.58%
8	12	10.33	90	7.00	1.00	7.00	10.33	72.31	13537.42	7.58E-05	2.58%
9	12	32.00	90	7.00	1.00	7.00	32.00	224.00	64408.42	1.61E-05	12.26%
10	12	32.00	90	7.00	1.00	7.00	32.00	224.00	64408.42	1.61E-05	12.26%
11	18	12.33	0	7.00	12.33	86.31	1.50	10.50	27421.50	3.75E-05	5.22%
12	18	12.33	0	7.00	12.33	86.31	1.50	10.50	27421.50	3.75E-05	5.22%
13	24	21.17	0	7.00	21.17	148.19	2.00	14.00	78900.71	1.31E-05	15.02%
* Assume that the general area of wall is rectangular								**∑Ri =**	**525438.84**		**100.00%**

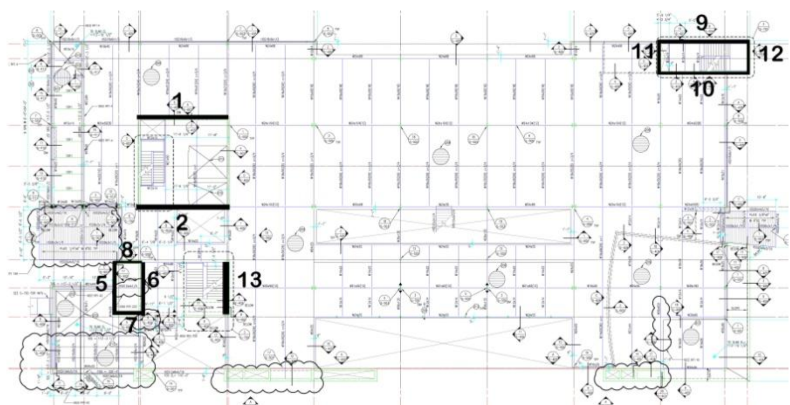
** Using a 1k load applied at the top of each LFRS system

Modified Special Reinforced Shear Wall Shear Wall Data* - Stair 3

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	∆(in)**	I =Ri/∑Ri
9	12	32.00	90	2.00	1.00	2.00	32.00	64.00	239119.06	4.36E-06	32.04%
10	12	32.00	90	2.00	1.00	2.00	32.00	64.00	239119.06	4.36E-06	32.04%
11	18	12.33	0	2.00	12.33	24.66	1.50	3.00	134053.03	7.77E-06	17.96%
12	18	12.33	0	2.00	12.33	24.66	1.50	3.00	134053.03	7.77E-06	17.96%
* Assume that the general area of wall in rectangular yet has openings								**∑Ri =**	**746344.18**		**100.00%**

** Using a 1k load applied at the top of each LFRS system

WALL	Height (ft)	Length (ft)
Wall 1_a	24.33	34.00
Wall 1_b	38.50	34.00
Wall 2_a	31.33	34.00
Wall 2_b	38.50	34.00
Wall 3_a	31.33	34.00
Wall 3_b	38.50	34.00
Wall 4_a	24.33	34.00
Wall 4_b	38.50	34.00
Wall 5_a	31.33	20.00
Wall 5_b	38.50	20.00
Wall 6_a	31.33	20.00
Wall 6_b	38.50	20.00
Wall 7_c	31.33	10.33
Wall 7_d	38.50	10.33
Wall 8_c	31.33	10.33
Wall 8_d	38.50	10.33
Wall 9_c	33.33	34.00
Wall 9_d	38.50	34.00
Wall 10_c	33.33	34.00
Wall 10_d	38.50	34.00
Wall 11_e	33.33	12.33
Wall 11_f	38.50	12.33
Wall 12_e	33.33	12.33
Wall 12_f	38.50	12.33
Wall 13_g	31.33	21.17
Wall 13_h	38.50	21.17



Type	Thickness (in)	Vertical Spacing (in)	Vertical Bar Size	Vertical Bar Diameter (in)	Vertical Bar Weight (plf)	Horizontal Spacing (in)	Horizontal Bar Size	Bar Diameter	Bar Weight (plf)
a	12	12	6	0.75	1.502	12	6	0.75	1.502
b	12	6	11	1.41	5.313	8	8	1	2.67
c	12	12	6	0.75	1.502	12	6	0.75	1.502
d	12	8	11	1.41	5.313	8	8	1	2.67
e	18	12	6	0.75	1.502	12	6	0.75	1.502
f	18	6	11	1.41	5.313	8	8	1	2.67
g	18	6	11	1.41	5.313	8	8	1	2.67
h	18	6	11	1.41	5.313	8	8	1	2.67

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 1 a**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	292.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	7344	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			929.0	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	2201	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	968	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	205	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	204.7	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip
	ΦVn	=	2786.9	kip
ACI 318-08 § 21.9.4	Vn	≤	124863.3	kip
$V_n \le A_{cv}$ (αc*$\sqrt{f_c'}$ + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	743.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	464.5	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	278.7	kip	
			Reinf. According to Ch 14		
Vs = Vu/(0.75) - Vc	Vs	=	-	kip	
S = (Av*fy*d)/Vs	Ag	=	7344	in^2	
	0.0025*Ag	=	18.4	in^2	
TRY #8	Abar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in	USE
S ≤ 3t = 54					
S ≤ 18" Governs	# bars required	=	40		
ρt = Av/(S*t)	ρt	=	0.2167	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.40781	>0.0025	OK
Max. Spacing S ≤ L/3 = 136	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" Governs					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	28		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	0.7156	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	28	#11	@	6	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.73	in^2
	a	=	0.72	in
jd = d - (a/2)	jd	=	326.04	in
	As	=	0.66	in^2

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	1		
Check Capacity:	a	=	0.65	in	
C=T 0.85*f'c*a*b =As*fy	c	=	0.76	in	
c = a/0.85	εt	=	1.61	>0.0025	OK
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)		Wall 1			

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho_{t\,provd.}$ =	0.2167	>	$(\rho_t)_{min.}$ =	0.0025	OK
$\rho_{l\,provd.}$ =	0.4078	>	$(\rho_l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	24	#11	@	6	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*$\sqrt{f_c'}$ + $\rho_t f_y$) αc = 2 (conservative) 93647 kips > V_u = 205 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 11,618 kip-ft > M_u = 968 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

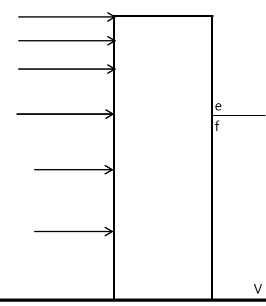
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 97.14 in. **No Boundary Element Needed**

where c = 1 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.0 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY — **Wall 1 b**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	7344	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥2*Acv*√(f'c) ; need at least two curtains (rows) =			929.0	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	3752	k
FACTORED MOMENT LOAD	M_u	=	1051	ft-k
FACTORED SHEAR LOAD	V_u	=	196	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	195.5	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip
	ΦVn	=	2786.9	kip
ACI 318-08 § 21.9.4	Vn	≤	124863.3	kip
$V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	743.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	464.5	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	278.7	kip	
			Reinf. According to Ch 14		
Vs = Vu/(0.75) - Vc	Vs	=	-	kip	
S = (Av*fy*d)/Vs	Ag	=	7344	in^2	
	0.0025*Ag	=	18.4	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in	USE
S ≤ 3t = 54	Av	=	7344.00		
S ≤ 18" **Governs**	# bars required	=	40		
ρt = Av/(S*t)	ρt	=	0.2167	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.36318	>0.0025	OK
Max. Spacing S ≤ L/3 = 136	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	25		

ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)

h/l	=	1.1324	ρl≥ρt
ρl≥ρt	is	OK	

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	25	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.80	in^2
	a	=	0.78	in
jd = d - (a/2)	jd	=	326.01	in
	As	=	0.72	in^2

	A/bar	=	1.56	in^2
	# bars required	=	0	
TRY #11				
Check Capacity:	a	=	0.70	in
C=T 0.85*f'c*a*b =As*fy	c	=	0.83	in
c = a/0.85	εt	=	1.48	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)		Wall 1		

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 0.83 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 6.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.2167 **>** $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.3632 **>** $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 2786.9 kips **>** $V_{u\,=}$ 195.5 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 12,612 kip-ft **>** $M_{u\,=}$ 1,051 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	28	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 2 a**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	292.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	7344	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			929.0	Need 2

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	2527	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	3371	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	1266	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	1265.6	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip
	ΦVn	=	2786.9	kip
ACI 318-08 § 21.9.4	Vn	≤	85557.6	kip
$V_n \le A_{cv}$ (αc*$\sqrt{f_c'}$ + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	743.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	464.5	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	278.7	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	944.3	kip	
S = (Av*fy*d)/Vs	Ag	=	7344	in^2	
	0.0025*Ag	=	18.4	in^2	
TRY #8	Abar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in	USE
S ≤ 3t = 54					
S ≤ 18" Governs	# bars required	=	23		
ρt = Av/(S*t)	ρt	=	0.1275	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.23903	>0.0025	OK
Max. Spacing S ≤ L/3 = 136	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" Governs					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	17		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	0.7156	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	23	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	17	#11	@	6	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	2.55	in^2
	a	=	2.50	in
jd = d - (a/2)	jd	=	325.15	in
	As	=	2.30	in^2

TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	1		
Check Capacity:	a	=	2.26	in	
C=T 0.85*f'c*a*b =As*fy	c	=	2.66	in	
c = a/0.85	εt	=	0.46	>0.0025	OK
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)					

Wall 1

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$	=	0.1275	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$	=	0.2390	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	23	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	24	#11	@	6	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*$\sqrt{f_c'}$ + $\rho_t f_y$) αc = 2 (conservative) 64168 kips > V_{u} = 1266 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 40,452 kip-ft > M_u = 3,371 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

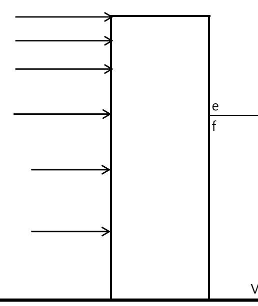
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 97.14 in. **No Boundary Element Needed**

where c = 3 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.0 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY — **Wall 2 b**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	7344	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			929.0	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	4123.0	k
FACTORED MOMENT LOAD	M_u	=	1268.0	ft-k
FACTORED SHEAR LOAD	V_u	=	114.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	114.0	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip
	ΦVn	=	2786.9	kip
ACI 318-08 § 21.9.4	Vn	≤	124863.3	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	743.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	464.5	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	278.7	kip
			Reinf. According to Ch 14	
Vs = Vu/(0.75) - Vc	Vs	=	-	kip
S = (Av*fy*d)/Vs	Ag	=	7344	in^2
	0.0025*Ag	=	18.4	in^2
TRY #8	A/bar	=	0.79	in^2
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in **USE**
S ≤ 3t = 54	Av	=	7344.00	
S ≤ 18" **Governs**	# bars required	=	40	
ρt = Av/(S*t)	ρt	=	0.2167	>0.0025 **OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.36318	>0.0025 **OK**
Max. Spacing S ≤ L/3 = 136	S	=	6	in
S ≤ 3t = 54				
S ≤ 18" **Governs**				
TRY #11	A/bar	=	1.56	in^2
	# bars required	=	25	
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)				
	h/l	=	1.1324	ρl≥ρt
	ρl≥ρt	is	**OK**	

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	25	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.96	in^2
	a	=	0.94	in
jd = d - (a/2)	jd	=	325.93	in
	As	=	0.86	in^2

TRY #11	A/bar	=	1.56	in^2
	# bars required	=	1	
Check Capacity:	a	=	0.85	in
C=T 0.85*f'c*a*b =As*fy	c	=	1.00	in
c = a/0.85	εt	=	1.22	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 1.00 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 23.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.2167	>	$(\rho t)_{min.}$ =	0.0025	**OK**
$\rho l_{provd.}$ =	0.3632	>	$(\rho l)_{min.}$ =	0.0025	**OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 2786.9 kips > V_u = 114.0 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 15,216 kip-ft > M_u = 1,268 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	28	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 5 a**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	376.0	in
LENGTH OF SHEAR WALL	L	=	240.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	4320	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			546.4	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	897	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	9415	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	353	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	353.0	kip
Vn = Vc + Vs	d	=	192.0	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2185.8	kip
	ΦVn	=	1639.3	kip
ACI 318-08 § 21.9.4	Vn	≤	30240.0	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	437.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	273.2	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	163.9	kip
			According to 11.9.9	
Vs = Vu/(0.75) - Vc	Vs	=	33.5	kip
S = (Av*fy*d)/Vs	Ag	=	4320	in^2
	0.0025*Ag	=	10.8	in^2
TRY #6	Abar	=	0.44	in^2
Max. Spacing S ≤ L/3 = 80	S	=	12.00	in USE
S ≤ 3t = 54				
S ≤ 18" Governs	# bars required	=	25	
ρt = Av/(S*t)	ρt	=	0.0500	>0.0025 OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.07217	>0.0025 OK
Max. Spacing S ≤ L/3 = 80	S	=	12	in
S ≤ 3t = 54				
S ≤ 18" Governs				
TRY #6	A/bar	=	0.44	in^2
	# bars required	=	35	
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)				
	h/l	=	1.5665	ρl≥ρt
	ρl≥ρt	is	OK	

WALL DIST. HORIZ. REINF.	25	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	35	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	172.80	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	12.11	in^2
	a	=	11.87	in
jd = d - (a/2)	jd	=	186.06	in
	As	=	11.24	in^2

TRY #8	A/bar	=	0.79	in^2
	# bars required	=	14	
Check Capacity: C=T 0.85*f'c*a*b =As*fy	a	=	11.02	in
c = a/0.85	c	=	12.97	in
	εt	=	0.05	>0.0025 OK
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)				

Wall 1

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$	=	0.0500	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$	=	0.0722	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	25	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	24	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 22680 kips > V_u = 353 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 112,980 kip-ft > M_u = 9,415 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

