

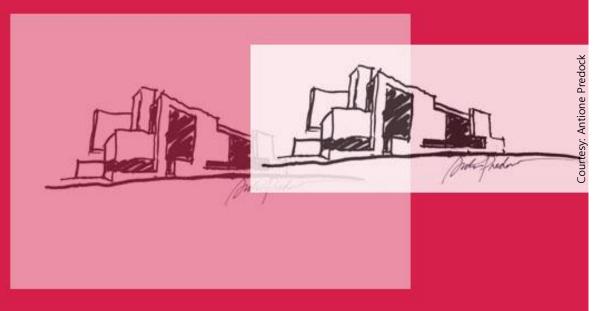


UNIVERSITY OF NEW MEXICO SCHOOL OF ARCHITECTURE + PLANNING

Albuquerque, NM

April 4, 2012

# Structural Final Report



Faculty Advisor: Dr. Richard Behr

**NICOLE TRUJILLO** 



# 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**

Size: Delivery:

Architecture School 108.000 GSF 65 Feet Nov 2005 - Sept 2007 \$22 Million 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 Design Architect: Executive Architect: Structural Engineer: **MEP Engineer Civil Engineer:** General Contractor: Mechanical Contractor: **Electrical Contractor:** 

University of New Mexico Antione Predock Jon Anderson Chavez-Grieves Bridges & Paxton Jeff Mortensen & Assoc. **Jaynes** Yearout Mechanical McDade-Woodcock



# MEP SYSTEMS

- - 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

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#### 4/4/2012

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# **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).

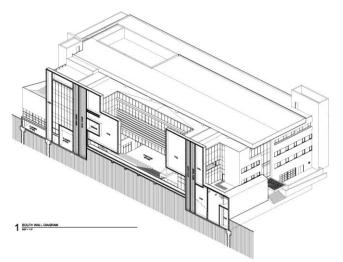


**Figure 3.** South View at Night (Courtesy: Kirk Gittings, Photographer)



**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)



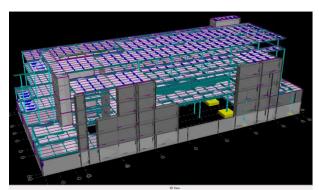


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# 2. Structural Overview

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).

Pearl Hall uses primarily steel frame construction with concrete shear walls. The south side walls are comprised of large cantilevered concrete and

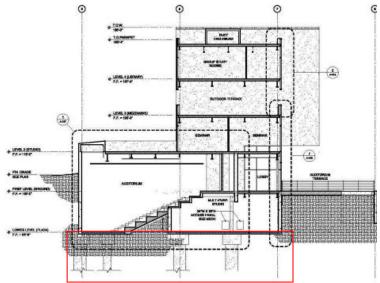


**Figure 8.** *RAM Structural Model of Pearl Hall.* (*Courtesy: Chavez-Grieves Consulting Engineers*)

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



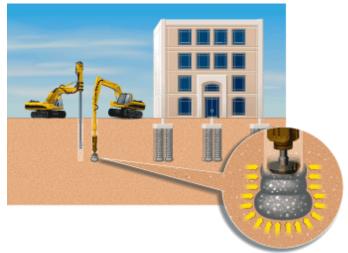
**Figure 9.** Section through Lower Level at Auditorium (Courtesy: Jon Anderson Architects)

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).

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



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

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.

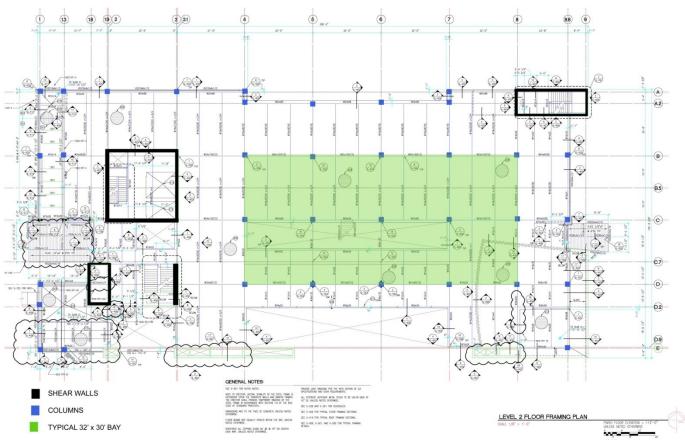
# 2.2 Floor Systems

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.

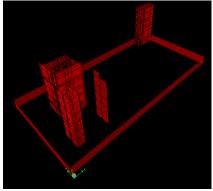
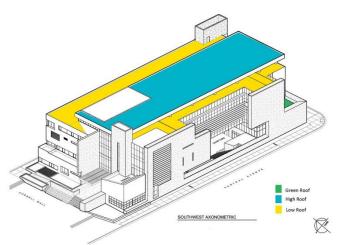


Figure 12. ETABS model of shear walls.

## 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

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, 9 <sup>th</sup> Edition (1989)	AISC Manual of Steel Construction –LRFD, 13 <sup>th</sup> 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

 Table 1. Design Codes

Building Code Analysis (Sheet G-100 Jon Anderson Architects)							
Assembley A-3							
Lower Level (Auditorium)							
Level 1 (Aud	Level 1 (Auditorium & Gallery)						
Level 4 (Libr	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 Wei	ght 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	d-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	Rectangular and square ASTM A500, Grade B					
structural tubing Fy = 46 ksi, Fu = 58 ksi						
Round Structural Tubing	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" VU, 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
<ul><li>1.5" Type B, 18 Gage, galvinized metal deck with nestable sidelaps.</li><li>2.5" Concrete pad reincorced with #4 @ 12" oc each way over 1.5" type B metal deck.</li></ul>
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

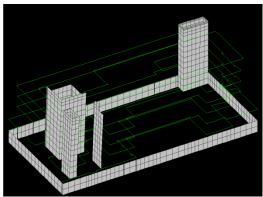
As it was briefly introduced before, Pearl Hall has an existing lateral system composed of shear walls as seen in Figure 17. This includes three shear wall cores and the lower level stem walls. The shear walls are made of 4 ksi concrete and varying thickness from 12-24 inches.

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.

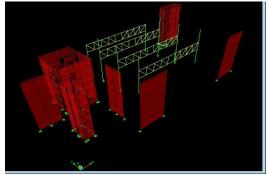
In order to evaluate lateral loads, an ETABS model was created to evaluate strength and serviceability requirements for the existing lateral force resisting system in Pearl Hall. The model was developed by the use of design drawings from Jon Anderson Architects (Appendix J).

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).

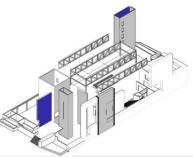
In addition, the shear walls were analyzed in ETABS to determine if serviceability requirements were met. Drift values were output from ETABS and considered very small. The values were on the magnitude of 0.001 in. This raised concern on the accuracy of the model. Therefore, it was desirable to verify 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.



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



**Figure 18.** Initial ETABS Model Showing lateral system



**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)

					Modal I	Period		ĺ				
					ETABS	0.52	S					
					RAM	0.56	S					
				% [	Difference	-(	6%					
Centers of	Mass & C	Centers of	Rigidity	Cente	rs of Mass &	Centers o	of Rigidity	,				
	ETABS				R	AM			%	Difference	from ETA	BS
XCM (ft)	YCM (ft)	XCR (ft)	YCR (ft)	XCM (ft)	YCM (ft)	XCR (f	t) YCR	(ft)	XCM (ft)	YCM (ft)	XCR (ft)	YCR (ft)
238.81	118.501	205.485	117.748	240.29	114.92	202.95	5 110	0.3	-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.

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

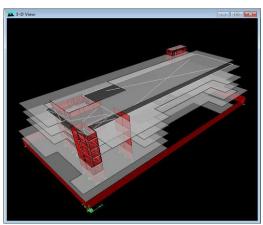
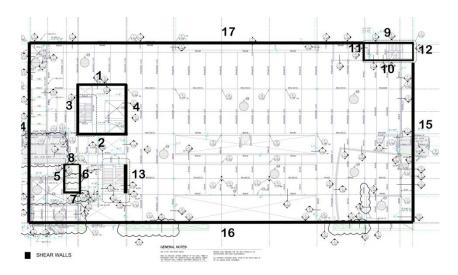


Figure 21. ETABS Model Showing lateral system and diaphragms

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 Cale	culations*	ETABS % Difference from ET					
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 Shea	r 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)	∆(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 rec	tangular					∑Ri =	513506.93		100.00%
** Using a 1k	load applied	at the top of each	h 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 Spe	ecified	100	PSF	
Assembly (auditorium, fixed seats)	Note Sp	ecified	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	С
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 + F	Parapet = 71.83
Fundamental Frequency, $n1 = 75/H = 1.044 > 1$	Structure is Rigid

Figure 29. Wind Load Design Criteria

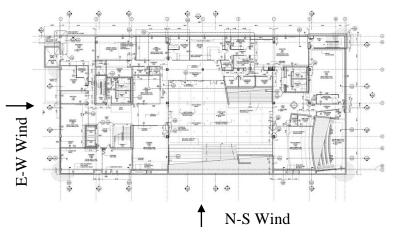


Figure 30. Wind Directions on Pearl Hall. (Courtesy of Jon Anderson Architect)

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.

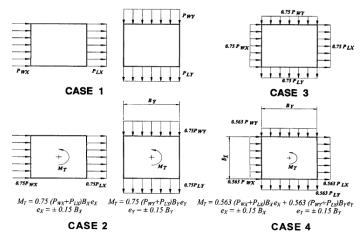


Figure 31. ASCE 7-05 Figure 6-9 Design Wind Load Cases

Floor	WIND 1X	WIND 2X	WIND 3X	WIND 4X
FIOOF	Story Force (k)	Story Force (k)	Story Force (k)	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
Eleer	WIND 1Y	WIND 2Y	WIND 3Y	WIND 4Y
Floor	WIND 1Y Story Force (k)	WIND 2Y Story Force (k)	WIND 3Y Story Force (k)	WIND 4Y Story Force (k)
Floor Stair 3			-	
	Story Force (k)	Story Force (k)	Story Force (k)	Story Force (k)
Stair 3	Story Force (k) 0.9	<b>Story Force (k)</b> 0.7	<b>Story Force (k)</b> 0.7	Story Force (k) 0.5
Stair 3 High Roof	<b>Story Force (k)</b> 0.9 22.9	<b>Story Force (k)</b> 0.7 17.1	<b>Story Force (k)</b> 0.7 17.1	<b>Story Force (k)</b> 0.5 13.4
Stair 3 High Roof Low Roof	Story Force (k)           0.9           22.9           60.4	Story Force (k)           0.7           17.1           45.3	Story Force (k)           0.7           17.1           45.3	Story Force (k)           0.5           13.4           47.4
Stair 3 High Roof Low Roof Level 4	<b>Story Force (k)</b> 0.9 22.9 60.4 75.4	<b>Story Force (k)</b> 0.7 17.1 45.3 56.6	<b>Story Force (k)</b> 0.7 17.1 45.3 56.6	<b>Story Force (k)</b> 0.5 13.4 47.4 89.8
Stair 3 High Roof Low Roof Level 4 Level 3	<b>Story Force (k)</b> 0.9 22.9 60.4 75.4 71.9	<b>Story Force (k)</b> 0.7 17.1 45.3 56.6 53.9	<b>Story Force (k)</b> 0.7 17.1 45.3 56.6 53.9	<b>Story Force (k)</b> 0.5 13.4 47.4 89.8 130.3

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

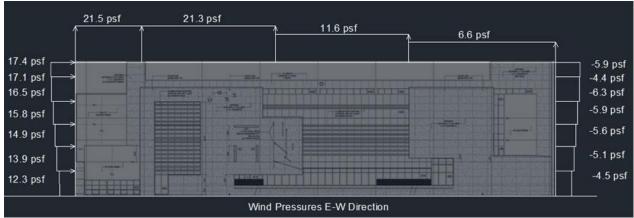


Figure 34. Total Base Shear from Windward Pressures in E-W Direction for Wind Case 1

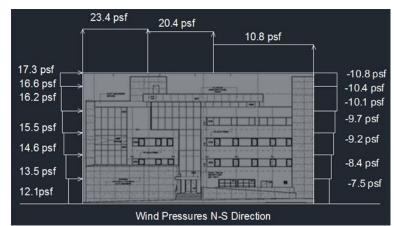


Figure 35. Total Base Shear from Windward Pressures in E-W Direction for Wind Case 1

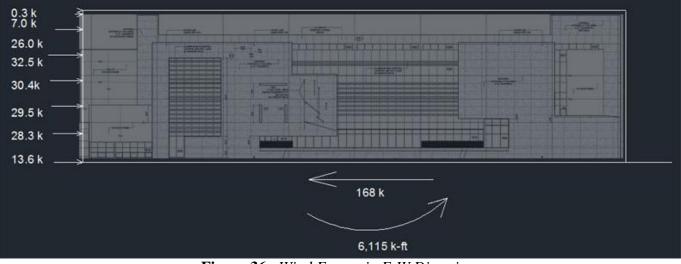


Figure 36 . Wind Forces in E-W Direction

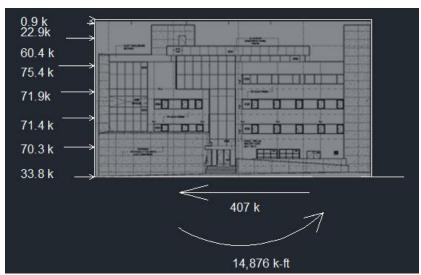


Figure 37 . Wind Forces in N-S Direction

WIND 1X	% Difference of	ETABS from H	and Calculations	[	WIND 1Y	% Difference of	ETABS from H	and Calculations
Level	Hand Calculated Story Force	ETABS Fx (k)	% Difference		Level	Hand Calculated Story Force (k)	ETABS Fx (k)	% Difference
Stair 3	(k) 0.3	0.3	-14,70%		Stair 3	0.9	0.9	5.60%
					High Roof	22.9	22.5	1.80%
High Roof	7.0	5.4	23.13%		Low Roof	60.4	60.5	-0.21%
Low Roof	26.0	21.3	18.06%					
Level 4	32.5	31.0	4.66%		Level 4	75.4	75.2	0.25%
Level 3	30.4	29.3	3.69%		Level 3	71.9	71.8	0.24%
Level 2	29.5	28.9	1.97%		Level 2	71.4	73.2	-2.43%
Level 1	28.3	28.2	0.60%		Level 1	70.3	74.2	-5.47%
< 10%, th	erefore can use	ETABS Calculat	ed Wind Forces		< 10%, t	herefore can use	ETABS Calculat	ed 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	S <sub>s</sub> =0.564
Accelerations	$S_1 = 0.170$

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

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Site Class Factors	$F_a=1.349$ $F_v=2.120$
$S_{MS} = F_a(S_s)$	0.761
$\mathbf{S}_{\mathbf{M}1} = \mathbf{F}_{\mathbf{v}}(\mathbf{S}_{1})$	0.360

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

$S_{\rm DS} = 2/3(S_{\rm MS})$	0.507
$S_{D1} = 2/3(S_{M1})$	0.240
Table 2. Modal Period for Existing Space	v
$T_a = C_t(h_n^x)$	0.493 s
-0.0019	$T_{a,X} = 0.420 \text{ s}$
$T_a = \frac{0.0019}{\sqrt{C_w}} h_n$	$T_{a,Y}^{a,H} = 0.430 \text{ s}$
	$T_{\rm X} = 0.295  {\rm s}$
T (ETABS Calculated)	$T_{Y} = 0.5243 \text{ s}$
$C_{w} = \frac{100}{A_{B}} \sum_{i=1}^{x} \left(\frac{h_{n}}{h_{i}}\right)^{2} \frac{A_{i}}{\left[1+0.83\left(\frac{h_{i}}{D_{i}}\right)^{2}\right]}$	$C_{w,X} = 0.11$ $C_{w,Y} = 0.10$
$C_{s} = \frac{\dot{S}_{DS}}{(R/I)}$ $C_{s} = \frac{S_{D1}}{T(R/I)}, T \leq T_{L}$	0.106
$S_{D1}$ T $T$	$C_{s,X} = 0.106$
$C_s = \frac{1}{T(R/I)}$ , $I \leq I_L$	$C_{s,Y} = 0.096$
$C_s \geq 0.01$	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.

<b>Table 5.</b> Seismic Design Criteria for Existing Special Reinforcea Shear Walls								
Special Reinforced C	Special Reinforced Concrete Shear Walls							
Response Modification Factor (R)	6							
Deflection Amplification Factor (C <sub>d</sub> )	5							
System Overstrength Factor ( $\Omega_0$ )	2.5							
<b>Building Height Limitation</b>	160 ft							
$\mathbf{S}_{\mathbf{M}1} = \mathbf{F}_{\mathbf{v}}(\mathbf{S}_1)$	0.360							
Diaphragm Type	Concrete filled metal deck							
Diaphragm Flexibility	Rigid							
$\mathbf{V} = \mathbf{C}_{\mathbf{s}^*} \mathbf{W}$	X: 1764 kip							
	Y: 1594 kip							

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

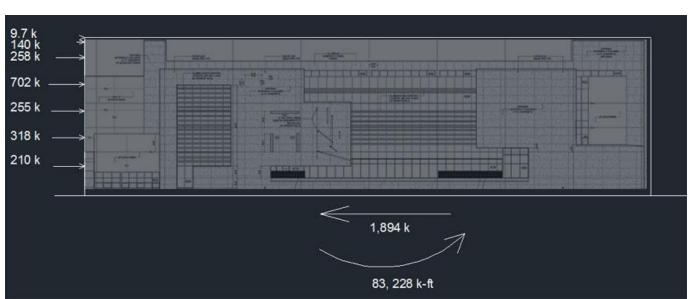


Figure 39 . Seismic Forces in E-W Direction

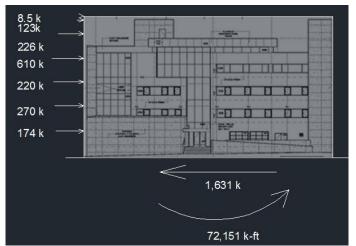


Figure 40 . Seismic Forces in N-S Direction

% Diff	erence of ETABS fro	om Hand Cal	culations	% Diff	erence of ETABS fr	om Hand Cal	culations
Level	Hand Calculated Fx (k) = V*C <sub>vx</sub>	ETABS Fx (k)	% Difference	Level	Hand Calculated Fy (k) = V*C <sub>vx</sub>	ETABS Fy (k)	% Difference
Stair 3	6.7	6.7	0.04%	Stair 3	6.1	6.1	-0.02%
High Roof	154.8	154.7	0.04%	High Roof	140.8	140.76	0.02%
Low Roof	228.6	228.5	0.04%	Low Roof	207.7	207.63	0.02%
Level 4	665.0	664.7	0.04%	Level 4	602.5	602.34	0.02%
Level 3	237.2	237.1	0.04%	Level 3	214.2	214.15	0.02%
Level 2	274.4	274.2	0.04%	Level 2	246.6	246.56	0.02%
Level 1	197.3	197.2	0.04%	Level 1	175.9	175.84	0.02%
Base Shear	1,764.0	1,763.2	0.04%	Base Shear	1593.7	1593.37	0.02%
< 10% ther	efore can use ETAB	S Calculated	Seismic Forces	< 10% ther	efore can use ETAB	S Calculated	Seismic Forces

.0%, therefore can use ETABS Calculated Seismic Forces <a>| < 10%, therefore can use ETABS Calculated Seismic Forces</a> Figure 41. Comparison of Seismic Forces in N-S and E-W Directions

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#### 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					Inherent 1	orsion in	the E-W	Direction w	ith Exisiting Sh	ear Walls	
Story	СОМ	COR	Eccentricity	Story Force (k)	Torsion(k-ft)	Story	СОМ	COR	Eccentricity	Story Force (k)	Torsion(ft-k)
Stair 3	118.06	117.75	-0.31	8.52	-3	Stair 3	240.13	205.49	-34.64	9.68	-335
High Roof	61.36	74.47	13.11	123.23	1,615	High Roof	151.41	54.05	-97.37	140.16	-13,647
Low Roof	76.78	95.40	18.62	225.73	4,202	Low Roof	121.09	69.61	-51.48	257.75	-13,270
Level 4	71.30	96.06	24.76	609.77	15,099	Level 4	134.71	72.74	-61.97	701.91	-43,499
Level 3	61.25	96.73	35.49	219.54	7,791	Level 3	130.33	75.66	-54.67	255.27	-13,956
Level 2	68.47	97.21	28.74	269.97	7,759	Level 2	129.42	79.94	-49.49	318.29	-15,751
Level 1	72.03	69.68	-2.34	174.17	-408	Level 1	120.01	97.77	-22.24	210.55	-4,682
				Total	36,055					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).

Speci	al Reinf.	Shear Wa	llAmplifi	cation Fa	ctor, Ao	in the E-W Di	rection
Story	δx	бхре	δavg	δmax	Ах	% torsion $\Delta$	<b>Torsion Irreg</b>
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
Specia	al Reinf.	Shear Wa	ll,Amplif	ication Fa	actor, Ao	in the N-S D	irection
Story	δy	буре	δavg	δmax	Ax	% torsion $\Delta$	<b>Torsion Irreg</b>
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	Accidental Torsion in the E-W Direction with Exisiting Shear Walls					Accident	al Torsion i	n the N-S D	irection wi	th Exisiting She	ar Walls
Story	Bx (ft)	%5 By (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)	Story	By(ft)	%5 By (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	12.00	0.60	2.10	9.68	12	Stair 3	32.00	1.60	3.10	8.52	42
High Roof	70.71	3.54	2.63	140.22	368	High Roof	232.08	11.60	3.11	123.23	383
Low Roof	120.00	6.00	2.20	257.86	568	Low Roof	236.34	11.82	3.12	225.73	705
Level 4	120.00	6.00	2.61	702.21	1,836	Level 4	244.67	12.23	3.12	609.77	1,902
Level 3	120.00	6.00	2.62	255.38	669	Level 3	244.67	12.23	3.12	219.54	685
Level 2	120.00	6.00	2.60	318.42	829	Level 2	256.00	12.80	3.12	269.97	843
Level 1	120.00	6.00	2.72	210.64	574	Level 1	256.00	12.80	3.02	174.17	526
				Total	4,856					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

	Horizontal Structural Irregularities								
Туре	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								
Туре	Irregularity	Comment	Status						
1a	Stiffness-Soft Story	See Appendix D.	<b>Not Good</b> (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	is 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 .
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Dr. Dichard Dohr	

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

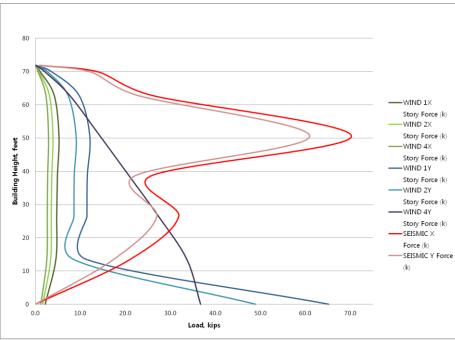


Figure 46. Factored Wind and Seismic Loads

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.

Exisit	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, Lef	t Side, 1								

Figure 47. Determination of Controlling Load Case

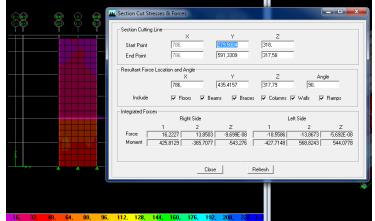


Figure 48. ETABS Output for Maximum Shear in Wall 13

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 <sub>vx</sub>	ETABS Fx (k)	% Difference	Hand Calculated ETABS V(k) 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%	0.04% 1,764.0		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 <sub>vy</sub>	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 1,593.7 1,593		0.02%	
Level 1	175.9	175.8	0.02%			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.

$$\delta x = \delta x e * C d / I$$
 (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	∆allowable(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 ET (in)	ABS δxe (in)	δx (in) = δxe*Cd/I	∆allowable(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.

Check Comment		Status
<b>Modal Period</b> 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.	ОК
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 $\rho$ 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.	ОК
Story Drift	Drift requirements are met in both orthogonal directions OK	OK

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

# **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.

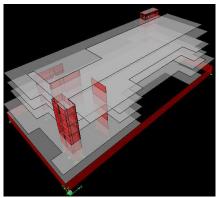
The goal of this structural depth is to design a lateral system that would impose the least cost for Pearl Hall. Yet, due to the structural irregularities of Pearl Hall, all designs must use modal analysis. Therefore, they will all be designed to meet strength and serviceability requirements for the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

# 3.3 Lateral Force Resisting Redesign System #1

# 3.3.1 ETABS Modeling

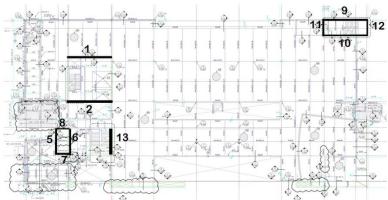
The ETABS model was used to design the modified shear wall lateral system (Figure 63). The ETABS output of shear, axial, and moment values were used during the design check and reinforcement design of the shear walls. In addition, PCAColumn was used to check the design reinforcement in the shear walls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

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.



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

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 Cal	culations*	ETA	ABS	% Difference from ETA						
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 th											

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

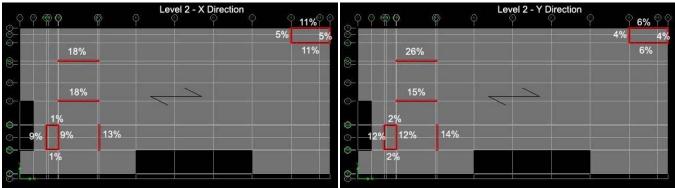


Figure 55. Relative Stiffness of Walls on Level 2 in both the X and Y Directions.

#### 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.

1 01

TT7 11

	or 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				
$\mathbf{S}_{\mathbf{M}1} = \mathbf{F}_{\mathbf{v}}(\mathbf{S}_1)$	0.360				
$\mathbf{S}_{\mathrm{DS}} = 2/3(\mathbf{S}_{\mathrm{MS}})$	0.507				
$S_{D1} = 2/3(S_{M1})$	0.240				
$\mathbf{C}_{\mathrm{t}}$	0.020				
$C_{s}$	X: 0.106				
	Y: 0.106				
Response Modification Factor (R)	6 (Special Reinforced Concrete Shear Walls)				
Deflection Amplification Factor (C <sub>d</sub> )	5				
System Overstrength Factor ( $\Omega_0$ )	2.5				
<b>Building Height Limitation</b>	160 ft				
Diaphragm Type	Concrete filled metal deck				
Diaphragm Flexibility	Rigid				
$Vx = C_{s} W$	1792 kip				
$Vy = C_s W$	1792 kip				

	. j 03				177 <b>-</b> mp				
% Diffe	erence of ETABS fro	om Hand Calc	ulations	% Difference of ETABS from Hand Calculations					
Level	Hand Calculated Fx (k) = V*C <sub>vx</sub>	ETABS Fx (k)	% Difference Level		Level Hand Calculated Fx (k) = V*C <sub>vx</sub>		% Difference		
Stair 3	6.9	6.7	2.89%	<u></u>	<u> </u>		2.000/		
High Roof	157.6	153.1	2.89%	Stair 3	6.9	6.7	2.89%		
Low Roof	228.5	221.9	2.89%	High Roof	157.6	153.1	2.89%		
Level 4	679.6	659.9	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 3	240.1	233.2	2.89%		
Level 2	278.8	270.7	2.89%	Level 2	278.8	270.7	2.89%		
Level 1	200.8	195.0	2.89%	Level 1	200.8	195.0	2.89%		
Base Shear	1,792.3	1,740.5	2.89%	Base Shear	1792.3	1740.53	2.89%		

<sup>&</sup>lt; 10%, therefore can use ETABS Calculated Seismic Forces <a></a> < 10%, therefore can use ETABS Calculated Seismic Forces</a> <a>Figure 56. Comparison of Seismic Forces in N-S and E-W Directions</a>

#### 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			
У	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 T	orsion in	the N-S	Direction w	ith Shear Wall	Design #2	Inherent T	orsion in	the E-W	Direction w	ith Exisiting Sh	near Walls
Story	сом	COR	Eccentricity	Story Force (k)	Torsion(k-ft)	Story	сом	COR	Eccentricity	Story Force (k)	Torsion(ft-k)
Stair 3	118.5	117.82	-0.68	6.92	-5	Stair 3	238.8	203.82	-35.00	6.92	-242
High Roof	60.8	81.25	20.50	157.60	3,231	High Roof	147.2	75.70	-71.49	157.60	-11,267
Low Roof	74.7	101.11	26.36	228.52	6,024	Low Roof	118.0	100.62	-17.38	228.52	-3,971
Level 4	70.8	100.51	29.67	679.56	20,162	Level 4	132.6	101.11	-31.51	679.56	-21,414
Level 3	61.2	99.11	37.88	240.15	9,098	Level 3	126.1	100.48	-25.61	240.15	-6,149
Level 2	68.0	96.88	28.91	278.79	8,059	Level 2	126.8	96.75	-30.04	278.79	-8,375
Level 1	71.3	69.71	-1.55	200.78	-310	Level 1	118.1	105.66	-12.45	200.78	-2,500
				Total	46,259					Total	-53,918

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

Accidenta	l Torsion ir	the N-S D	irection wi	th Shear Wall D	esian #2	Accidental	Torsion in	the E-W	/ Direction	with Exisiting S	hear Walls
Story	By(ft)	%5 By (ft)		Story Force (k)		Story	Bx (ft)	%5 Bx (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Stair 3	32.00	1.60	1.00	6.92	11	Stair 3	12.00	0.60	2.79	6.92	12
High Roof	232.08	11.60	1.00	157.60	158	High Roof	70.71	3.54	2.64	157.60	415
Low Roof	236.34	11.82	1.00	228.52	229	Low Roof	120.00	6.00	2.62	228.52	600
Level 4	244.67	12.23	1.00	679.56	680	Level 4	120.00	6.00	2.63	679.56	1,784
Level 3	244.67	12.23	1.00	240.15	240	Level 3	120.00	6.00	2.63	240.15	631
Level 2	256.00	12.80	1.00	278.79	279	Level 2	120.00	6.00	2.61	278.79	729
Level 1	256.00	12.80	1.00	200.78	201	Level 1	120.00	6.00	2.71	200.78	544
		· · ·		Total	1,796					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.

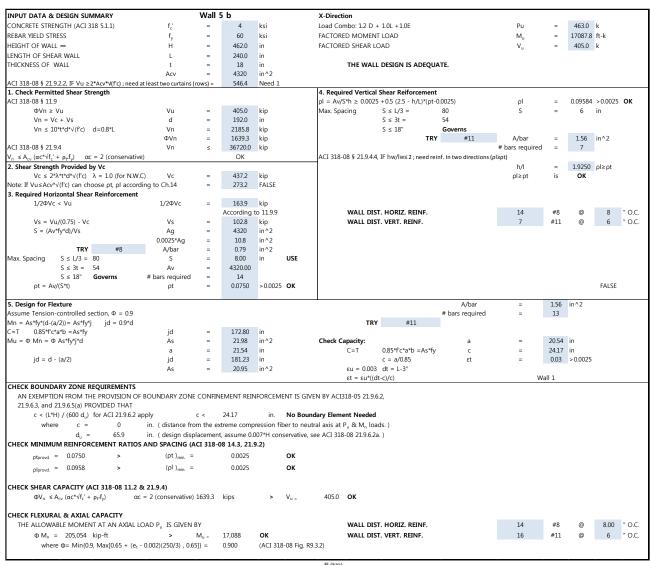
 $\delta x = \delta x e * C d / I (Amplified Drift)$ 

	Modified Spe	cial Reinforce	d Shear Wa	ll - Seismic	Drift X Directio	n
Story	Story Height (ft)	ETABS Displacement (in)	ETABS δxe (in)	δx (in) = δxe*Cd/I	∆allowable(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 Sp	ecial Reinforce	ed Shear Wa	ll - Seismic	Drift Y Direction	
Story	Story Height (ft)	ETABS Displacement ET (in)	ΓABS δxe (in)	δx (in) = δxe*Cd/I	∆allowable(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.



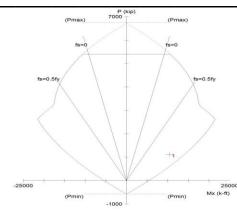


Figure 61. Design of Reinforcement for Wall 5a.

# 3.4 Lateral Force Resisting Redesign System #2

# 3.4.1 Introduction

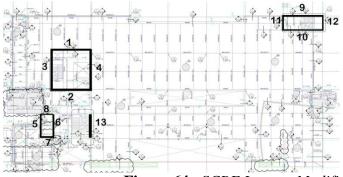
Pearl Hall is primarily a steel building. Therefore, it was desired to design the building with either braced frames or moment frames. It was decided that inverted V braces would be used to the redesign of system #2.

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.



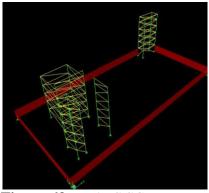


Figure 63. ETABS SCBFDesign

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

#### 3.4.3 Center of Rigidity

Special Brace Frame Design									
	CC	M	COR						
Level	Х	Υ	Х	Υ					
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.

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:

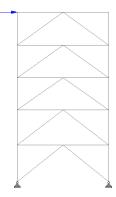


Figure 66. STAAD.Pro Frame

Torsional force = Fi = My(ki \* xi)/Ip + Mx(ki \* yi)/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.

				Stiffn	ess (k/in	)					Distribu	ution Fact	tors		
	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
	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
ç	4	-	-	22.73	28.57	37.04	55.56	111.11	-	-	0.11	0.11	0.11	0.12	0.13
ti	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
Direction	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
	1	-	-	22.08	27.77	36.68	53.56	106.50	-	-	0.13	0.13	0.14	0.14	0.14
o	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
Direction	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
Dir	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
<b>÷</b>	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 Cri	teria for Special Concentric Brace Frames
Occupancy Category	III
Importance Factor (I)	1.25
Seismic Design Category	D
Site Class	D
Site Class Factors	F <sub>a</sub> =1.349
	$F_v = 2.120$
$S_{MS} = F_a(S_s)$	0.761
$\mathbf{S}_{\mathbf{M}1} = \mathbf{F}_{\mathbf{v}}(\mathbf{S}_1)$	0.360
$\mathbf{S}_{\mathrm{DS}} = 2/3(\mathbf{S}_{\mathrm{MS}})$	0.506
$S_{D1} = 2/3(S_{M1})$	0.240
Ct	0.020
$C_s$	X: 0.069
	Y: 0.069
Response Modification Factor (R)	6 (Special Steel Concentric Brace Frames)
Deflection Amplification Factor (C <sub>d</sub> )	5
System Overstrength Factor ( $\Omega_0$ )	2
<b>Building Height Limitation</b>	160 ft
Diaphragm Type	Concrete filled metal deck
Diaphragm Flexibility	Rigid
$Vx = C_{s*}W$	853 kip
$Vy = C_{s} W$	853 kip

% Diff	erence of ETABS fro	om Hand Cal	culations	% Difference of ETABS from Hand Calculations					
Level	Hand Calculated Fx (k) = V*C <sub>vx</sub>	ETABS Fx (k)	% Difference	Level	Hand Calculated Fx (k) = V*C <sub>vx</sub>	ETABS Fy (k)	% Difference		
Stair 3	2.1	2.1	-0.71%	Stair 3	2.1	2.1	-0.71%		
High Roof	72.4	73.1	-0.88%	High Roof	72.4	73.1	-0.88%		
Low Roof	84.1	79.6	5.32%	Low Roof	84.1	79.6	5.32%		
Level 4	382.5	381.5	0.24%	Level 4	382.5	381.5	0.24%		
Level 3	107.7	108.7	-0.88%	Level 3	107.7	108.7	-0.88%		
Level 2	136.4	137.6	-0.89%	Level 2	136.4	137.6	-0.89%		
Level 1	68.1	68.9	-1.15%	Level 1	68.1	68.9	-1.15%		
Base Shear	853.2	851.4	0.21%	Base Shear	853.2	851.41	0.21%		

< 10%, therefore can use ETABS Calculated Seismic Forces < 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.

			in the N-S I entric Brace	Direction with d Frames		Inherent Torsion in the E-W Direction with Special Concentric Braced Frames					
Story	СОМ	COR	Eccentricity	Story Force (k)	Torsion(k-ft)	Story	СОМ	COR	Eccentricity	Story Force (k)	Torsion(ft-k)
Stair 3	118.06	120.67	2.61	2.08	5	Stair 3	240.13	220.48	-19.65	2.08	-41
High Roof	61.36	65.10	3.75	42.02	157	High Roof	151.42	50.74	-100.68	72.41	-7,290
Low Roof	76.78	97.81	21.03	45.02	947	Low Roof	121.13	70.14	-50.99	84.08	-4,288
Level 4	71.36	97.73	26.36	208.37	5,493	Level 4	134.69	73.98	-60.71	382.46	-23,220
Level 3	61.25	97.16	35.91	56.84	2,041	Level 3	130.38	77.95	-52.42	107.74	-5,648
Level 2	68.47	94.81	26.34	355.57	9,365	Level 2	129.44	76.96	-52.48	136.38	-7,158
Level 1	72.03	67.39	-4.65	28.39	-132	Level 1	120.03	96.75	-23.28	68.07	-1,584
				Total	17,877					Total	-49,228
							Accide	ntal Torsi	on in the E	-W Direction	
			ion in the N			with Special Concentric Braced Frames					
Story	By(ft)	•	ncentric Brad		Torsion(k-ft)	Story	Bx (ft)	%5 Bx (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Ctoir 2	32.00	1.60	3.14	2.08	10	Stair 3	12.00	0.60	2.81	2.08	4
Stair 3 High Roof	232.00	11.60		42.08	10	High Roof	70.71	3.54	2.63	72.41	190
Low Roof	236.34	11.82		45.02	141	Low Roof	120.00	6.00	2.63	84.08	221
Level 4	244.67	12.23		208.37	654	Level 4	120.00	6.00	2.63	382.46	1,007
Level 3	244.67	12.23		56.84	178	Level 3	120.00	6.00	2.64	107.74	284
Level 2	256.00	12.80	3.15	355.57	1,119	Level 2	120.00	6.00	2.62	136.38	358
Level 1	256.00	12.80	2.89	28.39	82	Level 1	120.00	6.00	2.74	68.07	186
				Total	2,316					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.

	SCBF - Seismic Drift X Direction											
Story	Story Height (ft)	ETABS Displacemen (in)	t ETABS δxe (in)	δx (in) = δxe*Cd/I	∆allowable(in) = 0.015h×	=						
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	/ Direction								
Story	Story Height (ft)	ETABS Displacement I (in)	ETABS δxe (in)	δx (in) = δxe*Cd/I	∆allowable(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						

#### $\delta x = \delta x e * Cd/I$ (Amplified Drift)

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.

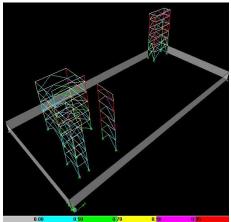
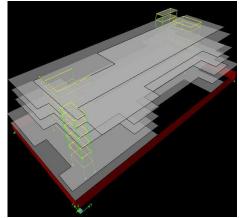


Figure 72. ETABS Model indicating the of the SCBF Design

#### 3.4.9 ETABS Modeling of Special Moment Resisting Frames

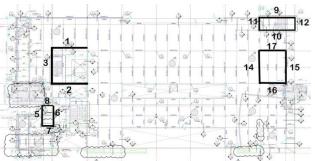
The strong column-weak beam design is required for special moment frames. For a system with weak columns, a mechanism is created when the columns of only one story reach their flexural capacities. This is because there is less dissipation of seismic energy prior to the collapse. Yet, for a system with 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

Figure 73. ETABS SMFDesign

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).



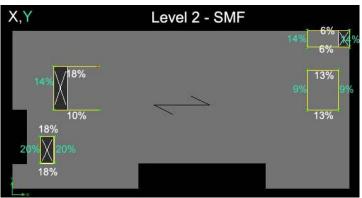
**Figure 74.** SMF Layout. Modified by N. Trujillo. (Courtesy of Jon Anderson Architect)

# 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.

Spe	Special Moment Frame Design											
	CC	M	COR									
Level	Х	Y	Х	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.

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:



**Figure 77** . *STAAD.Pro SMF Frame* 

Torsional force = Fi = My(ki \* xi)/Ip + Mx(ki \* yi)/Ip My = torsional moment in the y-direction, Mx = torsional moment in the x-direction, ki = frame stiffness<math>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.