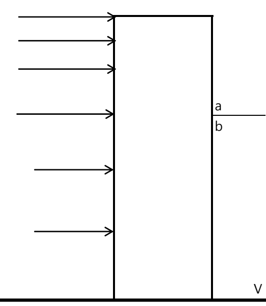
CHECK BOUNDARY ZONE REQUIREMENTS

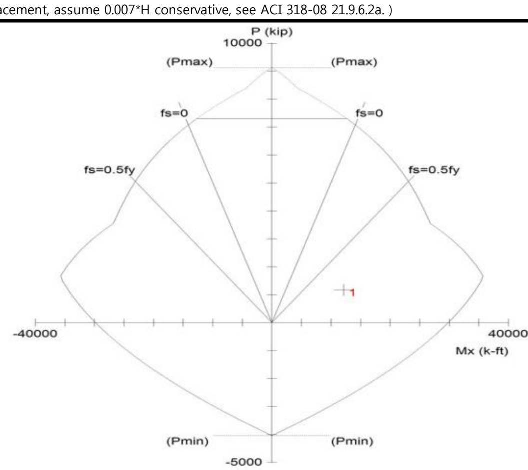
AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 57.14 in. **No Boundary Element Needed**

where c = 13 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)





INPUT DATA & DESIGN SUMMARY	Wall 5 b			
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	240.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	4320	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥2*Acv*√(f'c) ; need at least two curtains (rows) =			546.4	Need 1

X-Direction				
Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	463.0	k
FACTORED MOMENT LOAD	M_u	=	17087.8	ft-k
FACTORED SHEAR LOAD	V_u	=	405.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength				
ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	405.0	kip
Vn = Vc + Vs	d	=	192.0	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2185.8	kip
	ΦVn	=	1639.3	kip
ACI 318-08 § 21.9.4	Vn	≤	36720.0	kip
$V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc				
Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	437.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	273.2	FALSE

3. Required Horizontal Shear Reinforcement					
1/2ΦVc < Vu	1/2ΦVc	=	163.9	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	102.8	kip	
S = (Av*fy*d)/Vs	Ag	=	4320	in^2	
	0.0025*Ag	=	10.8	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 80	S	=	8.00	in	USE
S ≤ 3t = 54	Av	=	4320.00		
S ≤ 18" **Governs**	# bars required	=	14		
ρt = Av/(S*t)	ρt	=	0.0750	>0.0025	OK

4. Required Vertical Shear Reinforcement					
ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.09584	>0.0025	OK
Max. Spacing S ≤ L/3 = 80	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	7		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.9250	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	14	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	7	#11	@	6	" O.C.

FALSE

5. Design for Flexture				
Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	172.80	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	21.98	in^2
	a	=	21.54	in
jd = d - (a/2)	jd	=	181.23	in
	As	=	20.95	in^2

TRY #11	A/bar	=	1.56	in^2
	# bars required	=	13	
Check Capacity:	a	=	20.54	in
C=T 0.85*f'c*a*b =As*fy	c	=	24.17	in
c = a/0.85	εt	=	0.03	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 24.17 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 65.9 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0750 > $(\rho t)_{min.}$ = 0.0025 OK

$\rho l_{provd.}$ = 0.0958 > $(\rho l)_{min.}$ = 0.0025 OK

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 1639.3 kips > V_u = 405.0 OK

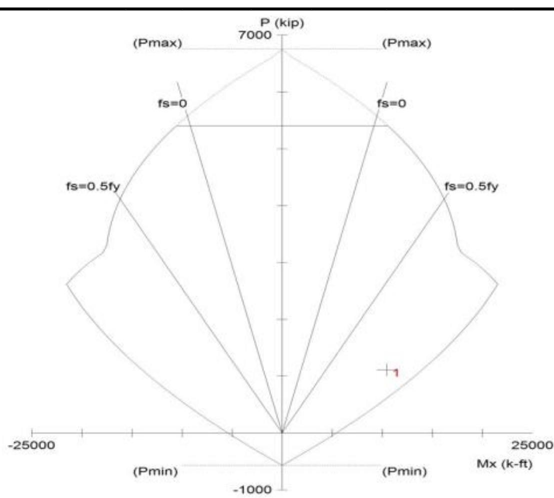
CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 205,054 kip-ft > M_u = 17,088 OK

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	14	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	16	#11	@	6	" O.C.



Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 6 a**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	376.0	in
LENGTH OF SHEAR WALL	L	=	240.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	2880	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			364.3	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1111	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	10463	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	291	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	291.0	kip
Vn = Vc + Vs	d	=	192.0	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1457.2	kip
	ΦVn	=	1092.9	kip
ACI 318-08 § 21.9.4	Vn	≤	20160.0	kip
V_n ≤ A_{cv} (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	291.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	182.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	109.3	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	96.6	kip	
S = (Av*fy*d)/Vs	Ag	=	2880	in^2	
	0.0025*Ag	=	7.2	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 80	S	=	12.00	in	USE
S ≤ 3t = 36					
S ≤ 18" **Governs**	# bars required	=	16		
ρt = Av/(S*t)	ρt	=	0.0500	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.07217	>0.0025	OK
Max. Spacing S ≤ L/3 = 80	S	=	12	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	24		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.5665	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	16	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	24	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	172.80	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	13.46	in^2
	a	=	19.79	in
jd = d - (a/2)	jd	=	182.11	in
	As	=	12.77	in^2

TRY #7	A/bar	=	0.6	in^2
	# bars required	=	21	
Check Capacity:	a	=	18.78	in
C=T 0.85*f'c*a*b =As*fy	c	=	22.09	in
c = a/0.85	εt	=	0.03	>0.0025 OK
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)				

Wall 1

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.0500	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$ =	0.0722	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	16	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	24	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

ΦV_n ≤ A_{cv} (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 15120 kips > V_u = 291 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 125,555 kip-ft > M_u = 10,463 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

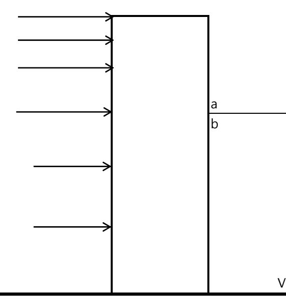
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 57.14 in. **No Boundary Element Needed**

where c = 22 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY — **Wall 6b**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	240.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	2880	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			364.3	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1165.1	k
FACTORED MOMENT LOAD	M_u	=	10224.2	ft-k
FACTORED SHEAR LOAD	V_u	=	101.2	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	101.2	kip
Vn = Vc + Vs	d	=	192.0	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1457.2	kip
	ΦVn	=	1092.9	kip
ACI 318-08 § 21.9.4	Vn	≤	67689.0	kip
$V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	291.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	182.1	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	109.3	kip
			Reinf. According to Ch 14	
Vs = Vu/(0.75) - Vc	Vs	=	-	kip
S = (Av*fy*d)/Vs	Ag	=	2880	in^2
	0.0025*Ag	=	7.2	in^2
TRY #8	A/bar	=	0.79	in^2
Max. Spacing S ≤ L/3 = 80	S	=	8.00	in **USE**
S ≤ 3t = 36	Av	=	2880.00	
S ≤ 18" **Governs**	# bars required	=	40	
ρt = Av/(S*t)	ρt	=	0.3251	>0.0025 **OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.41779	>0.0025 **OK**
Max. Spacing S ≤ L/3 = 80	S	=	6	in
S ≤ 3t = 36				
S ≤ 18" **Governs**				
TRY #11	A/bar	=	1.56	in^2
	# bars required	=	19	
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)				
	h/l	=	1.9250	ρl≥ρt
	ρl≥ρt	is	**OK**	

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	19	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	172.80	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	13.15	in^2
	a	=	19.34	in
jd = d - (a/2)	jd	=	182.33	in
	As	=	12.46	in^2

	A/bar	=	0.79	in^2
	# bars required	=	16	
TRY #8				
Check Capacity:	a	=	18.33	in
C=T 0.85*f'c*a*b =As*fy	c	=	21.56	in
c = a/0.85	εt	=	0.03	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 57.14 in. **No Boundary Element Needed**

where c = 22 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 3.2 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.3251 > $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.4178 > $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 1092.9 kips > V_u = 101.2 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 122,690 kip-ft > M_u = 10,224 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	48	#8	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 7 c**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	376.0	in
LENGTH OF SHEAR WALL	L	=	124.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	1487.952	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			188.2	Need 2

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	479	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	1723	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	516	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	516.0	kip
Vn = Vc + Vs	d	=	99.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	752.9	kip
	ΦVn	=	564.6	kip
ACI 318-08 § 21.9.4	Vn	≤	8258.1	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	150.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	94.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	537.4	kip	
S = (Av*fy*d)/Vs	Ag	=	1487.952	in^2	
	0.0025*Ag	=	3.7	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 41.332	S	=	12.00	in	**USE**
S ≤ 3t = 36					
S ≤ 18" **Governs**	# bars required	=	8		
ρt = Av/(S*t)	ρt	=	0.0258	>0.0025	**OK**

4. Required Vertical Shear Reiforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.0283	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 41.332	S	=	12	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	3.0320	FALSE	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	8	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	9	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	4.29	in^2
	a	=	6.31	in
jd = d - (a/2)	jd	=	96.04	in
	As	=	3.99	in^2

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
Check Capacity:	a	=	5.86	in	
C=T 0.85*f'c*a*b =As*fy	c	=	6.90	in	
c = a/0.85	εt	=	0.05	>0.0025	**OK**
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)			Wall 1		

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$	=	0.0258	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$	=	0.0283	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	8	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	12	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 6194 kips > V_{u} = 516 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 20,676 kip-ft > M_u = 1,723 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

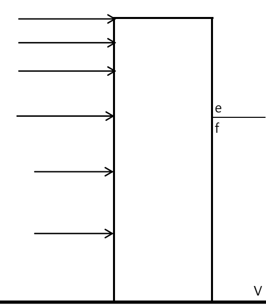
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 29.52 in. **No Boundary Element Needed**

where c = 7 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



Wall 7 d

INPUT DATA & DESIGN SUMMARY				
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	124.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	1487.952	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			188.2	Need 1

X-Direction				
Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	860.6	k
FACTORED MOMENT LOAD	M_u	=	1283.0	ft-k
FACTORED SHEAR LOAD	V_u	=	186.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	186.0	kip
Vn = Vc + Vs	d	=	99.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	752.9	kip
	ΦVn	=	564.6	kip
ACI 318-08 § 21.9.4	Vn	≤	9411.2	kip
V_n ≤ A_{cv} (αc*√f_c' + $ρ_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	150.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	94.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	97.4	kip	
S = (Av*fy*d)/Vs	Ag	=	1487.952	in^2	
	0.0025*Ag	=	3.7	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 41.332	S	=	8.00	in	**USE**
S ≤ 3t = 36	Av	=	1487.95		
S ≤ 18" **Governs**	# bars required	=	5		
ρt = Av/(S*t)	ρt	=	0.0387	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.05167	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 41.332	S	=	6	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	2		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	3.7259	FALSE	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	5	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	2	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	3.19	in^2
	a	=	4.70	in
jd = d - (a/2)	jd	=	96.85	in
	As	=	2.94	in^2

	A/bar	=	1.56	in^2
	# bars required	=	2	
TRY #11				
Check Capacity:	a	=	4.33	in
C=T 0.85*f'c*a*b =As*fy	c	=	5.09	in
c = a/0.85	εt	=	0.07	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 29.52 in. **No Boundary Element Needed**

where c = 5 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 3.2 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$ρt_{provd.}$ = 0.0387 **>** $(ρt)_{min.}$ = 0.0025 **OK**

$ρl_{provd.}$ = 0.0517 **>** $(ρl)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$ΦV_n$ ≤ A_{cv} (αc*√f_c' + $ρ_t f_y$) αc = 2 (conservative) 564.6 kips **>** V_u = 186.0 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 15,396 kip-ft **>** M_u = 1,283 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	5	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	14	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 8 c**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	376.0	in
LENGTH OF SHEAR WALL	L	=	124.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	1487.952	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			188.2	Need 2

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	438	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	1724	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	517	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	516.7	kip
Vn = Vc + Vs	d	=	99.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	752.9	kip
	ΦVn	=	564.6	kip
ACI 318-08 § 21.9.4	Vn	≤	8258.1	kip
$V_n \le A_{cv}$ (αc*$\sqrt{f_c'}$ + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	150.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	94.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	538.4	kip	
S = (Av*fy*d)/Vs	Ag	=	1487.952	in^2	
	0.0025*Ag	=	3.7	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 41.332	S	=	12.00	in	**USE**
S ≤ 3t = 36					
S ≤ 18" **Governs**	# bars required	=	8		
ρt = Av/(S*t)	ρt	=	0.0258	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.0283	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 41.332	S	=	12	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)	h/l	=	3.0320	FALSE	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	8	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	9	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	4.29	in^2
	a	=	6.31	in
jd = d - (a/2)	jd	=	96.04	in
	As	=	3.99	in^2

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
Check Capacity:	a	=	5.87	in	
C=T 0.85*f'c*a*b =As*fy	c	=	6.90	in	
c = a/0.85	εt	=	0.05	>0.0025	**OK**
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)		Wall 1			

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho_{t provd.}$	=	0.0258	>	$(\rho_t)_{min.}$ =	0.0025	OK
$\rho_{l provd.}$	=	0.0283	>	$(\rho_l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	8	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	12	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*$\sqrt{f_c'}$ + $\rho_t f_y$) αc = 2 (conservative) 6194 kips > V_u = 517 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 20,684 kip-ft > M_u = 1,724 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

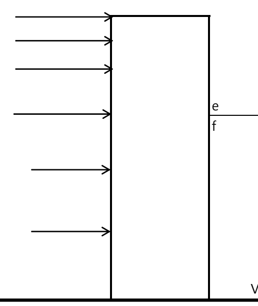
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 29.52 in. **No Boundary Element Needed**

where c = 7 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY — **Wall 8 d**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	124.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	1487.952	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			188.2	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	171.4	k
FACTORED MOMENT LOAD	M_u	=	1283.0	ft-k
FACTORED SHEAR LOAD	V_u	=	186.4	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	186.4	kip
Vn = Vc + Vs	d	=	99.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	752.9	kip
	ΦVn	=	564.6	kip
ACI 318-08 § 21.9.4	Vn	≤	9411.2	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	150.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	94.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	98.0	kip	
S = (Av*fy*d)/Vs	Ag	=	1487.952	in^2	
	0.0025*Ag	=	3.7	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 41.332	S	=	8.00	in	USE
S ≤ 3t = 36	Av	=	1487.95		
S ≤ 18" **Governs**	# bars required	=	5		
ρt = Av/(S*t)	ρt	=	0.0387	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.0412	>0.0025	OK
Max. Spacing S ≤ L/3 = 41.332	S	=	6	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	2		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	3.7259	FALSE	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	5	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	2	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	3.19	in^2
	a	=	4.70	in
jd = d - (a/2)	jd	=	96.85	in
	As	=	2.94	in^2