				Stiffn	ess (k/in	)					Distributi	on Facto	rs		
	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
	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
ч	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
Direction	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
Dir	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
	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
ч	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
Direction	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
Dir	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	a 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
$S_{\rm DS} = 2/3(S_{\rm MS})$	0.507
$S_{D1} = 2/3(S_{M1})$	0.240
$C_t$	0.028
Cs	X: 0.048
012	50   Page

George Pearl Hall	, The University of	New Mexico, Albuquerque,	NM   Nicole Trujillo
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	X 0.027
	Y: 0.037
Response Modification Factor (R)	8 (Special Steel Moment Frames)
Deflection Amplification Factor (C <sub>d</sub> )	5.5
System Overstrength Factor ( $\Omega_0$ )	3
<b>Building Height Limitation</b>	Not Limited
Diaphragm Type	Concrete filled metal deck
Diaphragm Flexibility	Rigid
$Vx = C_{s} W$	584 kip
$Vy=C_{s}W$	454 kip

% Diffe	erence of ETABS fro	om Hand Cal	culations	% Difference of ETABS from Hand Calculations					
Level	Hand Calculated ETABS Fx (k) = V*C <sub>vx</sub> Fx (k)		% Difference		Hand Calculated Fy (k) = V*C <sub>vx</sub>	ETABS Fy (k)	% Difference		
Stair 3	1.5	1.5	0.89%	Stair 3	1.2	1.2	0.63%		
High Roof	51.1	50.6	0.85%	High Roof	42.0	41.67	0.82%		
Low Roof	55.4	55.0	0.72%	Low Roof	45.0	44.71	0.68%		
Level 4	262.7	261.5	0.45%	Level 4	208.4	207.49	0.42%		
Level 3	73.9	73.8	0.10%	Level 3	56.8	56.8	0.07%		
Level 2	98.6	99.0	-0.38%	Level 2	72.7	73	-0.41%		
Level 1	41.5	42.0	-1.24%	Level 1	28.4	28.75	-1.29%		
Base Shear	584.6	583.4	0.21%	Base Shear	454.5	758.47	-66.87%		

< 10%, therefore can use ETABS Calculated Seismic Forces < 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		
	х	5.0313	581.3	386.6	6.42989	495.0	494.1	ok		
	у	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 To	h Special Mom	ent Frames	Inherent To	orsion in t	he E-W D	Direction wit	th Special Mom	ent Frames			
Story	сом	COR	Eccentricity	Story Force (k)	Torsion(k-ft)	Story	СОМ	COR	Eccentricity	Story Force (k)	Torsion(ft-k)
Stair 3	121.40	113.09	-8.31	1.21	-10	Stair 3	240.07	232.21	-7.86	1.46	-11
High Roof	62.36	70.74	8.38	42.02	352	High Roof	148.91	139.18	-9.73	51.07	-497
Low Roof	77.88	71.08	-6.80	45.02	-306	Low Roof	122.39	145.51	23.13	55.37	1,280
Level 4	71.99	69.65	-2.34	208.37	-487	Level 4	134.99	152.77	17.77	262.69	4,669
Level 3	64.58	67.24	2.66	56.84	151	Level 3	133.90	157.48	23.58	73.89	1,742
Level 2	68.39	64.49	-3.89	72.70	-283	Level 2	130.46	161.18	30.72	98.57	3,028
Level 1	73.42	64.12	-9.30	28.39	-264	Level 1	119.10	96.02	-23.08	41.53	-958
				Total	-847					Total	9,253
											-
Assistantal	<b>T</b>			h Cuarial Mana		Accidental T	orsion in	the E-W	Direction w	ith Special Mor	nent Frames
Accidental Story	Torsion in By(ft)			th Special Mome Story Force (k)		Accidental To Story	orsion in Bx (ft)	the E-W %5 Bx (ft)	Direction w Ax Factor	ith Special Mor Story Force (k)	nent Frames Torsion(k-ft)
Story	By(ft)	%5 By (	ft) Ax Factor	Story Force (k)	Torsion(k-ft)			%5 Bx		•	
Story Stair 3	<b>By(ft)</b> 32.00	% <b>5 By (</b> 1.60	ft) Ax Factor 3.15	Story Force (k)	Torsion(k-ft)	Story	Bx (ft)	%5 Bx (ft)	Ax Factor	Story Force (k)	Torsion(k-ft)
Story Stair 3 High Roof	<b>By(ft)</b> 32.00 232.08	%5 By ( 1.60 11.60	ft)         Ax Factor           3.15           3.15	Story Force (k) 1.21 42.02	Torsion(k-ft)	Story Stair 3	<b>Bx (ft)</b> 12.00	%5 Bx (ft) 0.60	Ax Factor 2.80	Story Force (k)	Torsion(k-ft)
Story Stair 3	<b>By(ft)</b> 32.00	% <b>5 By (</b> 1.60	ft)         Ax Factor           3.15           3.15           3.15           3.15           3.16	Story Force (k)	Torsion(k-ft) 6 132	Story Stair 3 High Roof	<b>Bx (ft)</b> 12.00 70.71	%5 Bx (ft) 0.60 3.54	<b>Ax Factor</b> 2.80 2.63	Story Force (k) 1.46 51.07	Torsion(k-ft) 2 135
Story Stair 3 High Roof Low Roof	<b>By(ft)</b> 32.00 232.08 236.34	%5 By ( 1.60 11.60 11.82	ft)         Ax Factor           3.15         3.15           2         3.16           3         3.15	Story Force (k) 1.21 42.02 45.02	Torsion(k-ft) 6 132 142	Story Stair 3 High Roof Low Roof	<b>Bx (ft)</b> 12.00 70.71 120.00	%5 Bx (ft) 0.60 3.54 6.00	Ax Factor 2.80 2.63 2.69	<b>Story Force (k)</b> 1.46 51.07 55.37	<b>Torsion(k-ft)</b> 2 135 149
Story Stair 3 High Roof Low Roof Level 4	<b>By(ft)</b> 32.00 232.08 236.34 244.67	%5 By ( 1.60 11.60 11.82 12.23	ft)         Ax Factor           3.15         3.15           2         3.16           3         3.15           3         3.15           3         3.15           3         3.15	Story Force (k) 1.21 42.02 45.02 208.37	Torsion(k-ft) 6 132 142 656	Story Stair 3 High Roof Low Roof Level 4	<b>Bx (ft)</b> 12.00 70.71 120.00 120.00	%5 Bx (ft) 0.60 3.54 6.00 6.00	Ax Factor 2.80 2.63 2.69 2.63	<b>Story Force (k)</b> 1.46 51.07 55.37 262.69	Torsion(k-ft) 2 135 149 692
Story Stair 3 High Roof Low Roof Level 4 Level 3	<b>By(ft)</b> 32.00 232.08 236.34 244.67 244.67	%5 By ( 1.60 11.60 11.82 12.23 12.23	ft)         Ax Factor           3.15         3.15           2         3.16           3         3.15           3         3.15           3         3.15           3         3.15           3         3.15           3         3.15           3         3.15	Story Force (k)           1.21           42.02           45.02           208.37           56.84	Torsion(k-ft) 6 132 142 656 179	Story Stair 3 High Roof Low Roof Level 4 Level 3	<b>Bx (ft)</b> 12.00 70.71 120.00 120.00 120.00	%5 Bx           (ft)           0.60           3.54           6.00           6.00           6.00	Ax Factor 2.80 2.63 2.69 2.63 2.63	<b>Story Force (k)</b> 1.46 51.07 55.37 262.69 73.89	Torsion(k-ft) 2 135 149 692 195

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.  $\delta x = \delta x a \times C d / L (Amplified Drift)$ 

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

		SMF - Se	ismic Drift )	( Direction		
Story	Story Height (ft)	ETABS Displacement (in)	ETABS δxe (in)	δx (in) = δxe*Cd/I	∆allowable(in)/p = 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 - Se	eismic Drift Y	<b>Direction</b>		
Story	Story Height (ft)	ETABS Displacement ET (in)	ΓABS δxe (in)	δx (in) = δxe*Cd/I	∆allowable(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.

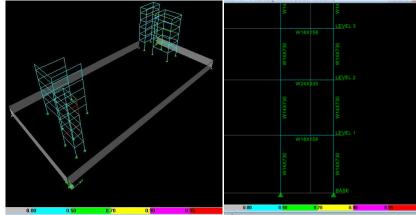


Figure 83. Comparison of the Center of Mass and Rigidity of ETABS versus Hand Calculations

# 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 Perio	d	•
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							
Total Cost	Loca	tion Factor*	Co	ost 2007**	Cost 2011		
\$ 5,874,944	\$	5,187,576	\$	5,874,944	\$ 6,535,566		
\$5,413,136	\$	4,779,799	\$	5,413,136	\$ 6,021,829		
\$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)							
	<b>Total Cost</b> \$ 5,874,944 \$ 5,413,136 \$ 1,696,717 on Factor)	Total Cost         Loca           \$ 5,874,944         \$           \$ 5,413,136         \$           \$ 1,696,717         \$           on Factor)         \$	Total Cost         Location Factor*           \$ 5,874,944         \$ 5,187,576           \$ 5,413,136         \$ 4,779,799           \$ 1,696,717         \$ 1,498,201           on Factor)         \$ 1,498,201	Total Cost         Location Factor*         Co           \$ 5,874,944         \$ 5,187,576         \$           \$ 5,413,136         \$ 4,779,799         \$           \$ 1,696,717         \$ 1,498,201         \$           on Factor)         \$         \$	Total Cost         Location Factor*         Cost 2007**           \$ 5,874,944         \$ 5,187,576         \$ 5,874,944           \$ 5,413,136         \$ 4,779,799         \$ 5,413,136           \$ 1,696,717         \$ 1,498,201         \$ 1,696,717           The cost         S 1,696,717         S 1,696,717		

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

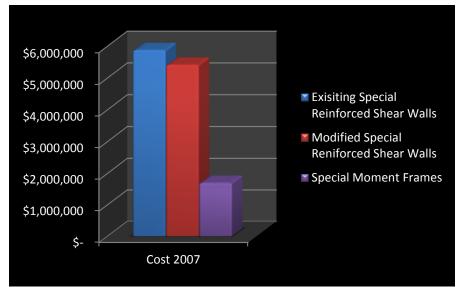


Figure 86. Seismic Design Criteria for Existing Special Reinforced Shear Walls

# 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.

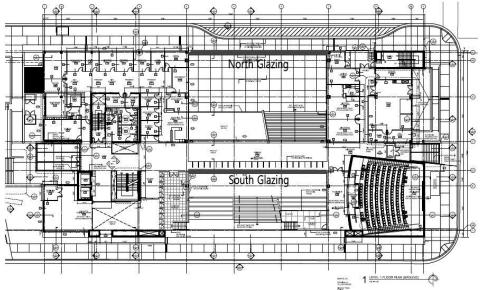
The breezeway on the lower level (See Figure 87) is rarely used. Classes use it for outside presentations occasionally, and it is used for a dance once a year. Most of the school year it is too cold to use the space, so students rarely gather there. Therefore, the breezeway can be closed in order to 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.



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

#### 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)

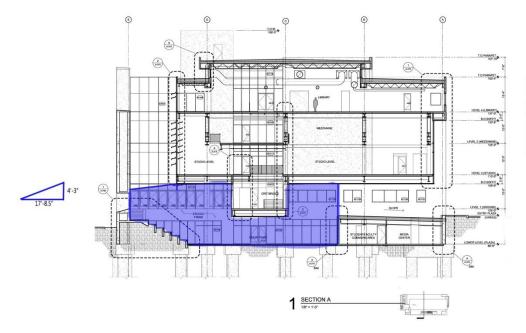
The first design aimed to extend a curtain wall to the outmost perimeter of the building in order to add as much additional usable space as possible. Yet, it was determined that the additional glazing would change the architectural look and design of the building which is undesirable.



**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).

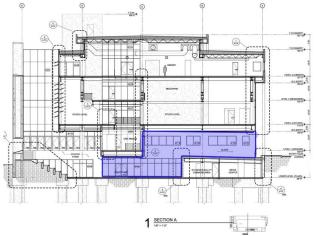


Figure 92. North Enclosure Design #2.



Figure 93. South Enclosure Design #2

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**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.

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.

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

West Entrand

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

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.



U-Value
0.25
0.18
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						
	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:					
	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 - Existi	ng Viracon 3-54 Glazing	Area	COOLING				HEATING			
Room	Туре	ft²	% OA	cfm/ft <sup>2</sup>	cfm/ton	ft²/ton	Btu/hr·ft <sup>2</sup>	% OA	cfm/ft <sup>2</sup>	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 - Vira	acon VRE 1-30 Glazing	Area	COOLING				HEATING			
Room	Туре	ft²	% OA	cfm/ft <sup>2</sup>	cfm/ton	ft²/ton	Btu/hr·ft <sup>2</sup>	% OA	cfm/ft <sup>2</sup>	Btu/hr·ft <sup>2</sup>
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					COOLIN	IG			HEATI	NG
Room	Туре	ft²	% OA	cfm/ft <sup>2</sup>	cfm/ton	ft²/ton	Btu/hr·ft <sup>2</sup>	% OA	cfm/ft <sup>2</sup>	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

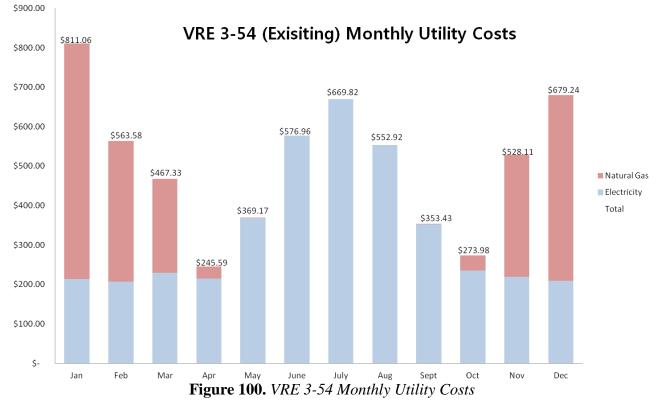
# 5.4 Cost

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.

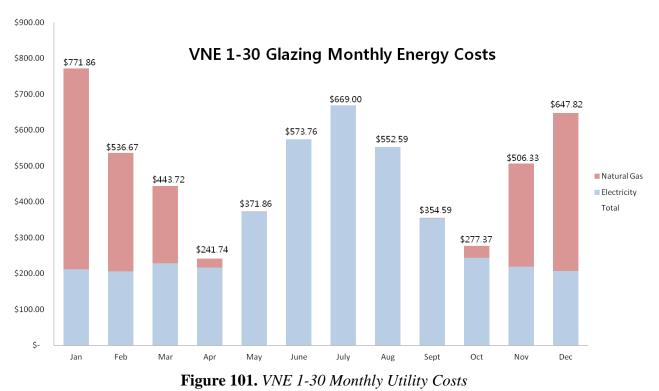
According to faculty at Pearl Hall, the proposed solution is to use large fans to blow the hot air from Level 3 to Level 1. In addition, there are plans to add vestibules to the East Entrance and the West Entrance.

Energy Consumption (BTU/ft<sup>2</sup>/year)

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



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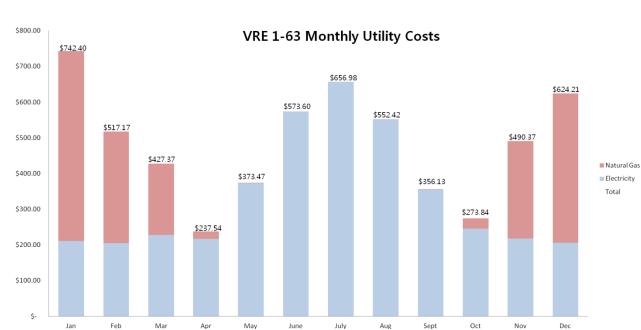


Figure 102. VRE 1-63 Monthly Utility Costs

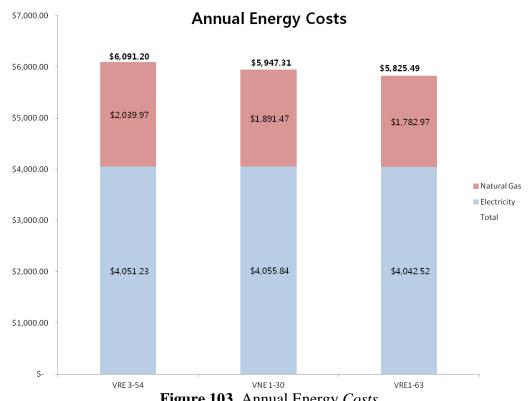


Figure 103. Annual Energy Costs

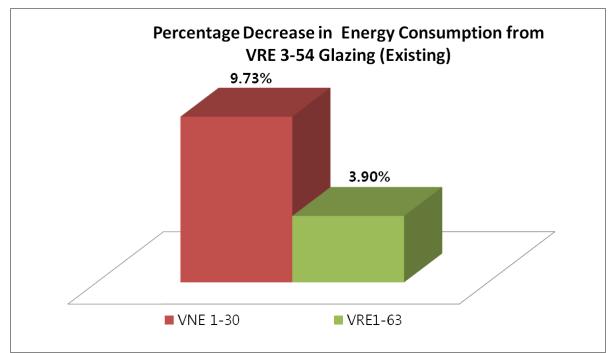


Figure 104. Percentage Decrease in Energy Consumption from Existing Glazing



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.

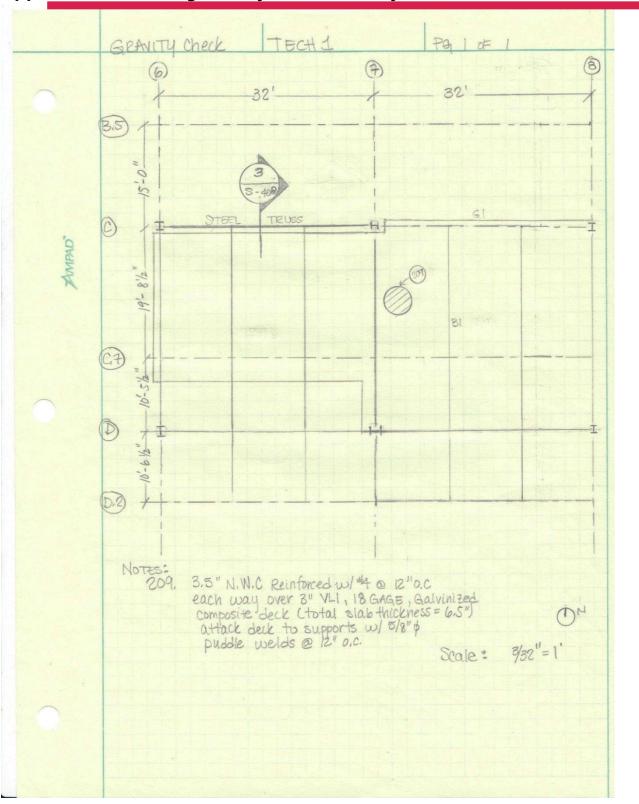
The mechanical breadth study focuses on the fact that Pearl Hall has been experiencing a performance issue in regards to occupant thermal comfort. The results of the study showed evidence using more insulating glazing, VNE 1-30 Glazing that it will provide the most energy cost savings for Pearl Hall. 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.

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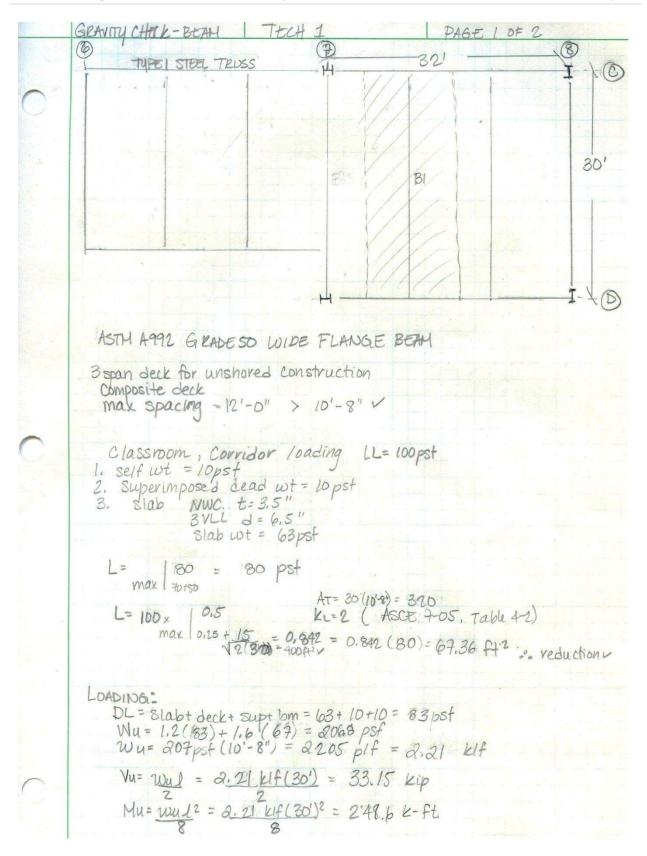
Appendix A – Existing Gravity and Lateral System Checks

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GRAVITY Check-SLAB TECH OF Ð B Floor LOAD 32 Dead: Slab+deck Q-1-41 Live load; U= 180 psf (corridors). U= 180 pst (classroom) loopst Use : SDI No. 31 Unshored 3 span 10'-6" span Composite deck 30 3 YLI -t=3.5 'Normal weight concrete Maximum clear span 6.5" total thickness D-Y "Max span = 13'-10" > 10'-6" span Vok ② Reinforced concrete allowable loads in Superimposed live load 229 psf ≥ 180psf Vok vegd = 229psf for 18 gage Slab information E.S depth -> Use 6x6-W2.1 x W2.1 Use 314 w/ 3.5" concrete reinforced with WWF 6x6 - W2:1 x W2.1

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	GRAVITY CHECK-BEAM TECHI PAGE 20F2
ń	· beams spaced @ 10'-8" O.C. W/ steel weight allowance (Op · beam simply supported with deck on flange : Lb=0
	→Using AISC Steel Manual, 13th edition . (Zx table 3-2) Using LRFD
	Mu < \$6 Mpx W18×40 \$6 Mpx = 294 kip-ft. > Mu= 248,6 k-ft
	$V_{u} \leq \phi_{v} Y_{nx}$ $W_{18x40} \phi_{v} Y_{nx} = 169 k > V_{u} = 33.157 k$
	• $\Delta L = \frac{4360}{5000}$ for $W18 \times 40$ Ix = $\frac{6}{2014}$ $WL = 0.084245f(10'-8'') = 0.8742$ $\Delta L = \frac{5}{384} \frac{14}{510} = \frac{5(0.714)(30'')^4(1728)}{384(49000)(6/2)} = 0.923''$
	$4360 = \frac{30(12)}{360} = 1" > \Delta u : ok -$ · $\Delta \pi \leq 4a40$ for W18 × 40 WTL = (0.0842+ 0.083)(10'-8") = 1.783 klf
С	$\Delta \pi = \frac{5}{384} = \frac{5(1.593 \times 30^{\circ})^{4}(1718)}{384(29000)(612)} = 1.642''$
	4240 = 30(12) = 1.5" < ATL :, DOES NOT WORK 240
	$W_{bm} = 4, 4\left(\frac{3}{10'-8''}\right)$
	DID NOT WORK: TRY W21×44, IX=843int d&Mpx = 358 kft > Mu DV Vnx = 215K > Vu
	$\Delta u = 5(0,714)(30)^{4}(1728 = 0.532" - 4360 = 1" = 0.64$
	$\Delta_{TL} = \frac{5(1.57)(30)^4(1728)}{384(17000)(612)} = 1.18'' \times 4240 = 1.5'' = 0k'$
0	There May be the discrepancy because they used such a higher classroom load than ASCE7-05. Therefore, they may have used judgement to use the W18×40 instead.

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#### George Pearl Hall , The University of New Mexico, Albuquerque, NM | Nicole Trujillo

TECH 1 PAGE 20F2 GRAVITY CHEEK -GIRDER Flange local buckling bf/2tf = 7.66  $\lambda p = 0.38 [29000/50]$  (.7 = 9.15  $\lambda p > 64/2tf$  Yes, flange compact (AISC Table 1-1) (Table 84.1) Web local buckling h/tw = 52.0 (Alsc To  $Jp = 3.76 \overline{157} \overline{14}$  (Table = 90.6 Jp > Wtw Yes, web compact (AISC Table 1-1) (Table B4:1) Shear capacity  $V_u = W_u L/2 = 36.5k$  d = 23.7in tw = 0.45in $Aw = d \cdot tw = 10, 67 in^{2}$   $\phi Vn = \phi vn (0, 6 Fyw) Aw = 320.1 K$   $\phi Vn = 1.0$ qVn>Vu ok~ Use W24x68]

# **Appendix B – Building Weight Calculations**

				<b>Building Weight</b>				
							Total Floor	
Level	Area (SF)	Beams (kip)	Columns (kip)	Floor (kip)	Superimposed (kip)	Walls (kip)	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	

St	tair	3												
Approx. Area	=	380	SF											
Ht.	=	2	ft	Total Weight	t =	37.06	k							
v	Vall	s		Supe	rim	posed			Fle	oor			Beams	
Height	=	0	ft	Partitions	=	0	psf	3VLI Deck	=	68	psf	Length (ft)	Joist	Weight (lb)
SW1, Length	=	0	ft	Misc.	=	10	psf	Weight	=	25.83864	k	11.5	10K1	57.5
SW2, Length	=	0	ft	Finishes	=	0	psf					11.5	10K1	57.5
SW3, Length	=	0	ft	Roof	=	20	psf					11.5	10K1	57.5
SW4, Length	=	0	ft	Weight	=	10.8798	k					11.5	10K1	57.5
SW5, Length	=	0	ft									11.5	10K1	57.5
SW6, Length	=	0	ft									11.5	10K1	57.5
SW7, Length	=	0	ft										Total Weight (k) =	0.345
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											

High	Ro	of			_												
Approx. Area	=	12,071	SF														
Ht.	=	7		Total Weight	=	1021.31 k											
w	alls		-			nposed		Flo	oor				Columns			Beams	
Height	=	2	ft		=		3VLI Deck	=		68	psf	Height	Shape	Weight	Length (ft)		Weigh
SW1, Length	=	0	ft		=	10 psf	Weight		820	.8418		7	HSS6X6X1/8	68.95	9.33	W10X12	111
SW2, Length	=	0	ft		=	0 psf						7	W10X33	231	9.33	W10X12	111
SW3, Length	=	0	ft		=	20 psf						7	W10X33	231	20.54	W12X14	287
SW4, Length	=	0	ft		=	145.492023 k						7	W10X33	231	7.58	W12X14	106
SW5, Length	=	0	ft									7	W10X33	231	7.58	W12X14	106
SW6, Length	=	0	ft									7	W10X33	231	7.58	W12X14	106
SW7, Length	=	0	ft									7	W10X39	273	30	W12X16	4
SW8, Length	=	0	ft									7	W10X45	315	30	W12X16	4
SW9, Length	=	32	ft									7	W12X136	952	30	W14X22	6
SW10, Length		32	ft	1								7	W12X150	280	30	W14X22	6
SW10, Length		12.33	ft	1								7	W12X40	280	30	W14X22	6
SW12, Length		12.33	ft	1								7	W12X53	371	30	W14X22	6
SW12, Length		0	ft									7	W12X79	553	23.67	W14X34	804
Unit Wt.	=	145	pcf									7	W12X73 W12X87	609	23.67	W14X34 W14X34	804
Weight	=		k									7	W12X87 W14X145	1015	23.67	W14X34 W14X43	1017
weight	-	0.20	ĸ									7	W14X145 W14X159	1013	30	W14X45 W16X26	1017
												7	W14X159 W14X159	1113	19.54	W16X26	508
												7	W14X133	1477	30	W16X26	10
												7	W14X211 W14X211	1477	30	W18X40	12
												7	W14X211 W14X43	301	32	W18X40 W18X40	12
													Total Weight (k) =		32	W18X40 W18X40	12
													rotal weight (k) =	11.35295	32	W18X40 W18X40	12
															32	W18X40 W18X40	12
															32	W18X40	1
															32	W18X40	12
															32	W18X40	12
															32	W18X40	1
															32	W18X40	12
															31.5	W21X50	15
															32	W21X50	16
															32	W21X50	16
															32	W21X50	1
															32	W21X50	16
															20.04		202.4
																24K7	1
															30	24K7	-
																24K7	3
															30 2	24K7	3
															29.5	24K7	297
															29.5	24K7	297
															29.5	24K7	297
															1080	24K7	109
				1											-	Fotal Weight (k) =	43.416

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

	Differenc	e in Dead Load fr	om Calculated to R	AM 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	С
	Internal Pressure Coefficient , GCpi	GCPI = 0.18
	Apply Directionality Factor	Kd = 0.85
	Topography Factor	Kzt = 1.00
Mean R	oof Height (ft): Top Story Height + P	arapet = 71.83
Fundamenta	al Frequency, n1 = 75/H = 1.044 >1	Structure is Rigid

	Wind For								All I	leights						Exte	mal F	ress	ure	Coe	effici	ents	(Cp	)		
. 0	e 6-6 (con				Coeffic	cients, C <sub>p</sub>		W	alls &	Roo	fs					W	all Pr	essu	re C	oeff	icien	ts (C	.p)			
Enclos	sed, Parti	ally Encl	osed Bui	0		Coefficie	unto C							Surfa	ace		L/B	(X)		L/B	(Y)		Ср (	X)	Cp	(Y)
ŀ	S	irface			L/B	Coemen	cnis, Cp C	n	Us	e With			v	Vindwa	rd W	all	All V	alues	5 /	ali v	alue	s	0.8	3	0	).8
F	Windward	l Wall		All	values		0.8	C		qz				Side \	Nall		All V	alues	5 /	all v	alue	s	-0.	7	-(	0.7
Γ					0-1		-0.5	;						Leeward	d Wa	all										
	Leeward	Wall			2		-0.3		4	$\boldsymbol{q}_h$				Stair	r 3		2.	50		0.	39		-0.2	70	-(	0.5
	Side Wall				≥4 values		-0.2							High I			3.8				26		-0.2			).5
L	Side wall			All	values		-0.7			qh				Low F			1.9				51	F	-0.3			0.5
			Roof F			ients, C <sub>p</sub> ,	for use	with q <sub>h</sub>						Leve			2.0				49		-0.2			).5 ).5
Wind					Vindwa	ird					Leeward			Leve			2.0				49 49		-0.2			).5 ).5
Direction				0	le, θ (de	<u> </u>					· · · ·	·		Leve			2.				49 47		-0.2			).5 ).5
	h/L	-0.7	-0.5	-0.3	-0.2	-0.2	35 0.0*	45	≥60#	10	15	≥ <b>20</b> -0.6														
Normal to	≤0.25	-0.18	0.0*	0.2	0.3	0.3	0.4	0.4	0.01 θ	-0.3	-0.5			Leve			2.				47		-0.2			0.5
ridge for	0.5	-0.18	8 -0.18 0.0* 0.2 0.2 0.3 0.4 0.					0.01 θ	-0.5	-0.5	-0.6	_	Base			2.130.47Roof Pressure Coefficient						-0.2	93	-(	0.5	
$\theta \ge 10^{\circ}$	≥1.0 -0.18 -0.18 -0.18 0.0* 0.2 0.2 0.3							0.01 θ	-0.7						Ro	oof P	ressu	ire C			nts (C	Cp)				
Normal			distance ard edge			Cp	*Vali purpo		ided for	interpolation			h/	L				X:	0.2	281			Y:	0.5	599	
to		0 to h				0.9, -0.18																	Cp (	(X)	Cp	(Y
ridge for θ < 10	≤ 0.5	h/2 to h to 2	h		-(	).9, -0.18 0.5, -0.18				ced linearly with area licable as follows						Roo	f - 0 t	o h/:	2				-0.9	00	-0.	97
and Parallel		> 21				0.3, -0.18		Area (sq	ft) [	Reduc	tion Fa	ctor				Roo	f - h/2	2 to	h				-0.9	00	-0.	86
to ridge	$\geq 1.0$	0 to h	/2		-1	1.3**, -0.1	8 ≤ 1(	)0 (9.3 sq	(m)		1.0					Roo	f - h	to 2ł	۱				-0.5	00	-0.	53
for all $\theta$		> h/	2		-(	0.7, -0.18		00 (23.2 s 000 (92.9			0.9					Ro	of - >	2h					-0.3	00	-0.	379
F	Impor	tance Fa	ctor, I (V	Vind Lo	ads)								╕┢╴	Terrain Ex	posure C	Constants						-				
	Table													Table 6-2												
																			_	_			_		ור	
														Exposure	α	z <sub>g</sub> (ft)	^ a	∧ b	ā	b	c	ℓ (ft)	Ē	z <sub>min</sub> (ft)*	┘║	
		C	itegory	N	on-Hur	rricane P ricane Pi	rone Re	gions			rone R - 100 m			в	7.0	1200	1/7	0.84	1/4.0	0.45	0.30	320	1/3.0	30		
			negory	a		h V = 85- and Ala	100 mph			iu v >	100 m	pu													╡║	
				-										С	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15		
			I		0	).87				0	).77			D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7	1∥	
			п		1	.00				1	.00			b	11.5	700	1/11.5	1.07	1/9.0	0.00	0.15	050	1/0.0	· · ·	」∥	
	III 1.15 1.15								$z_{min}$ = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of 0.6h or $z_{min}$ .																	
	IV 1.15 1.15										For building						-	-								

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

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

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

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

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

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

Gust Eff	fect 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			
ga	3.4	3.4			
gv	3.4	3.4			
Zmean	43.10	43.10			
Iz, mean	0.191	0.191			
Lz, 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 V	Vind (X)	N-S Wi	nd (Y)	Velocity Pre	ssure Coefficie	nts (Kz) and	Velocity
PEARL HALL	B (ft)	L (ft)	B (ft)	L (ft)	Level	Elevation (ft)	Kz	q <sub>z</sub> (psf)
Stair 3	12.33	32	32	12.33		( )		• • •
High Roof	60	232.08	232.08	60	Stair 3	71.83	1.177	23.86
Low Roof	120	236.34	236.34	120	High Roof	69.83	1.169	23.70
Level 4	120	244.67	244.67	120	Low Roof	62.83	1.141	23.13
Level 3	120	244.67	244.67	120		50.50	1.092	22.13
Level 2	120	256	256	120	Level 4	50.50	1.092	22.13
Level 1	120	256	256	120	Level 3	38.50	1.031	20.90
Base	120	256	256	120	Level 2	26.50	0.952	19.30
	6.5.10 Velocity Pressure. Velocity pressure, $q_z$ , eval				Level 1	13.50	0.850	17.23
height z shall	height z shall be calculated by the following equa				Base	0.00	0.850	17.23

height z shall be calculated by the following equation: (6-15)

 $q_z = 0.00256K_z K_{zt} K_d V^2 I (\text{lb/ft}^2)$ 

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

		W	ind Pressures E-W D	irection (X)			
Turne	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pr	essures (psf)	Net Pressu	res (psf)
Туре	FIOUI	Distances (It)	willia Plessule (psi)	(+)(Gcpi)	(-)(Gcpi)	(+)(Gcpi)	(-)(Gcpi)
	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
Windward Walls	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
	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
Leeward Walls	Level 4	50.50	-5.9	3.98	-3.98	-1.9	-9.9
Leeward wans	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

	-	W	ind Pressures N-S Dir	ection (Y)	•		
Tura	Floor	Distances (ft)	Wind Dracquire (not)	Internal P	ressures (psf)	Net Pres	ssures (psf)
Туре	FIOOI	Distances (ft)	Wind Pressure (psf)	(+)(Gcpi)	(-)(Gcpi)	(+)(Gcpi)	(-)(Gcpi)
	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
Windward Walls	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
	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
Leeward Walls	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

					Wind	ward H	Forces in	ı E-WD	Directi	on (C	Cases	s 1-I	V)				
	·						Wind Force	s E-W Directi	ion (WIN	D 1X)				·			
Floor	Bx (ft	)	Heigh	at (ft)	Elova	tion (ft)	Trib.	Below		Trib. A	bove		Story Fo	rco (k)	Ston	Shear (k)	Overturning
11001		/	5		LICVA		Height (ft)	Area (ft2)	Hei	ght (ft)	Area	(ft2)			Story	Shear (K)	Moment (k-ft)
Stair 3	12		4	2	7.	1.83	1.0	12.0		-	0.0	)	0.3	3		0.3	20.0
High Roof	70.708	33		7		9.83	3.5	247.5		1.0	70.		7.0			7.3	507.0
Low Roof	120		12.	333	6	2.83	6.2	740.0		3.5	420	.0	26.	0		33.2	2138.1
Level 4	120		1	.2	5	0.50	6.0	720.0		6.2	740	.0	32.	5		65.8	3781.7
Level 3	120		1	.2	3	8.50	6.0	720.0		6.0	720	.0	30.	4		96.2	4951.8
Level 2	120		1	.3	2	6.50	6.5	780.0		6.0	720	.0	29.	5		125.6	5732.5
Level 1	120		13	3.5	1	3.50	6.8	810.0		6.5	780	.0	28.	3		153.9	6115.0
Base	120			-	C	0.00	-	0.0		6.8	810	.0	13.	6		167.6	6115.0
														Tota	l Base	Shear (k) =	168
													Total O	/erturning	g Mom	ent (k-ft) =	6115
F							Wind For	ces E-W Directio	on (WIND 2	2X)							
Floor Bx (ft) Height (ft) Elevation (ft) Trib. Below Trib. Above Story Force (k) Story Shear (k) Moment (k, ft) Mr (+e_x) Mr (-e_x)																	
Floor $B_x$ (tt) Height (tt) Elevation (tt) Inb. Below Trib. Above Story Force (k) Story Shear (k) Moment (k-ft) $M_T$ (+e <sub>x</sub> ) $M_T$ (-e <sub>x</sub> )																	
Stair 3	12		2	7	1.83	1.0	12.0		0.0	0.	2	C	.2	15.0	)	4.5	-4.5
High Roof	70.7083		7		9.83	3.5	247.5	1.0	70.7	5.			.4	380.2		3922.0	-3922.0
Low Roof	120		2.333		2.83	6.2	740.0	3.5	420.0	19			4.9	1603.		42053.8	-42053.8
Level 4	120		12		0.50	6.0	720.0	6.2	740.0	24			9.3	2836.		52727.5	-52727.5
Level 3 Level 2	120 120		12 13		8.50 6.50	6.0 6.5	720.0 780.0	6.0 6.0	720.0 720.0	22			2.1 4.2	3713. 4299.		49235.2 47725.4	-49235.2 -47725.4
Level 1	120		.3.5		3.50	6.8	810.0	6.5	720.0	21			.5.5	4586.		45896.5	-45896.5
Base	120		-		0.00	-	0.0	6.8	810.0	10			5.7	4586.		22081.4	-22081.4
												al Base Sł		126			•
											verturnin	g Momer	nt (k-ft) =	4586	5		
	1							s E-W Directi	ion (WIN		,						<u> </u>
Floor	Bx (ft	)	Heigh	nt (ft)	Eleva	tion (ft)		Below		Trib. A		(1.0)	Story Fo	rce (k)	Story	Shear (k)	Overturning
				_	_		Height (ft)	Area (ft2)	Hei	ght (ft)	Area				-		Moment (k-ft)
Stair 3	12			2		1.83	1.0	12.0		-	0.0		0.2			0.2	15.0
High Roof	70.708	33		7		9.83	3.5	247.5		1.0	70.		5.2			5.4	380.2
Low Roof	120		12.			2.83	6.2	740.0		3.5	420		19.			24.9	1603.6
Level 4	120			.2		0.50	6.0	720.0		6.2	740		24.			49.3	2836.3
Level 3	120			.2		8.50	6.0	720.0		6.0	720		22.			72.1	3713.9
Level 2	120		1			6.50	6.5	780.0		6.0	720		22.			94.2	4299.4
Level 1	120		13	3.5 -		3.50	6.8	810.0		6.5	780		21.			115.5	4586.3
Base	120			-	L(	0.00	-	0.0		6.8	810	.0	10.			125.7 Shear (k) =	4586.3 126
													Total O			Snear (k) = ent (k-ft) =	4586
							Wind For	ces E-W Directio		(Y)	_			renturning	y iviom	ent (K-It) =	4300
I							wind For	les e-w Directio						_	. 1		
Floor	Bx (ft)	Heig	ght (ft)	Eleva	tion (ft)	Trib	Below	Trib. Ab	ove	Story Fo	orce (k)	Story S	hear (k)	Overtur		M <sub>T (-</sub> e <sub>x</sub> -e <sub>y</sub> )	M <sub>T (+</sub> e <sub>x</sub> -e <sub>y</sub> )
						Height (ft)	Area (ft2)	Height (ft)	Area (ft2)					Moment			
Stair 3	12		2		1.83	1.0	12.0	-	0.0	0.		-	.2	11.3		-81.3	-74.5
High Roof	70.7083 120		7 2.333		9.83	3.5	247.5 740.0	1.0 3.5	70.7 420.0	3. 14			.1	285.4		-106932.4	-101044.1 -253322.9
Low Roof Level 4	120		12		2.83 0.50	6.2 6.0	740.0	3.5 6.2	420.0 740.0	14			8.7 7.0	1203. 2129.		-316459.7 -420910.0	-253322.9 -341748.4
Level 3	120		12		8.50	6.0	720.0	6.0	720.0	10			4.1	2787.		-420510.0	-326645.1
Level 2	120		13	2	6.50	6.5	780.0	6.0	720.0	16	.6	70	0.7	3227.	.4	-431119.3	-359467.5
Level 1	120	1	.3.5		3.50	6.8	810.0	6.5	780.0	16			5.7	3442.		-423713.3	-354807.3
Base	120		-	0	0.00	-	0.0	6.8	810.0	7.			4.3	3442. 94		-203853.5	-170702.0
										Total O		al Base Sh g Momer	near $(k) =$	94 3443			
										TOtal U	vertunilli	y monter	it (K-It) =	5443	,		

#### Windwa roos in F. W. Direction (C. (1 W)

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

Wind Forces N-S Direction (WIND 1Y)												
				Wind Forces N	I-S Direction (W	/IND 1Y)						
Floor	By(ft)	Height (ft)	Elevation (ft)	Trib. Bel			oove	Story Force (k)	Story Shear (k)	Overturning Moment		
FIOOI	By(It)	neight (it)	Elevation (It)	Height (ft)	Area (ft2)	Height (ft)	Area (ft2)	Story Force (k)	Story Shear (k)	(k-ft)		
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		
								Fotal Overturning	Moment (k-ft) =	14876		

					١	Wind Forces N	-S Direction (WI	ND2Y)						
Floor	By(ft)	Height (ft)	Elevation (f		Trib. Bel			Above	Story Force (k)	Story Shear (k)	Overturning Moment	M <sub>T</sub> (+e <sub>v</sub> )	M <sub>T</sub> (-e <sub>v</sub> )	
		3		ł	leight (ft)	Area (ft2)	Height (ft)	Area (ft2)	,		(k-ft)	,	,	
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	
										Base Shear (k) =				
Total Overturning Moment (k-ft) = 11157 Wind Forces N-S Direction (WIND3Y)														
Trib Above Overturning Moment														
Floor	Floor Bufft) Height (tt) Flovation (tt)													
		inergine			Heig	ght (ft)	Area (ft2)	Height (ft)	Area (ft2)	Story Force	()		,	
Stair 3	32	2		71.83		1.0	32.0	-	0.0	0.7	0.7	48	3.5	
High Roof	232.08	7		59.83		3.5	812.3	1.0	32.0	17.1	17.8	124	45.9	
Low Roof	236.34	12.33	33	52.83		6.2	1457.4	3.5	812.3	45.3	63.1	409	92.0	
Level 4	244.67	12		50.50		6.0	1468.0	6.2	1457.4	56.6	119.7	694	18.9	
Level 3	244.67	12		38.50		6.0	1468.0	6.0	1468.0	53.9	173.6	903	25.7	
Level 2	256	13		26.50		6.5	1664.0	6.0	1468.0	53.6	227.2		45.2	
Level 1	256	13.5		L3.50		6.8	1728.0	6.5	1664.0	52.7	279.9		57.3	
Base	256	10.5	,	0.00		0.0	0.0	6.8	1728.0	25.4	305.3		57.3	
Dase	230			0.00		-	0.0	0.0	1726.0					
											otal Base Shear (k) =		05	
										Total Overturn	ning Moment (k-ft) =	11	157	
		1					-S Direction (WI		I				1	
Floor	By(ft)	Height (ft)	Elevation (f	H	Trib. Bel leight (ft)	ow Area (ft2)	Trib. / Height (ft)	Above Area (ft2)	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	M <sub>T (+</sub> e <sub>x</sub> ,+e <sub>y</sub> )	$M_{T(-)}e_{xy} + e_{y}$	
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	
										Base Shear (k) =	229			
								1	otal 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 pre-liminary 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. C6-49]. 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. C6-49] 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		e
$n_1 = 22.2/H^{0.8}$	(C6-14)	C
For concrete MRFs: $n_1 = 43.5/H^{0.9}$	(C6-15)	

For concrete shearwall systems:  $n_1 = 385(C_w)^{0.5}/H$ 

where

С

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

 $n_1$  = 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<sup>2</sup>)
- $A_i = \text{area of shear wall (ft<sup>2</sup>)}$
- $D_i = \text{length of shear wall "i" (ft)}$
- $b_i$  = height of shear wall "*i*" (ft)  $h_i$  = height of shear wall "*i*" (ft)

Observation from wind tunnel testing of buildings where fre-

quercy is calculated using tame team of the transfer to building. In the transfer team of team o

$n_1 = 100/H$ (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	n Criteria	ASCE 7-05	Period Determination	
Building Height (h), ft		71.830	C <sub>t</sub> 0.020 Table 1	12.8-2
Occupancy Category	III	Table 1-1	x 0.750 <i>Table</i> .	12.8-2
Ss	0.564 g	§11.4.1, Fig. 22-1	TL 6.000 sec Fig. 22	-15
S1	0.170 g	\$11.4.1, Fig.22-2	Cu 1.46 <i>Table</i> .	12.8-1
Importance Factor	1.250	Table 11.5-1	$T_a = C_t h_n^x$ 0.493 sec <i>EQ. 12.</i>	8-7
Soil Site Class	D	<i>§11.4.2</i>	$C_{w} = \frac{100}{A_{R}} \sum_{i=1}^{N} \left(\frac{h_{n}}{h_{i}}\right)^{2} \frac{A_{i}}{\left[1 + 0.83 \left(\frac{h_{i}}{h_{i}}\right)^{2}\right]}$	
Seismic Design Category	D	Table 11.6-1	$A_{R} = \frac{1}{1+0.83} \left( \frac{h_{i}}{D} \right)^{2}$	0.10
Fa	1.349	Table 11.4-1	$\begin{bmatrix} 1 + 0.03 \\ \overline{D}_{1} \end{bmatrix} = EQ. 12.$	.8-10
Fv	2.120	Table 11.4-2	Y 0.10	
Sms	0.761 g	EQ. 11.4-1		
Sm1	0.360 g	EQ. 11.4-2	$T_{xx} = \frac{0.0019}{\sqrt{L_{xx}}} h_{xx} \qquad X \qquad 0.420 \qquad \text{sec} \qquad EQ. \ 12$	.8-9
Sds	0.507 g	EQ. 11.4-3	Tx 0.295 sec <i>ETABS</i>	
Sd1	0.240 g	EQ. 11.4-4	Ty 0.5243 sec ETABS	

Calculation of Seismic Response Coefficient													
Special Reinfor	Special Reinforced Concrete Shear Walls												
R (Special reinforced													
concrete shear walls)		6.000		Table 122-1									
$C_s = \frac{S_{DS}}{(R/I)}$		0.106		EQ. 12.8-2									
$C_{q} = \frac{S_{pq}}{T(R/I)} \cdot T \leq T_{p}$	X Y	0.119 0.116		EQ. 12.8-3									
$C_s \ge 0.01$		ok		EQ. 12.8-5									
¢	Х	0.106		ETABS									
	Y	0.096		ETABS									
k	Х	1.00		<i>§12.8.3</i>									
k	Y	1.01		<i>§12.8.3</i>									
Base shear, V = C <sub>s*</sub> W		1764.0	kip	EQ. 12.8-1									
Base shear, $V = C_{s^*}W$		1593.7	kip	EQ. 12.8-1									
-		lation of Co.											

	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	х	62.83	34.00	34.00	11.59					
2	12	34.00	90	69.83	Х	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	х	68.83	10.33	10.33	0.30					
8	12	10.33	90	69.83	Х	69.83	10.33	10.33	0.28					
9	12	32.00	90	71.83	х	71.83	32.00	32.00	6.18					
10	12	32.00	90	71.83	Х	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					
<b>AB</b> =	30720	160 ~	$d_{m}$ <sup>2</sup> $A_{c}$					Shear	Walls 1-13					
hn=	71.83	~~ <u>~</u>	T) r	- C				CW - X =	0.106					
		~ 1=1	1+0.03	( <b>5</b> , <b>)</b>				CW -Y =	0.101					

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	Exisitin	g Special Reinfo	orced Shear Wa	alls - Seismi	c Forces (E-W Dir	ction, X)	·			
Level	Story Weight, wx (k)	Story Height, hx (ft)	w <sub>x</sub> h <sub>x</sub> <sup>k</sup>	C <sub>vx</sub>	Fx (k) = V*C <sub>vx</sub>	Stony Shoar	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	0.10	197.3	1764	78304			
Base	4055.5	0.00	0.0	0.00	0.0	1764	78304			
Dase	-									
		∑w <sub>x</sub> h <sub>x</sub> <sup>k</sup>	585894.3	1.0	Total Building		16,688			
	= 1.000					Base Shear,k =				
T =	= 0.295			•	Total	Moment,k-ft =	78,304			
			orced Shear W	alls - Seismi	c Forces (N-S Dir	ction, Y)				
Level	Story Weight,	Story Height,	w <sub>x</sub> h <sub>x</sub> <sup>k</sup>	C <sub>vx</sub>	$Fx(k) = V^*C_{vx}$	Story Shear	Overturning			
LUVUI	wx (k)	hx (ft)	vv <sub>X</sub> rr <sub>X</sub>	C <sub>VX</sub>	$(K) = V C_{VX}$	$(k) = Vx = \Sigma fi$	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			
		∑w <sub>x</sub> h <sub>x</sub> <sup>k</sup>	612813.1	1.0	Total Building	g Weight, k =	16,688			
k	= 1.012					Base Shear,k =	1,594			
T =	= 0.524				Total	Moment,k-ft =	70,900			
	% Difference of E	TABS from Hand C	alculations	% Diffe	rence of ETABS from	Hand Calculation	s			
	-evel Hand Cale	culated ETABS	% Difference	Laural	Hand Calculated	ETABS % Dif	ference			
	Fx (k) =	V*C <sub>vx</sub> Fx (k)	% Difference	Level	Fy (k) = $V^*C_{vx}$	Fy (k) % Dif	rerence			
	tair 3 6.7		0.04%	Stair 3	6.1		02%			
-	h Roof 154.		0.04%	High Roof	140.8		)2%			
	w Roof 228. evel 4 665.		0.04% 0.04%	Low Roof Level 4	207.7 602.5		)2% )2%			
	evel 3 237.		0.04%	Level 4 Level 3	214.2		)2% )2%			
	evel 2 274.		0.04%	Level 2	246.6		)2%			
	evel 1 197.		0.04%	Level 1	175.9		)2%			
Bas	e Shear 1,764	.0 1,763.2	0.04%	Base Shear	1593.7	1593.37 0.02%				
< 1	0%, therefore can u	se ETABS Calculate	d Seismic Forces	< 10%, therefore can use ETABS Calculated Seismic Forces						

< 10%, therefore can use ETABS Calculated Seismic Forces < 10%, therefore can use ETABS Calculated Seismic Forces

	L'qui valch	. 11	uttal.		munou	(DP		•					
							Calculation of		•				
							ecial Reinforced	Conc	rete Shear	r Wa	Ills (Design	#2)	
Calandia I				100			pecial reinforced						
	oad Design C	rite			E 7-05	conc	rete shear walls)		6.000		Table 12.2	1	
Building Heig				1.830	1 1	Tx			0.211	S	ETABS		
Occupancy C	ategory			Table		Ту	7		0.4105	S	ETABS		
Ss		0.56	5		l, Fig. 22-1	<u>C</u> , ≁	Sps		0.106		EQ. 128-2		
S1		0.17	-		l, Fig.22-2	_	(R/I)	V	0.237		EQ. 12.8-2 EQ. 12.8-3		
Importance		T	.250	Table		C <sub>s</sub> ==	$\frac{S_{D1}}{T(R/I)}, T \leq T_L$	X Y	0.237		LQ. 12.0-5		
Soil Site C			D	<i>§11.4.2</i>		63	: 0.01		ok		EQ. 128-5		
Seismic Design	Category		D	Table				Х	0.106		ETABS		
Fa			.349	Table		C <sub>s</sub> ==		Ŷ	0.100		ETABS		
Fv			.120	Table		k		x	1.00		§12.8.3		
Sms		0.76	5	EQ. 11		ћ k		Ŷ	0.96		§12.8.3		
Sm1		0.36		EQ. 11			shear, $V = C_{s^*}$		1792.3	kin	EQ. 12.8-1		
SDS		0.50	5	EQ. 11									
Sd1		0.24		EQ. 11			shear, $V = C_{s^*}$		1792.3		EQ. 12.8-1		
	1		l Specia	Reint	orced She	ar W	alls - Seismic Fo	orces	(E-W Dir	ctio	n, X)	1	
Level	Story Weig	ht,	Story H	•	w <sub>x</sub> h <sub>x</sub> <sup>k</sup>	:	C <sub>vx</sub>	Fx (k	$) = V C_{vx}$		tory Shear		erturning
Lever	wx (k)		hx (	ft)	VVXIIX		C <sub>vx</sub>		$) = \mathbf{v} \mathbf{c}_{VX}$	(k	) = Vx=Σfi	Mom	nent (k-ft)
Stair 3	32.0		71.8	33	2298.	2	0.00		6.9		7		497
High Roof	749.6		69.8	33	52342	.2	0.09		157.6		165	-	11502
Low Roof	5			75895		0.13	228.5			393		25860	
Level 4	4469.1		50.		225692		0.38		679.6		1073		50178
Level 3	2071.6		38.		79756		0.13		240.1		1313		59423
Level 2	3494.0		26.		92591		0.15		278.8		1515		76811
Level 1	4939.4		13.				0.10		200.8		1792		79522
	4959.4					.0			0.0				
Base	-		0.0				0.00				1792		79522
				∑w <sub>x</sub> h <sub>x</sub> k	595257	7.5	1.0	Tota	al Building	-	5		L6,964
	1.000										e Shear,k =		1,792
T =	0.211								Total	Мо	ment,k-ft =	7	79,522
	Mod	ified	d Specia	l Reinf	orced She	ear W	alls - Seismic F	orces	(N-S Dir	ctio	n, Y)		
	Story Weig	ht,	Story H	eight,						St	tory Shear	Ove	erturning
Level	wx (k)	ŕ	hx (	-	w <sub>x</sub> h <sub>x</sub> <sup>k</sup>		C <sub>vx</sub>	Fx (k	$) = V C_{vx}$		) = Vx=Σfi		-
Stair 3	32.0		71.8		2298.	า	0.00		6.9	(11	7		497
	749.6		69.8				0.00		0.9 157.6		165	-	497 11502
High Roof					52342								
Low Roof	1207.9		62.8		75895		0.13		228.5		393		25860
Level 4	4469.1		50.		225692		0.38		679.6		1073		50178
Level 3	2071.6		38.		79756		0.13		240.1		1313		59423
Level 2	3494.0		26.		92591		0.16		278.8		1592		76811
Level 1	4939.4		13.		66682	.0	0.11		200.8		1792		79522
Base	-		0.0		0.0		0.00		0.0		1792		79522
				∑w <sub>x</sub> h <sub>x</sub> <sup>k</sup>	595257	7.5	1.0	Tota	al Building	g We	eight, k =	1	L6,964
k =	1.000								-	-	e Shear,k =		, 1,792
	0.411										ment,k-ft =		79,522
-											-,		
4/4/2012									8	32	Page		

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

Dr. Richard Behr

Structural Option

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

									•	nse Coefficient
Sei	smic Load	d Design (	Criteria		ASCE 7-0	5	$T_a = C_t h_n^x$	ecial Concer	ntric Brac 0.493	e Frames sec <i>EQ. 12.8-7</i>
Buildin	ng Height	(h), ft		71	830		Ta = Cenn Tx		0.493	s ETABS
Occup	bancy Cate	egory	III		Table 1-1		Ту		0.720	s ETABS
	Ss		0.564	g	§11.4.1, Fig	22-1	C <sub>t</sub>		0.020	
	S1		0.170		§11.4.1, Fig.2	2-2	X		0.750	Table 12.8-2
Impo	ortance Fa	ctor	1.250	-	<i>Table 11.5-1</i>		R (Special Steel Concentric Brace Frames)		6.000	Table 122-1
	il Site Clas		D		<i>§11.4.2</i>		C = SDS	ee maneey		
	Design Ca		D		Table 11.6-1		"" (R/I) Sm	rran X	0.106 0.069	
o o ionni e	Fa	ategery	1.349		Table 11.4-1		C. T(R/I)'	7 <u>≪</u> γ <sub>2</sub> γ	0.069	
	Fv		2.120		Table 11.4-2		$C_s \ge 0.01$		ok	EQ. 12.8-5
	SMS		0.761		EQ. 11.4-1		A wax	X Y	0.069	
	SM1		0.360	5	-		k	X	1.11	§12.8.3
					EQ. 11.4-2		k	Y	1.11	<i>§12.8.3</i>
	SDS		0.507	5	EQ. 11.4-3		Base shear, V		853.2	kip <i>EQ. 128-1</i>
	Sd1		0.240	5	EQ. 11.4-4		Base shear, V		853.2	kip <i>EQ. 128-1</i>
		r	SCBF	- Sei	smic Forces	(E-W	/ Dirction, X	()		
	Story	Story						Story Sh	ear (k)	Overturning
Level	Weight,	Height, h	x w <sub>x</sub>	h <sub>x</sub> <sup>ĸ</sup>	C <sub>vx</sub>	Fx (	$(k) = V C_{vx}$	= Vx		Moment (k-ft)
	wx (k)	(ft)						- • • •	-211	Moment (k H)
Stair 3	14.2	71.83	163	3.7	0.00		2.1	2		149
High Roof	511	69.83	570	04.1	0.08		72.4	74	ŀ	5206
Low Roof	668	62.83	661	92.5	0.10		84.1	15	9	10489
Level 4	3,870	50.50	3010	)78.9	0.45		382.5	54	1	29803
Level 3	1,473	38.50	848	12.1	0.13		107.7	64	9	33951
Level 2	2,823	26.50	1073	362.4	0.16		136.4	78	5	37565
Level 1	2,979	13.50	535	84.2	0.08		68.1	85	3	38484
Base	-	0.00	0	.0	0.00		0.0	85.	3	38484
		Σw <sub>x</sub> h	, <sup>k</sup> 6716	67.9	1.0	Тс	otal Building	g Weight,	k =	12,339
k =	1.110						-	Base Sh	ear,k =	853
Τ=	0.720						Tota	l Momen	t,k-ft =	38,484
		•	SCB	F - Sei	ismic Forces	(N-S		-		
	Story	Story	T			<b>(</b>				
Level	Weight,	Height, h	x w <sub>x</sub>	h <sup>k</sup>	C <sub>vx</sub>	Fx (	$(k) = V C_{vx}$	Story Sh		Overturning
Level	wx (k)	(ft)		'x	CVX	17.		= Vx:	=Σfi	Moment (k-ft)
Stair 3	14.2	71.83	163	33.7	0.00		2.1	2		149
High Roof	511	69.83		04.1	0.08		72.4	74		5206
Low Roof	668	62.83		92.5	0.10		84.1	15		10489
Level 4	3,870	50.50	3010		0.45		382.5	54		29803
Level 4 Level 3	3,870 1,473	38.50	848		0.43		107.7	64		33951
Level 3	2,823	26.50		362.4	0.13		136.4	78		37565
Level 2 Level 1	2,823	13.50		84.2	0.10		68.1	85		38484
Base	2,313	0.00		04.2 .0	0.08		0.0	85.		38484
Dase	-					-				
		Σw <sub>x</sub> h	<sup>k</sup> 6716	067.9	1.0	ľ	otal Building			12,339
	1.110						_	Base Sh		
=	0.720						Iota	l Momen	t,k-ft =	38,484

1-03	nguiva	ient La	icial ro		ion (shec	ial Steel Mo					
										Coefficient	
							pecial Steel				
						$T_a = C_t h_n^x$		0.855		EQ. 12.8-7	
	- • • •					Tx		0.789		ETABS	
	Seismic Lo	_	n Criteria		CE 7-05	Ту		1.014 0.0		ETABS Table 128	
Buil	ding Heigł	nt (h), ft		71.830		C <sub>t</sub>		0.0		Table 12.8	
Oc	cupancy Ca	ategory	III	Table	• <i>1-1</i>	× R (Special Steel	Moment	0.0	00	TADIE 12.0	
	Ss		0.564	g <i>§11.4.</i>	1, Fig. 22-1	Frames)	MOMENT	8.000		Table 122	
	S1		0.170	g <i>§11.4.</i>	1, Fig.22-2	$C_s = \frac{S_{DS}}{(D_s/I)}$				10010 12.2	
In	nportance	Factor	1.250	) Table	11.5-1	$C_s = (R/I)$		0.0		EQ. 12.8-2	
	Soil Site C	lass	D	<i>§11.4.</i>	2		r <u>≾n</u> X	0.0		EQ. 128-3	
Seism	nic Design	Category	D	Table	11.6-1		- ~ Y	0.0		•	
	Fa	5 7	1.349		11.4-1	$C_s \ge 0.01$			k	EQ. 12.8-5	
	Fv		2.120		11.4-2	£1., and	X	0.0		ETABS	
	Sms		0.761	g <i>EQ. 1</i>		k	Y X	0.0 1.:		<i>ETABS</i> §12.8.3	
	Sm3		0.360	g EQ. 1		к k	X Y		.26	\$12.8.3 \$12.8.3	
	SDS		0.507	g <i>EQ. 1</i> g <i>EQ. 1</i>		ĸ Base shear, Vx	-	⊥ 584.6		912.8.5 EQ. 12.8-1	
	SD1		0.240			Base shear, Vy					
	301		0.240					454.5	kip	EQ. 12.8-1	
				SMF - Seis	mic Forces	(E-W Dirction, X	)				
		Story	Story	1.			Story She	ar (k)	Over	rturning	
	Level	Weight,	Height, hx	w <sub>x</sub> h <sub>x</sub> <sup>k</sup>	C <sub>vx</sub>	Fx (k) = $V^*C_{vx}$	= Vx=			ent (k-ft)	
		wx (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			0416	
	Level 3	1479	38.50	96415.2	0.13	73.9	444			3260	
	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			6433	
	Base	-	0.00	0.0	0.00	0.0	585			6433	
			∑w <sub>x</sub> h <sub>x</sub> <sup>k</sup>	762827.4	1.0	Total Building Weight, k				2,280	
		1.144				<b>T</b> .	Base She				
	=	0.789			· _		l Moment,	k-ft =	2	6,433	
		0		SMF - Sei	smic Forces	(N-S Dirction, Y)					
	Let 1	Story	Story		C		Story She	ar (k)	Over	rturning	
	Level	-	Height, hx	w <sub>x</sub> h <sub>x</sub> <sup>k</sup>	C <sub>vx</sub>	Fx (k) = V*C <sub>vx</sub>	= Vx=			ent (k-ft)	
	<u> </u>	wx (k)	(ft)	2001.0	0.00	10					
	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	88 707			5849 6372	
	Level 4 Level 3	3854 1470	50.50	533534.9 145541.0	0.46	208.4	297			6372 8560	
	Level 3 Level 2	1479 3025	38.50 26.50	145541.0 186159.0	0.13 0.16	56.8 72.7	353 426			8560 0487	
	Level 2 Level 1	2757	26.50 13.50	72679.9	0.16	28.4	426			0487 0870	
	Base	2131	0.00	0.0	0.00	0.0	455				
	Duse		<u>Σ</u> w <sub>x</sub> h <sub>x</sub> <sup>k</sup>					_	20870		
	L =	1 257	Σw <sup>x</sup> u <sup>x</sup>	1163849.5	1.0	Total Building	-			2,280 455	
		1.257 1.014				Tata	Base She I Moment,			455 0,870	
	1 =	1.014				1012	ii ivioment,	κ-π =	2	0,070	

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

ASCE 7-05 C	SCE 7-05 Chapter 12: Horizontal Irregularities Special Reinf. Shear WallAmplification Factor, Ao in the E-W Direction												
	Sp	pecial Rein	f. Shear	WallAr	nplificat	ion Facto							
Story		δx 8	бхре	δavg	δmax	Ах	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 ROO	F		0.22	0.23	0.45	2.7	2.7	2.0	Irregular, 1a				
LOW ROO	F	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				
	Sp	ecial Rein	f. Shear	Wall,A	mplifica	tion Facto	or, Ao in the N	N-S Direction					
Story	δy	буре	δavg	ο δm	ax	Ах	Ax (used)	% torsion Δ	Torsion Irreg.				
Stair 3	0.63	0.69	0.63	1.3	1	3.1	3.1	2.1	Irregular, 1a				
HGH ROOF	0.72	0.80	0.72	1.5	2	3.1	3.1	2.1	Irregular, 1a				
LOW ROOF	0.13	0.16	0.13	0.2	9	3.2	3.2	2.2	Irregular, 1a				
STORY4	0.39	0.43	0.39	0.8	2	3.1	3.1	2.1	Irregular, 1a				
STORY3	0.25	0.28	0.25	0.5	3	3.1	3.1	2.1	Irregular, 1a				
STORY2	0.04	0.05	0.04	0.0	9	3.3	3.3	2.2	Irregular, 1a				
STORY1	0.02	0.02	0.02	0.0	3	3.0	3.0	2.1	Irregular, 1a				
I	Modifie	ed Special	Reinf. S	hear Wa	all, Amp	lification	Factor, Ao in	the E-W Directi	ion				
Story		δx 8	бхре	δavg	δmax	Ах	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 ROO	F	0.26	0.24	0.26	0.50	2.6	2.6	1.9	Irregular, 1a				
LOW ROO	F	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				
N	<b>/lodifie</b>	d Special	Reinf. S	hear Wa	all, Amp	lification	Factor, Ao in	the N-S Direct	ion				
Story	δy	буре	δavg	ο δm	ах	Ах	Ax (used)	% torsion Δ	Torsion Irreg.				
Stair 3	0.56	0.61	0.58	0.6	1	0.8	1.0	1.0	Good				
HGH ROOF	0.63	0.69	0.66	0.6	9	0.8	1.0	1.0	Good				
LOW ROOF	0.47	0.52	0.49	0.5	2	0.8	1.0	1.1	Good				
STORY4	0.34	0.37	0.36	0.3	7	0.8	1.0	1.0	Good				
STORY3	0.21	0.24	0.22	0.2	4	0.8	1.0	1.0	Good				
STORY2	0.10	0.11	0.11	0.1	.1	0.8	1.0	1.0	Good				
STORY1	0.01	0.01	0.01	0.0	1	0.7	1.0	1.0	Good				

## ASCE 7.05 Chapter 12: Horizontal Irregularities

	SCBF, /	Amplifica	tion Facto	or, Ao in	the E-W	Direction	
Story	δx	бхре	δavg	δmax	Ax	% torsion $\Delta$	<b>Torsion Irreg.</b>
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

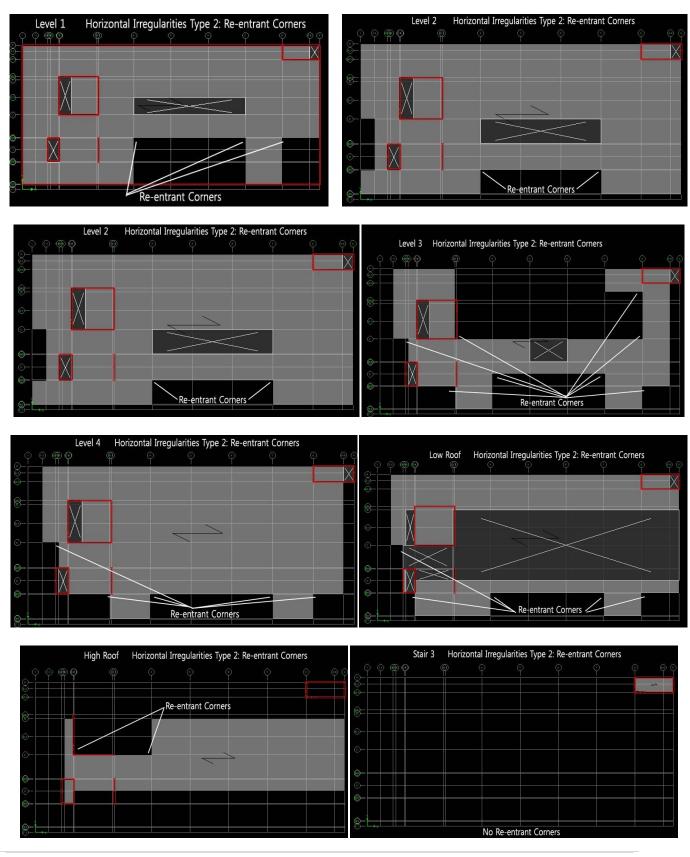
SCBF, Amplification I	Factor, Ao in	the N-S Direction
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Story	δy	буре	δavg	δmax	Ах	% torsion $\Delta$	Torsion Irreg.
Stair 3	0.48	0.54	0.48	1.03	3.1	2.1	Irregular, 1a
HGH ROOF	0.82	0.92	0.82	1.74	3.1	2.1	Irregular, 1a
LOW ROOF	0.43	0.48	0.43	0.91	3.1	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, A	mplificat	ion Facto	or, Ao in t	the E-W	Direction	
	Story	δx	бхре	δavg	δmax	Ax	% torsion $\Delta$	<b>Torsion Irreg.</b>
I	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,	Amplific	ation Facto	or, Ao in	the N-S	Direction

		· · · · · · · · · · · · · · · · · · ·		,			
Story	δy	буре	δavg	δmax	Ax	% torsion $\Delta$	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



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		AL STRUCTURAI phragm Disconti		
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	-	hear Wall - V		0		Speci	al Reinforced	Shear Wall -	Vertical Irre	gularity 1a Y I	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	Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift	Soft Story Status
Stair 3	1.9E-03	1.3E-03	1.5E-03		ok	Stair 3	6.1E-03	4.3E-03	4.9E-03		ok
High Roof	3.8E-04	2.7E-04	3.1E-04		ok	High Roof	1.8E-03	1.3E-03	1.5E-03		ok
Low Roof	2.1E-04	1.5E-04	1.7E-04		ok	Low Roof	3.7E-04	2.6E-04	2.9E-04		ok
Level 4	2.8E-04	2.0E-04	2.3E-04	6.6E-04	not ok	Level 4	5.9E-04	4.1E-04	4.7E-04	2.2E-03	not ok
Level 3	3.3E-04	2.3E-04	2.6E-04	2.3E-04	ok	Level 3	5.1E-04	3.6E-04	4.1E-04	7.5E-04	not ok
Level 2	3.0E-04	2.1E-04	2.4E-04	2.2E-04	ok	Level 2	4.2E-04	3.0E-04	3.4E-04	3.9E-04	ok
Level 1	5.0E-05	3.5E-05	4.0E-05	2.4E-04	not ok	Level 1	8.1E-05	5.7E-05	6.5E-05	4.1E-04	not ok
Modified Sp	ecial Reinford	ed Shear Wa	ll - Vertical I	rregularity	1a X Direction	Modified	Special Reinf	orced Shear V	Vall - Vertica	Irregularity 1	A 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	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	Stair 3	1.8E-03	1.2E-03	1.4E-03		ok
High Roof	5.5E-04	3.9E-04	4.4E-04		ok	High Roof	1.5E-03	1.0E-03	1.2E-03		ok
Low Roof	2.9E-04	2.0E-04	2.3E-04		ok	Low Roof	6.4E-04	4.5E-04	5.1E-04		ok
Level 4	3.1E-04	2.2E-04	2.5E-04	2.8E-03	not ok	Level 4	7.5E-04	5.3E-04	6.0E-04	1.3E-03	not ok
Level 3	3.4E-04	2.4E-04	2.7E-04	9.2E-04	not ok	Level 3	6.5E-04	4.6E-04	5.2E-04	9.5E-04	not ok
Level 2	2.5E-04	1.7E-04	2.0E-04	7.5E-04	not ok	Level 2	4.7E-04	3.3E-04	3.8E-04	6.8E-04	not ok
Level 1	2.5E-05	1.7E-05	2.0E-05	7.2E-04	not ok	Level 1	6.0E-05	4.2E-05	4.8E-05	6.3E-04	not ok
	SCBF - '	Vertical Irreg	ularity 1a X I	Direction			SCBF	- Vertical Irre	gularity 1a Y	Direction	
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next		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	4.4E-03	3.1E-03	3.5E-03		ok	Stair 3	2.4E-04	1.7E-04	1.9E-04		ok
High Roof	6.6E-04	4.7E-04	5.3E-04		ok	High Roof	4.8E-03	3.4E-03	3.9E-03		ok
Low Roof	2.7E-04	1.9E-04	2.2E-04		ok	Low Roof	2.5E-04	1.7E-04	2.0E-04		ok
Level 4	6.1E-04	4.3E-04	4.9E-04	4.3E-03	not ok	Level 4	7.8E-04	5.5E-04	6.3E-04	1.8E-03	not ok
Level 3	5.5E-04	3.9E-04	4.4E-04	1.2E-03	not ok	Level 3	7.6E-04	5.3E-04	6.1E-04	2.0E-03	not ok
Level 2	7.1E-04	5.0E-04	5.7E-04	1.2E-03	not ok	Level 2	1.0E-03	7.2E-04	8.2E-04	6.0E-04	ok
Level 1	2.0E-05	1.4E-05	1.6E-05	1.5E-03	not ok	Level 1	2.8E-05	1.9E-05	2.2E-05	8.6E-04	not ok
	SMF - V	ertical Irregu	larity 1a X D				SMF	- Vertical Irre	gularity 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	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	Stair 3	3.5E-03	2.5E-03	2.8E-03		ok
High Roof	3.0E-05	2.1E-05	2.4E-05		ok	High Roof	5.6E-05	3.9E-05	4.5E-05		ok
Low Roof	3.4E-05	2.4E-05	2.7E-05		ok	Low Roof	7.8E-05	5.4E-05	6.2E-05		ok
Level 4	5.6E-05	3.9E-05	4.5E-05	1.1E-04	not ok	Level 4	2.7E-04	1.9E-04	2.2E-04	1.2E-03	not ok
Level 3	7.8E-05	5.5E-05	6.3E-05	9.6E-05	not ok	Level 3	1.9E-04	1.3E-04	1.5E-04	1.3E-04	ok
Level 2	7.2E-05	5.0E-05	5.7E-05	1.3E-04	not ok	Level 2	4.0E-04	2.8E-04	3.2E-04	1.8E-04	ok
Level 1	1.2E-06	8.6E-07	9.9E-07	1.7E-04	not ok	Level 1	8.9E-05	6.2E-05	7.1E-05	2.8E-04	not ok

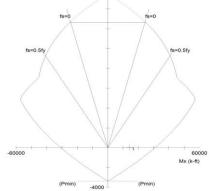
## Appendix E – Lateral Force Resisting System Design Checks-Existing System

				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)	∆(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 rec	tangular yet has c	penings				∑Ri =	1046672.10		100.00%
** Using a 1k	load applied	at the top of each	n LFRS system								

	Cent	er of Rigidity	
X Direction	kix (k/ft)	xi (ft)	kix Xi
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
Σ	1808816.31		118077107.89
	х	$t$ (ft) = $\sum k_{ix} x_i / k_{ix}$ =	65.28
Y Direction	<b>k</b> iy <b>(k/ft)</b>	yi (ft)	kiy yi
Y Direction SW1	<b>k</b> iy <b>(k/ft)</b> 372522.46	<b>y<sub>i</sub> (ft)</b> 97.00	<b>k</b> iy <b>y</b> i 36134678.63
SW1	372522.46	97.00	36134678.63
SW1 SW2	372522.46 372522.46	97.00 64.00	36134678.63 23841437.45
SW1 SW2 SW7	372522.46 372522.46 357612.53	97.00 64.00 24.00	36134678.63 23841437.45 8582700.82
SW1 SW2 SW7 SW8	372522.46 372522.46 357612.53 357612.53	97.00 64.00 24.00 44.00	36134678.63 23841437.45 8582700.82 15734951.50
SW1 SW2 SW7 SW8 SW9	372522.46 372522.46 357612.53 357612.53 210285.51	97.00 64.00 24.00 44.00 124.00	36134678.63 23841437.45 8582700.82 15734951.50 26075403.57
SW1 SW2 SW7 SW8 SW9 SW10	372522.46 372522.46 357612.53 357612.53 210285.51 210285.51	97.00 64.00 24.00 44.00 124.00 111.67	36134678.63 23841437.45 8582700.82 15734951.50 26075403.57 23482583.20
SW1 SW2 SW7 SW8 SW9 SW10 SW16 SW17	372522.46 372522.46 357612.53 357612.53 210285.51 210285.51 283841.76	97.00 64.00 24.00 44.00 124.00 111.67 4.00	36134678.63 23841437.45 8582700.82 15734951.50 26075403.57 23482583.20 1135367.03

#### Existing Special Reinforced Concrete Shear Wall Design Check

		Speci	ial Reinforce	ed Conc	rete Shear	Wall Design Based	on ACI 318-08 (	Ch. 21.9					
INPUT DATA & DESIGN SUMMARY		Wall				X-Direction							
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D +	+ 1.0L +1.0E		Pu	=	3718	k	at BASE
REBAR YIELD STRESS	f <sub>v</sub>	=	60	ksi		FACTORED BASE MC			M	=	46338		
HEIGHT OF WALL	Ĥ	=	376.0	in		FACTORED BASE SH			V,	=	330		
LENGTH OF SHEAR WALL	L	=	254.0	in					u				
THICKNESS OF WALL	t	=	24	in		THE WALL	DESIGN IS ADE	QUATE.					
	Acv	=		in^2									
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*V(f'c) ; need a	at least two curtains (r	ows) =	771.2	Need 1									
1. Check Permitted Shear Strength						4. Required Vertical							
ACI 318-08 § 11.9						ρl = Av/S*h ≥ 0.0025			ρΙ	=		>0.0025	ОК
ΦVn ≥ Vu	Vu	=	330.4	kip		Max. Spacing	S ≤ L/3 =	84.68	S	=	6	in	
Vn = Vc + Vs	d	=	203.2	in			S ≤ 3t =	72					
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3084.8	kip			S ≤ 18"	Governs					
	ΦVn	=	2313.6	kip			1	<b>RY</b> #11	A/bar	=		in^2	
ACI 318-08 § 21.9.4	Vn	≤	53429.2	kip			15 1 4 10		# bars required	=	11		
$V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)  \alpha c = 2 \text{ (conservative 2. Shear Strength Provided by Vc}$	9)		ОК			ACI 318-08 § 21.9.4.4	, IF NW/IW≤2; need	reinf. In two directions (د	ol≥pt) h/l	=	1 4700	-15 -4	
Vc $\leq 2^{\lambda}t^{+}d^{-}\sqrt{f'c}$ $\lambda = 1.0$ (for N.W.	C) Vc	=	617.0	kip						= is	1.4799 OK	ρι≥ρι	
Note: If $Vu \le Acv^* \sqrt{(f'c)}$ can choose pt, pl accord		=	385.6		ng to Ch.14				ρl≥ρt	15	UK		
3. Required Horizontal Shear Reinforcement		-	505.0	Accordi	g to CII.14								
1/2ΦVc < Vu	1/2ΦVc	=	231.4	kip									
	_, _ + • • •		According t			WALL DIS	T. HORIZ. REINF.		19	#8	0	8	" O.C.
Vs = Vu/(0.75) - Vc	Vs	=	-176.4	kip		WALL DIS	T. VERT. REINF.		11	#11	0	6	" O.C.
S = (Av*fy*d)/Vs	Ag	=	6096.96	in^2									
	0.0025*Ag	=	15.2	in^2									
<b>TRY</b> #8	Abar	=	0.79	in^2									
Max. Spacing $S \le L/3 = 84.68$ $S \le 3t = 72$	S	=	8.00	in	USE								
$S \leq St = 72$ $S \leq 18$ " Governs	# bars required	=	19										
$\rho t = Av/(S^*t)$	φt	-		> 0.0025	OK								
pr - 70(3 t)	pr		0.0754	20.0025	ÖR								
5. Design for Flexture													
Assume Tension-controlled section, $\Phi = 0.9$													
$Mn = As^{fy^{(d-(a/2))}} = As^{fy^{j}} jd = 0.9^{d}$						TRY	#11	A/bar	=		in^2		
C=T 0.85*f'c*a*b =As*fy	jd	=	182.91	in				# bars required	=	36			
$Mu = \Phi Mn = \Phi As*fy*j*d$	As	=	56.30	in^2		Check Capacity:		а	=	41.48			
	a	=	41.40	in		C=T	0.85*f'c*a*b =As		=	48.80		<b></b>	
jd = d - (a/2)	jd As	=		in i= A 2		εu = 0.003	c = a/0.85	εt	=	0.01	>0.0025	OK	
	AS	=	56.41	in^2		εu = 0.005 εt = εu*((d			14/	all 1			
CHECK MINIMUM REINFORCEMENT RATIOS	AND SPACING (A	CI 318-0	08 14.3. 21.9	9.2)		21 - 20 ((u	(-()/()		***				
ptprovd. = 0.0794 >	(pt ) <sub>min.</sub> =		0.0025		ок								
	(ρl) <sub>min.</sub> =		0.0025		ок		T. HORIZ. REINF.		19	#8	Ø	8	1.0.0
plprovd. = 0.1186 >	(p1) <sub>min.</sub> =		0.0025		OK		T. HORIZ. REINF. T. VERT. REINF.		19 36	#8 #11	@ @	-	" O.C.
CHECK SHEAR CAPACITY (ACI 318-08 11.2 8	8 21 0 4)					WALL DIS	I. VERI. REINF.		36	#11	(U)	6	" O.C.
	= 2 (conservative)	40072	kips		V., =	330 OK							
		10072	Kip3	-	• u =	550 OK							
CHECK FLEXURAL & AXIAL CAPACITY													
THE ALLOWABLE MOMENT AT AN AXIAL L	OAD P., IS GIVEN B	Y											
$\Phi M_{p} = 556,057$ kip-ft		M <sub>u =</sub>	46,338	ок									
where $\Phi = 0.900$	(ACI 318-08 Fig. R												
CHECK BOUNDARY ZONE REQUIREMENTS													
AN EXEMPTION FROM THE PROVISION OF	BOUNDARY ZONE	CONFIN	IEMENT REIN	IFORCEM	ent is give	N BY ACI318-05 21.9.6	5.2,						
21.9.6.3, and 21.9.6.5(a) PROVIDED THAT													
c < (L*H) / (600 $d_u$ ) for ACI 21.9.6.2 a		c <	60.49	in.		lary Element Needed							
where c = 49						al axis at P <sub>u</sub> & M <sub>n</sub> loa							
d <sub>u</sub> = 2.6	in. ( design displ	acemen	t, assume 0.0	107*H con	nservative, se	ee ACI 318-08 21.9.6.2a	a. )						
					(Pmax)	P (kip) 16000 — (Pma	x)						
					(ax)	(Pilla	1060.						
						$< \downarrow \setminus$ .							
					fs=0	fs=	0						
					A	+ /							