	A/bar	=	1.56	in^2
TRY #11	# bars required	=	2	
Check Capacity:	a	=	4.33	in
C=T 0.85*f'c*a*b =As*fy	c	=	5.09	in
c = a/0.85	εt	=	0.07	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L^*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 5.09 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 12.1 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0387 > $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.0412 > $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 564.6 kips > V_u = 186.4 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 15,396 kip-ft > M_u = 1,283 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	5	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	12	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 9 c**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	400.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	4896	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			619.3	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1225	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	635	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	72	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	71.8	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2477.2	kip
	ΦVn	=	1857.9	kip
ACI 318-08 § 21.9.4	Vn	≤	32748.8	kip
$V_n \le A_{cv}(\alpha c \sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	495.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	309.7	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	185.8	kip
			Reinf. According to Ch 14	
Vs = Vu/(0.75) - Vc	Vs	=	-	kip
S = (Av*fy*d)/Vs	Ag	=	4896	in^2
	0.0025*Ag	=	12.2	in^2
TRY #6	Abar	=	0.44	in^2
Max. Spacing S ≤ L/3 = 136	S	=	12.00	in USE
S ≤ 3t = 36				
S ≤ 18" Governs	# bars required	=	15	
ρt = Av/(S*t)	ρt	=	0.0448	>0.0025 OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.07697	>0.0025 OK
Max. Spacing S ≤ L/3 = 136	S	=	12	in
S ≤ 3t = 36				
S ≤ 18" Governs				
TRY #6	A/bar	=	0.44	in^2
	# bars required	=	25	
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)	h/l	=	0.9803	ρl≥ρt
	ρl≥ρt	is	OK	

WALL DIST. HORIZ. REINF.	15	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	25	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.48	in^2
	a	=	0.71	in
jd = d - (a/2)	jd	=	326.05	in
	As	=	0.43	in^2

TRY #6	A/bar	=	0.44	in^2
	# bars required	=	1	
Check Capacity:	a	=	0.64	in
C=T 0.85*f'c*a*b =As*fy	c	=	0.75	in
c = a/0.85	εt	=	1.63	>0.0025 OK
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho_{t provd.}$ =	0.0448	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho_{l provd.}$ =	0.0770	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	15	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	28	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c \sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 24562 kips > V_u = 72 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 7,620 kip-ft > M_u = 635 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

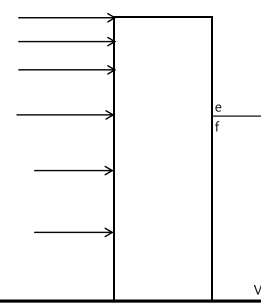
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 97.14 in. **No Boundary Element Needed**

where c = 1 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY **Wall 9 d**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	4896	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥2*Acv*√(f'c) ; need at least two curtains (rows) =			619.3	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	2774.9	k
FACTORED MOMENT LOAD	M_u	=	26.1	ft-k
FACTORED SHEAR LOAD	V_u	=	14.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	14.0	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2477.2	kip
	ΦVn	=	1857.9	kip
ACI 318-08 § 21.9.4	Vn	≤	115071.3	kip
$V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	495.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	309.7	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	185.8	kip	
			Reinf. According to Ch 14		
Vs = Vu/(0.75) - Vc	Vs	=	-	kip	
S = (Av*fy*d)/Vs	Ag	=	4896	in^2	
	0.0025*Ag	=	12.2	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in	USE
S ≤ 3t = 36	Av	=	4896.00		
S ≤ 18" Governs	# bars required	=	40		
ρt = Av/(S*t)	ρt	=	0.3251	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.54562	>0.0025	OK
Max. Spacing S ≤ L/3 = 136	S	=	6	in	
S ≤ 3t = 36					
S ≤ 18" Governs					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	25		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.1324	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	25	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.02	in^2
	a	=	0.03	in
jd = d - (a/2)	jd	=	326.39	in
	As	=	0.02	in^2

	A/bar	=	1.56	in^2
	# bars required	=	0	
TRY #11				
Check Capacity:	a	=	0.03	in
C=T 0.85*f'c*a*b =As*fy	c	=	0.03	in
c = a/0.85	εt	=	39.81	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)	Wall 1			

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L^*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 0.03 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 4.4 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.3251 > $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.5456 > $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 1857.9 kips > V_u = 14.0 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 313 kip-ft > M_u = 26 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	28	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 10 c**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	400.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	4896	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			619.3	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	974	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	6062	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	565	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	565.0	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2477.2	kip
	ΦVn	=	1857.9	kip
ACI 318-08 § 21.9.4	Vn	≤	44553.6	kip
$V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	495.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	309.7	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	185.8	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	257.9	kip	
S = (Av*fy*d)/Vs	Ag	=	4896	in^2	
	0.0025*Ag	=	12.2	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	12.00	in	USE
S ≤ 3t = 36					
S ≤ 18" Governs	# bars required	=	28		
ρt = Av/(S*t)	ρt	=	0.0850	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.1100	>0.0025	OK
Max. Spacing S ≤ L/3 = 136	S	=	12	in	
S ≤ 3t = 36					
S ≤ 18" Governs					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	36		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	0.9803	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	28	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	36	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	4.59	in^2
	a	=	6.74	in
jd = d - (a/2)	jd	=	323.03	in
	As	=	4.17	in^2

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
Check Capacity:	a	=	6.13	in	
C=T 0.85*f'c*a*b =As*fy	c	=	7.21	in	
c = a/0.85	εt	=	0.17	>0.0025	OK
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)			Wall 1		

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$	=	0.0850	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$	=	0.1100	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	28	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	36	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 33415 kips > $V_{u=}$ 565 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 72,739 kip-ft > $M_{u=}$ 6,062 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

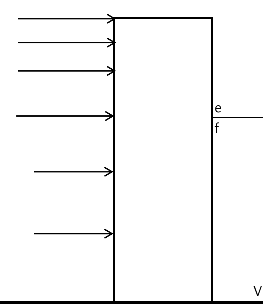
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 97.14 in. **No Boundary Element Needed**

where c = 7 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



Wall 10 d

INPUT DATA & DESIGN SUMMARY					**X-Direction**					
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi	Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	467	k	
REBAR YIELD STRESS	f_y	=	60	ksi	FACTORED MOMENT LOAD	M_u	=	6729	ft-k	
HEIGHT OF WALL	H	=	462.0	in	FACTORED SHEAR LOAD	V_u	=	81	k	
LENGTH OF SHEAR WALL	L	=	408.0	in						
THICKNESS OF WALL	t	=	12	in	**THE WALL DESIGN IS ADEQUATE.**					
	Acv	=	4896	in^2						
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			619.3	Need 1						

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	80.6	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2477.2	kip
	ΦVn	=	1857.9	kip
ACI 318-08 § 21.9.4	Vn	≤	115071.3	kip
$V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	495.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	309.7	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	185.8	kip	
			Reinf. According to Ch 14		
Vs = Vu/(0.75) - Vc	Vs	=	-	kip	
S = (Av*fy*d)/Vs	Ag	=	4896	in^2	
	0.0025*Ag	=	12.2	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in	**USE**
S ≤ 3t = 36	Av	=	4896.00		
S ≤ 18" **Governs**	# bars required	=	40		
ρt = Av/(S*t)	ρt	=	0.3251	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.54562	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 136	S	=	6	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	25		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.1324	ρl≥ρt	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	25	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9					
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d					
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in	
Mu = Φ Mn = Φ As*fy*j*d	As	=	5.09	in^2	
	a	=	7.49	in	
jd = d - (a/2)	jd	=	322.66	in	
	As	=	4.63	in^2	

	A/bar	=	1.56	in^2
	# bars required	=	3	
TRY #11				
Check Capacity:	a	=	6.81	in
C=T 0.85*f'c*a*b =As*fy	c	=	8.02	in
c = a/0.85	εt	=	0.15	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L^*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 8.02 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 42.4 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho_{t\,provd.}$	=	0.3251	>	$(\rho t)_{min.}$ =	0.0025	**OK**
$\rho_{l\,provd.}$	=	0.5456	>	$(\rho l)_{min.}$ =	0.0025	**OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 1857.9 kips > V_u = 80.6 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 80,742 kip-ft > M_u = 6,729 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	28	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 11 e**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	400.0	in
LENGTH OF SHEAR WALL	L	=	168.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	3024	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			382.5	Need 2

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1063	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	5498	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	2113	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	2113.0	kip
Vn = Vc + Vs	d	=	134.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1530.0	kip
	ΦVn	=	1147.5	kip
ACI 318-08 § 21.9.4	Vn	≤	18446.4	kip
$V_n \le A_{cv} (\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	306.0	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	191.3	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	114.8	kip
			According to 11.9.9	
Vs = Vu/(0.75) - Vc	Vs	=	2511.3	kip
S = (Av*fy*d)/Vs	Ag	=	3024	in^2
	0.0025*Ag	=	7.6	in^2
TRY #6	Abar	=	0.44	in^2
Max. Spacing S ≤ L/3 = 56	S	=	12.00	in USE
S ≤ 3t = 54				
S ≤ 18" Governs	# bars required	=	17	
ρt = Av/(S*t)	ρt	=	0.0350	>0.0025 OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.03694	>0.0025 OK
Max. Spacing S ≤ L/3 = 56	S	=	12	in
S ≤ 3t = 54				
S ≤ 18" Governs				
TRY #6	A/bar	=	0.44	in^2
	# bars required	=	18	
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)	h/l	=	2.3807	FALSE
	ρl≥ρt	is	OK	

WALL DIST. HORIZ. REINF.	17	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	18	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	120.96	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	10.10	in^2
	a	=	9.90	in
jd = d - (a/2)	jd	=	129.45	in
	As	=	9.44	in^2

TRY #6	A/bar	=	0.44	in^2
	# bars required	=	21	
Check Capacity:	a	=	9.25	in
C=T 0.85*f'c*a*b =As*fy	c	=	10.89	in
c = a/0.85	εt	=	0.04	>0.0025 OK
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)				

Wall 1

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$	=	0.0350	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$	=	0.0369	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	17	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	24	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv} (\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 13835 kips > V_u = 2113 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 65,974 kip-ft > M_u = 5,498 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

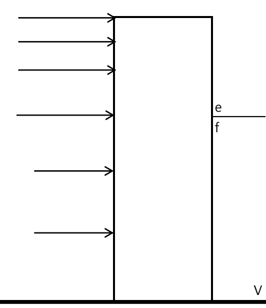
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L^*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 40.00 in. **No Boundary Element Needed**

where c = 11 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY — **Wall 11 f**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	148.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	2663.928	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			337.0	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	202.8	k
FACTORED MOMENT LOAD	M_u	=	443.4	ft-k
FACTORED SHEAR LOAD	V_u	=	151.5	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	151.5	kip
Vn = Vc + Vs	d	=	118.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1347.9	kip
	ΦVn	=	1010.9	kip
ACI 318-08 § 21.9.4	Vn	≤	18047.9	kip
$V_n \le A_{cv}(\alpha c \sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	269.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	168.5	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu		1/2ΦVc	=	101.1	kip
				According to 11.9.9	
Vs = Vu/(0.75) - Vc		Vs	=	-67.6	kip
S = (Av*fy*d)/Vs		Ag	=	2663.928	in^2
		0.0025*Ag	=	6.7	in^2
TRY	#8	A/bar	=	0.79	in^2
Max. Spacing S ≤ L/3 =	49.332	S	=	8.00	in **USE**
S ≤ 3t =	54	Av	=	2663.93	
S ≤ 18"	**Governs**	# bars required	=	8	
ρt = Av/(S*t)		ρt	=	0.0462	>0.0025 **OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)		ρl	=	0.03265	>0.0025 **OK**
Max. Spacing S ≤ L/3 =	49.332	S	=	6	in
S ≤ 3t =	54				
S ≤ 18"	**Governs**				
TRY	#11	A/bar	=	1.56	in^2
		# bars required	=	2	
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
		h/l	=	3.1217	FALSE
		ρl≥ρt	is	**NOT OK**	

WALL DIST. HORIZ. REINF.	8	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	2	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	106.56	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.92	in^2
	a	=	0.91	in
jd = d - (a/2)	jd	=	117.94	in
	As	=	0.84	in^2

	A/bar	=	1.56	in^2
	# bars required	=	1	
TRY #11				
Check Capacity:	a	=	0.82	in
C=T 0.85*f'c*a*b =As*fy	c	=	0.96	in
c = a/0.85	εt	=	0.46	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)		Wall 1		

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 0.96 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 38.5 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0462 > $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.0326 > $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c \sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 1010.9 kips > V_u = 151.5 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 5,321 kip-ft > M_u = 443 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	8	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	14	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 12 e**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	400.0	in
LENGTH OF SHEAR WALL	L	=	148.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	2663.9928	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			337.0	Need 2

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1131	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	5471	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	2105	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	2105.0	kip
Vn = Vc + Vs	d	=	118.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1347.9	kip
	ΦVn	=	1010.9	kip
ACI 318-08 § 21.9.4	Vn	≤	15584.3	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			**OK**	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	269.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	168.5	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	101.1	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	2537.1	kip	
S = (Av*fy*d)/Vs	Ag	=	2663.9928	in^2	
	0.0025*Ag	=	6.7	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 49.3332	S	=	12.00	in	**USE**
S ≤ 3t = 54					
S ≤ 18" **Governs**	# bars required	=	15		
ρt = Av/(S*t)	ρt	=	0.0308	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.06167	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 49.3332	S	=	12	in	
S ≤ 3t = 54					
S ≤ 18" **Governs**					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	30		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	2.7024	FALSE	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	15	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	30	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	106.56	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	11.41	in^2
	a	=	11.19	in
jd = d - (a/2)	jd	=	112.81	in
	As	=	10.78	in^2

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	24		
Check Capacity:	a	=	10.57	in	
C=T 0.85*f'c*a*b =As*fy	c	=	12.43	in	
c = a/0.85	εt	=	0.03	>0.0025	**OK**
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)			Wall 1		

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.0308	>	$(\rho t)_{min.}$ =	0.0025	**OK**
$\rho l_{provd.}$ =	0.0617	>	$(\rho l)_{min.}$ =	0.0025	**OK**

WALL DIST. HORIZ. REINF.	15	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	32	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 11688 kips > V_u = 2105 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 65,652 kip-ft > M_u = 5,471 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

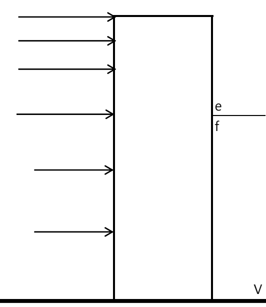
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 35.24 in. **No Boundary Element Needed**

where c = 12 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY					Wall 12 f	X-Direction				
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi		Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	703.0	k
REBAR YIELD STRESS	f_y	=	60	ksi		FACTORED MOMENT LOAD	M_u	=	2887.0	ft-k
HEIGHT OF WALL —	H	=	462.0	in		FACTORED SHEAR LOAD	V_u	=	187.0	k
LENGTH OF SHEAR WALL	L	=	148.0	in						
THICKNESS OF WALL	t	=	18	in		**THE WALL DESIGN IS ADEQUATE.**				
	Acv	=	2663.9928	in^2						
ACI 318-08 § 21.9.2.2, IF Vu ≥2*Acv*√(f'c) ; need at least two curtains (rows) =			337.0	Need 1						

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	187.0	kip
Vn = Vc + Vs	d	=	118.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1347.9	kip
	ΦVn	=	1010.9	kip
ACI 318-08 § 21.9.4	Vn	≤	18048.5	kip
$V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	269.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	168.5	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	101.1	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	-20.2	kip	
S = (Av*fy*d)/Vs	Ag	=	2663.9928	in^2	
	0.0025*Ag	=	6.7	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 49.3332	S	=	8.00	in	**USE**
S ≤ 3t = 54	Av	=	2663.99		
S ≤ 18" **Governs**	# bars required	=	8		
ρt = Av/(S*t)	ρt	=	0.0462	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.06167	>0.0025 **OK**
Max. Spacing S ≤ L/3 = 49.3332	S	=	6	in
S ≤ 3t = 54				
S ≤ 18" **Governs**				
TRY #11	A/bar	=	1.56	in^2
	# bars required	=	4	
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)				
	h/l	=	3.1216	FALSE
	ρl≥ρt	is	**OK**	

WALL DIST. HORIZ. REINF.	8	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	4	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	106.56	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	6.02	in^2
	a	=	5.90	in
jd = d - (a/2)	jd	=	115.45	in
	As	=	5.56	in^2

	A/bar	=	1.56	in^2
TRY #11	# bars required	=	4	
Check Capacity:	a	=	5.45	in
C=T 0.85*f'c*a*b =As*fy	c	=	6.41	in
c = a/0.85	εt	=	0.07	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 6.41 in. **Boundary Element Needed**

where c = 6 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 38.3 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0462 **>** $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.0617 **>** $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 1010.9 kips **>** $V_{u\,=}$ 187.0 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 34,644 kip-ft **>** $M_{u\,=}$ 2,887 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	8	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	14	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY			**Wall 13**			**X-Direction**						
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi		Load Combo: 1.2 D + 1.0L +1.0E		Pu	=	3716	k	at BASE
REBAR YIELD STRESS	f_y	=	60	ksi		FACTORED BASE MOMENT LOAD		M_u	=	41262	ft-k	
HEIGHT OF WALL	H	=	376.0	in		FACTORED BASE SHEAR LOAD		V_u	=	1317	k	
LENGTH OF SHEAR WALL	L	=	254.0	in								
THICKNESS OF WALL	t	=	24	in		**THE WALL DESIGN IS ADEQUATE.**						
	Acv	=	6096.96	in^2								
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			771.2	Need 2								

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	1317.0	kip
Vn = Vc + Vs	d	=	203.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3084.8	kip
	ΦVn	=	2313.6	kip
ACI 318-08 § 21.9.4	Vn	≤	53429.2	kip
$V_n \le A_{cv}$ (αc*$\sqrt{f_c'}$ + $\rho_t f_y$) αc = 2 (conservative)			**OK**	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	617.0	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	385.6	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	231.4	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	1139.0	kip	
S = (Av*fy*d)/Vs	Ag	=	6096.96	in^2	
	0.0025*Ag	=	15.2	in^2	
TRY #8	Abar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 84.68	S	=	8.00	in	**USE**
S ≤ 3t = 72					
S ≤ 18" **Governs**	# bars required	=	19		
ρt = Av/(S*t)	ρt	=	0.0794	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)		ρl	=	0.1186	>0.0025 **OK**
Max. Spacing S ≤ L/3 = 84.68		S	=	6	in
S ≤ 3t = 72					
S ≤ 18" **Governs**					
TRY #11		A/bar	=	1.56	in^2
		# bars required	=	11	
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
		h/l	=	1.4799	ρl≥ρt
		ρl≥ρt	is	**OK**	

WALL DIST. HORIZ. REINF.	19	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	11	#11	@	6	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d					**TRY** #11	A/bar	=	1.56	in^2
C=T 0.85*f'c*a*b =As*fy	jd	=	182.91	in		# bars required	=	32	
Mu = Φ Mn = Φ As*fy*j*d	As	=	50.13	in^2	**Check Capacity:**	a	=	36.48	in
	a	=	36.86	in	C=T 0.85*f'c*a*b =As*fy	c	=	42.92	in
jd = d - (a/2)	jd	=	184.80	in	c = a/0.85	εt	=	0.01	>0.0025 **OK**
	As	=	49.62	in^2	εu = 0.003 dt = L-3"				
					εt = εu*((dt-c)/c)		Wall 1		

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.0794	>	$(\rho t)_{min.}$ =	0.0025	**OK**	
$\rho l_{provd.}$ =	0.1186	>	$(\rho l)_{min.}$ =	0.0025	**OK**	

WALL DIST. HORIZ. REINF.	19	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	32	#11	@	6	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*$\sqrt{f_c'}$ + $\rho_t f_y$) αc = 2 (conservative) 40072 kips > V_u = 1317 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 495,139 kip-ft > M_u = 41,262 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 60.49 in. **No Boundary Element Needed**

where c = 43 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

Appendix G – Lateral Force Resisting System Design Checks-System #2

Special Moment Frame - Level 2					
Moment Frame #	kx cantilever (k/in)	D(in)**	ky cantilever (k/in)	Ix =Ri/∑Ri	I y=Ri/∑Ri
1	125.00	0.0		17.54%	
2	72.57	0.0		10.18%	
3		0.0	72.57		14.44%
5		0.0	100.00		19.91%
6		0.0	100.00		19.91%
7	125.00	0.0		17.54%	
8	125.00	0.0		17.54%	
9	41.67	0.0		5.85%	
10	41.67	0.0		5.85%	
11		0.0	71.43		14.22%
12		0.0	71.43		14.22%
14		0.0	43.48		8.65%
15		0.0	43.48		8.65%
16	90.91	0.0		12.76%	
17	90.91	0.0		12.76%	
∑Ri =	**712.72**		**502.38**	**100.00%**	**100.00%**

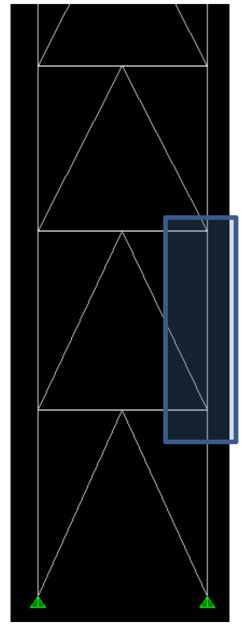
Center of Rigidity Special Moment Frame - Level 2			
X Direction	**k_{ix} (k/ft)**	**x_i (ft)**	**k_{ix} x_i**
SW3	870.83	31.50	27431.06
SW14	521.74	224.00	116869.57
SW5	1200.00	21.67	26000.00
SW6	1200.00	32.00	38400.00
SW11	857.14	224.00	192000.00
SW12	857.14	256.00	219428.57
SW15	500.00	247.67	123835.00
∑	6006.85		743964.20
		x (ft) = $\sum k_{ix}\, x_i / k_{ix}$ =	**123.85**
Y Direction	**k_{iy} (k/ft)**	**y_i (ft)**	**k_{iy} y_i**
SW1	1500.00	97.00	145500.00
SW2	870.83	64.00	55732.95
SW7	1500.00	24.00	36000.00
SW8	1500.00	44.00	66000.00
SW16	1090.91	64.00	69818.18
SW17	1090.91	94.00	102545.45
SW9	500.00	124.00	62000.00
SW10	500.00	111.67	55835.00
∑	8552.65		593431.58
		y (ft) = $\sum k_{iy}\, y_i / k_{iy}$ =	**69.39**

Center of Rigidity Special Concentric Braced Frame - Level 2			
X Direction	**k_{ix} (k/ft)**	**x_i (ft)**	**k_{ix} x_i**
SW3	1333.33	31.50	42000.00
SW4	1333.33	65.50	87333.33
SW5	1714.29	21.67	37142.86
SW6	1714.29	32.00	54857.14
SW11	2000.00	224.00	448000.00
SW12	2000.00	256.00	512000.00
SW13	1500.00	65.50	98250.00
SW14	1575082.14	0.00	0.00
SW15	1407353.53	256.00	360282502.54
∑	2994030.90		361562085.87
		x (ft) = $\sum k_{ix}\, x_i / k_{ix}$ =	**120.76**
Y Direction	**k_{iy} (k/ft)**	**y_i (ft)**	**k_{iy} y_i**
SW1	1277.96	97.00	123961.66
SW2	1333.33	64.00	85333.33
SW7	1333.33	24.00	32000.00
SW8	1333.33	44.00	58666.67
SW9	1714.29	124.00	212571.43
SW10	1714.29	111.67	191434.29
SW16	283841.76	4.00	1135367.03
SW17	248069.26	124.00	30760587.93
∑	540617.54		32599922.34
		y (ft) = $\sum k_{iy}\, y_i / k_{iy}$ =	**60.30**

Special Concentric Braced Frame - Level 2					
Moment Frame #	kx cantilever (k/in)	Δ (in)	ky cantilever (k/in)	Ix =Ri/∑Ri	Iy =Ri/∑Ri
1	53.56	0.02		13.30%	
2	55.56	0.02		13.79%	
3		0.02	55.56		11.78%
4		0.02	55.56		11.78%
5		0.01	71.43		15.15%
6	71.43	0.01		17.73%	
7	83.33	0.01		20.69%	
8	83.33	0.01		20.69%	
9	55.56	0.02		13.79%	
10		0.02	55.56		11.78%
11		0.01	83.33		17.68%
12		0.01	83.33		17.68%
13		0.02	66.67		14.14%
∑Ri =	**402.77**		**471.43**	**100.00%**	**100.00%**

Frame 11 for SCBF Design - Column Check on AISC Manual 13th Edition (AISC 360-05)

COLUMN SECTION	**W14x730**			r =	4.69	in	Table 1-1
COLUMN YIELD STRESS	F_y =	50.0	ksi	ΦPn =	8810	k	Table 4-1
HEIGHT	H =	13.0	ft				
AXIAL LOAD, Factored, Pu	P =	2600.4	kips				



CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05)

$$\frac{P_u}{\phi P_n} > 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

Pu/ΦPn = 0.30

$$\frac{P_u}{\phi P_n} \le 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

0.36 < 1.0 **OK**

Where KL_x =	13.0	ft		
$(KL / r)_{max}$ =	33.3	<	EI	
Pu =	2600.4	kips		
Mu_x =	135.6	ft-kips		
Mu_y =	144.9	ft-kips		
ΦP_c =	8810.0	kips, (AISC 360-05 Chapter E)		Table 4-1
>	Pu	**OK**		
ΦMn_x =	6230.0	ft-kips, (AISC 360-05 Chapter F)		Table 3-2
>	Mu_x	**OK**		
ΦMn_y =	3060.0	ft-kips, (AISC 360-05 Chapter F)		Table 3-4
>	Mu_y	**OK**		

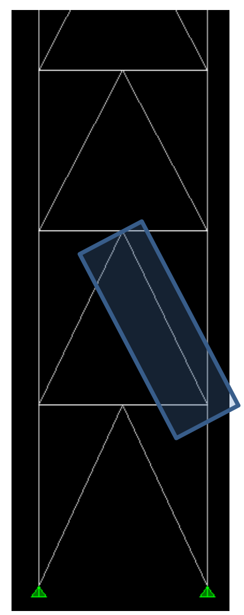
CHECK LATERAL DEFLECTION

D_{max} = 0.01 in

< L / 240 = 0.05 in **OK**

Frame 11 for SCBF Design - Brace Check on AISC Manual 13th Edition (AISC 360-05)

COLUMN SECTION	**W12x252**			r =	3.34	in	Table 1-1
COLUMN YIELD STRESS	F_y =	50.0	ksi	ΦPn =	2770	k	Table 4-1
HEIGHT	H =	14.4	ft				
AXIAL LOAD, Factored, Pu	P =	664.0	kips				



CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05)

$$\frac{P_u}{\phi P_n} > 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

Pu/ΦPn = 0.24

$$\frac{P_u}{\phi P_n} \le 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

0.25 < 1.0 **OK**

Where KL_x =	14.4	ft, for x-x axial load.		
$(KL / r)_{max}$ =	51.7		< 200	
Pu =	664.0	kips		
Mu_x =	0.0	ft-kips		
Mu_y =	9.1	ft-kips		
ΦP_c =	2770.0	kips, (AISC 360-05 Chapter E)		Table 4-1
>	Pu	**OK**		
ΦMn_x =	1610.0	ft-kips, (AISC 360-05 Chapter F)		Table 3-2
>	Mu_x	**OK**		
ΦMn_y =	735.0	ft-kips, (AISC 360-05 Chapter F)		Table 3-4
>	Mu_y	**OK**		