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

		Mc	dified Special Re	inforced Shear	Wall Shear W	/all 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)	∆(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%
		area of wall is rec						∑Ri =	1035637.35		100.00%
	-	at the top of each	• •	opennigs				Ζ			
oonig a 2k			dified Special Re	inforced Shear	Wall Shear V	/all Data*	- Level 2				
		1410			wan Shear v	Area in	- Level 2	Area in			
Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS- axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	NS- Dir. (ft2)	Length in EW-Dir (ft)	EW-dir (ft2)	k cantilever (k/in)	∆(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%
		area of wall in rec						ΣRi =	270442.77		100.00%
	-	at the top of each	• •	opennigs			L	Ζια -	2/0112.//		200.0070
Oshig u ik			dified Special Re	inforced Sheer	Wall Shoar M	/all Data*			·		
		1010		inforced Shear	wan Shear v		- Level 5	Area in			
Shear Wall	thickness	b, Length (ft)	Angle with	h Height (ft)	Length in	Area in	Length in	Area in EW-dir	k cantilever	∆(in)**	I - P:/SP:
Number	(in)	b, Length (ft)		h, Height (ft)	NS-Dir (ft)	NS- Dir. (ft2)	EW-Dir (ft)	Evv-air (ft2)	(k/in)	Δ(iii)	I =Ri/∑Ri
1	10	24.00	(deg)	12.00	1 50		24.00		E4429.04	1.005.05	17.050/
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%
15	<u> </u>		Ũ	12.00	21.17	231.01	2.00	24.00	50051.01	2.010 05	

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

		IVIC	odified Special Re	intorceu Shear	Tun Shear W		200014	A rea !			
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	t the general	area of wall in rec	tangular yet has	openings				∑Ri =	304977.93		100.00%
		at the top of each					L. L. L.				
		Mod	lified Special Reir	nforced Shear V	Vall Shear Wa	ll Data* -	Low Roof				
Chara Wall	41.2.1		A start start NC		1	Area in	Law and Law	Area in	I		
Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS- axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	NS- Dir. (ft2)	Length in EW-Dir (ft)	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%
10	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%
		area of wall in rec			21.17	201.05	2.00	<u>Σ</u> Ri =	292826.61	2.512 05	100.00%
Assume that		at the top of each		opennigs			L	<u>Z</u> ru =	252020.01		100.00%
* Using a 1k				forced Shear M	/all Shear Wa	Detet	High Roof				
** Using a 1k		Moc	lified Special Rein	nonceu snear w		II Data" -	<u> </u>				
		Мос	lified Special Rein	norceu snear w		Area in		Area in			
** Using a 1k Shear Wall Number	thickness (in)	Moc b, Length (ft)	lified Special Rein Angle with NS- axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir.	Length in EW-Dir (ft)	EW-dir	k cantilever (k/in)	∆(in)**	I =Ri∕∑Ri
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)	EW-Dir (ft)	EW-dir (ft2)	(k/in)		_
Shear Wall	thickness	<b>b, Length (ft)</b> 34.00	Angle with NS-		Length in NS-Dir (ft) 1.50	Area in NS- Dir. (ft2) 10.50	<b>EW-Dir (ft)</b> 34.00	<b>EW-dir</b> (ft2) 238.00	<b>(k/in)</b> 103386.85	1.01E-05	19.68%
Shear Wall Number 2 5	thickness (in) 18 18	<b>b, Length (ft)</b> 34.00 23.17	Angle with NS- axis (Y) (deg) 90 0	<b>h, Height (ft)</b> 7.00 7.00	Length in NS-Dir (ft) 1.50 23.17	Area in NS- Dir. (ft2) 10.50 162.19	<b>EW-Dir (ft)</b> 34.00 1.50	<b>EW-dir</b> (ft2) 238.00 10.50	<b>(k/in)</b> 103386.85 66208.30	1.01E-05 1.57E-05	19.68% 12.60%
Shear Wall Number 2 5 6	thickness (in) 18 18 18 18	<b>b, Length (ft)</b> 34.00 23.17 23.17	Angle with NS- axis (Y) (deg) 90 0 0	<b>h, Height (ft)</b> 7.00 7.00 7.00 7.00	Length in NS-Dir (ft) 1.50 23.17 23.17	Area in NS- Dir. (ft2) 10.50 162.19 162.19	<b>EW-Dir (ft)</b> 34.00 1.50 1.50	<b>EW-dir</b> (ft2) 238.00 10.50 10.50	(k/in) 103386.85 66208.30 66208.30	1.01E-05 1.57E-05 1.57E-05	19.68% 12.60% 12.60%
Shear Wall Number 2 5	thickness (in) 18 18	<b>b, Length (ft)</b> 34.00 23.17 23.17 10.33	Angle with NS- axis (Y) (deg) 90 0	h, Height (ft) 7.00 7.00 7.00 7.00	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00	<b>EW-Dir (ft)</b> 34.00 1.50 1.50 10.33	<b>EW-dir</b> (ft2) 238.00 10.50 10.50 72.31	(k/in) 103386.85 66208.30 66208.30 13537.42	1.01E-05 1.57E-05 1.57E-05 7.58E-05	19.68% 12.60% 12.60% 2.58%
Shear Wall Number 2 5 6 7 8	thickness (in) 18 18 18 18 12 12	<b>b</b> , Length (ft) 34.00 23.17 23.17 10.33 10.33	Angle with NS- axis (Y) (deg) 90 0 0 90 90 90	h, Height (ft) 7.00 7.00 7.00 7.00 7.00	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00	<b>EW-Dir (ft)</b> 34.00 1.50 1.50 10.33 10.33	<b>EW-dir</b> (ft2) 238.00 10.50 10.50 72.31 72.31	(k/in) 103386.85 66208.30 66208.30 13537.42 13537.42	1.01E-05 1.57E-05 1.57E-05 7.58E-05 7.58E-05	19.68% 12.60% 12.60% 2.58% 2.58%
Shear Wall Number 2 5 6 7 8 9	thickness (in) 18 18 18 18 12 12 12 12	<b>b</b> , Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00	Angle with NS- axis (Y) (deg) 90 0 0 90 90 90 90	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00	<b>EW-Dir (ft)</b> 34.00 1.50 1.50 10.33 10.33 32.00	<b>EW-dir</b> (ft2) 238.00 10.50 10.50 72.31 72.31 224.00	(k/in) 103386.85 66208.30 66208.30 13537.42 13537.42 64408.42	1.01E-05 1.57E-05 1.57E-05 7.58E-05 7.58E-05 1.61E-05	19.68% 12.60% 12.60% 2.58% 2.58% 12.26%
Shear Wall           Number           2           5           6           7           8           9           10	thickness (in) 18 18 18 12 12 12 12 12 12	<b>b</b> , Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00	Angle with NS- axis (Y) (deg) 90 0 0 90 90 90	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00 1.00	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 7.00 7.00	<b>EW-Dir (ft)</b> 34.00 1.50 1.50 10.33 10.33 32.00 32.00	EW-dir (ft2) 238.00 10.50 10.50 72.31 72.31 224.00 224.00	(k/in) 103386.85 66208.30 66208.30 13537.42 13537.42 64408.42 64408.42	1.01E-05 1.57E-05 1.57E-05 7.58E-05 7.58E-05 1.61E-05 1.61E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26%
Shear Wall           Number           2           5           6           7           8           9           10           11	thickness (in) 18 18 18 12 12 12 12 12 12 12 12 18	<b>b</b> , Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33	Angle with NS- axis (Y) (deg) 90 0 0 90 90 90 90 90 90 0 0	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00 1.00 12.33	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 7.00 86.31	EW-Dir (ft) 34.00 1.50 1.50 10.33 10.33 32.00 32.00 1.50	EW-dir (ft2) 238.00 10.50 10.50 72.31 72.31 224.00 224.00 10.50	(k/in) 103386.85 66208.30 66208.30 13537.42 13537.42 64408.42 64408.42 27421.50	1.01E-05 1.57E-05 1.57E-05 7.58E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22%
Shear Wall           Number           2           5           6           7           8           9           10           11           12	thickness (in)           18           18           18           12           12           12           12           12           12           12           13	<b>b</b> , Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33	Angle with NS- axis (Y) (deg) 90 0 0 90 90 90 90 90 0 0 0 0	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00 1.00 12.33 12.33	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 7.00 86.31 86.31	EW-Dir (ft) 34.00 1.50 1.50 10.33 10.33 32.00 32.00 1.50 1.50	EW-dir (ft2) 238.00 10.50 72.31 72.31 224.00 224.00 10.50 10.50	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 64408.42 27421.50 27421.50	1.01E-05 1.57E-05 1.57E-05 7.58E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05 3.75E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22% 5.22%
Shear Wall           Number           2           5           6           7           8           9           10           11           12           13	thickness (in)           18           18           18           12           12           12           12           12           12           12           12           12           12           12           12           12           12           12           12           12           12           12           13           14           15	b, Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33 21.17	Angle with NS- axis (Y) (deg) 90 0 90 90 90 90 90 0 0 0 0 0	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00 1.00 12.33	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 7.00 86.31	EW-Dir (ft) 34.00 1.50 1.50 10.33 10.33 32.00 32.00 1.50	EW-dir (ft2) 238.00 10.50 72.31 72.31 224.00 224.00 10.50 10.50 14.00	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 64408.42 27421.50 27421.50 78900.71	1.01E-05 1.57E-05 1.57E-05 7.58E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22% 5.22% 15.02%
Shear Wall           Number           2           5           6           7           8           9           10           11           12           13           Assume that	thickness (in) 18 18 18 12 12 12 12 12 12 12 12 12 18 18 18 24 t the general	<b>b</b> , Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33	Angle with NS- axis (Y) (deg) 90 0 90 90 90 90 90 0 0 0 0 0 0 tangular	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00 1.00 12.33 12.33	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 7.00 86.31 86.31	EW-Dir (ft) 34.00 1.50 1.50 10.33 10.33 32.00 32.00 1.50 1.50	EW-dir (ft2) 238.00 10.50 72.31 72.31 224.00 224.00 10.50 10.50	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 64408.42 27421.50 27421.50	1.01E-05 1.57E-05 1.57E-05 7.58E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05 3.75E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22% 5.22%
Shear Wall           Number           2           5           6           7           8           9           10           11           12           13           Assume that	thickness (in) 18 18 18 12 12 12 12 12 12 12 12 12 18 18 18 24 t the general	b, Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33 21.17 area of wall is rec at the top of eacl	Angle with NS- axis (Y) (deg) 90 0 90 90 90 90 90 0 0 0 0 0 0 tangular	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00 1.00 12.33 12.33 21.17	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 86.31 86.31 148.19	EW-Dir (ft) 34,00 1.50 1.50 10.33 10.33 32,00 32,00 1.50 1.50 2.00	EW-dir (ft2) 238.00 10.50 72.31 72.31 224.00 224.00 10.50 10.50 14.00	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 64408.42 27421.50 27421.50 78900.71	1.01E-05 1.57E-05 1.57E-05 7.58E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05 3.75E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22% 5.22% 15.02%
Shear Wall Number 2 5 6 7 8 9 10 11 12 13 * Assume that * Using a 1k Shear Wall	thickness (in) 18 18 18 12 12 12 12 12 12 12 12 12 12 12 12 12	b, Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33 21.17 area of wall is rec at the top of eacl	Angle with NS- axis (Y) (deg) 90 0 90 90 90 90 90 0 0 0 0 0 0 tangular tERS system odified Special Re Angle with NS-	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00 1.00 12.33 21.33 21.17 Wall Shear W Length in	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 86.31 86.31 148.19	EW-Dir (ft) 34.00 1.50 1.50 10.33 32.00 32.00 1.50 1.50 2.00 <b>4</b> <b>5</b> <b>5</b> <b>5</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b>	EW-dir           (ft2)           238.00           10.50           10.51           72.31           224.00           224.00           10.50           10.50           10.50           10.50           10.50           10.50           10.50           10.50           4.00           ΣRi =           Area in           EW-dir	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 64408.42 27421.50 27421.50 27421.50 78900.71 525438.84 k cantilever	1.01E-05 1.57E-05 1.57E-05 7.58E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05 3.75E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22% 5.22% 15.02%
Shear Wall           Number           2           5           6           7           8           9           10           11           12           13           Assume that           * Using a 1k           Shear Wall           Number	thickness (in) 18 18 18 12 12 12 12 12 12 12 18 18 24 t the general load applied	b, Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33 21.17 area of wall is rec at the top of each Mo	Angle with NS- axis (Y) (deg) 90 0 90 90 90 90 90 0 0 0 0 tangular 1 LFRS system odified Special Re Angle with NS- axis (Y) (deg)	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 1.00 1.00 1.00 1.00 12.33 12.33 21.17 Wall Shear W	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 86.31 86.31 148.19 ////////////////////////////////////	EW-Dir (ft) 34.00 1.50 1.50 10.33 32.00 32.00 1.50 1.50 2.00 - Stair 3	EW-dir (ft2) 238.00 10.50 72.31 72.31 224.00 224.00 10.50 10.50 14.00 ΣRi =	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 27421.50 27421.50 27421.50 78900.71 525438.84 K cantilever (k/in)	1.01E-05 1.57E-05 1.57E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05 1.31E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22% 5.22% 15.02% 100.00%
Shear Wall Number 2 5 6 7 8 9 10 11 12 13 Assume that * Using a 1k Shear Wall	thickness (in) 18 18 18 12 12 12 12 12 12 12 12 12 12 12 12 12	b, Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33 21.17 area of wall is rec at the top of each Mo	Angle with NS- axis (Y) (deg) 90 0 90 90 90 90 90 0 0 0 0 0 tangular tAFRS system odified Special Re Angle with NS-	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00 1.00 12.33 21.33 21.17 Wall Shear W Length in	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 86.31 86.31 148.19 /all Data* Area in NS- Dir.	EW-Dir (ft) 34.00 1.50 1.50 10.33 32.00 32.00 1.50 1.50 2.00 <b>4</b> <b>5</b> <b>5</b> <b>5</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b> <b>1</b>	EW-dir           (ft2)           238.00           10.50           10.51           72.31           224.00           224.00           10.50           10.50           10.50           10.50           10.50           10.50           10.50           10.50           4.00           ΣRi =           Area in           EW-dir	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 64408.42 27421.50 27421.50 27421.50 78900.71 525438.84 k cantilever	1.01E-05 1.57E-05 1.57E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05 1.31E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22% 5.22% 15.02% 100.00%
Shear Wall           Number           2           5           6           7           8           9           10           11           12           13           Assume that           * Using a 1k           Shear Wall           Number	thickness (in) 18 18 18 12 12 12 12 12 12 12 12 12 12 12 12 12	b, Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33 21.17 area of wall is rec at the top of each Mo b, Length (ft)	Angle with NS- axis (Y) (deg) 90 0 90 90 90 90 90 0 0 0 0 tangular 1 LFRS system odified Special Re Angle with NS- axis (Y) (deg)	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 23.17 1.00 1.00 1.00 1.00 12.33 12.33 21.17 Wall Shear W Length in NS-Dir (ft)	Area in NS- Dir. (ft2) 10.50 162.19 162.19 7.00 7.00 7.00 86.31 86.31 148.19 ////////////////////////////////////	EW-Dir (ft) 34.00 1.50 1.50 10.33 10.33 32.00 32.00 1.50 1.50 2.00 - Stair 3 Length in EW-Dir (ft)	EW-dir (ft2) 238.00 10.50 72.31 72.31 224.00 224.00 10.50 10.50 10.50 14.00 ΣRi =	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 27421.50 27421.50 27421.50 78900.71 525438.84 K cantilever (k/in)	1.01E-05 1.57E-05 1.57E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05 1.31E-05 1.31E-05	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22% 5.22% 15.02% 100.00%
Shear Wall           Number           2           5           6           7           8           9           10           11           12           13           Assume that           * Using a 1k           Shear Wall           Number           9	thickness (in) 18 18 18 12 12 12 12 12 12 12 12 12 12 18 18 24 the general load applied thickness (in)	b, Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33 21.17 area of wall is rec at the top of each Mo b, Length (ft) 32.00	Angle with NS- axis (Y) (deg) 90 0 90 90 90 90 90 0 0 0 0 tangular 1 LFRS system Dified Special Re Angle with NS- axis (Y) (deg) 90	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 1.00 1.00 1.00 12.33 12.33 21.17 Wall Shear W Length in NS-Dir (ft) 1.00	Area in NS- Dir. (ft2) 10.50 162.19 7.00 7.00 7.00 86.31 86.31 148.19 ////////////////////////////////////	EW-Dir (ft) 34.00 1.50 1.50 10.33 32.00 32.00 1.50 1.50 2.00 <b>. Stair 3</b> Length in EW-Dir (ft) 32.00	EW-dir           (ft2)           238.00           10.50           10.51           72.31           224.00           224.00           10.50           10.50           10.50           10.50           10.50           10.50           10.50           4.00           ΣRi =           Area in           EW-dir           (ft2)           64.00	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 27421.50 27421.50 27421.50 78900.71 525438.84 K cantilever (k/in) 239119.06	1.01E-05 1.57E-05 1.57E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05 3.75E-05 1.31E-05 4.36E-06	19.68% 12.60% 2.58% 2.58% 12.26% 12.26% 5.22% 5.22% 15.02% 100.00% I = Ri/ΣRi 32.04%
Shear Wall           Number           2           5           6           7           8           9           10           11           12           13           Assume that           * Using a 1k           Shear Wall           Number           9           10	thickness (in) 18 18 18 12 12 12 12 12 12 12 12 12 12 18 18 24 the general load applied thickness (in) 12 12	b, Length (ft) 34.00 23.17 23.17 10.33 10.33 32.00 32.00 12.33 12.33 21.17 area of wall is rec at the top of each b, Length (ft) 32.00 32.00 32.00	Angle with NS- axis (Y) (deg) 90 0 90 90 90 90 90 0 0 0 0 0 0 0 0 0	h, Height (ft) 7.00 7.00 7.00 7.00 7.00 7.00 7.00 7.0	Length in NS-Dir (ft) 1.50 23.17 1.00 1.00 1.00 12.33 12.33 21.17 Wall Shear W NS-Dir (ft) 1.00 1.00 1.00	Area in NS- Dir. (ft2) 10.50 162.19 7.00 7.00 7.00 86.31 86.31 148.19 ////////////////////////////////////	EW-Dir (ft) 34.00 1.50 1.50 10.33 10.33 32.00 32.00 1.50 2.00 - Stair 3 Length in EW-Dir (ft) 32.00 32.00	EW-dir           (ft2)           238.00           10.50           10.51           72.31           224.00           224.00           10.50           10.50           10.50           10.50           10.50           10.50           10.50           10.50           10.50           4.00 <b>ΣRi =</b> Carea in           EW-dir           (ft2)           64.00           64.00	(k/in) 103386.85 66208.30 13537.42 13537.42 64408.42 27421.50 27421.50 78900.71 525438.84 K cantilever (k/in) 239119.06 239119.06	1.01E-05 1.57E-05 1.57E-05 7.58E-05 1.61E-05 1.61E-05 3.75E-05 3.75E-05 1.31E-05 4.36E-06 4.36E-06	19.68% 12.60% 12.60% 2.58% 12.26% 12.26% 5.22% 5.22% 15.02% 100.00% I = Ri/∑Ri 32.04% 32.04%

WALL	Heigh	t (ft)	Leng	ıth (ft)						
Wall 1_a	a 24.	33	34	4.00						
Wall 1_b	38.	50	34	4.00						
Wall 2_a	31.	33	34	4.00						
Wall 2_b	38.	50	34	4.00						
Wall 3_a	31.	33	34	4.00						
Wall 3_b	38.	50	34	4.00						
Wall 4_a	a 24.	33	34	4.00						
Wall 4_b	38.	50	34	4.00						
Wall 5_a	31.	33	20	0.00						
Wall 5_b	38.	50	20	0.00						
Wall 6_a	31.	33	20	0.00						
Wall 6_b	38.	50	20	0.00						
Wall 7_c	31.	33	10	0.33						
Wall 7_d	38.	50	10	0.33					9	A
Wall 8_c	31.	33	10	0.33	2 Carlos Annar C	ease ease ease ease ease ease ease ease			14	12
Wall 8_d	38.	50	10	0.33		Miller Control of Cont		5	9 4A	Ŭ.
Wall 9_c	33.	33	34	4.00					1118	
Wall 9_d	38.	50	34	4.00					6 0	
Wall 10_	с 33.	33	34	4.00		<b>A</b>				
Wall 10_d	d 38.	50	34	4.00	-					O C V
Wall 11_	e 33.	33	1	2.33	and and		C A			5.
Wall 11_	f 38.	50	1	2.33	Lunn Pale	13				~
Wall 12_	e 33.	33	1	2.33						-
Wall 12_	f 38.	50	1	2.33	T		- 901		216 0	
Wall 13_	g 31.	33	2	1.17	<u> </u>	and the second	W.	Q. P.		•
Wall 13_	h 38.	50	2	1.17	higher	O. Cumun	5	den a	~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~	
				Vertical Bar	Vertical Bar	Vertical Bar	Horizontal	Horizontal Bar	Bar	Bar Weight
Туре	Thickness (in)	Vertical Spa	acing (in)	Size	Diameter (in)	Weight (plf)	Spacing (in)	Size	Diameter	(plf)
а	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 f	18 18	12 6		6 11	0.75 1.41	1.502 5.313	12 8	6 8	0.75	1.502 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

		Spec	ial Reinforc	ed Conc	rete Shear	Wall Design Based	on ACI 318-	08 Ch. 2	21.9					
INPUT DATA & DESIGN SUMMARY		Wall	1 a			X-Direction								
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E			Pu	=	2201	k	at BAS
REBAR YIELD STRESS	f <sub>v</sub>	=	60	ksi		FACTORED BASE M	IOMENT LOAD			M.	=	968	ft-k	
HEIGHT OF WALL	Ĥ	=	292.0	in		FACTORED BASE S	HEAR LOAD			V.	=	205	k	
LENGTH OF SHEAR WALL		=	408.0	in		Incroned bride o				• u		200	N.	
THICKNESS OF WALL	t	_	18	in		THE WAI	LL DESIGN IS A		TF					
THICKINESS OF WALL	Acv	-	7344	in^2			LE DESIGN IS F	LQUA						
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*v(f'c) ; need			929.0	Need 1										
1. Check Permitted Shear Strength	at least two cuitains (	10ws) =	525.0	Neeu 1		4. Required Vertica	al Shear Reifor	coment						
ACI 318-08 § 11.9						$pl = Av/S^*h \ge 0.002$			0025)	ρΙ	=	0.40781	> 0.0025	OK
ΦVn ≥ Vu	Vu	=	204.7	kip		Max. Spacing	S ≤ L/3 =		136	S	-		in	U.
Vn = Vc + Vs	d	-	326.4	in		wax. spacing	S ≤ 3t =		54	5		0		
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip			S ≤ 3t = S ≤ 18"		Governs					
VII \$ 10 LU V(IC) U=0.8 L	ΦVn	-	2786.9	kip			5 2 10	TRY	#11	A/bar	=	1.56	in A 2	
A CT 210 00 5 21 0 4								INT					1112	
ACI 318-08 § 21.9.4	Vn	≤	124863.3	кір		ACT 210 00 5 21 0 4	4.15			# bars required	=	28		
$V_n \leq A_{cv} (\alpha c^* \sqrt{f_c' + \rho_t} f_y)  \alpha c = 2 \text{ (conservative)}$	ve)		ОК			ACI 310-08 9 21.9.4.	.4, 1⊢ ⊓W/IW≤2;	need rein	f. In two directions (ρl≥			0.745		
2. Shear Strength Provided by Vc			749.6							h/l	=	0.7156	ρl≥ρt	
$Vc \le 2^{\lambda}t^{*}d^{*}\sqrt{f'c}$ $\lambda = 1.0$ (for N.W		=	743.2	kip						ρl≥ρt	is	ОК		
Note: If Vu≤Acv*√(f'c) can choose pt, pl accor		=	464.5	Accordi	ing to Ch.14									
3. Required Horizontal Shear Reinforcement			270.7	la la										
1/2ΦVc < Vu	1/2ΦVc	=	278.7	kip	~					10				
			Reinf. Acco		Ch 14		ST. HORIZ. RE			40	#8	@	8	" O.C.
Vs = Vu/(0.75) - Vc	Vs	=	-	kip		WALL DI	ST. VERT. REIN	NF.		28	#11	@	6	" O.C.
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 $S \le L/3 = 136$	S	=	8.00	in	USE									
S ≤ 3t = 54			10											
S ≤ 18" Governs	# bars required	=	40											
$\rho t = Av/(S^*t)$	ρt	=	0.2167	> 0.0025	OK									
5. Design for Flexture														
Assume Tension-controlled section, $\Phi = 0.9$														
$Mn = As^{fy^{(d-(a/2))}} = As^{fy^{j}}  jd = 0.9^{d}$						TR	Y #6		A/bar	=	0.44	in^2		
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in					# bars required	=	1			
$Mu = \Phi Mn = \Phi As^*fy^*j^*d$	As	=	0.73	in^2		Check Capacity:			а	=	0.65	in		
	а	=	0.72	in		C=T	0.85*f'c*a*b	=As*fy	с	=	0.76	in		
jd = d - (a/2)	jd	=	326.04	in			c = a/0.85		εt	=	1.61	> 0.0025	ок	
	As	=	0.66	in^2			3 dt = L-3"							
						εt = εu*((					all 1			
CHECK MINIMUM REINFORCEMENT RATIO	S AND SPACING (A	CI 318-	08 14.3, 21.9	9.2)		WALL DI	ST. HORIZ. RE	INF.		40	#8	@	8	" O.C.
<sub>ρtprovd.</sub> = 0.2167 >	(pt ) <sub>min.</sub> =		0.0025		ОК	WALL DI	ST. VERT. REIN	NF.		24	#11	@	6	" O.C.
plprovd. = 0.4078 >	(pl) <sub>min</sub> =		0.0025		ок									
piprova. Entorio P	SF 71106		2.0025						—————————————————————————————————————					
CHECK SHEAR CAPACITY (ACI 318-08 11.2	& 21 9 4)								<b>&gt;</b>					
-	c = 2 (conservative)	03647	king	>	V <sub>u =</sub>	205 <b>OK</b>								
$\Psi v_n \ge A_{cv} (\alpha c^{-v} v_c + \rho_{t^*} v_y)$ $\alpha$	c – z (conservative)	22047	kips	>	v <sub>u =</sub>	203 UK								
CHECK FLEXURAL & AXIAL CAPACITY														
		ov.							$\longrightarrow$		f f	-		
THE ALLOWABLE MOMENT AT AN AXIAL	-		0.00	<b>0</b> 11										
$\Phi$ M <sub>n</sub> = 11,618 kip-ft	>	M <sub>u =</sub>	968	ОК										
where Φ= 0.900	(ACI 318-08 Fig.	R9.3.2)							$\longrightarrow$					
CHECK BOUNDARY ZONE REQUIREMENTS														
AN EXEMPTION FROM THE PROVISION O 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT	F BOUNDARY ZONE	CONFIN	IEMENT REIN	NFORCEM	1ENT IS GIVE	N BY ACI318-05 21.9	.6.2,							
	apply	c <	97.14	in	No Bound	arv Element Needeo	d		1					
$c < (L^*H) / (600 d_u)$ for ACI 21.9.6.2 where $c = 1$		c < m the e	97.14 streme.comp	in. ression fi		dary Element Needeo al axis at P <sub>u</sub> & M <sub>n</sub> Io			1					

INPUT DATA &	DESIGN SU	MMAR	1		Wall	1 b			X-Direction									
CONCRETE STRE	NGTH (ACI	318 5.1.	i)	f <sub>c</sub> '	=	4	ksi		Load Combo:	1.2 D +	1.0L +1.0E			Pu	=	3752	k	
REBAR YIELD STR	RESS			f <sub>v</sub>	=	60	ksi		FACTORED M	10MENT	LOAD			Mu	=	1051	ft-k	
HEIGHT OF WAL				Н	=	462.0	in		FACTORED SH					V.	=	196	k	
LENGTH OF SHE				1	=	408.0	in				-			· u				
THICKNESS OF				t	=	18	in		тн	FWALL	DESIGN IS ADE	004	F					
				Acv	=	7344	in^2					20/1						
ACI 318-08 § 21.9	9.2.2. IF Vu ≥	2*Acv*1	(f'c) : need a	at least two curtains (	rows) =	929.0	Need 1	L										
1. Check Permit					,				4. Required \	Vertical S	hear Reiforcem	ent						
ACI 318-08 § 11.9											+0.5 (2.5 - h/L)*(		0025)	ρl	=	0.36318	3 > 0.0025	5 <b>ок</b>
ΦVn ≥				Vu	=	195.5	kip		Max. Spacing		S ≤ L/3 =		136	S	=	6	in	
Vn = V				d	=	326.4	in				S ≤ 3t =		54	-		-		
	0*t*d*√(f'c)	d=0.8 <sup>,</sup>	1	Vn	=	3715.8	kip				S ≤ 18"		Governs					
			-	ΦVn	=	2786.9	kip					TRY	#11	A/bar	=	1.56	in^2	
ACI 318-08 § 21.9	94			Vn	- 5	124863.3	kip		1				"11	# bars require		25	2	
√ <sub>n</sub> ≤ A <sub>cv</sub> (αc*√f <sub>c</sub> '		(c = ) (r	onservativo		2	0K	- YP		ACT 318-08 5	21 9 4 4 1	F hw/lw<2.noo	d rein	. In two directions (p			23		
2. Shear Strengt			Silseivaulve	1		UK			ACT 310-08 8	21.3.4.4, 1		u reill	. In two unections (p	l≥ρι) h/l	=	1.1324	ol> ot	
	th Provided *λ*t*d*√(f'c)		) (for NMC	C) Vc	=	743.2	kip		1					n/i pl≥pt	= is	0K	hi≤hr	
VC≤∠ Note: If Vu≤Acv*					_	464.5		ling to Ch.14						Pi≤ Pi	15	UK.		
3. Required Hor				ing to CII.14	-	404.3	Accord	ing to CII.14										
1/2ΦVc		a Kenn	Accinent	1/2ΦVc	=	278.7	kip		1									
1/2000	- • vu			1/2410	-	Reinf. Acco		Ch 14	14/4	דזת ווו	HORIZ. REINF			40	#8	0	8	" 0
V/c - V/	u/(0.75) - Vo	-		Vs	=	Reini. Acco	kip	CII 14			VERT. REINF.	•		40 25	#0 #11			" 0
		·			=	7344	кір in^2		VVA	NEL DIST.	VENT. REINF.			25	#11	ιψ.	U	0
5 = (AV	/*fy*d)/Vs			Ag 0.0025*Aq	=	/344 18.4	in^2		1									
	TRY		#8	A/bar	_	0.79	in^2											
Mau Caraina			#0					UCT										
Max. Spacing	S ≤ L/3 =			S	=	8.00	in	USE										
	S ≤ 3t = S ≤ 18"	54 Gover		Av # haan annuined	=	7344.00 40												
ρt = Av		Gover	ns	# bars required ρt	=	0.2167	> 0.002	5 <b>ОК</b>									FALSE	
5. Design for Fle													A/bar	=		in^2		
Assume Tension													# bars required	=	0			
Mn = As*fy*(d-(a			= 0.9*d							TRY	#11							
	*a*b =As*fy			jd	=	293.76	in								_			
$Mu = \Phi Mn = \Phi$	⊅ As*fy*j*d			As	=	0.80	in^2		Check Capac				а	=	0.70			
				а	=	0.78	in		C="		0.85*f'c*a*b =As	s*fy	с	=	0.83			
jd = d ·	- (a/2)			jd	=	326.01	in				c = a/0.85		εt	=	1.48	> 0.0025		
				As	=	0.72	in^2		εu ÷	= 0.003	dt = L-3"							
									εt =	= εu*((dt-	c)/c)				Wall 1			
CHECK BOUND																		
				BOUNDARY ZONE	E CONFI	NEMENT REIN	NFORCEN	/ent is give	N BY ACI318-0	05 21.9.6.2	2,							
21.9.6.3, and																		
c < (L*H	H) / (600 d <sub>u</sub> )	for AC	.I 21.9.6.2 ap	pply	c <	0.83	in.	No Bound	dary Element N	leeded								
wher	re c =		0	in. ( distance fro	om the e	extreme comp	ression f	fiber to neuti	ral axis at P <sub>u</sub> &	M <sub>n</sub> load	s. )							
	d <sub>u</sub> =	-	6.8	in. ( design disp	laceme	nt, assume 0.0	007*H co	nservative, se	ee ACI 318-08 2	21.9.6.2a.	)							
CHECK MINIMU	JM REINFO	RCEMEN	IT RATIOS	AND SPACING (A														
	= 0.2167		>	(pt ) <sub>min.</sub> =		0.0025	-	ок										
Otorowd						0.0025		ок										
	= 0.3632		>	(ρΙ) <sub>min.</sub> =		0.0025		OK										
ptprovd.		ACI 31	3-08 11.2 8	કે 21.9.4)														
	<b>CAPACITY</b> (			= 2 (conservative)	2786.9	kips	>	V <sub>u =</sub>	195.5 <b>OK</b>									
plprovd.	<b>CAPACITY (</b> A <sub>cv</sub> (αc*√f <sub>c</sub> '		αс	= 2 (conservative;														
plprovd. <b>CHECK SHEAR (</b> $\Phi V_n \leq r$	A <sub>cv</sub> (αc*√f <sub>c</sub> '	+ ρ <sub>t*</sub> f <sub>y</sub> )																
plprovd. CHECK SHEAR ( ΦV <sub>n</sub> ≤ . CHECK FLEXUR/	A <sub>cv</sub> (αc*√f <sub>c</sub> ' ·	+ ρ <sub>t*</sub> f <sub>y</sub> ) L <b>CAPA</b>	CITY											40	#0		0.00	
plprovd. CHECK SHEAR ( DVn ≤ CHECK FLEXURA THE ALLOWA	A <sub>cv</sub> (αc*√f <sub>c</sub> ' <b>AL &amp; AXIAI</b> ABLE MOME	+ ρ <sub>t*</sub> f <sub>y</sub> ) L <b>CAPAC</b> NT AT A	CITY	OAD P <sub>u</sub> IS GIVEN	BY						HORIZ. REINF			40	#8	@	8.00	" 0
$\begin{array}{l} \rho   provd. \end{array}$ CHECK SHEAR ( $\Phi V_n \leq I$ ) CHECK FLEXURA THE ALLOWA $\Phi M_n = I$	$A_{cv} (\alpha c^* \sqrt{f_c})^*$ <b>AL &amp; AXIAI</b> ABLE MOME = 12,612	+ ρ <sub>t*</sub> f <sub>y</sub> ) L <b>CAPA(</b> NT AT A kip-ft	<b>city</b> In axial LC		BY M <sub>u =</sub>	1,051 0.900	ок	18-08 Fig. R9.	WA		HORIZ. REINF. VERT. REINF.			40 28	#8 #11	-		" O " O

		Spec	ial Reinford	ed Conc	rete Shea	r Wall Design Based on ACI 318-08 Ch. 2	21.9					
INPUT DATA & DESIGN SUMMARY		Wall	2 a			X-Direction						
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D + 1.0L +1.0E		Pu	=	2527	k	at BAS
REBAR YIELD STRESS	f <sub>v</sub>	=	60	ksi		FACTORED BASE MOMENT LOAD		M	=	3371	ft-k	
HEIGHT OF WALL	Ĥ	=	292.0	in		FACTORED BASE SHEAR LOAD		V.	=	1266		
LENGTH OF SHEAR WALL	L	=	408.0	in				·u		1200	N.	
THICKNESS OF WALL	t	=	18	in		THE WALL DESIGN IS ADEQUA	TF					
INICKNESS OF WALL	Acv	-	7344	in^2		THE WALL DESIGN IS ADEQUA						
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*V(f'c) ; need a			929.0	Need 2								
1. Check Permitted Shear Strength	acticast two curtains (i	011/3/-	525.0	NCCU 2		4. Required Vertical Shear Reiforcement						
ACI 318-08 § 11.9						$pl = Av/S^*h \ge 0.0025 + 0.5 (2.5 - h/L)^*(pt-0.5)$	0025)	ρl	=	0 23903	> 0.0025	OK
ΦVn ≥ Vu	Vu	=	1265.6	kip		Max. Spacing $S \le L/3 =$	136	S	-		in	ÖR
Vn = Vc + Vs	d	=	326.4	in		S ≤ 3t =	54	5		0		
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip		S ≤ 18"	Governs					
VII S 10 ( d V(IC) - d=0.0 E	ΦVn	=	2786.9	kip		TRY	#11	A/bar	=	1.56	in A 2	
ACI 318-08 § 21.9.4	Vn					INT	#11		-	1.50	1112	
		≤	85557.6	kip		ACI 218 09 5 21 0 4 4 JE hundrund 2 mart and	• • • • • • • • • • • • • • • • • • •	# bars required	=	1/		
$V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_t f_y)  \alpha c = 2 \text{ (conservative})$	e)		OK			ACI 318-08 § 21.9.4.4, IF hw/lw≤2; need rein	ι. πι two airections (ρl			0.7154		
2. Shear Strength Provided by Vc	0		740.0	1.5				h/l	=	0.7156	ρι≥ρt	
$Vc \le 2^{\lambda}t^{*}d^{*}\sqrt{f'c}$ $\lambda = 1.0$ (for N.W.0		=	743.2	kip				pl≥pt	is	ок		
Note: If Vu≤Acv*√(f'c) can choose pt, pl accord	ing to Cn.14	=	464.5	FALSE								
3. Required Horizontal Shear Reinforcement	1/241/		270.7	1.5								
1/2ΦVc < Vu	1/2ΦVc	=	278.7	kip		WALL DICT LIGDLT DEINE		22	"0	~	0	
N/ N/ ((0.75) N/			According			WALL DIST. HORIZ. REINF.		23 17	#8 #11	@ @	8	" O.C.
Vs = Vu/(0.75) - Vc	Vs	=	944.3	kip		WALL DIST. VERT. REINF.		1/	#11	@	6	" O.C.
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							
$S \leq 3t = 54$			22									
$S \le 18$ " Governs $\rho t = Av/(S^*t)$	# bars required ρt	=	23 0.1275	> 0.0025	or							
•	μι	-	0.1275	20.0025	UK							
5. Design for Flexture Assume Tension-controlled section, $\Phi = 0.9$												
$Mn = As^*fy^*(d-(a/2)) = As^*fy^*j  jd = 0.9^*d$						<b>TRY</b> #11	A/bar	= 1	1.56	in^2		
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in		111 #11	# bars required	=	1.50	111 2		
$Mu = \Phi Mn = \Phi As^* fy^* j^* d$	As	-	293.70	in in^2		Check Capacity:	# bars required	=	2.26	in		
$ivid = \Phi ivin = \Phi As iy j d$	a	_	2.55	in		C=T 0.85*f'c*a*b =As*fy	c	=	2.20	in		
jd = d - (a/2)	jd	_	325.15	in		c = a/0.85	εt	=	0.46	> 0.0025	04	
Ju = u - (a/2)	As	_	2.30	in^2		$\epsilon u = 0.003 \text{ dt} = L-3"$	81	-	0.40	>0.0025	UK	
	AS	-	2.50	111.12				14/-	all 1			
CHECK MINIMUM REINFORCEMENT RATIOS		CT 210	00 14 2 21	0.2)		εt = εu*((dt-c)/c) WALL DIST. HORIZ. REINF.		23	#8	0	8	" O.C.
	-	CI 310-		9.2)								
ptprovd. = 0.1275 >	(pt ) <sub>min.</sub> =		0.0025		ОК	WALL DIST. VERT. REINF.		24	#11	@	6	" O.C.
plprovd. = 0.2390 >	(pl ) <sub>min.</sub> =		0.0025		ОК							
CHECK SHEAR CAPACITY (ACI 318-08 11.2 &	& 21.9.4)						>					
$\Phi V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t'} f_v) $ $\alpha c$	= 2 (conservative)	64168	kips	>	V <sub>u =</sub>	1266 <b>OK</b>						
							Í					
CHECK FLEXURAL & AXIAL CAPACITY									e			
THE ALLOWABLE MOMENT AT AN AXIAL LO	OAD P., IS GIVEN B	βY					>		f	_		
Φ M <sub>n</sub> = 40,452 kip-ft	-	M., _	3,371	ОК								
where $\Phi = 0.900$	(ACI 318-08 Fig. R	-	-,	••••								
CHECK BOUNDARY ZONE REOLITREMENTS												
CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF 21.0.6.2, and 21.0.6.5(2) PROVIDED THAT	BOUNDARY ZONE	CONFI	NEMENT REI	NFORCEM	ent is gi	VEN BY ACI318-05 21.9.6.2,						
AN EXEMPTION FROM THE PROVISION OF 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT												
AN EXEMPTION FROM THE PROVISION OF	ipply o	c <	97.14	in.	No Bou	VEN BY ACI318-05 21.9.6.2, <b>ndary Element Needed</b> ıtral axis at P <sub>u</sub> & M <sub>n</sub> loads. )	>					