CHECK LATERAL DEFLECTION

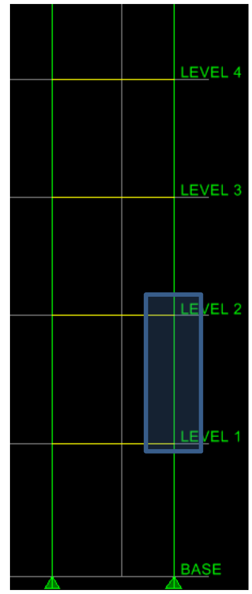
D_{max} = 0.09 in

< L / 240 = 0.06 in **NOT OK**

Where E_s =	29000	ksi
I_x =	272	in^4
I_y =	93.4	in^4

Frame 11 for SMF Design - Column Check on AISC Manual 13th Edition (AISC 360-05)

COLUMN SECTION	W14x730			r =	4.69 in	Table 1-1
COLUMN YIELD STRESS		F_y =	50.0 ksi	ΦPn =	8810 k	Table 4-1
HEIGHT		H =	13.0 ft			
AXIAL LOAD, Factored, Pu		P =	570.7 kips			



CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05)

$$\frac{P_u}{\phi P_n} \geq 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

Pu/ΦPn = 0.06

$$\frac{P_u}{\phi P_n} < 0.2 \Rightarrow \frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

0.15 < 1.0 **OK**

Where KL_x = 13.0 ft

$(KL / r)_{max}$ = 33.3 < 200

Pu = 570.7 kips

Mu_x = 501.8 ft-kips

Mu_y = 46.8 ft-kips

ΦP_c = 8810.0 kips, (AISC 360-05 Chapter E) Table 4-1

> Pu **OK**

ΦMn_x = 6230.0 ft-kips, (AISC 360-05 Chapter F) Table 3-2

> Mu_x **OK**

ΦMn_y = 3060.0 ft-kips, (AISC 360-05 Chapter F) Table 3-4

> Mu_y **OK**

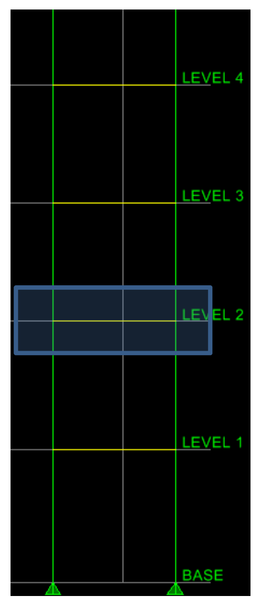
CHECK LATERAL DEFLECTION

D_{max} = 0.01 in

< L / 240 = 0.65 in **OK**

Frame 11 for SMF Design - Beam Check on AISC Manual 13th Edition (AISC 360-05)

COLUMN SECTION	W24x335			r =	3.34 in	Table 1-1
COLUMN YIELD STRESS		F_y =	50.0 ksi	ΦPn =	2770 k	Table 4-1
HEIGHT		L =	12.3 ft			
AXIAL LOAD, Factored, Pu		P =	0.0 kips			



CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05)

$$\frac{P_u}{\phi P_n} \geq 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

Pu/ΦPn = 0.00

$$\frac{P_u}{\phi P_n} < 0.2 \Rightarrow \frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

0.22 < 1.0 **OK**

Where KL_x = 12.3 ft, for x-x axial load.

$(KL / r)_{max}$ = 44.3 < 200

Pu = 0.0 kips

Mu_x = 948.0 ft-kips Vu_x = 0.0 ft-kips

Mu_y = 0.0 ft-kips Vu_y = 247.7 ft-kips

ΦP_c = 2770.0 kips, (AISC 360-05 Chapter E) Table 4-1

> Pu **OK**

ΦMn_x = 3830.0 ft-kips, (AISC 360-05 Chapter F) Table 3-2

> Mu_x **OK**

ΦMn_y = 893.0 ft-kips, (AISC 360-05 Chapter F) Table 3-4

> Mu_y **OK**

CHECK LATERAL DEFLECTION

D_{max} = 0.12 in

< L / 240 = 0.62 in **OK**

CHECK SHEAR CAPACITY ABOUT MAJOR AXIS (AISC 360-05 Chapter G2 or G5)

$V_{allowable}$ = ΦV_n = 1140.0 kips Table 3-2

> V_{Max} **OK**

Special Moment Frames Seismic Provisions , AISC 341-05 9.6

Column-Beam Moment Ratio $\left(\frac{\sum M_{pc}}{\sum M_{pb}}\right) > 1.0$ =

Mx	My
1.6	3.4
OK	**OK**

Appendix H –Cost Analysis for Lateral Force Resisting Systems

RS Means 2007

Exisiting Special Reinforced Shear Walls - Concrete Material and Labor Take-Off

Item	Length (ft)	Width (ft)	Height (ft)	Volume (cf)	Add 10% for waste	Material Unit Cost*	Material Cost [27 cf per cy]	Labor Unit Cost ($/CF) **	Labor Cost	Wall Finish Unit Cost (Matateria and Labor, $/SF)***	Wall Finish Cost	Total Cost
Wall 1	34.00	1.00	62.83	2136.22	2349.84	$108/CY	$ 8,544.88	$ 0.92	$ 536,874.81	$ 0.33	$ 1,409.91	$ 546,829.93
Wall 2	34.00	1.00	69.83	2374.22	2611.64	$108/CY	$ 9,496.88	$ 0.92	$ 663,167.13	$ 0.33	$ 1,566.99	$ 674,231.33
Wall 3	33.00	1.00	69.83	2304.39	2534.83	$108/CY	$ 9,217.56	$ 0.92	$ 643,662.21	$ 0.33	$ 1,520.90	$ 654,401.00
Wall 4	33.00	1.00	62.83	2073.39	2280.73	$108/CY	$ 8,293.56	$ 0.92	$ 521,084.37	$ 0.33	$ 1,368.44	$ 530,746.70
Wall 5	20.00	1.00	69.83	1396.60	1536.26	$108/CY	$ 5,586.40	$ 0.92	$ 390,098.31	$ 0.33	$ 921.76	$ 396,606.80
Wall 6	20.00	1.00	69.83	1396.60	1536.26	$108/CY	$ 5,586.40	$ 0.92	$ 390,098.31	$ 0.33	$ 921.76	$ 396,606.80
Wall 7	10.33	1.00	69.83	721.57	793.73	$108/CY	$ 2,886.30	$ 0.92	$ 201,550.14	$ 0.33	$ 476.24	$ 204,913.01
Wall 8	10.33	1.00	69.83	721.55	793.71	$108/CY	$ 2,886.21	$ 0.92	$ 201,544.29	$ 0.33	$ 476.23	$ 204,907.06
Wall 9	32.00	1.00	71.83	2298.56	2528.42	$108/CY	$ 9,194.24	$ 0.92	$ 660,422.26	$ 0.33	$ 1,517.05	$ 671,133.88
Wall 10	32.00	1.00	71.83	2298.56	2528.42	$108/CY	$ 9,194.24	$ 0.92	$ 660,422.26	$ 0.33	$ 1,517.05	$ 671,133.88
Wall 11	12.33	1.50	71.83	1328.85	1461.74	$108/CY	$ 5,315.41	$ 0.92	$ 381,805.59	$ 0.33	$ 584.69	$ 387,706.02
Wall 12	12.33	1.50	71.83	1328.85	1461.74	$108/CY	$ 5,315.41	$ 0.92	$ 381,805.59	$ 0.33	$ 584.69	$ 387,706.02
Wall 13	20.00	2.00	69.83	2793.20	3072.52	$108/CY	$ 11,172.80	$ 0.92	$ 780,196.62	$ 0.33	$ 921.76	$ 792,291.51
											Total Concrete Cost	$5,726,922

* Normal Weight Concrete, Ready Mix 4000psi (Agilia, Self Consolidating Concrete)

** Placing of concrete (Walls, pumped) for Labor and Equipment

*** Wall finish

RS Means 2007 COSTS - EXISTING SPECIAL REINFORCED SHEAR WALLS REBAR

Type	Thickness (in)	Vertical Spacing (in)	Vertical Bar Size	Vertical Bar Diameter (in)	Vertical Bar Weight (plf)	Horizontal Spacing (in)	Horizontal Bar Size	Horizontal Bar Diameter (in)	Bar Weight (plf)
a	12	12	6	0.75	1.502	12	6	0.75	1.502
b	12	6	11	1.41	5.313	8	8	1	2.67
c	12	12	6	0.75	1.502	12	6	0.75	1.502
d	12	8	11	1.41	5.313	8	8	1	2.67
e	18	12	6	0.75	1.502	12	6	0.75	1.502
f	18	6	11	1.41	5.313	8	8	1	2.67
g	18	6	11	1.41	5.313	8	8	1	2.67
h	18	6	11	1.41	5.313	8	8	1	2.67

WALL	Height (ft)	# Veritical bar spaces	# bars	Bar Length (ft) 3" cover	Total bar length (ft)	Total weight (pounds)	Add 10% for waste and lap	Length (ft)	# Horizontal bar spaces	# bars	Bar Length (ft) 3" cover	Total bar length	Total weight (pounds)	Add 10% for waste and lap
Wall 1_a	24.33	24.33	25	23.83	603.61	906.63	997.29	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 1_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 2_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 2_b	38.50	77.00	78	38.00	2964.00	4179.24	4597.16	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 3_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 3_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 4_a	24.33	24.33	25	23.83	603.61	906.63	997.29	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 4_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 5_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	20.00	20.00	21	19.50	409.50	615.07	676.58
Wall 5_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	20.00	30.00	31	19.50	604.50	1614.02	1775.42
Wall 6_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	20.00	20.00	21	19.50	409.50	615.07	676.58
Wall 6_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	20.00	30.00	31	19.50	604.50	1614.02	1775.42
Wall 7_c	31.33	31.33	32	30.83	996.73	1497.09	1646.80	10.33	10.33	11	9.83	111.44	167.38	184.12
Wall 7_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	10.33	15.50	16	9.83	162.24	433.18	476.50
Wall 8_c	31.33	31.33	32	30.83	996.73	1497.09	1646.80	10.33	10.33	11	9.83	111.44	167.38	184.12
Wall 8_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	10.33	15.50	16	9.83	162.15	432.93	476.22
Wall 9_c	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 9_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 10_c	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 10_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 11_e	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	12.33	12.33	13	11.83	157.77	236.97	260.67
Wall 11_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	12.33	18.50	19	11.83	230.74	616.07	677.68
Wall 12_e	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	12.33	12.33	13	11.83	157.78	236.98	260.68
Wall 12_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	12.33	18.50	19	11.83	230.75	616.10	677.71
Wall 13_g	31.33	62.66	64	30.83	1962.64	10427.49	11470.24	21.17	31.76	33	20.67	677.05	1016.92	1118.62
Wall 13_h	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	21.17	31.76	33	20.67	677.05	1807.71	1988.48

Existing Special Reinforced Shear Walls - Reinforcing Bars Material and Labor Cost

ITEM	Daily Output (tons)	Labor (Hours)	Material Cost	Labor Cost	Work Hours/ton	Material (tons)	Material Cost	Labor Cost
Walls, Rebar #3 to #7	3	10.67	$850/ton	$440/ton	3.56	16.59	$ 14,105.68	$ 7,301.76
Walls, Rebar #8 to #18	4	8.00	$850/ton	$330/ton	2.00	107.30	$ 91,205.32	$ 35,409.13
						Total Reinforcing Bar Cost		$ 148,021.89

RS Means 2007

Modified Special Reinforced Shear Walls - Concrete Material and Labor Take-Off

Item	Length (ft)	Width (ft)	Height (ft)	Volume (cf)	Add 10% for waste	Material Unit Cost*	Material Cost [27 cf per cy]	Labor Unit Cost ($/CF) **	Labor Cost	Wall Finish Unit Cost (Mataterial and Labor, $/SF)***	Wall Finish Cost	Total Cost
Wall 1	20.00	1.50	62.83	1884.90	2073.39	$108/CY	$ 7,539.60	$ 0.92	$ 473,713.07	$ 0.33	$ 829.36	$ 482,082.35
Wall 2	34.00	1.50	69.83	3561.33	3917.46	$108/CY	$ 14,245.32	$ 0.92	$ 994,750.70	$ 0.33	$ 1,566.99	$1,010,563.33
Wall 5	20.00	1.50	69.83	2094.90	2304.39	$108/CY	$ 8,379.60	$ 0.92	$ 585,147.47	$ 0.33	$ 921.76	$ 594,449.15
Wall 6	20.00	1.50	69.83	2094.90	2304.39	$108/CY	$ 8,379.60	$ 0.92	$ 585,147.47	$ 0.33	$ 921.76	$ 594,449.15
Wall 7	10.33	1.00	69.83	721.57	793.73	$108/CY	$ 2,886.30	$ 0.92	$ 201,550.14	$ 0.33	$ 476.24	$ 204,913.01
Wall 8	10.33	1.00	69.83	721.55	793.71	$108/CY	$ 2,886.21	$ 0.92	$ 201,544.29	$ 0.33	$ 476.23	$ 204,907.06
Wall 9	34.00	1.00	71.83	2442.22	2686.44	$108/CY	$ 9,768.88	$ 0.92	$ 701,698.65	$ 0.33	$ 1,611.87	$ 713,079.73
Wall 10	34.00	1.00	71.83	2442.22	2686.44	$108/CY	$ 9,768.88	$ 0.92	$ 701,698.65	$ 0.33	$ 1,611.87	$ 713,079.73
Wall 11	12.33	1.50	71.83	1328.85	1461.74	$108/CY	$ 5,315.41	$ 0.92	$ 381,805.59	$ 0.33	$ 584.69	$ 387,706.02
Wall 12	12.33	1.50	71.83	1328.85	1461.74	$108/CY	$ 5,315.41	$ 0.92	$ 381,805.59	$ 0.33	$ 584.69	$ 387,706.02
Wall 13	20.00	2.00	69.83	2793.20	3072.52	$108/CY	$ 11,172.80	$ 0.92	$ 780,196.62	$ 0.33	$ 921.76	$ 792,291.51
											Total Concrete Cost	**$5,292,936**

* Normal Weight Concrete, Ready Mix 4000psi (Agilia, Self Consolidating Concrete)

** Placing of concrete (Walls, pumped) for Labor and Equipment

*** Wall finish

****Agilia, Self-Consolidating Concrete from Lafarge Conrete

RS Means 2007 COSTS - MODIFIED SPECIAL REINFORCED SHEAR WALLS REBAR

Type	Thickness (in)	Vertical Spacing (in)	Vertical Bar Size	Vertical Bar Diameter (in)	Vertical Bar Weight (plf)	Horizontal Spacing (in)	Horizontal Bar Size	Horizontal Bar Diameter (in)	Horizontal Bar Weight
a	12	12	6	0.75	1.502	12	6	0.75	1.502
b	12	6	11	1.41	5.313	8	8	1	2.67
c	12	12	6	0.75	1.502	12	6	0.75	1.502
d	12	8	11	1.41	5.313	8	8	1	2.67
e	18	12	6	0.75	1.502	12	6	0.75	1.502
f	18	6	11	1.41	5.313	8	8	1	2.67
g	24	6	11	1.41	5.313	8	8	1	2.67
h	24	6	11	1.41	5.313	8	8	1	2.67

WALL	Height (ft)	# Veritical bar spaces	# bars	Bar Length (ft) 3" cover	Total bar length (ft)	Total weight (pounds)	Add 10% for waste and lap	Length (ft)	# Horizontal bar spaces	# bars	Bar Length (ft) 3" cover	Total bar length	Total weight (pounds)	Add 10% for waste and lap
Wall 1_f	24.33	48.66	50	23.83	1183.40	1777.46	1955.21	34.00	51.00	52	33.50	1742.00	2616.48	2878.13
Wall 1_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 2_f	31.33	62.66	64	30.83	1962.64	2947.88	3242.67	34.00	51.00	52	33.50	1742.00	2616.48	2878.13
Wall 2_f	38.50	77.00	78	38.00	2964.00	4179.24	4597.16	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 5_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	20.00	20.00	21	19.50	409.50	615.07	676.58
Wall 5_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	20.00	30.00	31	19.50	604.50	1614.02	1775.42
Wall 6_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	20.00	20.00	21	19.50	409.50	615.07	676.58
Wall 6_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	20.00	30.00	31	19.50	604.50	1614.02	1775.42
Wall 7_c	31.33	31.33	32	30.83	996.73	1497.09	1646.80	10.33	10.33	11	9.83	111.44	167.38	184.12
Wall 7_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	10.33	15.50	16	9.83	162.24	433.18	476.50
Wall 8_c	31.33	31.33	32	30.83	996.73	1497.09	1646.80	10.33	10.33	11	9.83	111.44	167.38	184.12
Wall 8_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	10.33	15.50	16	9.83	162.15	432.93	476.22
Wall 9_c	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 9_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 10_c	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 10_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 11_e	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	12.33	12.33	13	11.83	157.77	236.97	260.67
Wall 11_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	12.33	18.50	19	11.83	230.74	616.07	677.68
Wall 12_e	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	12.33	12.33	13	11.83	157.78	236.98	260.68
Wall 12_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	12.33	18.50	19	11.83	230.75	616.10	677.71
Wall 13_g	31.33	62.66	64	30.83	1962.64	10427.49	11470.24	21.17	31.76	33	20.67	677.05	1016.92	1118.62
Wall 13_h	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	21.17	31.76	33	20.67	677.05	1807.71	1988.48

Modified Special Reinforced Shear Walls - Reinforcing Bars Material and Labor Cost

ITEM	Daily Output (tons)	Labor (Hours)	Material Cost	Labor Cost	Work Hours/ton	Material (tons)	Material Cost	Labor Cost
Walls, Rebar #3 to #7	3	10.67	$850/ton	$440/ton	3.56	15.55	$ 13,220.46	$ 6,843.53
Walls, Rebar #8 to #18	4	8.00	$850/ton	$330/ton	2.00	84.86	$ 72,132.38	$ 28,004.34
						Total Reinforcing Bar Cost		$ 120,200.71

RS MEANS 2007 COSTS - SPECIAL MOMENT FRAMES				
FRAME	LEVEL	BEAM	LENGTH (ft)	Weight (lb)
1	LR	W24x370	34	62900.0
2	HR	W24x370	34	75480.0
3	HR	W24x370	33	73260.0
5	HR	W24x370	20	44400.0
6	HR	W24x370	20	44400.0
7	HR	W24x335	10.333	20769.3
8	HR	W18x158	10.333	9795.7
9	STAIR3	W18x158	32	30336.0
10	STAIR3	W18x158	32	30336.0
11	STAIR3	W18x158	12.333	11691.7
12	STAIR3	W18x175	12.333	12949.7
15	HR	W24x370	30	66600.0
16	HR	W24x370	23.667	52540.7
17	HR	W24x370	23.667	52540.7
			Total Weight (tons)	294

RS MEANS 2007 COSTS - SPECIAL MOMENT FRAMES			
Grid Line	Column	LENGTH (ft)	Weight (lb)
8-A	W14x730	71.83	52435.9
9-A	W14x730	71.83	52435.9
8-SW10	W14x730	71.83	52435.9
9-SW10	W14x730	71.83	52435.9
8-B	W14x730	69.93	51048.9
8.8-B	W14x730	69.93	51048.9
8-C	W14x730	69.93	51048.9
8.8-C	W14x730	69.93	51048.9
3.1-SW1	W14x730	62.83	45865.9
3.1-C	W14x730	62.83	45865.9
1.9-SW1	W14x730	69.93	51048.9
1.9-C	W14x730	69.93	51048.9
1.9-SW8	W14x730	69.93	51048.9
1.9-D2	W14x730	69.93	51048.9
SW5-SW8	W14x730	69.93	51048.9
SW5-D2	W14x730	69.93	51048.9
		Total (tons) =	406

Special Moment Frame					
Iteam	Tonnage of Steel	Material($/ton)	Labor ($/ton)	Equipment ($/ton)	Total
Beams	294	2050	225	115	$ 702,660
Columns	406	2050	225	115	$ 970,297

Connection Fabrication				
#MF's	# of Connections	Fabrication Time (hrs)	Cost ($/Fabr.hr)	Total
15	95	4.8 Ea.	45	$ 20,520

Connection Installation			
Installation Time (days)	Installation Time (hrs)	Cost ($/Labor hr)	Total
15	95	4.8 Ea.	$ 3,240

Appendix I – Mechanical Breadth Cost Analysis

Electricity Costs (PNM)		Summer	Winter
Utility		June-Aug	Sept-May
Electricity consumption/KW		$9.56	$8.19
Electricity demand per month/KWh		$0.0821025	$0.064170

Gas Costs (New Mexico Gas Company)	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
Gas distribution/ therm	$ 0.49100	$ 0.53090	$ 0.47790	$0.50080	$ 0.46870	$ 0.52350	$ 0.56990	$ 0.53470	$ 0.49170	$ 0.53750	$ 0.49010	$ 0.45170

VRE 3-54 Glazing

Monthly Costs	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	
Elec(KWh)	2,051	1,819	2,170	1,955	3,576	4,815	5,946	4,755	3,458	2,271	2,022	1,981	
Consumption ($)	$ 131.61	$ 116.72	$ 139.25	$ 125.45	$ 229.47	$ 395.32	$ 488.18	$ 390.40	$ 221.90	$ 145.73	$ 129.75	$ 127.12	
Peak(KW)	10	11	11	11	17	19	19	17	16	11	11	10	
Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$181.64	$162.52	$131.04	$90.09	$90.09	$81.90	
Gas(therms)	1217	672	498	60	1	0	0	0	1	71	629	1041	
Gas Dist ($)	$597.55	$356.76	$237.99	$30.05	$0.47	$0.00	$0.00	$0.00	$0.49	$38.16	$308.27	$470.22	
Total Elec. Cons ($)	$ 131.61	$ 116.72	$ 139.25	$ 125.45	$ 229.47	$ 395.32	$ 488.18	$ 390.40	$ 221.90	$ 145.73	$ 129.75	$ 127.12	
Total Elec. Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$181.64	$162.52	$131.04	$90.09	$90.09	$81.90	
Total gas dist ($)	$597.55	$356.76	$237.99	$30.05	$0.47	$0.00	$0.00	$0.00	$0.49	$38.16	$308.27	$470.22	TOTAL
Total Elect. Costs	$ 213.51	$ 206.81	$ 229.34	$ 215.54	$ 368.70	$ 576.96	$ 669.82	$ 552.92	$ 352.94	$ 235.82	$ 219.84	$ 209.02	$ 4,051.23
Total Gas Costs	$597.55	$356.76	$237.99	$30.05	$0.47	$0.00	$0.00	$0.00	$0.49	$38.16	$308.27	$470.22	$ 2,039.97
Total	$ 811.06	$ 563.58	$ 467.33	$ 245.59	$ 369.17	$ 576.96	$ 669.82	$ 552.92	$ 353.43	$ 273.98	$ 528.11	$ 679.24	$ 6,091.20