INPUT DATA & DE	SIGN SU	MMAR	,		Wall	2 b			X-Directio	n								
CONCRETE STRENG	TH (ACI 3	18 5.1.1	.)	f <sub>c</sub> '	=	4	ksi		Load Com	bo: 1.2 D +	+ 1.0L +1.0E			Pu	=	4123.0	k	
REBAR YIELD STRESS	s			f <sub>v</sub>	=	60	ksi		FACTORED	MOMEN	T LOAD			M.	=	1268.0	ft-k	
HEIGHT OF WALL	-			Н	=	462.0	in		FACTORED					V.	=	114.0		
LENGTH OF SHEAR	WALL			1	_	408.0	in		INCIONEL	> SITE/AR EX	OND			۴u	-	114.0	ĸ	
THICKNESS OF WAI				t	_	408.0	in			TI IF 14/41 1			<b>TF</b>					
THICKNESS OF WAI	LL			Acv	=	7344	in in^2			THE WALL	L DESIGN IS ADE	QUA	ATE.					
	o																	
ACI 318-08 § 21.9.2.2			(f'c) ; need at	t least two curtains	(rows) =	929.0	Need 1		4	1.4	cl p.:(							
1. Check Permitted	Shear St	rength									Shear Reiforcen							
ACI 318-08 § 11.9											5 +0.5 (2.5 - h/L)*	(pt-0		ρl	=		> 0.0025	ок
ΦVn ≥ Vu				Vu	=	114.0	kip		Max. Spac	ing	S ≤ L/3 =		136	S	=	6	in	
Vn = Vc +				d	=	326.4	in				S ≤ 3t =		54					
Vn ≤ 10*t*	*d*√(f'c)	d=0.8*	L	Vn	=	3715.8	kip				S ≤ 18"		Governs					
1				ΦVn	=	2786.9	kip					TRY	#11	A/bar	=		in^2	
ACI 318-08 § 21.9.4				Vn	≤	124863.3	kip							# bars required	=	25		
$V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho)$	ρ <sub>t*</sub> f <sub>y</sub> ) αα	: = 2 (c	onservative	)		OK			ACI 318-08	8 § 21.9.4.4	, IF hw/lw≤2; nee	d reir	nf. In two directions (ρ	l≥pt)				
2. Shear Strength P	Provided	y Vc							7					h/l	=	1.1324	ρl≥ρt	
Vc ≤ 2*λ*t*	t*d*√(f'c)	$\lambda = 1.0$	(for N.W.C	.) Vc	=	743.2	kip							ρl≥ρt	is	ок		
Note: If Vu≤Acv*√(f'					=	464.5		ling to Ch.14										
3. Required Horizon				-				5										
1/2ΦVc < 1				1/2ΦVc	=	278.7	kip											
, .						Reinf. Acco		Ch 14		WALL DIS	T. HORIZ. REINF			40	#8	@	8	" O.C.
Vs = Vu/(0)	075) - Vc			Vs	=	-	kip				T. VERT. REINF.			25	#11	@		" O.C.
S = (Av*fy*)				Ag	=	7344	in^2							25		e	Ū	0.0.
5 (/////	u)/ • 5			0.0025*Ag	=	18.4	in^2											
	TRY		#8	A/bar	_	0.79	in ^2											
Max. Spacing S	$S \le L/3 =$	136	#0	S	-	8.00	in	USE										
	$S \leq L/S =$ $S \leq 3t =$	54		Av	-	7344.00		USE										
-	S ≤ 31 = S ≤ 18"	54 Gover		# bars required		40												
		Gover	15	•	=	40 0.2167	> 0.002										FALSE	
$\rho t = Av/(S^3)$	o~t)			ρt	=	0.2107	>0.002	5 UK									FALSE	
5. Design for Flextu													A/bar	=	1.56	in^2		
Assume Tension-con		ection	m – ng										# bars required	-	1.50			
Mn = As*fy*(d-(a/2))			≠ = 0.5 = 0.9*d							TRY	#11		" bars required	-	-			
C=T 0.85*f'c*a*b		ju	= 0.5 u	jd	=	293.76	in			INI	#11							
				As	-	0.96	in^2		Charle Car				_	=	0.85			
$Mu = \Phi Mn = \Phi As$	s^ty^j^a								Check Cap		0.05+0 + +	+6	а					
	(2)			a	=	0.94	in			C=T	0.85*f'c*a*b =A	s^ty	c	=	1.00	in 0.0005		
jd = d - (a,	4/2)			jd	=	325.93	in				c = a/0.85		εt	=	1.22	>0.0025		
				As	=	0.86	in^2				dt = L-3"							
									1	εt = εu*((d	lt-c)/c)			Wa	all 1			
CHECK BOUNDARY																		
AN EXEMPTION				BOUNDARY ZON	ie confii	NEMENT REIN	NFORCEN	<b>MENT IS GIVE</b>	EN BY ACI31	.8-05 21.9.6	5.2,							
21.9.6.3, and 21.9																		
c < (L*H) /	/ (600 d <sub>u</sub> )	for AC	l 21.9.6.2 ap	ply	c <	1.00	in.	No Bound	dary Elemen	nt Needed								
where	c =		0	in. ( distance f	rom the e	extreme comp	ression	fiber to neut	ral axis at P	. & M <sub>n</sub> loa	ids. )							
	d <sub>u</sub> =		23.6	in. ( design dis	placemer	nt, assume 0.0	007*H co	nservative, s	ee ACI 318-0	08 21.9.6.2a	a. )							
CHECK MINIMUM		CEMEN						.,.										
			>	(ρt ) <sub>min.</sub>		0.0025		ок										
ptprovd. =																		
plprovd. =	0.3632		>	(pl) <sub>min.</sub>	=	0.0025		ОК										
CHECK SHEAR CAP																		
$\Phi V_n \le A_{cv}$	(αc*√f <sub>c</sub> ' +	$\rho_{t^{\star}}f_{y})$	ας	= 2 (conservative	e) 2786.9	kips	>	V <sub>u =</sub>	114.0	ок								
CHECK FLEXURAL &																		
THE ALLOWABLE	e momei	IT AT A	N AXIAL LC	DAD P <sub>u</sub> IS GIVEN	I BY					WALL DIS	T. HORIZ. REINF			40	#8	@	8.00	" O.C.
Φ M <sub>n</sub> =	15,216	kip-ft		>	M <sub>u =</sub>	1,268	ок			WALL DIS	T. VERT. REINF.			28	#11	@	6	" O.C.
			) (F . /-	0.000)/050/0	6E11 -	0.900	(ACT 2	18-08 Fig. R9	2 2)									
where Φ	D = Min{0.	9, IVIAXII	J.05 + (e <sub>t</sub> -	0.002)(250/3), 0.	03]} =	0.500	(ACL ).	10-00 FIQ. N.9.	.3.2)									

$\begin{array}{c c c c c c c c c c c c c c c c c c c $	MOMENT LOAD M <sub>u</sub> = 9415 ft-k
CONCRETE STRENGTH (ACI 318 5.1.1) ft' = 4 ksi Lead Combo 1.2 REBAR YIELD STRESS ft = 60 ksi FACTORED BASE HGCH OF SHEAR WALL L = 240.0 in FACTORED BASE LEAGTH OF SHEAR WALL L = 440.0 in Acv = 4320 in^2 I. Check Permitted Shear Strength ACI 318-08 519-22. IF Vu 22*Av*Vffc); need at least wordrains (rows) = 546.4 Need 1 I. Check Permitted Shear Strength ACI 318-08 519-22. IF Vu 22*Av*Vffc); need at least wordrains (rows) = 546.4 Need 1 I. Check Permitted Shear Strength ACI 318-08 519-4 V V U = 353.0 kip Wn = Vv + VS d = 192.0 in Wn = Vv + VS d = 10 (for NW.C) Vc = 437.2 kip Wv = 2000 kip Vv = 20000 kip Vv = 2000 k	$\begin{array}{cccccccccccccccccccccccccccccccccccc$
BEBAR WELD STRESS       f, HEIGHT OF WALL       H       =       60       ksi       FACTORED BASE         HEIGHT OF WALL       L       =       2400       in       FACTORED BASE         HEIGHT OF SHAR WALL       L       =       2400       in       FACTORED BASE         VIC 318-08 5 21.92.2, IF Vu 2 2*Av/*(ftc); need at least two curtains (rows) =       546.4       Need 1       Image: Comparison of the curtains (rows) =       546.4       Need 1         1. Check Permitted Shear Strength       A       a       =       192.0       in       Max. Spacing         Vin < 10: 10: 00 Vin 2 Vu       Vu       =       353.0       kip       Max. Spacing       Max. Spacing         Vin < 10: 10: 00 Vin 2 Vu       Vu       =       353.0       kip       Max. Spacing         Vin < 10: 00 Vin 2 Vu       Vu       =       30240.0       kip       Max. Spacing         Vin < 2 Vin Vin (Vic) (A = 10.10 (for NWC)       Vc       =       233.8       kip       Max. Spacing       Vin (Vic) (A = 10.40 (for NWC)       Vc       =       243.0       in ^2         Vis = Vur(70: 7) - Vc       Vs       S       335. kip       WWALL I       WALL I       WALL I       WALL I       WALL I       Max. Spacing       S L 31.8       S L	$\begin{array}{cccccccccccccccccccccccccccccccccccc$
HEIGHT OF WALL H = 376.0 in FACTORED BASE LENGTH OF SHEAR WALL L = 240.0 in THE W Acv = 4320 in $^{2}$ THE WINCKNESS OF WALL Acv = 138 in THE WINCKNESS OF WALL Acv = 4320 in $^{2}$ L Check Permitted Shear Strength Act 318-08 § 21.92.2, IF Vu $\geq 2^{*Acv^{*}V(T_{i}^{*}), need at least two curtains (new) = 566.4 Need 1 L Check Permitted Shear Strength Act 318-08 § 21.92.2, IF Vu \geq 2^{*Acv^{*}V(T_{i}^{*}), need at least two curtains (new) = 566.4 Need 1 L Check Permitted Shear Strength Act 318-08 § 21.94.4 Vu = 353.0 kip Vn \leq 10^{*}t" ^{*} p. f.) \alpha c = 2 (conservative) OKAct 318-08 § 21.94.4 Vn = 1285.8 kipAct 318-08 § 21.94.4 Vn = 1285.8 kipVn \leq 2^{*}Art"4' (^{*}(r) \alpha = -0.08 to 1 ^{*} ^{*$	SHEAR LOAD $V_u = 353$ k ALL DESIGN IS ADEQUATE. Cal Shear Reiforcement $025 + 0.5 (2.5 - h/1)^*(pt - 0.0025)$ $pl = 0.07217 > 0.0025$ OK $S \le 1/3 = 80$ $S = 12$ in $S \le 1/3 = 54$ $S \le 18^*$ Governs TRY #6 A/bar = 0.44 in^2 # bars required = 35 4.4, IF hw/lw≤2; need reinf. In two directions (pl≥pt) h/l = 1.5665 pl≥pt
ENGTH OF SHEAR WALL t = 2400 in HICKNESS OF WALL $A cv = 4220$ in $^{\circ}$ 2 KCI 318-08 5 21.92. IF Vu 2*AcV*(If C); need at least two curtains (now) = 5664 Need 1 LCheck Permitted Shear Stength $A cv = 4320$ in $^{\circ}$ 2 KCI 318-08 5 21.92. IF Vu 2*AcV*(If C); need at least two curtains (now) = 5664 Need 1 LCheck Permitted Shear Stength $A cv = 4320$ in $^{\circ}$ 2 ACI 318-08 5 21.94 $Vu = 335.0$ kip $P = 4.05\% h \ge 00$ Vn = Vv + Vs = d = 1320 in $A cv = 15838$ kip $Ovn = 15833$ kip $Ovn = 15833$ kip $Ovn = 15833$ kip $Ovn = 10000$ kip $A cv = 100000$ kip $A cv = 1000000$ kip $A cv = 100000000$ kip $A cv = 100000000000$ kip $A cv = 100000000000000000000000000000000000$	ALL DESIGN IS ADEQUATE.         cal Shear Reiforcement         025 $+0.5$ (2.5 $-h/L$ )*(pt-0.0025)       pl       =       0.07217 > 0.0025       OK         S $\leq L/3 =$ 80       S       =       12       in         S $\leq L/3 =$ 54         S $\leq 18^{\circ}$ Governs         TRY       #6       A/bar       =       0.44       in^2         # bars required       =       35         A/4, IF hw/lws2; need reinf. In two directions (pl2pt)         h/l       =       1.5665       pl2 pt
HICKNESS OF WALL       t       =       18       in       THE Walk         Acv       =       4320       in^2       in^2         Act 318-08 5 21.9 2.2. IF Vu ≥ 2*Acy*((Pc) ; need at least two ourtains (rows) =       546.4       Need 1         A. Check Permitted Sheer Strength       Vu       =       353.0       kip         VI 318-08 5 11.9       0       0       9.0       in A. Spacing       Max. Spacing         Vn < 10*V Vs	cal Shear Reiforcement         025 + 0.5 (2.5 - h/L)*(pt-0.0025) $pl$ =       0.07217 > 0.0025 OK         S ≤ L/3 =       80       S =       12 in         S ≤ 3t =       54       S       S         S ≤ 18"       Governs       Governs       a         TRY       #6       A/bar       a         # bars required       35       a         4.4, IF hw/lws2; need reinf. In two directions (pl2pt)       h/l       =       1.5665 pl ≥ pt
$\begin{array}{rcl} Ac & = & 4320 & in^2 2 \\ 5.0 \mbox{ for } 5 & 19.22 \mbox{ if } V & 22^*Acy^*M(T_0) : need at least two curtains (rows) = & 5464 & Need 1 \\ \hline \begin{tabular}{lllllllllllllllllllllllllllllllllll$	cal Shear Reiforcement         025 + 0.5 (2.5 - h/L)*(pt-0.0025) $pl$ =       0.07217 > 0.0025 OK         S ≤ L/3 =       80       S =       12 in         S ≤ 3t =       54       S       S         S ≤ 18"       Governs       Governs       a         TRY       #6       A/bar       a         # bars required       35       a         4.4, IF hw/lws2; need reinf. In two directions (pl2pt)       h/l       =       1.5665 pl ≥ pt
ACI 318-08 § 219.22. IF Vu = 27Aqr <sup>4</sup> (f <sup>2</sup> ); need at least two curtains (rows) = 546.4 Need 1 1. Check Permitted Shear Strength ACI 318-08 \$19 $0 = 0 \le 10^{-1} = 10^{-1} \le 10^{-1} = $	$\begin{array}{cccccccccccccccccccccccccccccccccccc$
1. Check Permitted Shear Strength ACI 318-08 § 11.94. Required Verti $pl = Av/S^{+} \ge 0.0$ Max. Spacing4. Required Verti $pl = Av/S^{+} \ge 0.0$ $pl = Coording to Ch.144. Required Vertipl = Av/S^{+} \ge 0.0pl = Coording to Ch.144. Required Vertipl = Av/S^{+} \ge 0.0pl = Coording to Ch.144. Required Vertipl = Av/S^{+} \ge 0.0pl = 0.0^{+} Av/S^{+} \ge 0.0^{+} Av/S^{+} \ge 0.0^{+} Av/S^{+} = 0.0^{+} Av/S^{+} $	$\begin{array}{cccccccccccccccccccccccccccccccccccc$
ACI 318-08 § 119 $@Vn \ge Vu$ $Vu$ $Vu$ $u$ $Vu$ $u$ $2353.0$ kip $Vn \le 10^{rtd-N}(fc) d=0.8^{rL}$ $Vn$ $u$ $2185.8$ kip $Vn \le 10^{rtd-N}(fc) d=0.8^{rL}$ $Vn$ $u$ $2185.8$ kip ACI 318-08 § 21.9.4 $Vn$ $s$ $30240.0$ kip $Vs \le 2^{N}td^{-N}(fc) A = 1.0$ (for NWC) $Vc$ $u$ $437.2$ kip $Vs \le 2^{N}td^{-N}(fc) A = 1.0$ (for NWC) $Vc$ $s$ $s$ $33.5$ kip $Note: If VusAcv^{-N}(Fc) can choose pt pl according to Ch.14 = 273.2 FALSE3.$ Required Horizontal Shear Reliforcement $1/2^{2VV < Vu}$ $1/2^{2VV} = 163.9$ kip Vs = Vu/(0.75) Vc $Vs$ $s$ $=$ $33.5$ kip $S = (Avf)^{rh}(d)/Vs$ $Ag$ $=$ $10.8$ in $^{-2}$ $Max. Spacing$ $S \le 1/3 = 80$ $S = 12.00$ in $USE$ $S \le 318^{-5}$ Governs # bars required $=$ $25$ $s \le 318^{-5}$ Governs # bars required $=$ $25$ $s \le 318^{-5}$ Governs # bars required $=$ $211.1$ in $^{-2}$ Check Capacity: $C=T$ $0.575'Cra^{+0} = As^{+}fy'$ jd $= 0.9^{*}d$ $T$ $c = T$ $0.575'Cra^{+}b = As^{+}fy'$ jd $= 0.9^{*}d$ $T$ $c = 0.00500 > 0K$ $Creck Capacity: da s = 11.24 in ^{-2} cau = 0.00ct = atu'ct$ CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2) WALL I $phrend. = 0.00500 > (pt)_{min} = 0.00225 OK$ $WALL I phrend. = 0.00500 > (pt)_{min} = 0.00225 OK WALL I phrend. = 0.00500 > (pt)_{min} = 0.00225 OK WALL I phrend. = 0.00500 > (pt)_{min} = 0.00225 OK WALL I QV_n \le A_{Ci}(ac^{+}f_i' + p_if_j) ac = 21.9.4QV_n \le A_{Ci}(ac^{+}f_i' + p_if_j) ac = 21.9.4$	$\begin{array}{cccccccccccccccccccccccccccccccccccc$
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	$\begin{array}{cccccccccccccccccccccccccccccccccccc$
$ \begin{array}{ccccc} \mbox{Vn} = Vc + Vs & d & = 1920 & in & Vn & S & S & S & S & S & S & S & S & S & $	$\begin{array}{cccccccccccccccccccccccccccccccccccc$
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	$S \le 18^{\circ}$ <b>Governs</b> <b>TRY</b> #6 A/bar = 0.44 in^2 # bars required = 35 A.4, IF hw/lws 2; need reinf. In two directions (pl2et) h/l = 1.5665 pl2 pt
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $	TRY#6A/bar=0.44in^2# bars required=354.4, IF hw/lws 2; need reinf. In two directions (pl>pt) $h/l$ =1.5665 $pl \ge pt$
$\begin{array}{c c c c c c c c c c c c c c c c c c c $	# bars required = 35 4.4, IF hw/lws 2; need reinf. In two directions (pl>pt) h/l = 1.5665 pl>pt
ACI 318-08 § 21.9.4 Vn s 302400 kip $V_{n} \leq A_{n} (ac^{n})(t_{n}^{+} + p_{n}t_{n}) = acc = 2 (conservative) OK ACI 318-08 § 21.9.4 CI 318-08 $ 21.9.4 CI 318-08 $ $	# bars required = 35 4.4, IF hw/lws 2; need reinf. In two directions (pl>pt) h/l = 1.5665 pl>pt
Vn $\leq A_{co}$ (ac*Vfc <sup>+</sup> + p, f <sub>0</sub> ) $\alpha c = 2$ (conservative) $OK$ ACI 318-08 § 21.9.         2. Shear Strength Provided by Vc       Vc       =       437.2       kip         Vc       S2 * 2V*t4*V(fc) = 1.0 (for NWC)       Vc       =       437.2       kip         Note: If VusAcv*V(fc) can choose pt, pl according to Ch.14       =       273.2       FALSE       FALSE         3. Required Horizontal Shear Reinforcement       1/2@Vc       =       163.9       kip       According to 11.9.9       WALL E         Vs       Vu(0.75) - Vc       Vs       =       33.5       kip       WALL E         Vs       Vu(0.75) - Vc       Vs       =       33.5       kip       WALL E         Max. Spacing       S L/3 = 80       S       =       12.00       in ^2       MAL E         S S 31 =       54       S       =       12.00       in ^2       Second in ^2       Sec	4.4, IF hw/lw≤2; need reinf. In two directions (ρl≥pt) h/l = $1.5665$ pl≥pt
2. Shear Strength Provided by Vc V c $2^{2}\lambda^{1+1}^{4}v'(f_{C}) \lambda = 1.0 (for NW.C) Vc = 437.2 kip Vc 2^{2}\lambda^{1+1}^{4}v'(f_{C}) \lambda = 1.0 (for NW.C) Vc = 437.2 kip Vc 2^{2}\lambda^{1+1}v'(f_{C}) \lambda = 1.0 (for NW.C) Vc = 437.2 kip Vs = Vu/(0.75) Vc V = 163.9 kip According to 119.9 WALL I Vs = Vu/(0.75) Vc Vs = 33.5 kip Vs = Vu/(0.75) Vc Vs = 33.5 kip Vs = Vu/(0.75) Vc Vs = 33.5 kip Vs = Vu/(0.75) Vc Vs = 0.00025^{4}M = 0.44 in^{2} = 0.044 in^{2} = 0.0500 > 0.00025 OKS. Design for FlextureAssume Tension-controlled section, \Phi = 0.9Mn = Asfty'(1-(a/2)) = Asfty'j jd = 0.9'dT = 0.85fty'(1-(a/2)) = Asfty'j jd = 0.9'dT = 0.0500 > (pt)_{min} = 0.0025 OKCHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)VWALL Iplyrood. = 0.0500 > (pt)_{min} = 0.0025 OKCHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)\Phi V_n \le A_{Cv} (ac^{n}(t_c^{1} + p, f_t)  \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) \Phi V_n \le A_{Cv} (ac^{n}(t_c^{1} + p, f_t)  \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) \Phi V_n \le A_{Cv} (ac^{n}(t_c^{1} + p, f_t)  \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK CHECK BOWNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-08 11.2 46 51.9 21.2 1.9.5 (a) POVIDED THAT c < (L^+H) / (600 d_u) for ACI 21.9.6 2 app) c < 57.14 in. No Boundary Element Need where c = 13 in. (distance from the extreme compression fiber to neutral axis at P, @ AM, d_u = 2.6 in. (design displacement, assume$	h/l = 1.5665 pl≥pt
$ V_{c} \le \frac{2^{2} N_{T} (d_{c}')}{(2c_{c}')} = 1.0 (for N.W.C) V_{c} = \frac{437.2}{2} kip \\ V_{c} (d_{c}') (f_{c}') (can choose pt, pl according to Ch.14 = 273.2 FALSE \\ S. Required Horizontal Shar Reinforcement \\ 1/20V_{c} < Vu 1/20V_{c} = 163.9 kip \\ According to 119.9 WALL I \\ S = (Av^{+}f_{y}'^{+}d)/V_{s} Ag = 4320 in^{2} \\ 0.0025^{+}Ag = 10.8 in^{2} \\ 0.0025^{+}Ag = 0.44 in^{2} \\ S \le 18^{+} 6 A_{bar} = 0.44 in^{2} \\ S \le 18^{+} 6 A_{bar} = 0.44 in^{2} \\ S \le 18^{+} 6 A_{bar} = 0.44 in^{2} \\ S \le 18^{+} 6 V_{s}' = 0.0500 \\ S \le 31 = 54 \\ S \le 18^{+} 6 V_{s}' = 0.0500 \\ S \ge 18^{+} 6 V_{s}' = 0.0005^{+} 0.0025^{+} 0K \\ S = 10^{+} (A_{c}/2) \\ A \le 18^{+} 6 V_{s}' = 0.0500 \\ S = 10^{-} (A_{c}/2) \\ S = 10^{+} 6 V_{s}' = 0.9 \\ M_{1} = A_{s}' f_{s}' (-(a_{c}/2)) \\ A = 11.24 \\ M_{1} = A_{s}' f_{s}' (-(a_{c}/2)) \\ A = 11.24 \\ M_{1} = A_{s}' f_{s}' (-(a_{c}/2)) \\ A = 11.24 \\ M_{1} = 0 M_{1} = 0.0500 \\ A = 11.24 \\ M_{2} = 0.0500 \\ C = T \\ D_{1} = 0.0500 \\ C = T \\ D_{1} = 0.0500 \\ C = T \\ D_{1} = 0.0500 \\ C = C \\ C$	
Note: If VusAcv <sup>+</sup> /(fc) can choose pt, pl according to Ch.14 = 273.2 FALSE <b>3. Required Horizontal Shear Reinforcement</b> 1/2 $\Phi$ Vc < Vu 1/2 $\Phi$ Vc = 163.9 kip Vs = Vu/(0.75) - Vc Vs = 33.5 kip Vs = Vu/(0.75) - Vc Vs = 4320 in^22 0.0025*Ag = 10.8 in^22 0.0025*Ag = 10.8 in^2 0.0025*Ag = 0.8 in^2 0.0025*Ag = 0.08 in^2 Vax. Spacing S < L/3 = 80 S = 1200 in USE S $\leq 31 = 54$ S $\leq 31 = 54$ S $\leq 318^{\circ}$ Governs # bars required = 25 pt = Av/(S*t) pt = 0.0500 > 0.0025 OK S. Design for Flexture Assume Tension-controlled section, $\Phi = 0.9$ Min = As 'fy'(d-(a/2)) = As 'fy' j jd = 0.9'd T = T 0.85*(r^a/b = As+fy' j jd = 0.9'd Min = As 'fy'(d'(a/2)) = As 'fy' j jd = 0.9'd T = T 0.85*(r^a/b = As+fy' j jd = 0.9'd Min = $\Phi$ As 'fy' j' d As = 12.211 in^2 Check Capacity: jd = 11.24 in^2 cu = 0.0 ct = cu' ct = 0.0500 > (pt) <sub>min</sub> = 0.0025 OK CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2) WALL D plprod. = 0.0722 > (p1) <sub>min</sub> = 0.0025 OK CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $\Phi$ V <sub>n</sub> $\leq A_{cv} (\alpha c^{+} f_c^{+} + p_r f_v) \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK CHECK KINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2) WALL D \PhiVn \leq A_{cv} (\alpha c^{+} f_c^{+} + p_r f_v) \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) \PhiVn \leq A_{cv} (\alpha c^{+} f_c^{+} + p_r f_v) \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE RPROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY \Phi where \Phi = 0.900 (ACI 318-08 Fig. R9.3.2)CHECK BOUNDARY ZONE REQUIREMENTSAN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 2121.9.6.3, and 21.9.6.5(a) PROVIDED THATc < (L*H) / (600 d_j) for ACI 21.9.6.2 apply c < 57.14 in. No Boundary Element Needwhere c = 13 in. (distance from the extreme compression fiber to neutral axis at Pa & Midi$	pl≥pt is <b>OK</b>
3. Required Horizontal Shear Reinforcement $1/2\Phi V_{C} < V_{U}$ $1/2\Phi V_{C} = 163.9$ kip According to 11.9.9 $V_{S} = VU(0.75) \cdot V_{C}$ $V_{S} = 33.5$ kip $V_{S} = (Av'fy'd)/V_{S}$ $Ag = 4320$ in^2 $0.0025'Ag = 10.8$ in^2 $0.0025'Ag = 0.44$ in^2 $0.0025'Ag = 0.44$ in^2 $V_{S} = 31.8'$ Governs # bars required = 25 $pt = Av/(S^{+})$ $pt = 0.0500$ > 0.0025 OK 5. Design for Flexture Assume Tension-controlled section, $\Phi = 0.9$ Mm = As $f_{Y}(1-(a_{2})) = As^{+}f_{Y}$ $jd = 0.9^{-}M$ $a = 11.87$ in C =T jd = d - (a/2) $jd = 172.80$ in C + Check Capacity: $CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)$ $WALL C plgrood = 0.05500 > (pt)_{min} = 0.0025 OKCHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)0V_{n} \le A_{c_{n}} (ac^{+}f_{c_{n}}^{+} + p_{n}f_{n}) ac = 2 (conservative) 22680 kips > V_{u} = 353 OKCHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)plgrood = 0.0722 > (pl)_{min} = 0.0025 OKCHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)0V_{n} \le A_{c_{n}} (ac^{+}f_{c_{n}}^{+} + p_{n}f_{n}) ac = 2 (conservative) 22680 kips > V_{u} = 353 OKCHECK BOUNDARY ZONE REQUIREMENTSAN EXEMPTION FROM THA TAN AXIAL LOAD Pu IS GIVEN BY0 M_{m} = 112.980 kip-ft > Mu 9.415 OKwhere 0 = 0.900 (ACI 318-08 Fig. R9.3.2)CHECK BOUNDARY ZONE REQUIREMENTSAN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-08 Fig. R9.3.2)CHECK BOUNDARY ZONE REQUIREMENTSAN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 2121.9.6.3, and 21.9.6.5(a) PROVIDED THATc < (L^{+}) / (600 d_{a}) for ACI 21.9.6.2 apply c < 57.14 in. No Boundary Element Needwhere c = 13 in (distance from the extreme compression fiber to neutral axis at Pa, 8, Mad_{a} = 2.6 in. (design displacement, assume 0.007+H conservative, see ACI 318-08 21.94$	
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TRY#6Abar=0.44in^2Max. SpacingS $\leq 1/3 = 80$ S=12.00inUSES $\leq 31 = 54$ S1110.0500> 0.0025OKpt = Av/(St)pt=0.0500> 0.0025OKS. Design for FlextureAssume Tension-controlled section, $\Phi = 0.9$ Mn = $\Delta sfy'(d-(a/2)) = As^+fy'j$ jd = 0.9°dTCheck Capacity:jd = d - (a/2)jd =jd = d - (a/2)jd = 1124in^2Check Capacity:jd = 0.0500>(pt )min. =0.0025OKWALL Cptiorond =0.0500>(pt )min. =0.0025OKWALL Cptiorond =0.0500>(ptiorond =0.0500>(ptiorond =0.0500>(pt )min. =0.0025OKWall COKWall COUT22>(pt )min. =0.0025OKOKOKOKOKOK<	
Max. Spacing $S ≤ L/3 = 80$ $S = 12.00$ in USE S ≤ 3t = 54 S ≤ 18" Governs # bars required = 25 $pt = Av/(S^{+})$ $pt = 0.0500 > 0.0025$ OK 5. Design for Flexture Assume Tension-controlled section, $\Phi = 0.9$ Mn = As*fy*(d-(a/2)) = As*fy*j jd = 0.9*d T $C^{-T}$ 0.85*fC*a*b = As*fy jd = 172.80 in Mu = $\Phi$ Mn = $\Phi$ As*fy*j*d As = 12.11 in ^2 Check Capacity: a = 11.87 in C=T jd = d - (a/2) jd = 186.06 in As = 11.24 in ^2 Eu = 0.0 ECECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2) WALL C $ptprovd. = 0.0500$ > $(pt J_{min.} = 0.0025$ OK WALL C $ptprovd. = 0.0722$ > $(pl J_{min.} = 0.0025$ OK WALL C $ptprovd. = 0.0722$ > $(pl J_{min.} = 0.0025$ OK WALL C CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $\Phi V_n ≤ A_{cv}$ (ac*Nf_c' + $p_nf_y$ ) $\alpha c = 2$ (conservative) 22680 kips > $V_u = 353$ OK CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL LOAD $P_u$ IS GIVEN BY $\Phi M_n = 112.980$ kip-ft > $M_u = 9.415$ OK where $\Phi = 0.900$ (ACI 318-08 Fig. R9.3.2) CHECK DINDRAPY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY $\Phi M_n = 112.980$ kip-ft > $M_u = 9.415$ OK where $\Phi = 0.900$ (ACI 218-08 Fig. R9.3.2) CHECK BUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT $c < (1*H) / (600 d_u)$ for ACI 21.9.6.2 apply $c < 57.14$ in. No Boundary Element Need where $c = 13$ in. (distance from the extreme compression fiber to neutral axis at Pu, & M_u $d_u = 2.6$ in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.4	
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S. Design for Flexture Assume Tension-controlled section, $\Phi = 0.9$ Mn = As fly'(d-(a/2)) = As fly'i jd = 0.9'd C=T 0.85 ffc*a*b = As fly Mu = $\Phi$ Mn = $\Phi$ As fly'j*d As = 1172.80 in T jd = d - (a/2) jd = 1187.0 in C=T jd = d - (a/2) jd = 1186.06 in As = 11.124 in ^2 Check Capacity: CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2) WALL C ptprovd. = 0.0500 > (pt ) <sub>min</sub> . = 0.0025 OK WALL C ptprovd. = 0.0722 > (pl ) <sub>min</sub> . = 0.0025 OK WALL C ptprovd. = 0.0722 > (pl ) <sub>min</sub> . = 0.0025 OK WALL C CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $\Phi V_n \le A_{cv} (ac^{+N}f_c' + \rho_{v}f_y) \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK$ CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL LOAD Pu IS GIVEN BY $\Phi$ M <sub>n</sub> = 112.980 kip-ft > M_u = 9.415 OK where $\Phi = 0.900$ (ACI 318-08 Fig. R9.3.2) CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION ROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT $c < (1^{+H}) / (600 d_u)$ for ACI 21.9.6.2 apply $c < 57.14$ in. No Boundary Element Need where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at Pu & 8.40 where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at Pu & 8.40 where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at Pu & 8.40 where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at Pu & 8.40 where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at Pu & 8.40 where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at Pu & 8.40 where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at Pu & 8.40 where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at Pu & 8.40 where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at Pu & 8.40 where $c = 1.3$ in. (distance from the extreme compression fiber to neutral ax	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As'fy'(d-(a/2)) = As'fy'j jd = 0.9°d C=T 0.85°fc*a*b = As'fy j jd = 0.9°d As = 11211 in^2 Check Capacity: a = 11.87 in C=T jd = d - (a/2) jd = 186.06 in As = 11.24 in^2 cu = 0.0 ct = cu' CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2) VALL C ptprovd. = 0.0500 > (pt ) <sub>min</sub> . = 0.0025 OK WALL C ptprovd. = 0.0722 > (pl ) <sub>min</sub> . = 0.0025 OK CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $\Phi V_n \le A_{cv} (\alpha c^* v_1 c'_1 + p_e f_y) \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK$ CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY $\Phi M_n = 112,980 kip-ft > M_u = 9,415 OK$ where $\Phi = 0.900$ (ACI 318-08 Fig. R9.3.2) CHECK BIONARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT $c < (L^+H) / (600 d_u)$ for ACI 21.9.6.2 apt) $c < 57.14$ in. No Boundary Element Need where $c = 1.3$ in. (distance from the extreme compression fiber to neutral axis at P_u & M_u d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.4	
Assume Tension-controlled section, $\Phi = 0.9$ $Mn = As^4fy'(d-(A/2)) = As^4fy'j$ jd = 0.9*d         T Z=T         0.85*fc*a*b = As^4fy         As 's 's'' j' d = 0.9*d         As 's 's'' j' d = 0.9*d         As 's 's'' j' d = 0.0*d         As 's 's 's'' j' d = 0.0*d         As 's 's 's'' j' d = 0.0*d         As 's 's'' j' d = 0.0*d         As 's 's 's'' j' d = 0.0*d         Second (ACI 318-08 14.3, 21.9.2)         VALL 0         As 's 's 's'' (ACI 318-08 11.2 & 21.9.4)         ptrovd. = 0.0722 > (pl) <sub>min.</sub> = 0.0025 OK         Vu = 0.0025         OK         CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $\Phi V_n \le A_{cv} (\alpha c^* \sqrt{t}_c + \rho_e f_y)  \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK         CHECK FLEXURAL & AXIAL CAPACITY         THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY         \Phi M_n = 112,980 kip-ft > M_u = 9,415 OK         where \Phi = 0.900 (ACI 318-08 Fig. R9.3.2)         CHECK BIDARARY ZONE REQUIREMENTS         AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21         21.9.6.3, and 21.9.6.5(a) PROVIDED THAT         c < (L*H) / (600 d_u) for ACI 21.9.6.2 apr) c < 57.14 in. No Boundary Element Need         where c = 13 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u         d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 12.9.4         Si = 0.000*H conservative, see ACI 318-08 21.9.4         Si = 0.000*H conservativ$	
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As=11.24in^2 $\mathfrak{eu} = 0.0$ $\mathfrak{et} = \mathfrak{eu}^2$ CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)WALL C $\mathfrak{p}_{lprovd.} = 0.0500 > (pt)_{min.} = 0.0025 OKplprovd.=0.0722 > (pl)_{min.} =0.0025 OKWALL Cplprovd.=0.0722 > (pl)_{min.} =0.0025 OKCHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)\Phi V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_v f_y)  \alpha c = 2 (conservative) 22680 kips > V_u =353 OKCHECK FLEXURAL & AXIAL CAPACITYTHE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY\Phi M_n =11.2,980 kip-ftM_u =9,415 OKWhere \Phi =0.9000 (ACI 318-08 Fig. R9.3.2)CHECK BOUNDARY ZONE REQUIREMENTSAN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 2121.9.6.5.(a) PROVIDED THATc < (L*H) / (600 d_u) for ACI 21.9.6.2 apply$	0.85*f'c*a*b =As*fy c = 12.97 in
As=11.24in^2 $\mathfrak{eu} = 0.0$ $\mathfrak{et} = \mathfrak{eu}^2$ CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)WALL C $\mathfrak{p}$ (provd. = $\mathfrak{p}$ (provd. =0.0500>(pt ) min. =0.0025OKWALL C $\mathfrak{p}$ $\mathfrak{p}$ (provd. =0.0722>(pl ) min. =0.0025OKWALL C <b>CHECK SHEAR CAPACITY (ACI 318-08 11.2 &amp; 21.9.4)</b> $\Phi$ V <sub>n</sub> $\leq$ Acy ( $\alpha c^{+}\sqrt{f_c} + \mathfrak{p}_r f_y$ ) $\alpha c = 2$ (conservative) 22680kips> $V_u =$ 353OKCHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL LOAD P <sub>u</sub> IS GIVEN BY $\Phi$ M <sub>n</sub> =112.980kip-ftMu_u =9.415OKCHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL LOAD P <sub>u</sub> IS GIVEN BY $\Phi$ M <sub>n</sub> =112.980kip-ftMu_u =9.415OKCHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21 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 Need $where c = 13$ in. (distance from the extreme compression fiber to neutral axis at P <sub>u</sub> & M_u $d_u = 2.6$ in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.1	c = a/0.85
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CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)       WALL IC         phyroud. = 0.0500 > (pt )min. = 0.0025 OK       WALL IC         phyroud. = 0.0722 > (pl )min. = 0.0025 OK         CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $\Phi V_n < A_{cv} (\alpha c^* h_{c}^+ + p_r f_y)  \alpha c = 2 (conservative) 22680 kips > V_u = 353 OK         CHECK FLEXURAL & AXIAL CAPACITY         THE ALLOWABLE MOMENT AT AN AXIAL LOAD Pu IS GIVEN BY         \Phi M_n = 112,980 kip-ft > M_u = 9,415 OK         where \Phi = 0.900 (ACI 318-08 Fig. R9.3.2)         CHECK BOUNDARY ZONE REQUIREMENTS         AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21         21.9.6.3 and 21.9.6.5(a) PROVIDED THAT         c < 57.14$	
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THE ALLOWABLE MOMENT AT AN AXIAL LOAD P <sub>u</sub> IS GIVEN BY $\Phi$ M <sub>n</sub> = 112,980 kip-ft > M <sub>u</sub> 9,415 OK where $\Phi$ 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 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 Need where $c = 13$ in. (distance from the extreme compression fiber to neutral axis at P <sub>u</sub> & M <sub>n</sub> $d_u = 2.6$ in. (design displacement, assume 0.007 <sup>+</sup> H conservative, see ACI 318-08 21.9.4	a
where 0 =       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         21.9.6.3, and 21.9.6.5(a) PROVIDED THAT         c < (1*H) / (600 d <sub>u</sub> ) for ACI 21.9.6.2 apply       c < 57.14	b
CHECK BOUNDARY ZONE REQUIREMENTS         AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21 21.96.3, and 21.9.6.5(a) PROVIDED THAT         c < (L*H) / (600 d <sub>u</sub> ) for ACI 21.9.6.2 apply         c < 57.14	
AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21 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 Need where $c = 13$ in. (distance from the extreme compression fiber to neutral axis at P <sub>u</sub> & M <sub>n</sub> $d_u = 2.6$ in. (design displacement, assume 0.007 <sup>+</sup> H conservative, see ACI 318-08 21.9.4	$\longrightarrow$
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	
$ \begin{array}{rrrr} c < (L^+H) / (600 \ d_u) \ \ \ \ or \ \ ACI \ 21.9.62 \ apply \ \ \ c < 57.14 \ \ \ \ or \ \ No \ \ Boundary \ Element \ Need \\ where \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ $	.9.6.2,
where         c         =         13         in. (distance from the extreme compression fiber to neutral axis at $P_u \& M_n$ $d_u$ =         2.6         in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.94	$\longrightarrow$
d <sub>u</sub> = 2.6 in. ( design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6	ed
d <sub>u</sub> = 2.6 in. ( design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6	loads. )
10000 -	
(Pmax) (Pma	ax)
	I
fs=0f	
	's=0
	's=0
fs=0.5fy	
$\wedge$ $\wedge$ $\uparrow$ $/$	fs=0 fs=0.5fy
	rs=0.5fy
	rs=0.5fy
-40000	rs=0.5fy