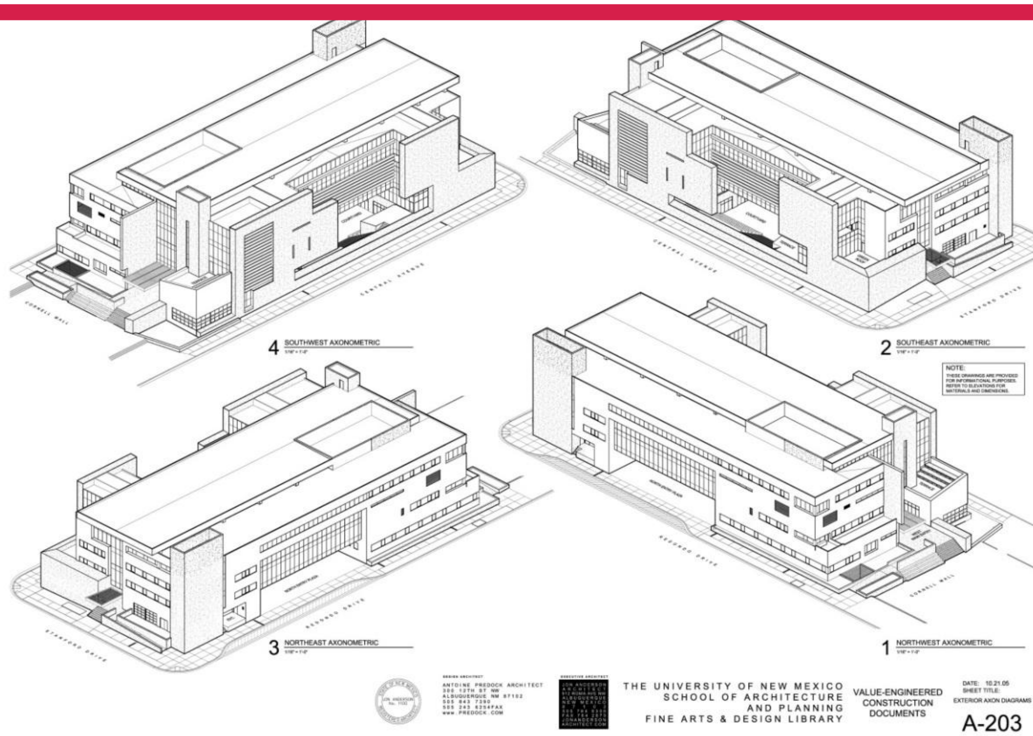
VNE 1-30 Glazing

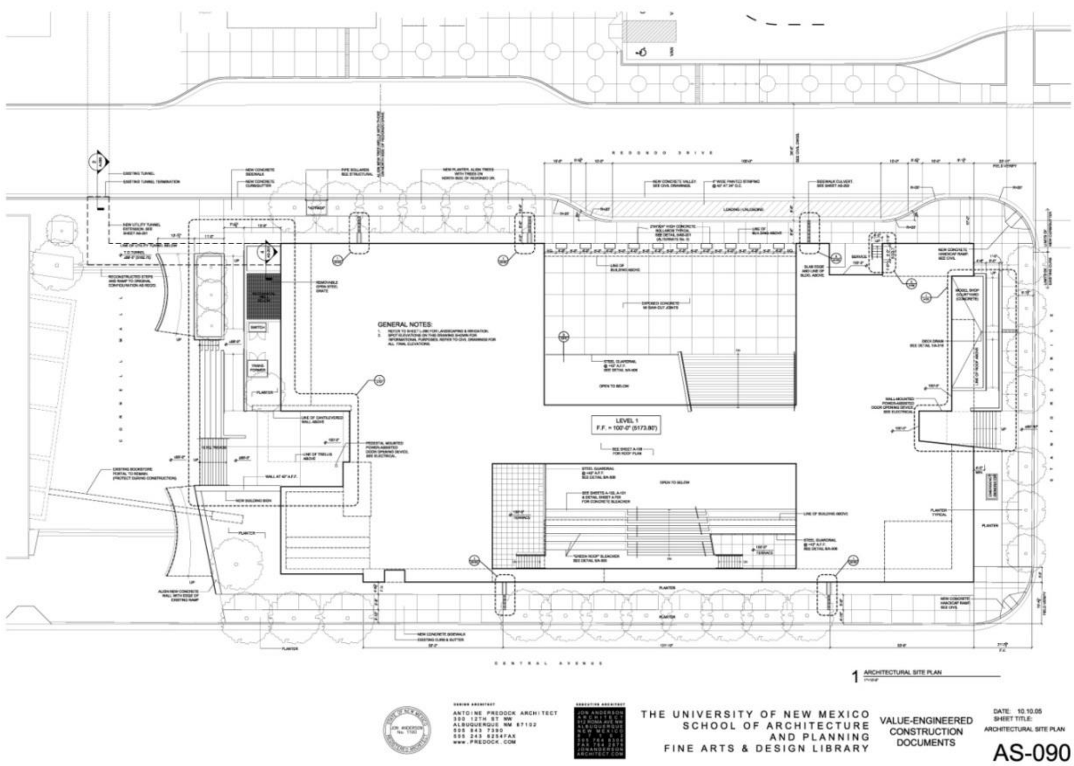
Monthly Costs	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	
Elec(KWh)	2,037	1,805	2,152	1,973	3,618	4,776	5,936	4,751	3,476	2,280	2,011	1,963	
Consumption ($)	$ 130.71	$ 115.83	$ 138.09	$ 126.61	$ 232.17	$ 392.12	$ 487.36	$ 390.07	$ 223.05	$ 146.31	$ 129.05	$ 125.96	
Peak(KW)	10	11	11	11	17	19	19	17	16	12	11	10	
Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$181.64	$162.52	$131.04	$98.28	$90.09	$81.90	
Gas(therms)	1139	623	451	50	1	0	0	0	1	61	586	974	
Gas Dist ($)	$559.25	$330.75	$215.53	$25.04	$0.47	$0.00	$0.00	$0.00	$0.49	$32.79	$287.20	$439.96	
Total Elec. Cons ($)	$ 130.71	$ 115.83	$ 138.09	$ 126.61	$ 232.17	$ 392.12	$ 487.36	$ 390.07	$ 223.05	$ 146.31	$ 129.05	$ 125.96	
Total Elec. Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$181.64	$162.52	$131.04	$98.28	$90.09	$81.90	
Total gas dist ($)	$559.25	$330.75	$215.53	$25.04	$0.47	$0.00	$0.00	$0.00	$0.49	$32.79	$287.20	$439.96	TOTAL
Total Elect. Costs	$ 212.61	$ 205.92	$ 228.18	$ 216.70	$ 371.40	$ 573.76	$ 669.00	$ 552.59	$ 354.09	$ 244.59	$ 219.14	$ 207.86	$ 4,055.84
Total Gas Costs	$559.25	$330.75	$215.53	$25.04	$0.47	$0.00	$0.00	$0.00	$0.49	$32.79	$287.20	$439.96	$ 1,891.47
Total	$ 771.86	$ 536.67	$ 443.72	$ 241.74	$ 371.86	$ 573.76	$ 669.00	$ 552.59	$ 354.59	$ 277.37	$ 506.33	$ 647.82	$ 5,947.31

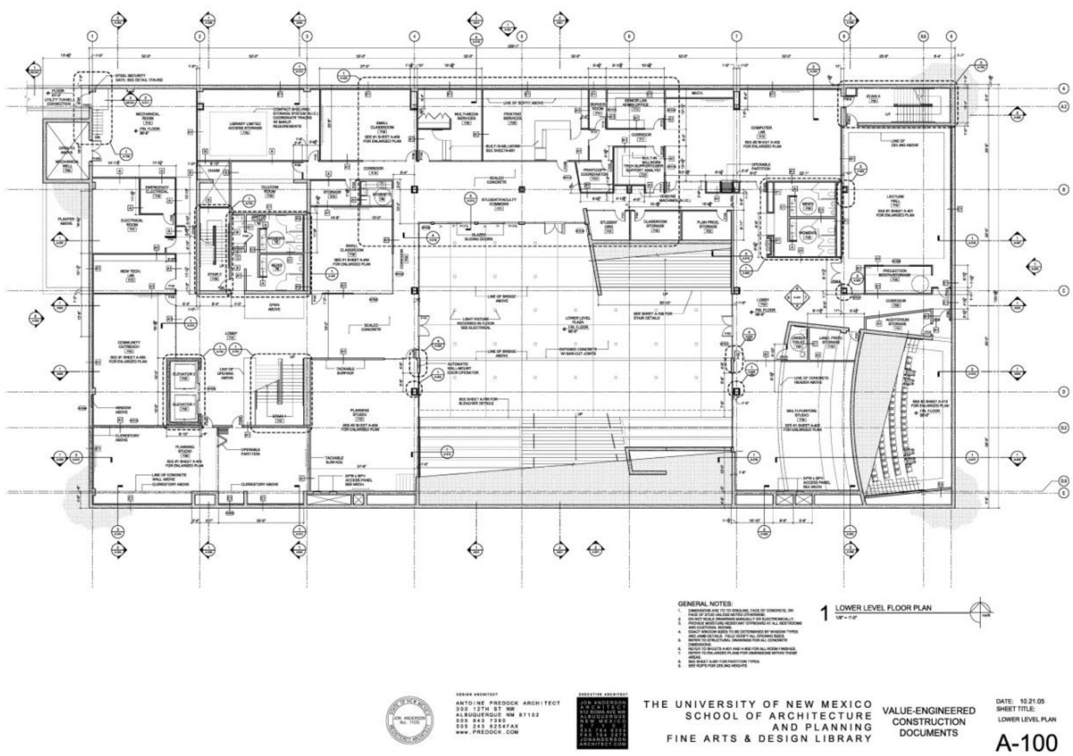
VRE1-63 Glazing

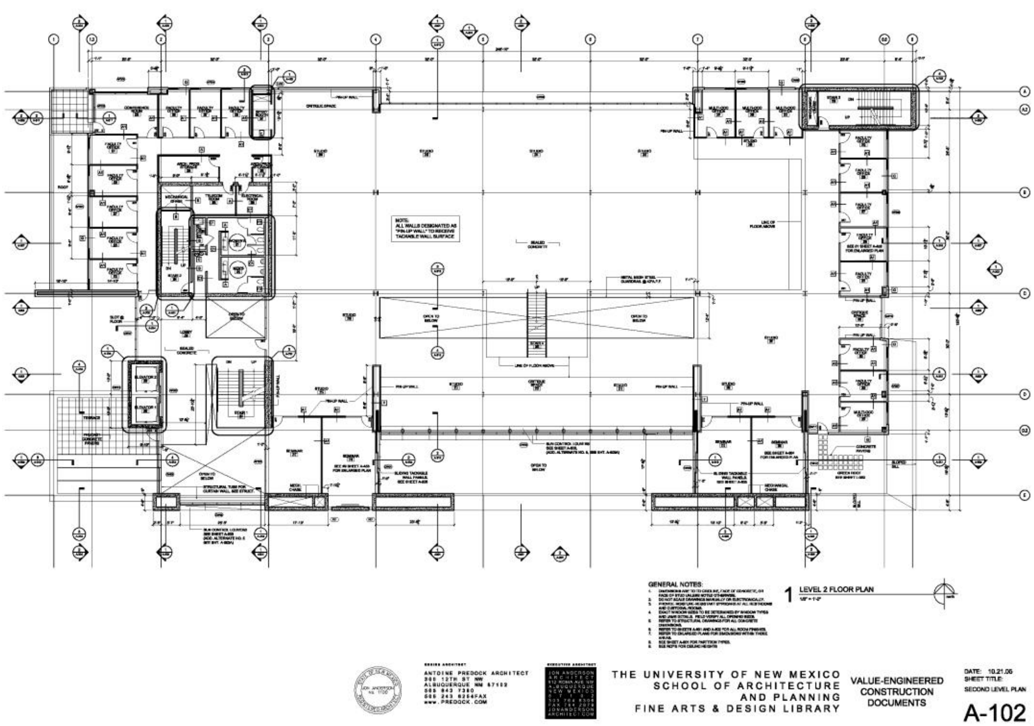
Monthly Costs	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	
Elec(KWh)	2,014	1,799	2,143	1,970	3,643	4,774	5,906	4,749	3,500	2,292	1,999	1,940	
Consumption ($)	$ 129.24	$ 115.44	$ 137.52	$ 126.41	$ 233.77	$ 391.96	$ 484.90	$ 389.90	$ 224.59	$ 147.08	$ 128.28	$ 124.49	
Peak(KW)	10	11	11	11	17	19	18	17	16	12	11	10	
Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$172.08	$162.52	$131.04	$98.28	$90.09	$81.90	
Gas(therms)	1082	587	418	42	1	0	0	0	1	53	555	925	
Gas Dist ($)	$531.26	$311.64	$199.76	$21.03	$0.47	$0.00	$0.00	$0.00	$0.49	$28.49	$272.01	$417.82	
Total Elec. Cons ($)	$ 129.24	$ 115.44	$ 137.52	$ 126.41	$ 233.77	$ 391.96	$ 484.90	$ 389.90	$ 224.59	$ 147.08	$ 128.28	$ 124.49	
Total Elec. Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$172.08	$162.52	$131.04	$98.28	$90.09	$81.90	
Total gas dist ($)	$531.26	$311.64	$199.76	$21.03	$0.47	$0.00	$0.00	$0.00	$0.49	$28.49	$272.01	$417.82	TOTAL
Total Elect. Costs	$ 211.14	$ 205.53	$ 227.61	$ 216.50	$ 373.00	$ 573.60	$ 656.98	$ 552.42	$ 355.63	$ 245.36	$ 218.37	$ 206.39	$ 4,042.52
Total Gas Costs	$531.26	$311.64	$199.76	$21.03	$0.47	$0.00	$0.00	$0.00	$0.49	$28.49	$272.01	$417.82	$ 1,782.97
Total	$ 742.40	$ 517.17	$ 427.37	$ 237.54	$ 373.47	$ 573.60	$ 656.98	$ 552.42	$ 356.13	$ 273.84	$ 490.37	$ 624.21	$ 5,825.49