(Pmin)

-5000

(Pmin)

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INPUT DATA & DESIGN SUMMARY	W	/all 5 b	)			X-Direction							
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D +	+ 1.0L + 1.0E		Pu	=	463.0	k	
REBAR YIELD STRESS		-	60	ksi		FACTORED MOMEN			M	-	17087.8		
HEIGHT OF WALL -	,		462.0						-				
				in		FACTORED SHEAR LO	JAD		Vu	=	405.0	к	
LENGTH OF SHEAR WALL			240.0	in									
THICKNESS OF WALL	t	=	18	in		THE WALL	DESIGN IS ADEQUA	TE.					
	Acv	=	4320	in^2									
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*v(f'c) ; need a	t least two curtains (rows	s) =	546.4	Need 1									
1. Check Permitted Shear Strength						4. Required Vertical	Shear Reiforcement						
ACI 318-08 § 11.9						ρl = Av/S*h ≥ 0.0025	+0.5 (2.5 - h/L)*(pt-0.	0025)	ρl	=	0.09584	> 0.0025	ок
ΦVn ≥ Vu	Vu	=	405.0	kip		Max. Spacing	S ≤ L/3 =	80	S	=	6	in	
Vn = Vc + Vs			192.0	in				54					
Vn ≤ 10*t*d*√(f'c) d=0.8*L			2185.8	kip			S ≤ 18"	Governs					
VII 3 10 ( d V(i c) - d=0.0 E								#11	A /l= = =	=	1.50	in^2	
A CT 210 00 5 21 0 4			1639.3	kip			TRY		A/bar			in <sup>2</sup>	
ACI 318-08 § 21.9.4		≤ 3	36720.0	kip					# bars required	=	7		
$V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)  \alpha c = 2 \text{ (conservative})$	)		OK			ACI 318-08 § 21.9.4.4	, IF hw/lw≤2; need rein	f. In two directions (pl	≥pt)				
2. Shear Strength Provided by Vc									h/l	=	1.9250	ρl≥ρt	
$Vc \le 2^{*}\lambda^{*}t^{*}d^{*}\sqrt{f'c}$ $\lambda = 1.0$ (for N.W.C	C) Vc	=	437.2	kip					ρl≥ρt	is	ок		
Note: If Vu≤Acv*√(f'c) can choose pt, pl accord	ing to Ch.14	=	273.2	FALSE									
3. Required Horizontal Shear Reinforcement													
1/2ΦVc < Vu	1/2ΦVc	=	163.9	kip									
,			cording t			WALL DIS	T. HORIZ. REINF.		14	#8	0	8	" O.C.
Vs = Vu/(0.75) - Vc	Vs		102.8	kip			T. VERT. REINF.		7	#11	@		" O.C.
$S = (Av^*fy^*d)/Vs$		2	4320	in^2		WALL DIS	····ENT. INERNE.		,		3	5	0.c.
3 = (AV Iý U)/VS													
	2	=	10.8	in^2									
TRY #8		-	0.79	in^2									
Max. Spacing $S \le L/3 = 80$	-	=	8.00	in	USE								
$S \leq 3t = 54$			4320.00										
S ≤ 18" Governs	# bars required	-	14										
$\rho t = Av/(S^*t)$		-	0.0750	> 0.0025	ок							FALSE	
												-	
5. Design for Flexture						1		A/bar	=	1.56	in^2		
Assume Tension-controlled section, $\Phi = 0.9$								# bars required	=	13			
						TRY	#11	Sub required	-	15			
	: 4		172.80	in		IRY	#11						
C=T 0.85*f'c*a*b =As*fy	,												
$Mu = \Phi Mn = \Phi As*fy*j*d$			21.98	in^2		Check Capacity:		а	=	20.54			
	-		21.54	in		C=T	0.85*f'c*a*b =As*fy	с	=	24.17			
jd = d - (a/2)	jd	=	181.23	in			c = a/0.85	εt	=	0.03	> 0.0025		
	As	=	20.95	in^2		εu = 0.003	dt = L-3"						
$\begin{array}{rcl} \mbox{where} & c = & 0 \\ \mbox{$d_u$} & = & 65.9 \\ \mbox{CHECK MINIMUM REINFORCEMENT RATIOS} \\ \mbox{$\rho_{lprovd.}$} & = & 0.0750 & > \\ \mbox{$\rho_{lprovd.}$} & = & 0.0958 & > \\ \end{array}$	in. ( design displace	ement, as: 318-08 1	sume 0.0	07*H cons		ral axis at P <sub>u</sub> & M <sub>n</sub> loa ee ACI 318-08 21.9.6.2a							
CHECK SHEAR CAPACITY (ACI 318-08 11.2 8	21.9.4)												
$\Phi V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y) $ ac	= 2 (conservative) 163	89.3 kip	05	>	V <sub>u =</sub>	405.0 <b>OK</b>							
CHECK FLEXURAL & AXIAL CAPACITY													
THE ALLOWABLE MOMENT AT AN AXIAL LO	DAD P <sub>u</sub> IS GIVEN BY						T. HORIZ. REINF.		14	#8	@		" O.C.
$\Phi$ M <sub>n</sub> = 205,054 kip-ft	> M <sub>u</sub>		,088	ок			T. VERT. REINF.		16	#11	@	6	" O.C.
where $\Phi$ = Min{0.9, Max[0.65 + (e <sub>t</sub> -	0.002)(250/3), 0.65]} =	= 0.9	00	(ACI 318	-08 Fig. R9	.3.2)							
	-				-								
		J.	fs=0.5fy	(Pmax)	/		(Pmax) /s=0 /s=0.5fy						
		25000	+	(Pmin)			(Pmin) Mx (P	25000 -ft)					
4/4/2012		25000	+	(Pmin)		000		-ft)	Pag	е			

Dr. Richard Behr Structural Option

		Spe	ecial Reinfo	rced Cor	ncrete Sh	ear Wall Design Base	d on ACI 318-08	Ch. 21.9					
INPUT DATA & DESIGN SUMMARY		Wall	6 a			X-Direction							
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D +	+ 1.0L +1.0E		Pu	=	1111	k	at BAS
REBAR YIELD STRESS	fy	=	60	ksi		FACTORED BASE MC			M	=	10463		
HEIGHT OF WALL	, H	=	376.0	in		FACTORED BASE SH			V	=	291		
LENGTH OF SHEAR WALL	L	=	240.0	in			Li ili Londo		• 0		2.72	ĸ	
THICKNESS OF WALL	t	=	12	in		THE WALL	DESIGN IS ADEQU	JATE.					
	Acv	=	2880	in^2									
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need a		ows) =	364.3	Need 1									
1. Check Permitted Shear Strength						4. Required Vertical	Shear Reiforcemen	nt					
ACI 318-08 § 11.9						ρI = Av/S*h ≥ 0.0025	+0.5 (2.5 - h/L)*(pt	-0.0025)	ρΙ	=	0.07217	> 0.0025	ок
ΦVn ≥ Vu	Vu	=	291.0	kip		Max. Spacing	S ≤ L/3 =	80	S	=	12	in	
Vn = Vc + Vs	d	=	192.0	in			S ≤ 3t =	36					
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1457.2	kip			S ≤ 18"	Governs					
	ΦVn	=	1092.9	kip			TR		A/bar	=	0.44	in^2	
ACI 318-08 § 21.9.4	Vn	≤	20160.0	kip					# bars required	=	24		
$V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)  \alpha c = 2 \text{ (conservative})$			ОК			ACI 318-08 § 21.9.4.4	, IF hw/lw≤2; need r	einf. In two directions (p					
2. Shear Strength Provided by Vc									h/l	=	1.5665	ρl≥ρt	
$Vc \le 2^*\lambda^*t^*d^*\sqrt{f'c}$ $\lambda = 1.0$ (for N.W.6	.C) Vc	=	291.4	kip					ρl≥ρt	is	ОК		
Note: If Vu≤Acv*√(f'c) can choose pt, pl accord		=	182.1	FALSE									
3. Required Horizontal Shear Reinforcement													
1/2ΦVc < Vu	1/2ΦVc	=	109.3	kip									
			According t	to 11.9.9		WALL DIS	T. HORIZ. REINF.		16	#6	@	12	" O.C.
Vs = Vu/(0.75) - Vc	Vs	=	96.6	kip		WALL DIS	T. VERT. REINF.		24	#6	@	12	" O.C.
S = (Av*fy*d)/Vs	Ag	=	2880	in^2									
	0.0025*Ag	=	7.2	in^2									
<b>TRY</b> #6	Abar	=	0.44	in^2									
Max. Spacing $S \le L/3 = 80$	S	=	12.00	in	USE								
S ≤ 3t = 36													
$S \le 18$ " Governs	# bars required	=	16										
$\rho t = Av/(S^*t)$	ρt	=	0.0500	>0.0025	ок								
5. Design for Flexture													
Assume Tension-controlled section, $\Phi = 0.9$													
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d						TRY	#7	A/bar	=	0.6	in^2		
$ v_{11} - n_{2} y_{1}(u_{1}(d/2)) = n_{2} y_{1}  =  0  = 0.9^{\circ} 0$	id	=	172.80	in				# bars required	=	21			
$C=T = 0.85 \text{ f}^{-}(a^{-}(a^{-}2)) = As^{-}(y^{-})$ $Ja = 0.9^{-}a$	ju												
	As	=	13.46	in^2		Check Capacity:		а	=	18.78	in		
C=T 0.85*f'c*a*b =As*fy	,	= =	13.46 19.79	in^2 in		Check Capacity: C=T	0.85*f'c*a*b =As*f		=	18.78 22.09	in in		
C=T 0.85*f'c*a*b =As*fy	As						0.85*f'c*a*b =As*f c = a/0.85					ок	
$C=T \qquad 0.85^{+t}c^*a^*b = As^*fy$ Mu = $\Phi$ Mn = $\Phi$ As <sup>*</sup> fy <sup>*</sup> d	As a	=	19.79	in			c = a/0.85	у с	=	22.09	in	ок	
$C=T \qquad 0.85^{+t}c^*a^*b = As^*fy$ Mu = $\Phi$ Mn = $\Phi$ As <sup>*</sup> fy <sup>*</sup> d	As a jd	= =	19.79 182.11	in in		C=T	c = a/0.85 dt = L-3"	у с	= =	22.09	in	ок	
$C=T \qquad 0.85^{+t}c^*a^*b = As^*fy$ Mu = $\Phi$ Mn = $\Phi$ As <sup>*</sup> fy <sup>*</sup> d	As a jd As	= =	19.79 182.11 12.77	in in in^2		C=T εu = 0.003 εt = εu*((d	c = a/0.85 dt = L-3"	у с	= =	22.09 0.03	in	<b>ОК</b> 12	" O.C.
C=T 0.85*fc*a*b =As*fy $Mu = \Phi Mn = \Phi As*fy^d$ *d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIOS	As a jd As	= =	19.79 182.11 12.77	in in in^2	ок	C=T εu = 0.003 εt = εu*((d WALL DIS	c = a/0.85 dt = L-3" lt-c)/c)	у с	=	22.09 0.03 Wall 1	in >0.0025		" O.C. " O.C.
$C=T \qquad 0.85^{*f}c^*a^*b = As^*fy$ $Mu = \Phi Mn = \Phi As^*fy'j^*d$ $jd = d - (a/2)$ CHECK MINIMUM REINFORCEMENT RATIOS $\rho_{tprovet.} = 0.0500 > $	As a jd As S AND SPACING (Ad (pt ) <sub>min.</sub> =	= =	19.79 182.11 12.77 08 14.3, 21.9 0.0025	in in in^2		C=T εu = 0.003 εt = εu*((d WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
C=T 0.85*fc*a*b =As*fy $Mu = \Phi Mn = \Phi As*fy^d$ *d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIOS	As a jd As S AND SPACING (Ad	= =	19.79 182.11 12.77 08 14.3, 21.9	in in in^2	ок ок	C=T εu = 0.003 εt = εu*((d WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$C=T  0.85^{*f}c^*a^*b = As^*fy$ $Mu = \Phi  Mn = \Phi  As^*fy^*j^*d$ $jd = d - (a/2)$ $CHECK  MINIMUM  REINFORCEMENT  RATIOS$ $\rho tprovd. = 0.0500  >$ $\rho tprovd. = 0.0722  >$	As a jd As S AND SPACING (A (ρt ) <sub>min.</sub> = (ρl ) <sub>min.</sub> =	= =	19.79 182.11 12.77 08 14.3, 21.9 0.0025	in in in^2		C=T εu = 0.003 εt = εu*((d WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$C=T   0.85^{*f}c^*a^*b = As^*fy$ $Mu = \Phi   Mn = \Phi   As^*fy^*j^*d$ $jd = d - (a/2)$ $CHECK   MINIMUM   REINFORCEMENT   RATIOS$ $\rho_{tprovel.} =   0.0500   >$ $\rho_{provel.} =   0.0722   >$ $CHECK   SHEAR   CAPACITY  (ACI   318-08   11.2   40)$	As a jd As 5 AND SPACING (A( (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = & 21.9.4)	= = = CI 318-0	19.79 182.11 12.77 08 14.3, 21.9 0.0025 0.0025	in in in^2 <b>9.2)</b>	ок	C=T εu = 0.003 εt = εu*((d WALL DIS WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$C=T   0.85^{*f}c^*a^*b = As^*fy$ $Mu = \Phi   Mn = \Phi   As^*fy^*j^*d$ $jd = d - (a/2)$ $CHECK   MINIMUM   REINFORCEMENT   RATIOS$ $\rho_{tprovel.} =   0.0500   >$ $\rho_{provel.} =   0.0722   >$ $CHECK   SHEAR   CAPACITY  (ACI   318-08   11.2   40)$	As a jd As S AND SPACING (A (ρt ) <sub>min.</sub> = (ρl ) <sub>min.</sub> =	= = = CI 318-0	19.79 182.11 12.77 08 14.3, 21.9 0.0025	in in in^2		C=T εu = 0.003 εt = εu*((d WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$\begin{array}{llllllllllllllllllllllllllllllllllll$	As a jd As 5 AND SPACING (A( (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = & 21.9.4)	= = = CI 318-0	19.79 182.11 12.77 08 14.3, 21.9 0.0025 0.0025	in in in^2 <b>9.2)</b>	ок	C=T εu = 0.003 εt = εu*((d WALL DIS WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$\begin{array}{llllllllllllllllllllllllllllllllllll$	As a jd As <b>5 AND SPACING (A</b> ( (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = & 21.9.4) c = 2 (conservative)	= = CI 318-C	19.79 182.11 12.77 08 14.3, 21.9 0.0025 0.0025	in in in^2 <b>9.2)</b>	ок	C=T εu = 0.003 εt = εu*((d WALL DIS WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$C=T \qquad 0.85^{*f}c^{*a^*b} = As^{*fy}$ $Mu = \Phi Mn = \Phi As^{*fy}j^{*d}$ $jd = d - (a/2)$ $CHECK MINIMUM REINFORCEMENT RATIOS$ $\rho tprovd. = 0.0500 >$ $\rho tprovd. = 0.0722 >$ $CHECK SHEAR CAPACITY (ACI 318-08 11.2 4)$ $\Phi V_n \le A_{cv} (\alpha c^* \sqrt{t_c^*} + \rho_r f_y) \qquad \alpha c$ $CHECK FLEXURAL & AXIAL CAPACITY$ $THE ALLOWABLE MOMENT AT AN AXIAL LOWABLE AXIAL $	As a jd As 5 AND SPACING (Ad (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = & 21.9.4) c = 2 (conservative)	= = = <b>CI 318-C</b> 15120	19.79 182.11 12.77 08 14.3, 21.9 0.0025 0.0025 kips	in in in^2 <b>3.2)</b>	ок	C=T εu = 0.003 εt = εu*((d WALL DIS WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$\begin{array}{llllllllllllllllllllllllllllllllllll$	As a jd As <b>5 AND SPACING (A</b> (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = & 21.9.4) c = 2 (conservative) CAD P <sub>u</sub> IS GIVEN B > 1	= = CI 318-C 15120 3Y M <sub>u</sub> =	19.79 182.11 12.77 08 14.3, 21.9 0.0025 0.0025	in in in^2 <b>3.2)</b>	ок	C=T εu = 0.003 εt = εu*((d WALL DIS WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$C=T \qquad 0.85^{*f}c^*a^*b = As^*fy'$ $Mu = \Phi Mn = \Phi As^*fy'j^*d$ $jd = d - (a/2)$ $CHECK MINIMUM REINFORCEMENT RATIOS$ $ptproved. = 0.0500 >$ $ptproved. = 0.0722 >$ $CHECK SHEAR CAPACITY (ACI 318-08 11.2 d)$ $\Phi V_n \leq A_{cv} (\alpha c^* \sqrt{f_c^*} + \rho_r f_y) \qquad \alpha c$ $CHECK FLEXURAL & AXIAL CAPACITY$ $THE ALLOWABLE MOMENT AT AN AXIAL LS$ $\Phi M_n = 125,555 \qquad kip-ft$ $where \Phi = 0.900$	As a jd As 5 AND SPACING (Ad (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = & 21.9.4) c = 2 (conservative)	= = CI 318-C 15120 3Y M <sub>u</sub> =	19.79 182.11 12.77 08 14.3, 21.9 0.0025 0.0025 kips	in in in^2 <b>3.2)</b>	ок	C=T εu = 0.003 εt = εu*((d WALL DIS WALL DIS	c = a/0.85 dt = L-3" lt-c)/c) <b>T. HORIZ. REINF.</b>	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$C=T \qquad 0.85^{+p}c^{+}a^{+}b = As^{+}fy'$ $Mu = \Phi Mn = \Phi As^{+}fy'j^{+}d$ $jd = d - (a/2)$ $CHECK MINIMUM REINFORCEMENT RATIOS$ $ptproved. = 0.0500 >$ $ptproved. = 0.0722 >$ $CHECK SHEAR CAPACITY (ACI 318-08 11.2 d)$ $\Phi V_n \leq A_{cv} (\alpha c^{+}\sqrt{f_c'} + \rho_{tr}f_y) \qquad \alpha c$ $CHECK FLEXURAL & AXIAL CAPACITY$ $THE ALLOWABLE MOMENT AT AN AXIAL LOP \Phi M_n = 125,555  kip-ft where \Phi = 0.900 CHECK BOUNDARY ZONE REQUIREMENTS$	As a jd As <b>5 AND SPACING (A</b> (4 (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = <b>&amp; 21.9.4</b> ) c = 2 (conservative) CAD P <sub>u</sub> IS GIVEN B > 1 (ACI 318-08 Fig. R	= = CI 318-C 15120 3Y M <sub>u</sub> = 89.3.2)	19.79 182.11 12.77 08 14.3, 21.5 0.0025 0.0025 kips	in in^2 Э.2) > ОК	OK V <sub>u =</sub>	C=T EU = 0.003 ET = EU <sup>4</sup> (Id WALL DIS WALL DIS 291 OK	c = a/0.85 dt = L-3" lt-c)/c) T. HORIZ. REINF. T. VERT. REINF.	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$C=T  0.85^{\rm eff}c^{\rm ea}b = As^{\rm eff}y$ $Mu = \Phi Mn = \Phi As^{\rm eff}y^{\rm eff}d$ $jd = d - (a/2)$ $CHECK MINIMUM REINFORCEMENT RATIOS$ $\rho_{\rm Iproved.} = 0.0500 >$ $\rho_{\rm Iproved.} = 0.0722 >$ $CHECK SHEAR CAPACITY (ACI 318-08 11.2 8$ $\Phi V_n \le A_{\rm ex} (\alpha c^{\rm eff} d_c^{\rm eff} + \rho_{\rm eff} d_{\rm y})  \alpha cc$ $CHECK FLEXURAL & XIAL CAPACITY$ $THE ALLOWABLE MOMENT AT AN AXIAL LG \Phi M_n = 125,555  kip-ft where \Phi = 0.900 CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF$	As a jd As <b>5 AND SPACING (A</b> (4 (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = <b>&amp; 21.9.4</b> ) c = 2 (conservative) CAD P <sub>u</sub> IS GIVEN B > 1 (ACI 318-08 Fig. R	= = CI 318-C 15120 3Y M <sub>u</sub> = 89.3.2)	19.79 182.11 12.77 08 14.3, 21.5 0.0025 0.0025 kips	in in^2 Э.2) > ОК	OK V <sub>u =</sub>	C=T EU = 0.003 ET = EU <sup>4</sup> (Id WALL DIS WALL DIS 291 OK	c = a/0.85 dt = L-3" lt-c)/c) T. HORIZ. REINF. T. VERT. REINF.	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$\begin{array}{llllllllllllllllllllllllllllllllllll$	As a jd As <b>5 AND SPACING (A</b> ( (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = <b>&amp; 21.9.4</b> ) c = 2 (conservative) COAD P <sub>u</sub> IS GIVEN B COAD P <sub>u</sub> IS GIVEN B S I (ACI 318-08 Fig. R F BOUNDARY ZONE	= = CI 318-C 15120 3Y M <sub>u</sub> = {9.3.2} CONFIN	19.79 182.11 12.77 08 14.3, 21.9 0.0025 0.0025 0.0025 kips 10,463	in in^2 <b></b>	OK V <sub>u =</sub>	C = T EU = 0.003 Et = EU*((d WALL DIS WALL DIS 291 OK 291 OK	c = a/0.85 dt = L-3" lt-c)/c) T. HORIZ. REINF. T. VERT. REINF.	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	
$C=T  0.85^{\rm eff}c^{\rm ea}b = As^{\rm eff}y$ $Mu = \Phi Mn = \Phi As^{\rm eff}y^{\rm eff}d$ $jd = d - (a/2)$ $CHECK MINIMUM REINFORCEMENT RATIOS$ $\rho_{\rm Iproved.} = 0.0500 >$ $\rho_{\rm Iproved.} = 0.0722 >$ $CHECK SHEAR CAPACITY (ACI 318-08 11.2 8$ $\Phi V_n \le A_{\rm ex} (\alpha c^{\rm eff} d_c^{\rm eff} + \rho_{\rm eff} d_{\rm y})  \alpha cc$ $CHECK FLEXURAL & XIAL CAPACITY$ $THE ALLOWABLE MOMENT AT AN AXIAL LG \Phi M_n = 125,555  kip-ft where \Phi = 0.900 CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF$	As a jd As <b>5 AND SPACING (A</b> ( (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = <b>&amp; 21.9.4</b> ) <b>:</b> COAD P <sub>u</sub> IS GIVEN B COAD P <sub>u</sub> IS GIVEN B (ACI 318-08 Fig. R F BOUNDARY ZONE (ACI 318-08 Fig. R	= = = CI 318-C 15120 3Y M <sub>u</sub> = 89.3.2) CONFIN c <	19.79 182.11 12.77 8 14.3, 21.9 0.0025 0.0025 0.0025 kips 10.463 EMENT REIN 57.14	in in^2 Э.2) > ОК ИFORCEM in.	OK V <sub>u</sub> = ENT IS GIV No Bour	C=T EU = 0.003 ET = EU <sup>4</sup> (Id WALL DIS WALL DIS 291 OK	c = a/0.85 dt = L-3" it-c)/c) T. HORIZ.REINF. T. VERT. REINF.	у с	= = 16	22.09 0.03 Wall 1 #6	in >0.0025 @	12	

	DESIGN SUM	IMARY		Wall	6b			X-Direction							
CONCRETE STRE			f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D -	+ 1.0L +1.0E		Pu	=	1165.1	k	
REBAR YIELD STE		,	f <sub>v</sub>	=	60	ksi		FACTORED MOMEN			M	=	10224.2		
HEIGHT OF WAL			'y H	_	462.0	in		FACTORED SHEAR L			V.	=	1022-4.2		
ENGTH OF SHE			1	_	240.0	in		TACTORED STIEAR D	ORD		¥u	-	101.2	ĸ	
HICKNESS OF			t	=	240.0	in		TUE 14/411		TE					
HICKNESS OF	WALL			=				THE WAL	L DESIGN IS ADEQUA	ATE.					
			Acv		2880	in^2									
			at least two curtains (	rows) =	364.3	Need 1									
1. Check Permit		ength							Shear Reiforcement						
ACI 318-08 § 11.9									5 +0.5 (2.5 - h/L)*(pt-0		ρΙ	=		> 0.0025	ок
ΦVn ≥			Vu	=	101.2	kip		Max. Spacing	S ≤ L/3 =	80	S	=	6	in	
Vn = V	c + Vs		d	=	192.0	in			S ≤ 3t =	36					
Vn ≤ 1	0*t*d*√(f'c)	d=0.8*L	Vn	=	1457.2	kip			S ≤ 18"	Governs					
			ΦVn	=	1092.9	kip			TRY	#11	A/bar	=	1.56	in^2	
ACI 318-08 § 21.9	9.4		Vn	≤	67689.0	kip					# bars required	=	19		
/n ≤ A <sub>cv</sub> (αc*√f <sub>c</sub> '	+ ρ <sub>t*</sub> f <sub>v</sub> ) αc	= 2 (conservative	2)		OK			ACI 318-08 § 21.9.4.4	, IF hw/lw≤2; need rei	nf. In two directions (					
2. Shear Strengt			,		-			1			h/l	=	1.9250	ol>ot	
		$\lambda = 1.0$ (for N.W.0	C) Vc	=	291.4	kip					ρl≥ρt	is	OK		
		oose pt, pl accord		_	182.1		ng to Ch.14				PPr	13	UK.		
		Reinforcement		-	102.1	Accord	g to C11.14								
2. κειμίτεα ποι 1/2ΦVα		Kennorcement	1/2ΦVc	=	109.3	kip									
1/2010	. < vu		1/2010	-	Reinf. Accor		Ch 14		T. HORIZ. REINF.		40	#8	0	8	" 0.0
N/- N/	u/(0.75) - Vc		Vs	=	Reint. Accol		Ch 14	-	T. VERT. REINF.		40	#0 #11	@ @		" O.
						kip		WALL DIS	I. VERI. REINF.		19	#11	w	0	0.0
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 =		S	=	8.00	in	USE								
		36	Av	=	2880.00										
	S ≤ 18"	Governs	# bars required	=	40										
ρt = Av	//(S*t)		ρt	=	0.3251	> 0.002	5 <b>OK</b>							FALSE	
5. Design for Fle	exture									A/bar		0.79	in^2		
Assume Tension		ection $\Phi = 0.9$								# bars required	-	16			
Mn = As*fy*(d-(a								TRY	#8		-	10			
	*a*b =As*fy	ju = 0.5 u	jd	=	172.80	in		IKI	"0						
C=1 0.85*1C Mu = Φ Mn = Φ			Ja As	-	172.80	in in^2		Check Capacity:		а	=	18.33	in		
$v_{10} = \Psi v_{10} = \Psi$	As Ty J'd			-	13.15	in^2		Cneck Capacity: C=T	0.05+6-+-+- * **		-	18.33 21.56	in		
	( 10)		a					C=1	0.85*f'c*a*b =As*fy	c					
jd = d	- (a/2)		jd	=	182.33	in			c = a/0.85	εt	=	0.03	>0.0025		
			As	=	12.46	in^2		εu = 0.003							
								εt = εu*((d	t-c)/c)		1	Wall 1			
CHECK BOUND		•													
			BOUNDARY ZONE	E CONFIN	IEMENT REIN	IFORCEN	IENT IS GIVE	N BY ACI318-05 21.9.6	5.2,						
21.9.6.3, and	21.9.6.5(a) PR	OVIDED THAT													
c < (L*I	H) / (600 d <sub>u</sub> )	for ACI 21.9.6.2 a	pply	c <	57.14	in.	No Bound	ary Element Needed							
wher	re c =	22	in. ( distance fro	om the ex	treme comp	ression f	iber to neutr	al axis at P <sub>u</sub> & M <sub>n</sub> loa	ids. )						
	d., =	3.2	in. ( design disp	lacemen	t, assume 0.0	07*H coi	nservative, se	e ACI 318-08 21.9.6.2	a. )						
			AND SPACING (A				,								
		>	(ρt ) <sub>min.</sub> =		0.0025		ок								
CHECK MINIMU	- 0.2251	,													
CHECK MINIMU	= 0.3251		(ρl) <sub>min.</sub> =		0.0025		ок								
CHECK MINIMU	= 0.3251 = 0.4178	>													
CHECK MINIMU ptprovd. plprovd.	= 0.4178		2 21 9 <i>4</i> )												
CHECK MINIMU ptprovd. plprovd. CHECK SHEAR (	= 0.4178 CAPACITY (A	CI 318-08 11.2 8		1002.0	king		V	101.2 04							
CHECK MINIMU ptprovd. plprovd. CHECK SHEAR (	= 0.4178	CI 318-08 11.2 8	<b>21.9.4)</b> = 2 (conservative)	1092.9	kips	>	V <sub>u =</sub>	101.2 <b>OK</b>							
CHECK MINIMU ptprovd. plprovd. CHECK SHEAR (	= 0.4178 <b>CAPACITY (A</b> A <sub>cv</sub> (αc*√f <sub>c</sub> ' +	<b>CI 318-08 11.2 δ</b> ρ <sub>t</sub> , f <sub>y</sub> ) αc		1092.9	kips	>	V <sub>u =</sub>	101.2 <b>OK</b>							
CHECK MINIMU ptprovd. plprovd. CHECK SHEAR ( DVn ≤ CHECK FLEXURA	= 0.4178 <b>CAPACITY (Α</b> Α <sub>cv</sub> (αc*√f <sub>c</sub> ' + <b>AL &amp; AXIAL</b>	<b>CI 318-08 11.2 δ</b> ρ <sub>t</sub> .f <sub>y</sub> ) ας <b>CAPACITY</b>	= 2 (conservative)		kips	>	V <sub>u =</sub>		T. HORIZ. REINF		40	#8	Ø	8.00	" 0
CHECK MINIMU         ρtprovd.           ρtprovd.         φ           CHECK SHEAR (         ΦVn         ≤           CHECK FLEXURA         THE ALLOWA         Φ	= $0.4178$ <b>CAPACITY (A</b> $A_{cv} (\alpha c^* \sqrt{f_c'} +$ <b>AL &amp; AXIAL</b> ABLE MOMEN	<b>CI 318-08 11.2 δ</b> ρ <sub>t</sub> ·f <sub>y</sub> ) ας <b>CAPACITY</b> T AT AN AXIAL LO	= 2 (conservative) DAD P <sub>u</sub> IS GIVEN	BY	·		V <sub>u =</sub>	WALL DIS	T. HORIZ. REINF.		40	#8	Ø	8.00	" O.
CHECK MINIMU plprovd. plprovd. CHECK SHEAR ( $\Phi V_n \leq$ CHECK FLEXUR, THE ALLOWA $\Phi M_n =$	= 0.4178 <b>CAPACITY (A</b> $A_{cv}$ (αc*√f <sub>c</sub> ' + <b>AL &amp; AXIAL</b> MBLE MOMEN = 122,690	<b>CI 318-08 11.2 δ</b> ρ <sub>t</sub> -f <sub>y</sub> ) αc <b>CAPACITY</b> T AT AN AXIAL LC kip-ft	= 2 (conservative)	BY M <sub>u =</sub>	kips 10,224 0.900	ок	V <sub>u =</sub> 8-08 Fig. R9.	WALL DIS WALL DIS	T. HORIZ. REINF. T. VERT. REINF.		40 48	#8 #8	@ @		" O. " O.

		Spec	ial Reinforc	ed Conc	rete Shea	r Wall Design Based	on ACI 318-08 0	Ch. 21.9					
INPUT DATA & DESIGN SUMMARY		Wall	7 c			X-Direction							
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E		Pu	=	479	k	at BAS
REBAR YIELD STRESS	f <sub>v</sub>	=	60	ksi		FACTORED BASE M	OMENT LOAD		M	=	1723	ft-k	
HEIGHT OF WALL	Ĥ	=	376.0	in		FACTORED BASE S	HEAR LOAD		V,	=	516	k	
LENGTH OF SHEAR WALL	L	-	124.0	in					u				
THICKNESS OF WALL	t	=	12	in		THE WAL	L DESIGN IS ADEC	DUATE.					
	Acv	-	1487.952					<b>C</b>					
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*v(f'c) ; need		rows) =	188.2	Need 2									
1. Check Permitted Shear Strength						4. Required Vertica	Shear Reiforcem	ent					
ACI 318-08 § 11.9						ρI = Av/S*h ≥ 0.002	5 +0.5 (2.5 - h/L)*(	pt-0.0025)	ρΙ	=	0.0283	> 0.0025	ок
ΦVn ≥ Vu	Vu	=	516.0	kip		Max. Spacing	S ≤ L/3 =	41.332	S	=	12	in	
Vn = Vc + Vs	d	=	99.2	in			S ≤ 3t =	36					
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	752.9	kip			S ≤ 18"	Governs					
	ΦVn	=	564.6	kip			1	<b>'RY</b> #6	A/bar	=	0.44	in^2	
ACI 318-08 § 21.9.4	Vn	≤	8258.1	kip					# bars required	=	9		
$V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_v)  \alpha c = 2 (conservati)$	ve)		ок			ACI 318-08 § 21.9.4.4	4, IF hw/lw≤2; need	l reinf. In two directions (ρl	≥pt)				
2. Shear Strength Provided by Vc						1			h/l	=	3.0320	FALSE	
$Vc \le 2^{*}\lambda^{*}t^{*}d^{*}\sqrt{(f'c)}$ $\lambda = 1.0$ (for N.W	V.C) Vc	=	150.6	kip					ρl≥ρt	is	ОК		
Note: If Vu≤Acv*√(f'c) can choose pt, pl acco		=	94.1	FALSE									
3. Required Horizontal Shear Reinforcement	t												
1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip									
			According	to 11.9.9		WALL DIS	T. HORIZ. REINF.		8	#6	@	12	" O.C.
Vs = Vu/(0.75) - Vc	Vs	=	537.4	kip		WALL DIS	ST. VERT. REINF.		9	#6	0	12	" O.C.
S = (Av*fy*d)/Vs	Ag	=	1487.952										
	0.0025*Ag	=	3.7	in^2									
<b>TRY</b> #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										
$\rho t = Av/(S*t)$	ρt	=	0.0258	> 0.0025	OK								
5. Design for Flexture						-1							
Assume Tension-controlled section, $\Phi = 0.9$													
$Mn = As^{+}fy^{+}(d^{-}(a/2)) = As^{+}fy^{+}j \qquad jd = 0.9^{+}d$						TR'	<b>Y</b> #6	A/bar	=	0.44	in^2		
C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in				# bars required	=	9			
$Mu = \Phi Mn = \Phi As*fy*j*d$	As	=	4.29	in^2		Check Capacity:		a	=	5.86	in		
	a	=	6.31	in		C=T	0.85*f'c*a*b =As		=	6.90	in		
jd = d - (a/2)	jd		96.04	in		0.00	c = a/0.85	εt	=	0.05	> 0.0025	OK	
	As	=	3.99	in^2			3 dt = L-3"		14/	-11 1			
CHECK MINIMUM REINFORCEMENT RATIO		CT 210	00 14 2 21	0.2)		εt = εu*((	ST. HORIZ. REINF.		8	all 1 #6	0	12	" O.C.
		CI 510-		5.2)			ST. VERT. REINF.		12				
ptprovd. = 0.0258 >	(pt ) <sub>min.</sub> =		0.0025		ок	WALL DI	SI. VERI. REINF.		12	#6	@	12	" O.C.
plprovd. = 0.0283 >	(pl ) <sub>min.</sub> =		0.0025		ок			、					
CHECK SHEAR CAPACITY (ACI 318-08 11.2								>					
$\Phi V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)$ c	xc = 2 (conservative)	6194	kips	>	V <sub>u =</sub>	516 OK		<b>&gt;</b>					
CHECK FLEXURAL & AXIAL CAPACITY										e			
THE ALLOWABLE MOMENT AT AN AXIAL	-							-		f			
$\Phi$ M <sub>n</sub> = 20,676 kip-ft	>	M <sub>u =</sub>	1,723	ОК									
where Φ= 0.900	(ACI 318-08 Fig.	R9.3.2)						$\longrightarrow$					
CHECK BOUNDARY ZONE REQUIREMENTS													
AN EXEMPTION FROM THE PROVISION C	OF BOUNDARY ZONE	CONFIN	NEMENT REIN	NFORCEM	ient is gi	VEN BY ACI318-05 21.9.	6.2,						
21.9.6.3, and 21.9.6.5(a) PROVIDED THAT								<b>&gt;</b>					
c < (L*H) / (600 $d_u$ ) for ACI 21.9.6.2	apply	c <	29.52	in.	No Bou	ndary Element Needeo	I	-					
where c = 7	in. ( distance fro	m the e	xtreme comp	pression fi	iber to neu	utral axis at P <sub>u</sub> & M <sub>n</sub> lo	ads. )						
d, = 2.6						see ACI 318-08 21.9.6.2					/		