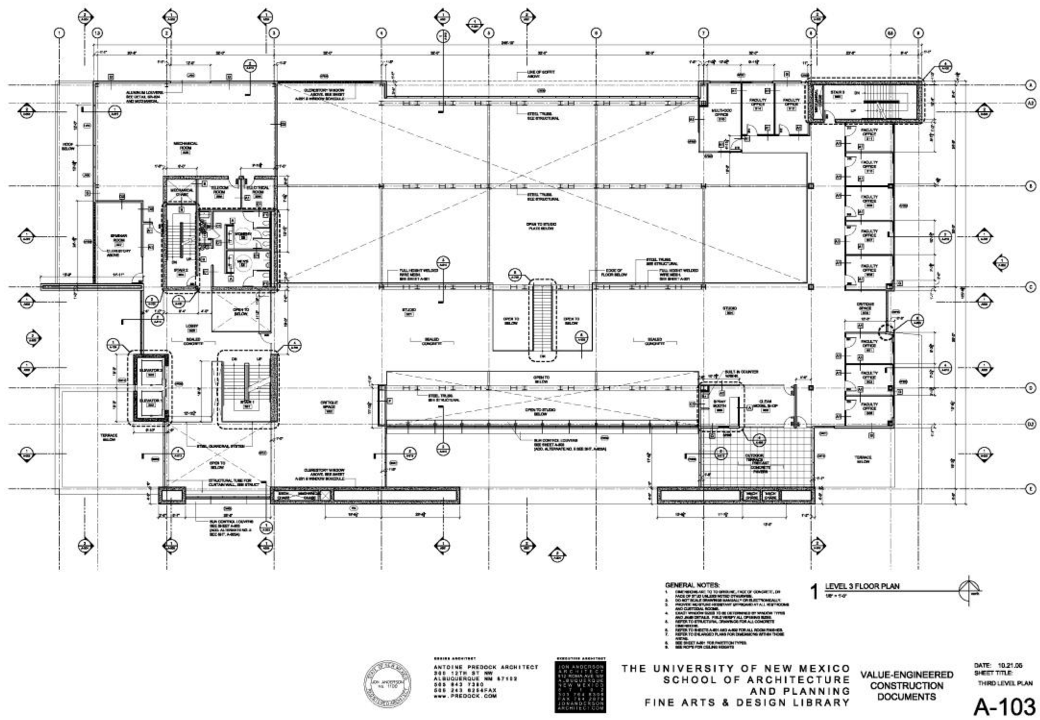
Appendix J –Construction Documents

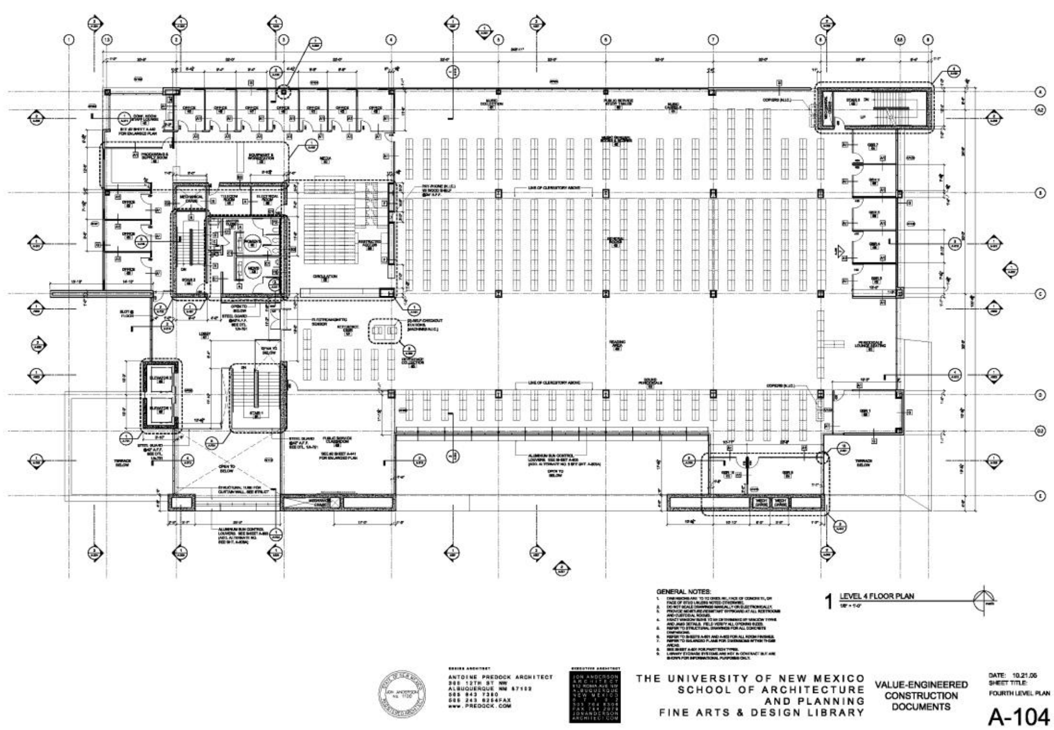


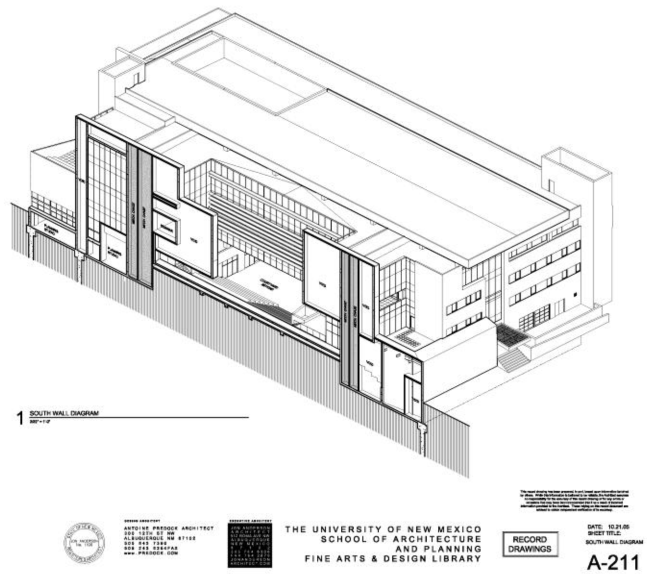


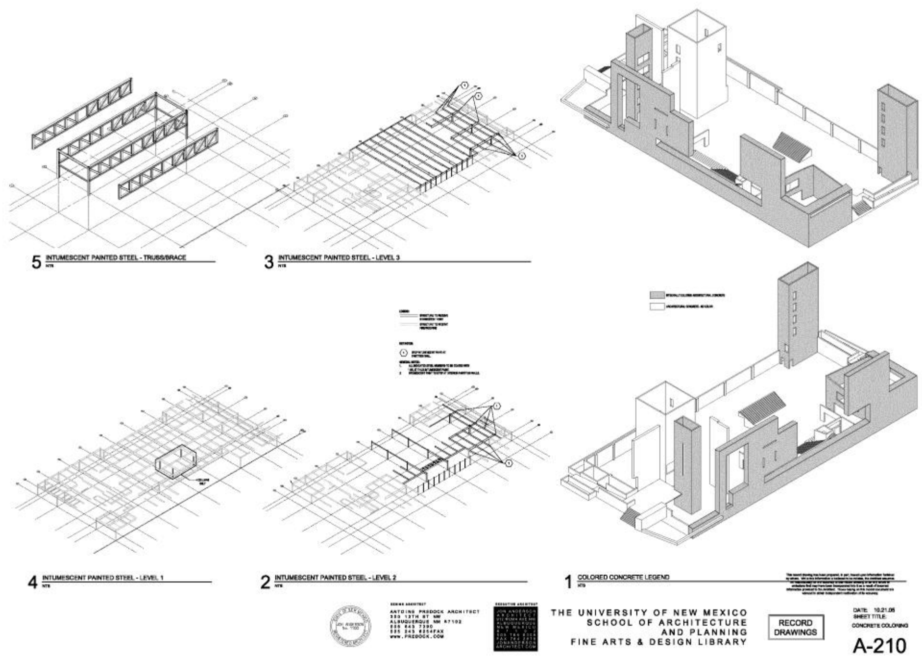

GENERAL NOTES:
1 LOWER LEVEL FLOOR PLAN
ANTOINE PREDOCK ARCHITECT
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
DATE: 10.21.05
SHEET TITLE:
LOWER LEVEL PLAN
A-100

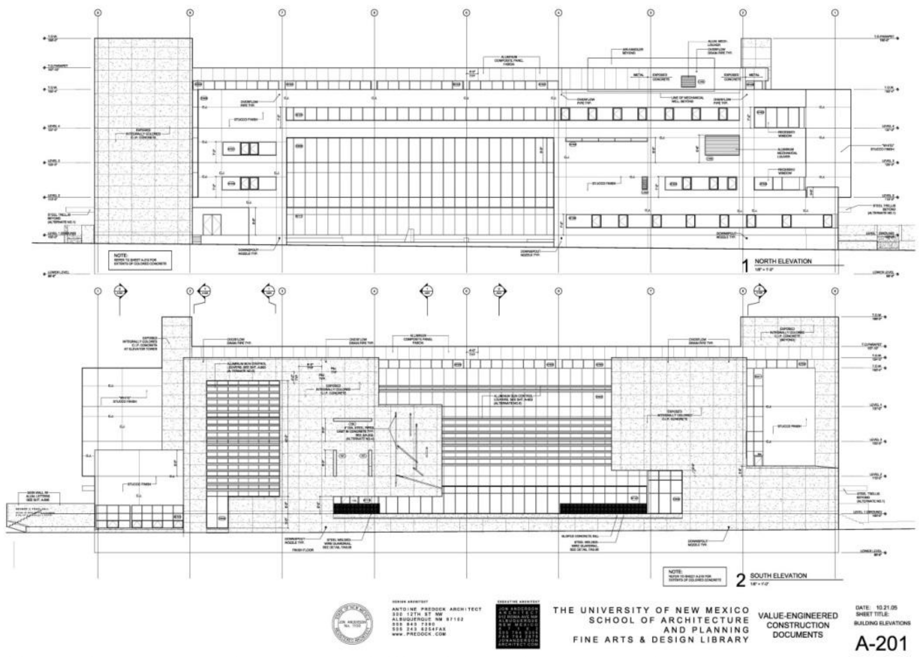

GENERAL NOTES:
1 LEVEL 2 FLOOR PLAN
ANTOINE PREDOCK ARCHITECT
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
DATE: 10.21.05
SHEET TITLE:
SECOND LEVEL PLAN
A-102

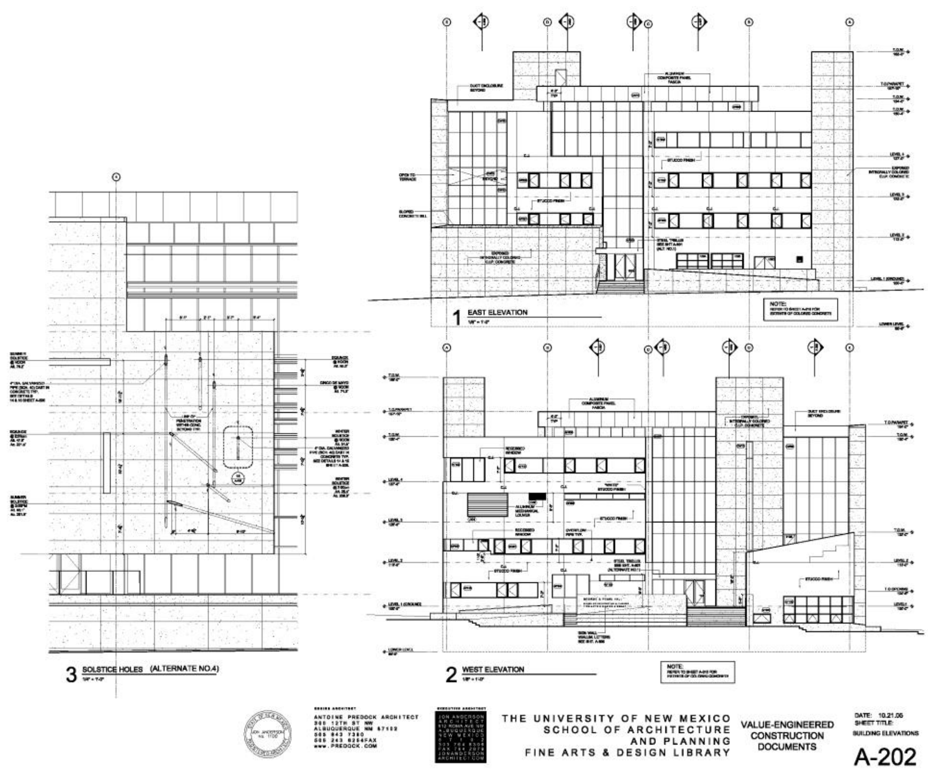

GENERAL NOTES:
1 LEVEL 3 FLOOR PLAN
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
THIRD LEVEL PLAN
A-103

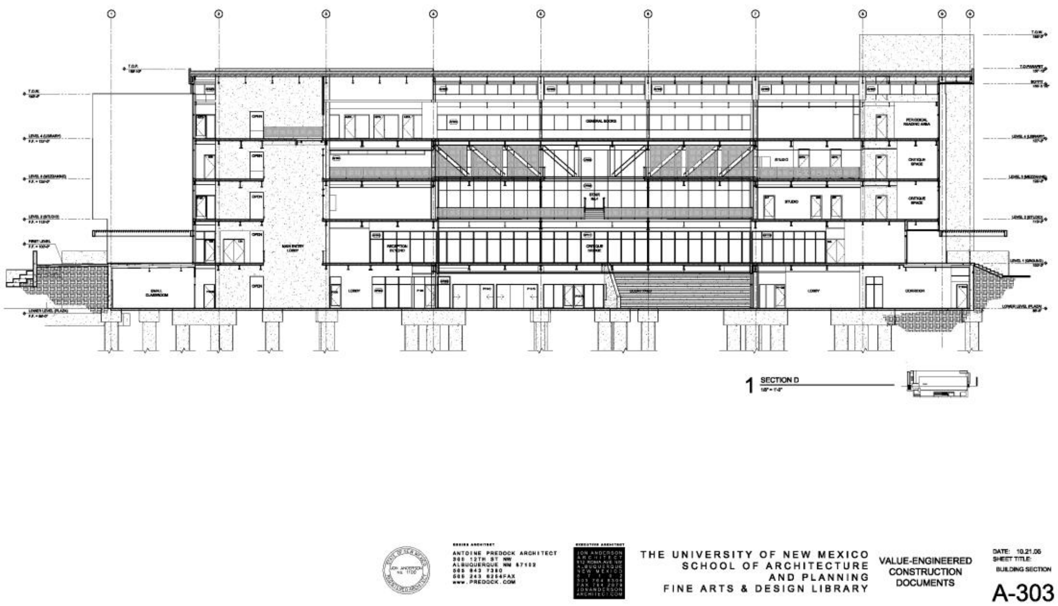

GENERAL NOTES:
1 LEVEL 4 FLOOR PLAN
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
FOURTH LEVEL PLAN
A-104

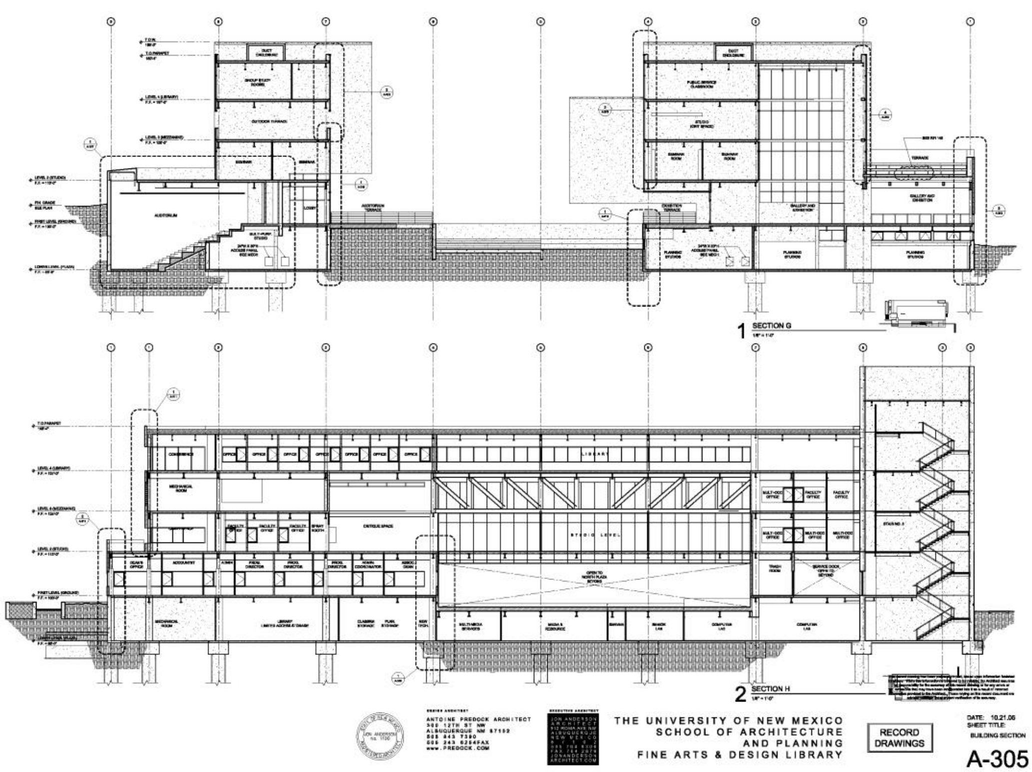

1 SOUTH WALL DIAGRAM
ANTOINE PREDOCK ARCHITECT
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
RECORD DRAWINGS
DATE: 10.21.05
SHEET TITLE:
SOUTH WALL DIAGRAM
A-211


5 INTUMESCENT PAINTED STEEL - TRUSS/BRACE
3 INTUMESCENT PAINTED STEEL - LEVEL 3
4 INTUMESCENT PAINTED STEEL - LEVEL 1
2 INTUMESCENT PAINTED STEEL - LEVEL 2
1 COLORED CONCRETE LEGEND
ANTOINE PREDOCK ARCHITECT
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
RECORD DRAWINGS
DATE: 10.21.05
SHEET TITLE:
CONCRETE COLORING
A-210


1 NORTH ELEVATION
2 SOUTH ELEVATION
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
A-201


1 EAST ELEVATION
2 WEST ELEVATION
3 SOLSTICE HOLES (ALTERNATE NO.4)
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
A-202


1 SECTION D
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
A-303


1 SECTION G
2 SECTION H
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
RECORD
DRAWINGS
A-305