INPUT DATA &	DESIGN SU	MMARY		Wall	7 d			X-Direction							
CONCRETE STRE	NGTH (ACI	318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E		Pu	=	860.6	k	
REBAR YIELD ST	RESS		f <sub>v</sub>	=	60	ksi		FACTORED MOMEN	IT LOAD		Mu	=	1283.0	ft-k	
HEIGHT OF WAL	L —		Ĥ	=	462.0	in		FACTORED SHEAR	OAD		V.,	=	186.0	k	
LENGTH OF SHE			L	=	124.0	in					ŭ				
THICKNESS OF			t	=	12	in		THE WAI	L DESIGN IS ADEQ	JATE.					
			Acv	=	1487.952	in^2									
ACI 318-08 § 21.	9.2.2. IF Vu ≥	: 2*Acv*v(f'c) : ne	ed at least two curtains	(rows) =	188.2	Need 1									
1. Check Permit				/				4. Required Vertica	I Shear Reiforcemer	nt					
ACI 318-08 § 11.		J							25 +0.5 (2.5 - h/L)*(pt		ρΙ	=	0.05167	> 0.0025	ок
ΦVn ≥	Vu		Vu	=	186.0	kip		Max. Spacing	S ≤ L/3 =	41.332	S	=	6	in	
Vn = V	c + Vs		d	=	99.2	in			S ≤ 3t =	36					
Vn ≤ 1	0*t*d*√(f'c)	d=0.8*L	Vn	=	752.9	kip			S ≤ 18"	Governs					
			ΦVn	=	564.6	kip			TR		A/bar	=	1.56	in^2	
ACI 318-08 § 21.	94		Vn	≤	9411.2	kip					# bars required	=	2		
$V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'})$		c = 2 (conserva			OK			ACT 318-08 § 21.9.4	4. IF hw/lw<2 : need r	einf. In two directions (pl					
2. Shear Strengt			aute)		0.0				,,,	cinit in two directions (p	h/l	=	3 7 2 5 9	FALSE	
		λ = 1.0 (for N	I.W.C) Vc	=	150.6	kip					ρl≥ρt	is –	OK	TALSE	
Note: If Vu≤Acv				-	94.1	FALSE					bi-bi		JR		
3. Required Hor				-	JH.1	TALUL									
1/2ΦVa			1/2ΦVc	=	56.5	kip									
1/2410			1/2440	_	According 1			WALL DI	ST. HORIZ. REINF.		5	#8	0	8	" 0.0
$V_{c} = V$	u/(0.75) - Vo		Vs	=	97.4	kip			ST. VERT. REINF.		2	#11	@		" 0.0
	u/(0.75) = vc /*fy*d)/Vs		Ag	-	1487.952			WALL DI	JI. VERI. REINT.		2	#11	e	0	0.0
5 - (AI	/ 1y u)/ vs		0.0025*Ag	=	3.7	in^2									
	TRY	#8	A/bar	-	0.79	in^2									
Max. Spacing	S ≤ L/3 =		S	_	8.00	in	USE								
wax. spacing	S ≤ L/S = S ≤ 3t =	41.552 36	Av	-	1487.95		USE								
	S ≤ 5t = S ≤ 18"	Governs	# bars required	-	1407.93										
pt = Av		Governs	<i>#</i> bais required ρt	=	0.0387	> 0.0025	ок							FALSE	
5. Design for Fl	ovturo									A/bar	=	1.56	in A 2		
Assume Tension		ection Φ = 0	9							# bars required	=	2	111 2		
Mn = As*fy*(d-(a								TR	Y #11	" buis required		~			
	*a*b =As*fy	j ja = 0.5 v	jd	=	89.28	in		in the second seco	. "11						
Mu = Φ Mn = Φ			As	=	3.19	in^2		Check Capacity:		a	=	4.33	in		
1010 - ¢ 1011 - 4	r na iy j u		a	=	4.70	in		C=T	0.85*f'c*a*b =As*f		=	5.09	in		
id = d	- (2/2)		id	-	96.85	in		C=1	c = a/0.85	εt	_		> 0.0025		
ju – u	- (a/2)		As	-	2.94	in^2		cu = 0.00	3 dt = L-3"	21	-	0.07	20.0025		
			AS	=	2.94	In^2		εu = 0.00 εt = εu*((			14/	all 1			
CHECK BOUND	A DV ZONE		rc .					ει – ευ ((	ut-c)/c)		VV.	111 1			
			OF BOUNDARY ZON		NEMENT REIN		ENT IS GIV	EN BY ACT318-05-21.9	62						
		ROVIDED THAT		L CONI		I ONCEN	2141 15 014	EN DI ACISIO OS ZI.S	.0.2,						
		for ACI 21.9.6		c <	29.52	in	No Boun	dary Element Needeo	4						
whe								•							
whe								ral axis at P <sub>u</sub> & M <sub>n</sub> lo							
CUTCK	d <sub>u</sub> =						iservative, s	ee ACI 318-08 21.9.6.2	(d.)						
			IOS AND SPACING (A			9.2)									
ptprovd.	= 0.0387	>	(pt ) <sub>min.</sub> =	-	0.0025		ОК								
plprovd.	= 0.0517	>	(pl ) <sub>min.</sub> =		0.0025		ОК								
CHECK SHEAR	<b>CAPACITY</b> (	ACI 318-08 11													
	A <sub>cv</sub> (αc*√f <sub>c</sub> ' ·		$\alpha c = 2$ (conservative)	) 564.6	kips	>	V <sub>u =</sub>	186.0 <b>OK</b>							
CHECK FLEXUR	AL & AXIAI	. CAPACITY													
			LLOAD P., IS GIVEN	BY				WALL DI	ST. HORIZ. REINF.		5	#8	0	8.00	" 0.0
	APPER INICIAL		LE LOMB I U IS GIVEN	51							-		-		
	- 15 204	kin ft		N.4	1 2 2 2				CT VEDT DEINE		14	#11	0		
$\Phi M_n$	= 15,396		> (e <sub>t</sub> - 0.002)(250/3) , 0.6	M <sub>u =</sub>	1,283 0.900	OK	8-08 Fig. R9		ST. VERT. REINF.		14	#11	0	6	" 0.0

		Spec	ial Reinforc	ed Conc	rete Shea	r Wall Design Based on ACI 318-08 Ch.	21.9					
INPUT DATA & DESIGN SUMMARY		Wall	8 c			X-Direction						
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D + 1.0L +1.0E		Pu	=	438	k	at BAS
REBAR YIELD STRESS	f <sub>v</sub>	=	60	ksi		FACTORED BASE MOMENT LOAD		M	=	1724	ft-k	
HEIGHT OF WALL	Н	=	376.0	in		FACTORED BASE SHEAR LOAD		V	=	517		
LENGTH OF SHEAR WALL	L	-	124.0	in		THETONED DAGE SHEAR EOAD		۴u	_	517	ĸ	
THICKNESS OF WALL	t	-	124.0	in		THE WALL DESIGN IS ADEQUA	TE					
THICKNESS OF WALL	Acv	_	1487.952			THE WALL DESIGN IS ADEQUA	(TE.					
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*V(f'c) ; need a			1467.952	Need 2								
1. Check Permitted Shear Strength	at least two curtains (ro	ows) =	100.2	Need 2		4. Required Vertical Shear Reiforcement						
ACI 318-08 § 11.9						$pl = Av/S^*h \ge 0.0025 + 0.5 (2.5 - h/L)^*(pt-0.5)^*(pt$	0025)	-1	=	0 0 2 0 2	> 0.0025	01
ΦVn ≥ Vu	Vu	=	516.7	kip		$p_1 = Av_1s^{-n} \ge 0.002s + 0.5 (2.5 - n/L)^{-1}(p_1 - 0.002s + $	41.332	ρl S	=		>0.0025	UK
								5	=	12	in	
Vn = Vc + Vs	d	=	99.2	in		S ≤ 3t =	36					
$Vn \le 10*t*d*\sqrt{f'c}$ $d=0.8*L$	Vn	=	752.9	kip		S ≤ 18"	Governs					
	ΦVn	=	564.6	kip		TRY	#6	A/bar	=		in^2	
ACI 318-08 § 21.9.4	Vn	≤	8258.1	kip				# bars required	=	9		
$V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)  \alpha c = 2 \text{ (conservative})$	e)		ОК			ACI 318-08 § 21.9.4.4, IF hw/lw≤2; need rei	nf. In two directions (ρΙ					
2. Shear Strength Provided by Vc								h/l	=	3.0320	FALSE	
$Vc \le 2^{*}\lambda^{*}t^{*}d^{*}\sqrt{(f'c)}$ $\lambda = 1.0$ (for N.W.0		=	150.6	kip				pl≥pt	is	ОК		
Note: If Vu≤Acv*√(f'c) can choose pt, pl accord	ling to Ch.14	=	94.1	FALSE								
3. Required Horizontal Shear Reinforcement												
1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip								
			According	to 11.9.9		WALL DIST. HORIZ. REINF.		8	#6	@	12	" O.C.
Vs = Vu/(0.75) - Vc	Vs	=	538.4	kip		WALL DIST. VERT. REINF.		9	#6	@	12	" O.C.
S = (Av*fy*d)/Vs	Ag	=	1487.952	in^2								
	0.0025*Ag	=	3.7	in^2								
<b>TRY</b> #6	Abar	=	0.44	in^2								
Max. Spacing S ≤ L/3 = 41.332	S	=	12.00	in	USE							
S ≤ 3t = 36												
$S \le 18"$ Governs	# bars required	=	8									
$\rho t = Av/(S^*t)$	ρt	=	0.0258	>0.0025	ОК							
5. Design for Flexture												
Assume Tension-controlled section, $\Phi = 0.9$												
Mn = As*fy*(d-(a/2)) = As*fy*j jd = 0.9*d						<b>TRY</b> #6	A/bar	=	0.44	in^2		
C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in			# bars required	=	9			
$Mu = \Phi Mn = \Phi As*fy*j*d$	As	=	4.29	in^2		Check Capacity:	а	=	5.87	in		
	а	=	6.31	in		C=T 0.85*f'c*a*b =As*fy	с	=	6.90	in		
jd = d - (a/2)	jd	=	96.04	in		c = a/0.85	εt	=	0.05	> 0.0025	ок	
<u>.</u>	As	=	3.99	in^2		εu = 0.003 dt = L-3"						
						$\epsilon t = \epsilon u^*((dt-c)/c)$		Wa	all 1			
CHECK MINIMUM REINFORCEMENT RATIOS	AND SPACING (AC	CI 318-	08 14.3, 21.9	9.2)		WALL DIST. HORIZ. REINF.		8	#6	0	12	" O.C.
	(pt ) <sub>min.</sub> =		0.0025		ок	WALL DIST. VERT. REINF.		12	#6	@	12	" O.C.
piprova.						WALL DIST. VENT. REINF.		12	#0	ιψ.	12	0.C.
plprovd. = 0.0283 >	(pl ) <sub>min.</sub> =		0.0025		ок							
CHECK SHEAR CAPACITY (ACI 318-08 11.2 &	ፄ 21.9.4)						$\longrightarrow$					
$\Phi V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)$ $\alpha c$	= 2 (conservative) 6	5194	kips	>	V <sub>u =</sub>	517 <b>OK</b>	<b>&gt;</b>					
					-		-					
CHECK FLEXURAL & AXIAL CAPACITY									e			
THE ALLOWABLE MOMENT AT AN AXIAL LO	OAD P., IS GIVEN B	Y					$\rightarrow$		f	_		
	-	M <sub>11 =</sub>	1,724	ок								
Φ M <sub>2</sub> = 20.684 kip-ft		-	1,724	2			、					
$\Phi$ M <sub>n</sub> = 20,684 kip-ft	(ACT 318-08 Eig D						$\rightarrow$					
where $\Phi = 0.900$	(ACI 318-08 Fig. R	9.3.2)										
where Φ= 0.900 CHECK BOUNDARY ZONE REQUIREMENTS						/EN BY ACT218.05 21 0 6 2						
where Φ= 0.900 CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF			NEMENT REIN	NFORCEM	ent is gi	/EN BY ACI318-05 21.9.6.2,						
where $\Phi$ = 0.900 CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT	BOUNDARY ZONE	CONFIN					<b>&gt;</b>					
where Φ= 0.900 CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF	BOUNDARY ZONE	CONFIN	29.52	in.	No Bou	/EN BY ACI318-05 21.9.6.2, ndary Element Needed Itral axis at P <sub>u</sub> & M <sub>n</sub> loads. )	<b>&gt;</b>					

INPUT DATA &	DESIGN SU	MMARY		Wall	8 d			X-Direction							
CONCRETE STREE	NGTH (ACI 3	318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	0 + 1.0L + 1.0E		Pu	=	171.4	k	
REBAR YIELD STR	ESS		f <sub>v</sub>	=	60	ksi		FACTORED MOME	NT LOAD		M.	=	1283.0	ft-k	
HEIGHT OF WALL	_		Ĥ	=	462.0	in		FACTORED SHEAR	LOAD		V.,	=	186.4	k	
LENGTH OF SHEA			1	=	124.0	in					· u				
THICKNESS OF V			t	=	12	in		THE WA	LL DESIGN IS ADEQ	UATE					
			Acv	=		in^2				071121					
ACT 318-08 & 21 G	22 IE VII >	2*∆cu*√(f'c) · neec	l at least two curtains (		188.2	Need 1									
1. Check Permitt			at least two curtains (	101037-	100.2	NCCU I		4 Required Vertic	al Shear Reiforceme	nt					
ACI 318-08 § 11.9		. engen							125 +0.5 (2.5 - h/L)*(p		ρΙ	=	0.0412	> 0.0025	OK
ΦVn ≥			Vu	=	186.4	kip		Max. Spacing	S ≤ L/3 =	41.332	S	=		in	•
Vn = Vo			d	=	99.2	in			S ≤ 3t =	36	-		-		
	)*t*d*√(f'c)	d=0.8*I	Vn	=	752.9	kip			S ≤ 18"	Governs					
	, t u ((t c)	G 0.0 L	ΦVn	=	564.6	kip				RY #11	A/bar	=	1.56	in^2	
ACI 318-08 § 21.9	14		Vn	_ ≤	9411.2	kip					# bars required	-	2		
$V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'})$		2 (concentatio		-	OK	кір		ACT 318-08 5 21 0/	14 IE hw/lwc 2 pood	reinf. In two directions (p		-	2		
			<i>(e)</i>		OK			ACI 510-00 3 21.5.4	+.+, 11 11W/1WS2,11eeu	renn. in two unections (p		=	2 7250	FALSE	
2. Shear Strengt		<b>by vc</b> λ = 1.0 (for N.W	I.C) Vc	=	150.6	kip					h/l pl≥pt	= is	3.7259 OK	FALSE	
Note: If Vu≤Acv*				-	94.1	FALSE					hi≤hr	15	UK		
3. Required Hori				-	34.L	TALSE									
1/2ΦVc		r Kennorcement	1/2ΦVc	=	56.5	kip									
1/2000	< vu		1/20VC	-	According t			WALLD	IST. HORIZ. REINF.		5	#8	0	8	" O.C
Vc - V	u/(0.75) - Vc		Vs	=	98.0	kip			IST. VERT. REINF.		2	#11	@		" O.C
	*fy*d)/Vs		Ag	=		in^2		WALL D	IST. VENT. REINF.		2	#11	ιψ.	0	0.C
5 - (AV	iy u)/vs		0.0025*Ag	=	3.7	in ^2									
	TRY	#8	A/bar	=	0.79	in ^2									
Max. Spacing	S ≤ L/3 =		S	=	8.00	in	USE								
Max. spacing	S ≤ 2/5 = S ≤ 3t =	36	Av	-	1487.95		USL								
	S ≤ 5t = S ≤ 18"	Governs	# bars required	=	5										
pt = Av		Governs	pt	=	0.0387	> 0.0025	ок							FALSE	
PT 11.	, (= -,		P .				•								
5. Design for Fle	exture									A/bar	=	1.56	in^2		
Assume Tension-	-controlled s	ection, $\Phi = 0.9$								# bars required	=	2			
Mn = As*fy*(d-(a	1/2))= As*fy*	j jd = 0.9*d						TI	RY #11						
	*a*b =As*fy		jd	=	89.28	in									
$Mu = \Phi Mn = \Phi$	As*fy*j*d		As	=	3.19	in^2		Check Capacity:		a	=	4.33			
			а	=	4.70	in		C=T	0.85*f'c*a*b =As*	fy c	=	5.09	in		
jd = d -	· (a/2)		jd	=	96.85	in			c = a/0.85	εt	=	0.07	> 0.0025		
			As	=	2.94	in^2		εu = 0.00	03 dt = L-3"						
								εt = εu*(	((dt-c)/c)		Wa	ll 1			
CHECK BOUNDA				CONT		FORCEN		N DV A CI210 OF 21							
			F BOUNDARY ZOINE	CONFI	NEIVIEINI REIN	IFORCEIVI	EINT IS GIV	EN BY ACI318-05 21.9	9.0.2,						
		ROVIDED THAT			5.00										
		for ACI 21.9.6.2		c <	5.09			dary Element Neede							
when		0						ral axis at P <sub>u</sub> & M <sub>n</sub> l							
	d <sub>u</sub> =						iservative, s	ee ACI 318-08 21.9.6	.2a. )						
CHECK MINIMU	M REINFOR	CEMENT RATIO	S AND SPACING (A	CI 318-	08 14.3, 21.9	9.2)									
CHECK WINNING	= 0.0387	>	(pt ) <sub>min.</sub> =		0.0025		ок								
		>	(ρΙ ) <sub>min.</sub> =		0.0025		ок								
ρtprovd. ⁼	= 0.0412														
ρtprovd. ⁼ ρlprovd. ⁼		ACT 318-08 11 2	& 21 9 4)												
ptprovd. = plprovd. =	CAPACITY (A			EGAG	king		V	1964 04							
ptprovd. = plprovd. = CHECK SHEAR C			& 21.9.4) ac = 2 (conservative)	564.6	kips	>	V <sub>u =</sub>	186.4 <b>OK</b>							
ptprovd. <sup>=</sup> plprovd. <sup>=</sup> CHECK SHEAR C ⊕V <sub>n</sub> ≤ /	CAPACITY (/ A <sub>cv</sub> (αc*√f <sub>c</sub> ' +	· ρ <sub>t*</sub> f <sub>y</sub> ) α		564.6	kips	>	V <sub>u =</sub>	186.4 <b>OK</b>							
ptprovd. = plprovd. = CHECK SHEAR C ⊕Vn ≤ A CHECK FLEXURA	<b>CAPACITY (/</b> Α <sub>cv</sub> (αc*√f <sub>c</sub> ' + <b>ΔL &amp; AXIAL</b>	· ρ <sub>t*</sub> f <sub>y</sub> ) α CAPACITY			kips	>	V <sub>u =</sub>		IST. HORIZ. REINF.		5	#8	@	8.00	" O.C
ptprovd. = plprovd. = <b>CHECK SHEAR C</b> $\Phi V_n \le I$ <b>CHECK FLEXURA</b> THE ALLOWA	<b>CAPACITY (/</b> Α <sub>cv</sub> (αc*√f <sub>c</sub> ' + <b>ΔL &amp; AXIAL</b>	· ρ <sub>t*</sub> f <sub>y</sub> ) α <b>CAPACITY</b> NT AT AN AXIAL	c = 2 (conservative)		kips 1,283	> ок	V <sub>u =</sub>	WALL D	IST. HORIZ. REINF. IST. VERT. REINF.		5 12	#8 #11	@		" 0.C

		Spec	ial Reinforc	ed Conc	rete Shear	Wall Design Based	d on ACI 318-	08 Ch. 2	21.9					
INPUT DATA & DESIGN SUMMARY		Wall	9 c			X-Direction					_			
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E			Pu	=	1225	k	at BAS
REBAR YIELD STRESS	f <sub>v</sub>	=	60	ksi		FACTORED BASE M	10MENT LOAD			M	=	635	ft-k	
HEIGHT OF WALL	Ĥ	=	400.0	in		FACTORED BASE S	SHEAR LOAD			V,	=	72	k	
LENGTH OF SHEAR WALL	L	=	408.0	in						ŭ				
THICKNESS OF WALL	t	=	12	in		THE WAI	LL DESIGN IS	ADEQUA	TE.					
	Acv	=	4896	in^2				•						
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*v(f'c) ; nee	d at least two curtains (	rows) =	619.3	Need 1										
1. Check Permitted Shear Strength						4. Required Vertica	al Shear Reifor	cement						
ACI 318-08 § 11.9						$\rho I = Av/S*h \ge 0.002$				ρl	=		> 0.0025	ок
ΦVn ≥ Vu	Vu	=	71.8	kip		Max. Spacing	S ≤ L/3 =		136	S	=	12	in	
Vn = Vc + Vs	d	=	326.4	in			S ≤ 3t =		36					
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2477.2	kip			S ≤ 18"		Governs					
	ΦVn	=	1857.9	kip				TRY	#6	A/bar	=		in^2	
ACI 318-08 § 21.9.4	Vn	≤	32748.8	kip						# bars required	=	25		
$V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_t f_y)  \alpha c = 2 \text{ (conservat})$	ive)		OK			ACI 318-08 § 21.9.4	.4, IF hw/lw≤2;	need rein	f. In two directions (ρl					
2. Shear Strength Provided by Vc			405.1							h/l	=	0.9803	ρl≥ρt	
Vc ≤ $2^{\lambda}t^{*}d^{*}\sqrt{(f'c)}$ $\lambda = 1.0$ (for N.V		=	495.4	kip						ρl≥pt	is	ОК		
Note: If Vu≤Acv*√(f'c) can choose pt, pl acco		=	309.7	Accord	ing to Ch.14									
<ol> <li>Required Horizontal Shear Reinforcement 1/2ΦVc &lt; Vu</li> </ol>	1/2ΦVc	=	185.8	kip										
1/2000 < 00	1/2010	-	Reinf. Acco		Ch 14	WALL DI	IST. HORIZ. RE	INF		15	#6	0	12	" O.C.
Vs = Vu/(0.75) - Vc	Vs	=	Kelili. Acco	kip	CII 14		IST. VERT. REIM			25	#6	@		" O.C.
$S = (Av^*fy^*d)/Vs$	Ag	=	4896	in^2			SI. VENI. REI	<b>N</b> .		23	#0	e.	12	0.c.
5 (, t, t, y a), t 5	0.0025*Ag	=	12.2	in^2										
<b>TRY</b> #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											
$\rho t = Av/(S^*t)$	ρt	=	0.0448	> 0.0025	5 <b>OK</b>									
5. Design for Flexture														
Assume Tension-controlled section, $\Phi = 0.9$														
Mn = As*fy*(d-(a/2)) = As*fy*j jd = 0.9*d						TR	<b>RY</b> #6		A/bar	=	0.44	in^2		
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in					# bars required	=	1			
$Mu = \Phi Mn = \Phi As*fy*j*d$	As	=	0.48	in^2		Check Capacity:			а	=	0.64	in		
	а	=	0.71	in		C=T	0.85*f'c*a*b	=As*fy	с	=	0.75	in		
jd = d - (a/2)	jd	=	326.05	in			c = a/0.85		εt	=	1.63	> 0.0025	ок	
	As	=	0.43	in^2			03 dt = L-3"							
						εt = εu*((					all 1			_
CHECK MINIMUM REINFORCEMENT RATIO	OS AND SPACING (A	CI 318-		9.2)			IST. HORIZ. RE			15	#6	@	12	" O.C.
<sub>ptprovd.</sub> = 0.0448 >	(pt ) <sub>min.</sub> =		0.0025		ОК	WALL DI	ST. VERT. REIM	NF.		28	#6	@	12	" O.C.
plprovd. = 0.0770 >	(pl ) <sub>min.</sub> =		0.0025		ок									
P. P. C. C.									$\longrightarrow$					
CHECK SHEAR CAPACITY (ACI 318-08 11.2	2 & 21.9.4)								>					
	$\alpha c = 2$ (conservative)	24562	kips	>	V <sub>u =</sub>	72 OK								
					-				ſ					
CHECK FLEXURAL & AXIAL CAPACITY											е			
THE ALLOWABLE MOMENT AT AN AXIAL	LOAD P <sub>u</sub> IS GIVEN I	3Y									f	-		
Φ M <sub>n</sub> = 7,620 kip-ft	>	M <sub>u =</sub>	635	ок										
where $\Phi = 0.900$	(ACI 318-08 Fig. I								<b>&gt;</b>					
CHECK BOUNDARY ZONE REQUIREMENTS														
AN EXEMPTION FROM THE PROVISION		CONFIN	NEMENT REIN	NFORCEN	IENT IS GIVE	N BY ACI318-05 21.9	9.6.2,							
21.9.6.3, and 21.9.6.5(a) PROVIDED THAT									>					
c < (L*H) / (600 d <sub>u</sub> ) for ACI 21.9.6.2	2 apply	c <	97.14	in.	No Boun	ary Element Neede	d		-					
where c = 1	in. ( distance fro	m the e	xtreme comp	pression f	iber to neut	al axis at P <sub>u</sub> & M <sub>n</sub> Ic	oads. )							
d <sub>u</sub> = 2.8			t, assume 0.0								\			

INPUT DATA &	DESIGN SU	MMARY			Wall	9 d			X-Direction							
CONCRETE STRE	NGTH (ACI	318 5.1.1		f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E		Pu	=	2774.9	k	
REBAR YIELD STI	RESS			fv	=	60	ksi		FACTORED MOMEN			M.,	=	26.1	ft-k	
HEIGHT OF WAL				H	=	462.0	in		FACTORED SHEAR L			V.,	=	14.0		
LENGTH OF SHE				1	-	402.0	in		TACTORED SHEAR E	.OAD		v <sub>u</sub>	-	14.0	~	
				t L	-	408.0										
THICKNESS OF	WALL			•	=		in 		THE WAL	L DESIGN IS ADEQ	UATE.					
				Acv		4896	in^2									
			'c) ; need at	t least two curtains (	rows) =	619.3	Need 1									
1. Check Permit		trength							4. Required Vertical							
ACI 318-08 § 11.									$\rho I = Av/S*h \ge 0.002$			ρl	=		> 0.0025	ок
ΦVn ≥				Vu	=	14.0	kip		Max. Spacing	S ≤ L/3 =	136	S	=	6	in	
Vn = V				d	=	326.4	in			S ≤ 3t =	36					
Vn ≤ 1	0*t*d*√(f'c)	d=0.8*l		Vn	=	2477.2	kip			S ≤ 18"	Governs					
				ΦVn	=	1857.9	kip			T	RY #11	A/bar	=	1.56	in^2	
ACI 318-08 § 21.	9.4			Vn	≤	115071.3	kip					# bars required	=	25		
$V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'})$	$+ \rho_{t*}f_{v}) \alpha$	c = 2 (co	nservative)	)		OK			ACI 318-08 § 21.9.4.4	4, IF hw/lw≤2; need	einf. In two directions (p	l≥pt)				
2. Shear Strengt									1			h/l	=	1.1324	ρl≥ρt	
	*λ*t*d*√(f'c)		(for N.W.C	) Vc	=	495.4	kip					ρl≥ρt	is	OK		
Note: If Vu≤Acv <sup>3</sup>					=	309.7		ing to Ch.14				F F.				
3. Required Hor						505.1	· iccord									
1/2ΦVa			cement	1/2ΦVc	=	185.8	kip									
1/2010				1/2000	-	Reinf. Acco		Ch 14		T. HORIZ. REINF.		40	#8	Ø	8	" O.C.
$V_{c} = V$	u/(0.75) - Vc			Vs	=		kip	CII 14	-	ST. VERT. REINF.		25	#11	@		" O.C
	u/(0.75) - VC /*fy*d)/Vs			Aq	-	4896	in^2		WALL DIS	DI. VENI. KEINF.		25	#11	(LU)	0	0.C.
5 = (A)	/*iy*u)/vs					4696	in^2									
n in the second s				0.0025*Ag	=											
	TRY	100	#8	A/bar	=	0.79	in^2									
Max. Spacing	S ≤ L/3 =			S	=	8.00	in	USE								
	S ≤ 3t =			Av	=	4896.00										
	S ≤ 18"	Govern	S	# bars required	=	40										
pt = Av	//(S*t)			ρt	=	0.3251	> 0.002	5 <b>OK</b>							FALSE	
· · · -											A.(1		1.50			
5. Design for Fl											A/bar	=	1.56	in^2		
Assume Tension											# bars required	=	0			
Mn = As*fy*(d-(a		ʻj jd	= 0.9*d			000 70			TRY	<b>Y</b> #11						
	*a*b =As*fy			jd	=	293.76	in									
Mu = Φ Mn = Φ	> As*fy*j*d			As	=	0.02	in^2		Check Capacity:		а	=	0.03			
				а	=	0.03	in		C=T	0.85*f'c*a*b =As*	·	=		in		
jd = d	- (a/2)			jd	=	326.39	in			c = a/0.85	εt	=	39.81	> 0.0025		
1				As	=	0.02	in^2		εu = 0.003	3 dt = L-3"						
									εt = εu*((c	dt-c)/c)		Wa	all 1			
CHECK BOUND																
				BOUNDARY ZONE	CONFI	NEMENT REIN	IFORCEN	1ENT IS GIVE	N BY ACI318-05 21.9.0	6.2,						
21963 and	21.9.6.5(a) P	ROVIDE	THAT													
21.5.0.5, unu	H) / (600 d )	for ACI	21.9.6.2 ap	ply	c <	0.03	in.	No Bound	lary Element Needed							
	(000 a <sub>u</sub> )		0	in. ( distance fro	om the e	xtreme comp	ression f	iber to neutr	al axis at P <sub>u</sub> & M <sub>n</sub> loa	ads. )						
					lacemer	it, assume 0.0	07*H co	nservative. se	e ACI 318-08 21.9.6.2	a.)						
c < (L*I			4.4	in. ( design disp												
c < (L*l when	re c = d <sub>u</sub> =					08 14.3. 21 9										
c < (L*I when	re c = d <sub>u</sub> = JM REINFOR		RATIOS	AND SPACING (A	CI 318-			OK								
c < (L*I when CHECK MINIMU ptprovd.	re c = d <sub>u</sub> = JM REINFOF = 0.3251		RATIOS -	AND SPACING (A (pt ) <sub>min.</sub> =	CI 318-	0.0025		ок								
c < (L*I when CHECK MINIMU ptprovd.	re c = d <sub>u</sub> = JM REINFOR		RATIOS	AND SPACING (A	CI 318-			ок ок								
c < (L*I when CHECK MINIMU ptprovd. plprovd.	re c = d <sub>u</sub> = JM REINFOF = 0.3251 = 0.5456	RCEMEN	RATIOS / >	AND SPACING (Α (ρt ) <sub>min.</sub> = (ρl ) <sub>min.</sub> =	CI 318-	0.0025										
c < (L*I when CHECK MINIMU ptprovd. plprovd. CHECK SHEAR	re c = d <sub>u</sub> = JM REINFOF = 0.3251 = 0.5456 CAPACITY (A	RCEMEN ACI 318	F RATIOS / > > 08 11.2 &	AND SPACING (A (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = : 21.9.4)	ACI 318-	0.0025 0.0025		ок								
c < (L*I when CHECK MINIMU ptprovd. plprovd. CHECK SHEAR	re c = d <sub>u</sub> = JM REINFOF = 0.3251 = 0.5456	RCEMEN ACI 318	F RATIOS / > > 08 11.2 &	AND SPACING (Α (ρt ) <sub>min.</sub> = (ρl ) <sub>min.</sub> =	ACI 318-	0.0025	>		14.0 <b>OK</b>							
$c < (L^{4})$ when CHECK MINIMU ptprovd. plprovd. CHECK SHEAR ( $\Phi V_n \leq$	re c = $d_u$ = JM REINFOF = 0.3251 = 0.5456 CAPACITY ( $\alpha c^* \sqrt{f_c}$ -	ACI 318 + ρ <sub>t</sub> , f <sub>y</sub> )	r RATIOS / > > 08 11.2 & αc :	AND SPACING (A (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = : 21.9.4)	ACI 318-	0.0025 0.0025		ок	14.0 <b>ОК</b>							
c < (L <sup>4</sup> whee CHECK MINIMU ptprovd. ptprovd. ptprovd. CHECK SHEAR ( ⊕Vn ≤ CHECK FLEXUR.	re c = $d_u =$ JM REINFOF = 0.3251 = 0.5456 CAPACITY ( $A_{cv}$ ( $\alpha c^* \sqrt{f_c}$ ' · AL & AXIAL	ACI 318 + ρ <sub>t*</sub> f <sub>y</sub> ) . CAPAC	<b>RATIOS</b> > 08 11.2 & αc =	AND SPACING (A (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = 2 21.9.4) = 2 (conservative)	<b>CI 318-</b> 1857.9	0.0025 0.0025		ок		T HORIZ REINE		40	#8	۵	8.00	" 0.0
c < (L <sup>41</sup> when plprovd. plprovd. CHECK SHEAR ØV <sub>n</sub> ≤ CHECK FLEXUR THE ALLOWA	re c = $d_u =$ JM REINFOF = 0.3251 = 0.5456 CAPACITY ( $_{ac}$ $A_{cv}$ ( $\alpha c^* \sqrt{f_c}$ ) AL & AXIAL ABLE MOMEN	<b>ACI 318</b> ⊢ρt•fy) - <b>CAPAC</b> ΝΤ ΑΤ Α	<b>RATIOS</b> > 08 11.2 & αc =	AND SPACING (A (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = : 21.9.4)	<b>ACI 318-</b> 1857.9 BY	0.0025 0.0025 kips	>	ок	WALL DIS	ST. HORIZ. REINF.		40	#8	@	8.00	" 0.0
c < (L <sup>4</sup> when ptprovd. ptprovd. CHECK SHEAR OV <sub>n</sub> ≤ CHECK FLEXUR THE ALLOW/ D M <sub>n</sub>	re c = $d_u =$ JM REINFOF = 0.3251 = 0.5456 CAPACITY ( $_{ac}$ $A_{cv}$ ( $\alpha c^* \sqrt{f_c}^{-1}$ AL & AXIAL ABLE MOMEI = 313	ACI 318 + ρ <sub>t</sub> -f <sub>y</sub> ) - CAPAC NT AT A kip-ft	<b>7 RATIOS</b> > 08 11.2 & αc = TY N AXIAL LC	AND SPACING (A (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = 2 21.9.4) = 2 (conservative)	A <b>CI 318-</b> 1857.9 BY M <sub>u =</sub>	0.0025 0.0025	>	ок	WALL DIS	ST. HORIZ. REINF. ST. VERT. REINF.		40 28	#8 #11	@		" 0.C

		Spec	ial Reinford	ed Conci	rete Shea	r Wall Design Based on ACI 318-08 Ch.	. 21.9					
INPUT DATA & DESIGN SUMMARY		Wall :	10 c			X-Direction						
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D + 1.0L +1.0E		Pu	=	974	k	at BAS
REBAR YIELD STRESS	f <sub>v</sub>	=	60	ksi		FACTORED BASE MOMENT LOAD		M	=	6062	ft-k	
HEIGHT OF WALL	H	=	400.0	in		FACTORED BASE SHEAR LOAD		V	=	565		
LENGTH OF SHEAR WALL	L	-	408.0	in		TACTORED BASE STIEAR LOAD		۴u	-	505	ĸ	
THICKNESS OF WALL	t	-	408.0	in		THE WALL DESIGN IS ADEQU	ATE					
THICKINESS OF WALL	Acv	_	4896	in^2		THE WALL DESIGN IS ADEQU						
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*V(f'c) ; need ;			4896 619.3	Need 1								
1. Check Permitted Shear Strength	at least two curtains (	10ws) =	019.5	Need 1		4. Required Vertical Shear Reiforcement						
ACI 318-08 § 11.9						$\rho l = Av/S^*h \ge 0.0025 + 0.5 (2.5 - h/L)^*(\rho t-$		-1	=	0 1 1 0 0	> 0.0025	01
ΦVn ≥ Vu	Vu	=	565.0	kip		$p_1 = Av_1s^{-n} \ge 0.002s + 0.5 (2.5 - n/L)^{-}(p_1 - Max. Spacing S \le L/3 =$	136	ρl S	=	12		UK
Vn = Vc + Vs	d	-	326.4			S ≤ 3t =	36	3	-	12		
				in								
$Vn \le 10*t*d*\sqrt{f'c}$ $d=0.8*L$	Vn	=	2477.2	kip		S ≤ 18"	Governs					
	ΦVn	=	1857.9	kip		TRY	<b>/</b> #6	A/bar	=		in^2	
ACI 318-08 § 21.9.4	Vn	≤	44553.6	kip				# bars required	=	36		
$V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)  \alpha c = 2 \text{ (conservative})$	e)		ОК			ACI 318-08 § 21.9.4.4, IF hw/lw≤2; need re	int. In two directions (pl					
2. Shear Strength Provided by Vc								h/l	=	0.9803	ρl≥ρt	
$Vc \le 2^{*}\lambda^{*}t^{*}d^{*}\sqrt{(f'c)}$ $\lambda = 1.0$ (for N.W.		=	495.4	kip				ρl≥ρt	is	OK		
Note: If Vu≤Acv*√(f'c) can choose pt, pl accord	ding to Ch.14	=	309.7	FALSE								
3. Required Horizontal Shear Reinforcement												
1/2ΦVc < Vu	1/2ΦVc	=	185.8	kip								
			According			WALL DIST. HORIZ. REINF.		28	#6	@	12	" O.C.
Vs = Vu/(0.75) - Vc	Vs	=	257.9	kip		WALL DIST. VERT. REINF.		36	#6	@	12	" O.C.
S = (Av*fy*d)/Vs	Ag	=	4896	in^2								
	0.0025*Ag	=	12.2	in^2								
<b>TRY</b> #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									
$\rho t = Av/(S^*t)$	ρt	=	0.0850	>0.0025	ОК							
5. Design for Flexture												
Assume Tension-controlled section, $\Phi = 0.9$												
Mn = As*fy*(d-(a/2)) = As*fy*j jd = 0.9*d						<b>TRY</b> #6	A/bar	=	0.44	in^2		
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in			# bars required	=	9			
$Mu = \Phi Mn = \Phi As*fy*j*d$	As	=	4.59	in^2		Check Capacity:	а	=	6.13	in		
	а	=	6.74	in		C=T 0.85*f'c*a*b =As*fy	c c	=	7.21	in		
jd = d - (a/2)	jd	=	323.03	in		c = a/0.85	εt	=	0.17	> 0.0025	ок	
	As	=	4.17	in^2		εu = 0.003 dt = L-3"						
						$\epsilon t = \epsilon u^*((dt-c)/c)$		Wa	all 1			
CHECK MINIMUM REINFORCEMENT RATIOS	AND SPACING (A	CI 318-	08 14.3. 21.	9.2)		WALL DIST. HORIZ. REINF.		28	#6	0	12	" O.C.
ptprovd. = 0.0850 >	(pt ) <sub>min.</sub> =		0.0025		ок	WALL DIST. VERT. REINF.		36	#6	@	12	" O.C.
						WALL DIST. VERT. REINT.		50	#0	<sup>w</sup>	12	0.c.
plprovd. = 0.1100 >	(pl ) <sub>min.</sub> =		0.0025		ок							
CHECK SHEAR CAPACITY (ACI 318-08 11.2 &	& 21.9.4)						$\rightarrow$					
$\Phi V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)$ ac	= 2 (conservative)	33415	kips	>	V <sub>u =</sub>	565 <b>OK</b>	>					
-												
CHECK FLEXURAL & AXIAL CAPACITY							、		е			
THE ALLOWABLE MOMENT AT AN AXIAL L	OAD P <sub>u</sub> IS GIVEN	BY					,		f	_		
Φ M <sub>n</sub> = 72,739 kip-ft	>	M., _	6,062	ок								
where $\Phi = 0.900$	(ACI 318-08 Fig.	-	-,				、					
	, .c. 510 00 rig.											
•	BOUNDARY ZONF		VEMENT REI	NFORCEM	ent is gi	VEN BY ACI318-05 21.9.6.2,						
21.9.6.3, and 21.9.6.5(a) PROVIDED THAT			97.14	in	No Bou	ndary Element Needed	$\longrightarrow$					
	pply	c <	97.14	in.		ndary Element Needed utral axis at P <sub>u</sub> & M <sub>n</sub> loads. )						

INPUT DATA 8	& DESIGN SU	MMAR	Y		Wall	10 d			X-Direction							
CONCRETE STR	ENGTH (ACI	318 5.1.	1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E		Pu	=	467	k	
REBAR YIELD ST			,	f	=	60	ksi		FACTORED MOMEN	IT LOAD		M	=	6729	ft-k	
HEIGHT OF WA				·y H	=	462.0	in		FACTORED SHEAR L			V.,	=	81	k	
LENGTH OF SHI				1	-	402.0	in		TACTORED STIEAR	OND		v <sub>u</sub>	-	01	ĸ	
				t	-	408.0			T115 14/41							
THICKNESS OF	WALL			Acv	=	4896	in in^2		THE WAL	L DESIGN IS ADEQU	ATE.					
				at least two curtains	(rows) =	619.3	Need 1	L								
1. Check Permi		trength								I Shear Reiforcemen						
ACI 318-08 § 11										25 +0.5 (2.5 - h/L)*(pt		ρl	=		> 0.0025	ок
ΦVn ≥				Vu	=	80.6	kip		Max. Spacing	S ≤ L/3 =	136	S	=	6	in	
	Vc + Vs			d	=	326.4	in			S ≤ 3t =	36					
Vn ≤ 3	10*t*d*√(f'c)	d=0.8	*L	Vn	=	2477.2	kip			S ≤ 18"	Governs					
				ΦVn	=	1857.9	kip			TR	Y #11	A/bar	=	1.56	in^2	
ACI 318-08 § 21	L.9.4			Vn	≤	115071.3	kip					# bars required	=	25		
$V_n \leq A_{cv} (\alpha c^* \sqrt{f_c})$	('+ ρ <sub>t*</sub> f <sub>v</sub> ) α	c = 2 (d	onservative	e)		OK			ACI 318-08 § 21.9.4.4	4, IF hw/lw≤2; need re	inf. In two directions (p	l≥pt)				
2. Shear Streng									-			h/l	=	1.1324	ol≥ot	
	2*λ*t*d*√(f'c)		0 (for N.W.C	C) Vc	=	495.4	kip					ρl≥ρt	is	ОК		
Note: If Vu≤Acv					_	309.7		ling to Ch.14				F.= b.		5		
3. Required Ho					-	505.7	Accolu									
•	/c < Vu		cement	1/2ΦVc	=	185.8	kip									
1/244	c < vu			1/2010	-	Reinf. Acco		Ch 14	WALL DI	ST. HORIZ. REINF.		40	#8	Ø	8	" O.C.
V(c - )	Vu/(0.75) - Vo			Vs	=		kip	CII 14		ST. VERT. REINF.		25	#11	@		" O.C.
	vu/(0.75) - vu v*fv*d)/Vs				-	4896			WALL DI	SI. VERI. REINF.		25	#11	ιψ.	0	0.c.
5 = (A	w"ly"d)/vs			Ag			in^2									
n in the second s				0.0025*Ag	=	12.2	in^2									
	TRY		#8	A/bar	=	0.79	in^2									
Max. Spacing	S ≤ L/3 =			S	=	8.00	in	USE								
	S ≤ 3t =			Av	=	4896.00										
	S ≤ 18"	Gove	rns	# bars required	=	40										
ρt = A	Av/(S*t)			ρt	=	0.3251	> 0.002	5 OK							FALSE	
5. Design for F											A/bar	=	1.56	in^2		
Assume Tension											# bars required	=	3			
Mn = As*fy*(d-		*j jc	= 0.9*d						TR	Y #11						
	c*a*b =As*fy			jd	=	293.76	in									
$Mu = \Phi Mn = 0$	Φ As*fy*j*d			As	=	5.09	in^2		Check Capacity:		а	=	6.81	in		
				а	=	7.49	in		C=T	0.85*f'c*a*b =As*fy	с с	=	8.02	in		
jd = d	l - (a/2)			jd	=	322.66	in			c = a/0.85	εt	=	0.15	> 0.0025		
				As	=	4.63	in^2		εu = 0.003	3 dt = L-3"						
									εt = εu*((	dt-c)/c)		Wa	all 1			
CHECK BOUND	DARY ZONE	REQUIF	EMENTS													
AN EXEMPT	TON FROM T	HE PRC	VISION OF	BOUNDARY ZON	e confii	NEMENT REIM	NFORCEN	/ENT IS GIVE	N BY ACI318-05 21.9.	.6.2,						
21.9.6.3, and	d 21.9.6.5(a) P	ROVIDE	D THAT													
	*H) / (600 d,,)			pply	с <	8.02	in.	No Bound	lary Element Needec	i						
whe			0						al axis at P <sub>11</sub> & M <sub>0</sub> Io							
WIIC	d, =		42.4						e ACI 318-08 21.9.6.2							
	-							iservative, se	C ACI 310-00 21.9.0.2	.a. )						
		CEIVIEI		AND SPACING (			9.2)									
ρtprovd.	= 0.3251		>	(pt ) <sub>min.</sub> =	:	0.0025		ОК								
plprovd.	= 0.5456		>	(ρΙ) <sub>min.</sub> =	-	0.0025		ок								
CHECK SHEAR		ACT 31	8-08 11 2 4	2 21 9 4)												
	≤ A <sub>cv</sub> (αc*√f <sub>c</sub> ' ·			= 2 (conservative)	) 1857.9	kips	>	V <sub>u =</sub>	80.6 <b>OK</b>							
CHECK FLEXUE																
	ABLE MOME	NT AT /	AN AXIAL LO	DAD P <sub>u</sub> IS GIVEN	BY				WALL DIS	ST. HORIZ. REINF.		40	#8	@	8.00	" O.C.
THE ALLOW						6,729	ок			T VERT DENIE		28	#11	@	6	" O.C.
	= 80,742	kip-ft		>	M <sub>u =</sub>	0,729	UK.		WALL DI	ST. VERT. REINF.		20	#11	w		
$\Phi M_n$		-	0.65 + (e	> 0.002)(250/3) , 0.6	-	0,729		18-08 Fig. R9.		SI. VERI. REINF.		20	#11	w	Ū	

$ \begin{array}{c c c c c c c c c c c c c c c c c c c $			Spec	ial Reinford	ed Conc	rete Shea	r Wall Design Based	on ACI 318-08	Ch. 21.9					
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	INPUT DATA & DESIGN SUMMARY		Wall '	L1 e			X-Direction							
REAM NUMBER SING STRESS $I_{r}$ <		f'			ksi			+ 1 0L + 1 0E		Pu	=	1063	k	at BAS
HIGH TO WALL         # <t< td=""><td></td><td>-</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>ut bris</td></t<>		-												ut bris
ENCIT-O STRAM WALL         L         =		,								u				
Histocks Cor Wall to t = a b in control (1997) A so t = a b in control (1997) A so t = a b in control (1997) C 138 dis 11922, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 11922, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 11922, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 11922, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 1192, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 1192, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 1192, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 1192, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 1192, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 1192, EV us 27 An (1997) (19) ere at lates to control (1997) C 138 dis 1192, EV us 27 An (1997) (19) ere at lates to control (1997) C 1993 FALSE According to 11.19 fill (19) ere at lates to control (1997) C 1993 FALSE C 1993 FALSE							FACTORED BASE SF	IEAK LOAD		V u	-	2115	ĸ	
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NCI 38:08 51:05:000000000000000000000000000000000	Internets of Whee								QUATE.					
L check Premitted Shear Strength Q 13 Bel 5112 Q 13 Bel 512 Q 13 Bel	ACT 318-08 § 21 9 2 2 IF Vu > 2*Acv*v/(f'c) : need :													
$ \begin{tabular}{ c c c c c }  c c c c c c c c c c c c c $			10113)	562.5	Heed 2		4. Required Vertical	Shear Reiforcen	nent					
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $	ACI 318-08 § 11.9									ρΙ	=	0.03694	> 0.0025	ок
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $		Vu	=	2113.0	kip						=			
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$		d	=											
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1530.0	kip			S ≤ 18"	Governs					
KC) 318-08 \$ 21.9.4       Vn       \$ <ul> <li>18</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.4.1 F hw/hw2; meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.2.1 F meed relind. In two directions (ubp)</li> <li>47.3 18-08 \$ 21.9.2.1 F m</li></ul>		ΦVn	=	1147.5	kip				TRY #6	A/bar	=	0.44	in^2	
$\frac{1}{\sqrt{2}} \leq A_{1} (c^{+})_{1}^{+} (c^{+})_{2}^{-} (c^{+})_{$	ACI 318-08 § 21 9 4		<							,				
2 here strength Provided by VC VC 2 24747(r) A 3 10 for NWC, VC $=$ 3060 kp phone I VoxAA-V(P)(r) A 3 10 for NWC, VC $=$ 3060 kp phone I VoxAA-V(P)(r) A 3 10 for NWC VC $=$ 3060 kp phone I VoxAA-V(P)(r) A 3 10 for AC 2 VS = VW(073) · VC VS $=$ 2113 kp VALL DIST. HORIZ, REINF. 17 #6 $\oplus$ 12 ° OC. VALL DIST. VERT. REINF. 18 #6 $\oplus$ 12 ° OC. VALL DIST. VERT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VERT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VERT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VERT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VERT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VERT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VORT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VORT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VORT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VORT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VORT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VORT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VORT. REINF. 24 #6 $\oplus$ 12 ° OC. VALL DIST. VORT. REINF. 24			-		- P.		ACI 318-08 § 21.9.4.4	, IF hw/lw≤2;nee	ed reinf. In two directions (p					
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$				•			-	. ,			=	2.3807	FALSE	
We if YuskAnVifC can choose pt, p1 according to Ch.14 = $\frac{1913}{1208}$ FAUSE <b>Required Horizontal Share Reinforcement</b> $\frac{1}{229Vc < Vu}$ <b>J</b> $229Vc < Vu$ <b>J</b> $229Vc < 2$ <b>J</b> $148$ <b>k</b> (p) <b>Wall DIST. HORIZ. REINF. J</b> $7$ <b>f</b> $6$ <b>e 1</b> $2$ <b>o</b> C. <b>Wall DIST. VERT. REINF. J</b> $7$ <b>f</b> $6$ <b>e 1</b> $22$ <b>o</b> C. <b>Wall DIST. VERT. REINF. J</b> $7$ <b>f</b> $6$ <b>e 1</b> $22$ <b>o</b> C. <b>TRY f</b> $6$ <b>A</b> $bar$ <b>f</b> $6$ <b>b</b> $122$ <b>o</b> C. <b>S</b> $c \sqrt{3} = 56$ <b>S s s s</b> $1200$ <b>in USE</b> <b>S</b> $c \sqrt{3} = 56$ <b>S s s s s</b> $1200$ <b>in USE</b> <b>S</b> $c \sqrt{3} = 54$ <b>S s</b> $12$ <b>s s s s s s s s s s</b>	5	C) Vc	=	306.0	kip									
3. Required Horizontal Shear Reinforcement $1/28V < V_{ii}$ $V_{ii} = Vu(075) - V_{ii}$ $S = (Avfy'd)V_{ii}$ $V_{ii} = vu(075) - V_{ii}$ $S = (Avfy'd)V_{ii}$ Ag = 23034 Ag = 23035 $G = 12^{\circ} O.C.$ Val L DIST. VERT. REINF. Val L DIST. V			=							F F.	-			
$ \frac{1}{1/2} \frac{2}{9} \sqrt{c} < \sqrt{u} $ $ \frac{1}{1/2} \frac{1}{9} \sqrt{c} = \frac{1148}{c} \frac{k}{p} $ $ \frac{k}{kcording to 113.9} $ $ \frac{k}{kcording to 112.9} $ $ \frac$	3. Required Horizontal Shear Reinforcement	<b>J</b>												
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	•	1/2ΦVc	=	114.8	kip									
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$				According	to 11.9.9		WALL DIS	T. HORIZ. REINF	<b>.</b>	17	#6	0	12	" O.C.
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	Vs = Vu/(0.75) - Vc	Vs	=				WALL DIS	T. VERT. REINF.		18	#6		12	
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $	S = (Av*fy*d)/Vs	Ag	=	3024	in^2									
Max. Spacing $S \le IJ = \frac{56}{56}$ $S = 12.00$ in USE $S \le 31 = 54$ $S \le 13^{\circ}$ <b>Governs</b> # bars required = 17 $pt = Av(S^{\circ})$ $pt = 0.0350 > 0.0025$ OK Mn = Asthy(d-(a)2) = Asthy) jd = 0.9^{\circ}d $E^{=T} = 0.85th(2^{\circ}ab = Asthy)$ jd = 120.96 in <b>TRY</b> #6 A/bar = 0.44 in^2 $E^{=T} = 0.85th(2^{\circ}ab = Asthy)$ jd = 120.96 in Cell Capacity: a = 9.25 in 10.0025 OK $TRY$ #6 # bars required = 21 in 10.00 in^2 Check Capacity: a = 9.25 in 10.0025 OK a = 9.900 in Cell Capacity: a = 9.25 in 10.00 solved > 0.0025 OK $As = 9.944$ in^2 $E = 0.0035$ $et = 0.0035$ $et = 0.004 > 0.0025 OK$ CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2) WALL DIST. HORIZ, REINF. 24 #6 @ 12 ° O.C. $ptword = 0.0350 > (pt)_{me.} = 0.0025 OK$ CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $QV_n \le Ax3AL CAPACITY$ THE ALLOWABLE MOMENT AT AN AXAL LOAD $P_u$ IS GIVEN BY THE ALLOWABLE MOMENT AT AN AXAL LOAD $P_u$ IS GIVEN BY $DW_m = 65.974$ kip-ft $X = M_u$ . 5,498 OK where $0 = 0.000$ (ACI 318-08 12.8 2.9.2) CHECK FLEXURAL & AXAL CAPACITY THE ALLOWABLE MOMENT AT AN AXAL LOAD $P_u$ IS GIVEN BY THE ALLOWABLE MOMENT AT AN AXAL LOAD $P_u$ IS GIVEN BY AN EVENTFORM TREPORCISION OF BOUNDARY ZONE CONTINUEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5.974 kip-ft $X = M_u$ . 5,498 OK where $0 = 0.000$ (ACI 313-08 Fig. P3.3.2) CHECK FLEXURAL & AXAL CAPACITY AN EVENTFORM TREPORCISION OF BOUNDARY ZONE CONTINUEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5.974 kip-ft $X = 40.00$ in. No Boundary Element Needed where $c = 1$ 1 in (distance from the externe compression fiber to neutral axis at P, & M_n (ads.))		0.0025*Ag	=	7.6	in^2									
$S \le S : 5^{\circ}$ $S \le S^{\circ} : 5^{\circ}$ $S = $	<b>TRY</b> #6	Abar	=	0.44	in^2									
$S \le 18^{\circ} \text{ Governs} \qquad \text{\# bars required} = 17 \\ pt = Av/(St) \qquad pt = 0.0025 \text{ OK}$ S. Design for Flexture Sscuence franciscontrolled section, $\Phi = 0.9$ $Mn = As^{+}fy'(1/a(2)) = As^{+}fy')  jd = 0.9^{\circ}d$ $= 10.35^{\circ}f(2\pi^{+}b) = As^{+}fy')  jd = 0.9^{\circ}d$ $= 10.35^{\circ}f(2\pi^{+}b) = As^{+}fy')  jd = 0.9^{\circ}d$ $= 0.035^{\circ}f(2\pi^{+}b) = As^{+}fy'  c = 0.038^{\circ}f(2\pi^{+}b) = As^{+}fy'  c = 0.044^{\circ}f(2\pi^{+}b) = As^{+}fy'  c = 0.038^{\circ}f(2\pi^{+}b) = As^{+}fy'  c = 0.0025^{\circ}f(2\pi^{+}b) = As^{+}fy'  c = 0.002^{\circ}f(2\pi^{+}b) = As^{+}fy'  c = 0.002^{\circ}f$	Max. Spacing $S \le L/3 = 56$	S	=	12.00	in	USE								
$ pt = Av/(S^{*}) \qquad pt = 0.0350 > 0.0025 \text{ OK} $ S. Design for Flexture $ Sasume Tension-controlled section, \Phi = 0.9 \\ \text{in a Astry(-(1/2)) - Astry()}  jd = 0.9^{\circ} \text{ diam a set by (-(1/2)) - Astry()}  jd = 0.9^{\circ} \text{ diam a set by (-(1/2)) - Astry()}  jd = 0.9^{\circ} \text{ diam a set by (-(1/2)) - Astry()}  jd = 120.96  \text{in } \\ \text{ in a Astry(-(1/2)) - Astry()}  jd = 120.96  \text{in } \\ \text{ a } = 9.90  \text{in } \\ \text{ C = T } 0.85^{+}fc^{+}a^{+}b - As^{+}fy  c = 10.08  \text{in } \\ \text{ a } = 9.25  \text{in } \\ \text{ a } = 9.20  \text{in } \\ \text{ As } = 0.003  \text{ be characterized in } \\ \text{ As } = 0.003  \text{ be characterized in } \\ \text{ As } = 0.003  \text{ be characterized in } \\ \text{ As } = 0.003  \text{ be characterized in } \\ \text{ As } = 0.003  \text{ be characterized in } \\ \text{ As } = 0.003  \text{ be characterized in } \\ \text{ As } = 0.003  \text{ be characterized in } \\ \text{ As } = 0.003  \text{ be characterized in } \\ \text{ As } = 0.0035  \text{ be characterized in } \\ \text{ c = u'(dit-0/c)} \qquad \text{ Wall } 1 \\ \text{ c = u'(dit-0/c)} \qquad \text{ Wall } 1 \\ \text{ c = u'(dit-0/c)} \qquad \text{ Wall } 1 \\ \text{ c = u'(dit-0/c)} \qquad \text{ Wall } 1 \\ \text{ c = u'(dit-0/c)} \qquad \text{ Wall } 1 \\ \text{ c = u'(dit-0/c)} \qquad \text{ Wall } 1 \\ \text{ c = u'(dit-0/c)} \qquad \text{ well } 1 \\ \text{ c = u'(dit-0/c)} \qquad \text{ well } 1 \\ \text{ c = u'(dit-0/c)} \qquad \text{ c = u = 0.0025 } \text{ o K } \\ \text{ Wall DIST. FORIZ.REINF.} \qquad 17  \# 6  \textcircled{ e } 12  \circ \text{ O.C} \\ \text{ plyrewist } 0.0025  \text{ o K } \qquad \text{ WALL DIST. FORIZ.REINF.} \qquad 17  \# 6  \textcircled{ e } 12  \circ \text{ O.C} \\ \text{ plyrewist } 0.00369  \text{ (pl )}_{\text{inh.}} = 0.0025  \text{ O K } \qquad \text{ Wall } 1 \\ \text{ o C = C (cherk SHEAR CAPACITY (ACI 318-08 112 & 2.19.4) \\ \text{ o W_n } \leq 6.5974  \text{ kipt B M_3 } \text{ o } 5.498  \text{ O K } \\ \text{ where } 0 = 0.900 \qquad (ACI 318-08 \text{ Fig. R9.3.2) \\ \text{ the C ADUIDARY ZONE REQUIREMENTS } \\ \text{ AN EXEMPTION FROM THE REVOVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY \\ \text{ o K where } c = 11  \text{ in } . (distance from the extreme compression fiber to neutral axis at P_4.8 M_3 loas.) \\ \text{ where } c = 11  \text{ in } . (distance from the extreme compression fiber to neutral axis a$														
Solve in the set of t														
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As thy ('d-(A/2)) = As thy ('d-(A/	$\rho t = Av/(S^*t)$	ρt	=	0.0350	> 0.0025	ОК								
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As thy ('d-(A/2)) = As thy ('d-(A/	5. Design for Flexture													
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	Assume Tension-controlled section, $\Phi = 0.9$													
$\begin{aligned} Mu = \Phi  As^{r} f^{r} f^{r} d & As & = 10.10  in^{A} s & = 0.010  in^{A} s & c & = 10.89  in \\ a & = 9.90  in & C^{CT} d O S^{CT} d^{CT} a^{T} b a^{CT} f^{C} d^{C} s d^{C} s^{C} d^{C} d^{C$	$Mn = As^{fy^{(d-(a/2))}} = As^{fy^{j}}  jd = 0.9^{d}$						TRY	#6	A/bar	=	0.44	in^2		
$a = 9.90 \text{ in} \qquad C=T  0.85^{+}fc^{+}a^{+}b = As^{+}fy  c = 10.89 \text{ in} \\ c = a/0.85  et = 0.04  > 0.04  > 0.025  OK \\ As = 9.44  in^{+}2 \qquad eu = 0.003  dt = 1.3^{*} \\ et = eu^{+}((dt-c)/c) \qquad Wall 1 \\ \hline CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2) \qquad WALL DIST. HORIZ. REINF. \qquad 17  \#6  @  12  * 0.c. \\ plyrowd = 0.0350  >  (pt)_{min.} = 0.0025  OK \qquad WALL DIST. HORIZ. REINF. \qquad 24  \#6  @  12  * 0.c. \\ plyrowd = 0.0369  >  (pl)_{min.} = 0.0025  OK \qquad WALL DIST. VERT. REINF. \qquad 24  \#6  @  12  * 0.c. \\ checK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) \\ @V_n \leq A_{cv} (\alpha c^{+}b_{1}'c^{+} + p_{i}f_{i})  \alpha c = 2 (conservative) 13835 \\ where \ \phi = 0.090  (ACI 318-08 Fig. R9.3.2) \\ CHECK HEXURAL & AXIAL CAPACITY \\ THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_{u} IS GIVEN BY \\ @ M_{min} = 65.974  kip.ft \qquad > M_{u_{in}}  5.498  OK \\ where \ \phi = 0.900  (ACI 318-08 Fig. R9.3.2) \\ CHECK BOUNDARY ZONE REQUIREMENTS \\ AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, \\ 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT \\ c < (L^+h) / (600 d_u) for ACI 21.9.6.2 apply \qquad c < 40.00  in. No Boundary Element Needed \\ where \ c = 11  in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.) \\ \end{array}$	C=T 0.85*f'c*a*b =As*fy	jd	=	120.96	in				# bars required	=	21			
$jd = d \cdot (a/2)$ $jd = 129.45 in c = a/0.85 et = 0.04 > 0.0025 OK$ $ks = 9.44 in^{2} eu = 0.003 dt = t-3^{*} et = u^{*}((dt-c)/c)$ $Wall 1$ $CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)$ $WALL DIST. HORIZ. REINF.$ $I7  #6 @ 12 ° O.C.$ $plprovd. = 0.0369 > (pt)_{min.} = 0.0025 OK$ $WALL DIST. VERT. REINF.$ $I7  #6 @ 12 ° O.C.$ $plprovd. = 0.0369 > (pl)_{min.} = 0.0025 OK$ $CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)$ $\Phi V_n \le A_{cv} (ac^{*}\sqrt{f_c} + p_rf_y) ac = 2 (conservative) 13835 kips > V_u = 2113 OK$ $CHECK FLEXURAL & AXIAL CAPACITY$ $THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY \Phi M_n = 65.974 kip \cdot ft > M_u = 5.498 OK where \Phi = 0.900 (ACI 318-08 Fig. R93.2) CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT c < (1^{*}H) / (600 d_y) \text{ for ACI 21.9.6.2 apply } c < 40.00 in. No Boundary Element Needed where c = 11 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)$	$Mu = \Phi Mn = \Phi As*fy*j*d$	As	=	10.10	in^2		Check Capacity:		a	=	9.25	in		
As = 9.44 in ^2 $eu = 0.003 \text{ dt} = 1-3^{\circ}$ $et = eu^{\circ}((dt-c)/c)$ Wall 1 CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2) WALL DIST. HORIZ. REINF. ptprovel. = 0.0350 > (pt) <sub>min</sub> . = 0.0025 OK WALL DIST. HORIZ. REINF. ptprovel. = 0.0369 > (pt) <sub>min</sub> . = 0.0025 OK CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $\Phi V_n \le A_{cv}$ (act <sup>-v</sup> t <sup>-</sup> <sub>c</sub> + p <sub>e</sub> t <sup>-</sup> <sub>p</sub> ) ac = 2 (conservative) 13835 kips > V_u = 2113 OK CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL LOAD P <sub>u</sub> IS GIVEN BY $\Phi M_n = 65,974$ kip-ft > M <sub>u</sub> 5,498 OK where $\Phi = 0.900$ (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 $c < (t^{+})/(600 d_0)$ 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 <sub>u</sub> & M <sub>n</sub> loads.)		а	=	9.90	in		C=T	0.85*f'c*a*b =A	is*fy c	=	10.89	in		
$t = \varepsilon u^{*}((dt-c)/c) \qquad \text{Wall 1}$ $c = \varepsilon u^{*}((dt-c)/c) \qquad \text{Wall DIST. HORIZ. REINF.} \qquad 17 \qquad \#6 \qquad @ \qquad 12 \qquad ^{\circ} O.C.$ $p_{\text{proved.}} = 0.0350 \qquad > \qquad (pt)_{min.} = \qquad 0.0025 \qquad \text{OK} \qquad \text{WALL DIST. VERT. REINF.} \qquad 24 \qquad \#6 \qquad @ \qquad 12  ^{\circ} O.C.$ $p_{\text{proved.}} = 0.0369 \qquad > \qquad (p1)_{min.} = \qquad 0.0025 \qquad \text{OK} \qquad \text{WALL DIST. VERT. REINF.} \qquad 24 \qquad \#6 \qquad @ \qquad 12  ^{\circ} O.C.$ $p_{\text{proved.}} = 0.0369 \qquad > \qquad (p1)_{min.} = \qquad 0.0025 \qquad \text{OK} \qquad \text{Wall DIST. VERT. REINF.} \qquad 24 \qquad \#6 \qquad @ \qquad 12  ^{\circ} O.C.$ $p_{\text{proved.}} = 0.0369 \qquad > \qquad (p1)_{min.} = \qquad 0.0025 \qquad \text{OK} \qquad \qquad$	jd = d - (a/2)	jd	=	129.45	in				εt	=	0.04	> 0.0025	ОК	
CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)WALL DIST. HORIZ. REINF.17#6012° O.C. $ptgrowt. = 0.0350 > (pt)_{min.} = 0.0025 OKOKWALL DIST. VERT. REINF.17#6@12° O.C.plgrowt. = 0.0369 > (pl)_{min.} = 0.0025 OKOKWALL DIST. VERT. REINF.24#6@12° O.C.QV_n \leq A_{cv} (ac*\fc' + p,rfy)ac = 2 (conservative) 13835 kips > V_u =2113 OKOKQV_n \leq A_{cv} (ac*\fc' + p,rfy)ac = 2 (conservative) 13835 kips > V_u =2113 OKIf is a conservative is conservative is a conservati$		As	=	9.44	in^2		εu = 0.003	dt = L-3"						
$p_{\text{turout.}} = 0.0350 \Rightarrow (\text{pt})_{\text{min.}} = 0.0025 \text{ OK} \text{ WALL DIST. VERT. REINF.} 24 \#6 @ 12 ° O.C.$ $p_{\text{planot.}} = 0.0369 \Rightarrow (\text{pl})_{\text{min.}} = 0.0025 \text{ OK}$ $CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)$ $\Phi V_n \leq A_{cv} (\alpha c^{h} f_c^{+} + p_r f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } > V_u = 2113 \text{ OK}$ $CHECK FLEXURAL & AXIAL CAPACITY$ THE ALLOWABLE MOMENT AT AN AXIAL LOAD P <sub>u</sub> IS GIVEN BY $\Phi M_n = 65.974 \text{ kip-ft} \Rightarrow M_u = 5.498 \text{ OK}$ where $\Phi = 0.900$ (ACI 318-08 Fig. R93.2) CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION 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 $c < (t^{h}) / (600 d_{u}) \text{ for ACI 21.9.6.2 apply } c < 40.00 \text{ in. No Boundary Element Needed}$ where $c = 11$ in. (distance from the extreme compression fiber to neutral axis at P <sub>u</sub> & M <sub>n</sub> loads.)														
$p_{ provid.} = 0.0369 > (pl)_{min.} = 0.0025 \text{ OK}$ $CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)$ $\Phi V_n \le A_{cv} (\alpha c^{n} \sqrt{f_c} + p_r f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } > V_u = 2113 \text{ OK}$ $CHECK FLEXURAL & AXIAL CAPACITY$ THE ALLOWABLE MOMENT AT AN AXIAL LOAD P <sub>u</sub> IS GIVEN BY $\Phi M_n = 65974 \text{ kip-ft} \qquad > M_u = 5,498 \text{ OK}$ where $\Phi = 0.900$ (ACI 318-08 Fig. R93.2) CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION 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 $c < (1^{+H}) / (600 d_u) \text{ for ACI 21.9.6.2 apply } c < 40.00 \text{ in. No Boundary Element Needed}$ where $c = 11$ in. (distance from the extreme compression fiber to neutral axis at P <sub>u</sub> & M <sub>n</sub> loads.)	CHECK MINIMUM REINFORCEMENT RATIOS	AND SPACING (A	CI 318-	08 14.3, 21.	9.2)		WALL DIS	T. HORIZ. REINF		17	#6	0	12	" O.C.
CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $\Phi V_n \le A_{cv} (\alpha c^{h} f_c^+ + \rho_r f_y)$ $\alpha c = 2 (conservative) 13835$ kips > $V_u = 2113$ OK CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL LOAD $P_u$ IS GIVEN BY $\Phi M_n = 65.974$ kip-ft > $M_u = 5.498$ OK where $\Phi = 0.900$ (ACI 318-08 Fig. R93.2) CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION 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 $c < (1^{+h}) / (600 d_y)$ 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.)	ptprovd. = 0.0350 >	(pt ) <sub>min.</sub> =		0.0025		ок	WALL DIS	T. VERT. REINF.		24	#6	@	12	" O.C.
CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4) $\Phi V_n \le A_{cv} (\alpha c^{h} f_c^+ + \rho_r f_y)$ $\alpha c = 2 (conservative) 13835$ kips > $V_u = 2113$ OK CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL LOAD $P_u$ IS GIVEN BY $\Phi M_n = 65.974$ kip-ft > $M_u = 5.498$ OK where $\Phi = 0.900$ (ACI 318-08 Fig. R93.2) CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION 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 $c < (1^{+h}) / (600 d_y)$ 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.)	olorovd = 0.0369 >	(ρΙ) <sub>min.</sub> =		0.0025		ок								
$ \Phi V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_u \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 $	an an an an								>		1			
$ \Phi V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_n \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 \text{ kips } V_u = 2113 \text{ OK} $ $ P V_u \leq A_{c_v} (\alpha c^h f_c' + \rho_t f_y)  \alpha c = 2 \text{ (conservative) } 13835 $	CHECK SHEAR CAPACITY (ACI 318-08 11.2 &	& 21.9.4)									1			
CHECK FLEXURAL & AXIAL CAPACITY       e         THE ALLOWABLE MOMENT AT AN AXIAL LOAD $P_u$ IS GIVEN BY       f $\Phi M_n = 65,974$ kip-ft       > $M_u = 5,498$ OK         where $\Phi = 0.900$ (ACI 318-08 Fig. R9.3.2)       CHECK BOUNDARY ZONE REQUIREMENTS         AN EXEMPTION THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2,       21.9.6.5.(a) PROVIDED THAT         c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply			13835	kips	>	V	2113 OK			l	1			
THE ALLOWABLE MOMENT AT AN AXIAL LOAD $P_u$ IS GIVEN BY $\Phi M_n = 65,974$ kip-ft > $M_u = 5,498$ OK where $\Phi = 0.900$ (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 $c < (1^{+H}) / (600 d_u)$ 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.)	in considering and					-			_		1			
$ \Phi M_n = 65,974 \text{ kip-ft } > M_u = 5,498 \text{ OK} $ where $\Phi = 0.900 \text{ (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_w) \text{ for ACI 21.9.6.2 apply } c < 40.00 \text{ in. No Boundary Element Needed} $ where $c = 11$ in. (distance from the extreme compression fiber to neutral axis at P <sub>u</sub> & M <sub>n</sub> loads.)	CHECK FLEXURAL & AXIAL CAPACITY										e			
$ \Phi M_n = 65,974 \text{ kip-ft } > M_u = 5,498 \text{ OK} $ where $\Phi = 0.900 \text{ (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_w) \text{ for ACI 21.9.6.2 apply } c < 40.00 \text{ in. No Boundary Element Needed} $ where $c = 11$ in. (distance from the extreme compression fiber to neutral axis at P <sub>u</sub> & M <sub>n</sub> loads.)	THE ALLOWABLE MOMENT AT AN AXIAL L	OAD P., IS GIVEN	BY								f	_		
where $\Phi = 0.900$ (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 <sub>a</sub> ) for ACI 21.9.6.2 apply         where c = 11       in. (distance from the extreme compression fiber to neutral axis at P <sub>a</sub> & M <sub>n</sub> loads.)		-		5,498	ОК						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 <sub>u</sub> ) for ACI 21.9.6.2 apply         c < 11											1			
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 < (1*H) / (600 d <sub>u</sub> ) for ACI 21.9.6.2 apply c < 40.00 in. <b>No Boundary Element Needed</b> where c = 11 in. (distance from the extreme compression fiber to neutral axis at P <sub>u</sub> & M <sub>n</sub> loads. )		(									1			
21.9.6.3, and 21.9.6.5(a) PROVIDED THAT c < (1*H) / (600 d <sub>u</sub> ) 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 <sub>u</sub> & M <sub>n</sub> loads.)	•	BOUNDARY ZONE		EMENT REI	NFORCEM	ENT IS GP	/EN BY ACI318-05 21 9 F	5.2.						
$c < (t^{+}H) / (600 d_u)$ for ACI 21.9.6.2 apply $c < 40.00$ in. <b>No Boundary Element Needed</b> where $c = 11$ in. (distance from the extreme compression fiber to neutral axis at P <sub>u</sub> & M <sub>n</sub> loads.)		2.2.51157111 2014			Onceivi			,			1			
where $c = 11$ in. (distance from the extreme compression fiber to neutral axis at $P_u \& M_n$ loads.)		vlaa	c <	40.00	in	No Bou	ndarv Element Needed		$\longrightarrow$	1				
		FF')					, Lieinene iteeded			1	1			
	where c = 11	in (distance fre	m the e	treme com	maccion fi	her to no:	tral avic at P & M los	de )						

INPUT DATA & I	DESIGN SU	MMARY		Wall	11 f			X-Direction							
CONCRETE STREM	NGTH (ACI 3	318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E		Pu	=	202.8	k	
REBAR YIELD STR	ESS		f <sub>v</sub>	=	60	ksi		FACTORED MOMEN	IT LOAD		M.,	=	443.4	ft-k	
HEIGHT OF WALL	-		Ĥ	=	462.0	in		FACTORED SHEAR L	.OAD		V.	=	151.5	k	
LENGTH OF SHEA			1	=	148.0	in					- u				
THICKNESS OF W			t	=	18	in		THE WAI	L DESIGN IS ADEQU	ATF					
THICKINESS OF V	WILL .		Acv	_		in^2			L DESIGN IS ADEQU						
ACT 218-08 & 21 0	22 TE VIL >	2*A au*s//f'a) : no	ed at least two curtains		337.0	Need 1									
1. Check Permitt				(10ws) =	557.0	INCEU 1		4 Poquirod Vortica	I Shear Reiforcement	•					
ACI 318-08 § 11.9		rength							5 +0.5 (2.5 - h/L)*(pt-		ρΙ	=	0.03265	>0.0025	OK
$\Phi Vn \geq 1$			Vu	=	151.5	kip		Max. Spacing	S ≤ L/3 =	49.332	S	-		in	UK
Vn = Vc			d	=	118.4	in		Max. spacing	S ≤ 3t =	49.532 54	3	-	0		
	. + vs *t*d*√(f'c)	d=0.9*I	Vn	=	1347.9	kip			S ≤ 18"	Governs					
VII 2 10	(IC)	u=0.0 L	ΦVn	-							A /h = =	=	1.56	:- 4 2	
					1010.9	kip			TR	f #11	A/bar			in^2	
ACI 318-08 § 21.9			Vn	≤	18047.9	kip					# bars required	=	2		
$V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'})$			tive)		OK			ACI 318-08 § 21.9.4.4	4, IF hw/lw≤2; need re	inf. In two directions (pl					
2. Shear Strength					0.00.0						h/l	=	3.1217		
		λ = 1.0 (for N.		=	269.6	kip					ρl≥ρt	is	NOT OK		
Note: If Vu≤Acv*				=	168.5	Accordi	ng to Ch.14								
3. Required Horiz		r Reinforceme													
1/2ΦVc	< Vu		1/2ΦVc	=	101.1	kip									
					According t				ST. HORIZ. REINF.		8	#8	@	8	" O.C
	/(0.75) - Vc		Vs	=	-67.6	kip		WALL DIS	ST. VERT. REINF.		2	#11	@	6	" O.C
$S = (Av^*)$	'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	$S \leq L/3 =$	49.332	S	=	8.00	in	USE								
	S ≤ 3t =	54	Av	=	2663.93										
	S ≤ 18"	Governs	# bars required	=	8										
pt = Av/	′(S*t)		ρt	=	0.0462	> 0.0025	ОК							FALSE	
5. Design for Fle										A/bar	=	1.56	in^2		
Assume Tension-										# bars required	=	1			
Mn = As*fy*(d-(a,		j jd = 0.9*d						TRY	<b>Y</b> #11						
	a*b =As*fy		jd	=	106.56	in									
$Mu = \Phi Mn = \Phi$	As*fy*j*d		As	=	0.92	in^2		Check Capacity:		а	=	0.82			
			а	=	0.91	in		C=T	0.85*f'c*a*b =As*fy		=		in		
jd = d -	(a/2)		jd	=	117.94	in			c = a/0.85	εt	=	0.46	>0.0025		
			As	=	0.84	in^2			3 dt = L-3"						
								εt = εu*((α	dt-c)/c)		Wa	all 1			
CHECK BOUNDA															
			OF BOUNDARY ZONE	E CONFI	NEMENT REIN	IFORCEM	ENT IS GIVE	N BY ACI318-05 21.9.	6.2,						
21.9.6.3, and 2	1.9.6.5(a) Pl	ROVIDED THAT													
c < (L*H	) / (600 d <sub>u</sub> )	for ACI 21.9.6.	2 apply	c <	0.96	in.	No Bound	ary Element Needed	l						
where	e c =	0	in. ( distance fro	om the e	extreme comp	ression fi	ber to neutr	al axis at P <sub>u</sub> & M <sub>n</sub> loa	ads. )						
	d <sub>u</sub> =	38.5	in. ( design disr	placemer	nt, assume 0.0	07*H cor	nservative, se	e ACI 318-08 21.9.6.2	a.)						
CHECK MINIMU	-		OS AND SPACING (A												
		>	(pt ) <sub>min.</sub> =		0.0025		ок								
ptprovd. =															
plprovd. =	0.0326	>	(ρΙ ) <sub>min.</sub> =		0.0025		ок								
			2 2 21 2 4												
CHECK SHEAR C															
ΦV <sub>n</sub> ≤ A	λ <sub>cv</sub> (αc*√f <sub>c</sub> ' +	· ρ <sub>t*</sub> f <sub>y</sub> )	$\alpha c = 2$ (conservative)	1010.9	kips	>	V <sub>u =</sub>	151.5 <b>OK</b>							
		CAPACITY													
	I & AVIAL	CAPACITI													
CHECK FLEXURA			LOAD D. TO CRIEN	D1/					T		0	"0	0	0.00	
CHECK FLEXURA THE ALLOWAI	BLE MOMEI	NT AT AN AXIA	l load P <sub>u</sub> IS given						T. HORIZ. REINF.		8	#8	@	8.00	" O.C
<b>CHECK FLEXURA</b> THE ALLOWAI Φ M <sub>n</sub> =	BLE MOMEI 5,321	NT AT AN AXIA kip-ft	L LOAD P <sub>u</sub> IS GIVEN > e <sub>t</sub> - 0.002)(250/3) , 0.6	M <sub>u =</sub>	443 0.900	ок	8-08 Fig. R9.	WALL DIS	ST. HORIZ. REINF. ST. VERT. REINF.		8 14	#8 #11	@ @		" 0.0 " 0.0

		Specia	al Reinforc	ed Concr	ete Shea	r Wall Design Based	on ACI 318-08 Ch	. 21.9					
INPUT DATA & DESIGN SUMMARY	W	Vall 1	.2 e			X-Direction							
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E		Pu	=	1131	k	at BAS
REBAR YIELD STRESS	f <sub>v</sub>	=	60	ksi		FACTORED BASE M	OMENT LOAD		M <sub>ii</sub>	=	5471	ft-k	
HEIGHT OF WALL	Ĥ	=	400.0	in		FACTORED BASE S	HEAR LOAD		V	=	2105	k	
LENGTH OF SHEAR WALL		=	148.0	in					- u				
THICKNESS OF WALL	t	=	18	in		THE WAI	L DESIGN IS ADEQU	IATE.					
	Acv	=	2663.9928										
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*v(f'c) ; need			337.0	Need 2									
1. Check Permitted Shear Strength		,				4. Required Vertica	I Shear Reiforcemen	t		-			
ACI 318-08 § 11.9							5 +0.5 (2.5 - h/L)*(pt-		ρΙ	=	0.06167	> 0.0025	ок
ΦVn ≥ Vu	Vu	=	2105.0	kip		Max. Spacing	S ≤ L/3 =	49.3332	S	=	12		
Vn = Vc + Vs	d	=	118.4	in		5	S ≤ 3t =	54					
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1347.9	kip			S ≤ 18"	Governs					
	ΦVn	=	1010.9	kip			5 _ 10 TR'		A/bar	=	0.44	in^2	
ACI 318-08 § 21.9.4	Vn	≤	15584.3	kip					# bars required	=	30	2	
$V_n \leq A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_v)  \alpha c = 2 (conservative)$		-	0K	NIP		ACT 318-08 § 21 9.4	1 IF hw/lw<2 · need re	einf. In two directions (pl		-	50		
2. Shear Strength Provided by Vc	<i>i</i> c <i>j</i>		UN				.,,		∠pt) h/l	=	2.7024	FALSE	
2. Snear Strength Provided by VC $Vc \le 2^{\lambda}t^{*}d^{*}\sqrt{f'c}$ $\lambda = 1.0$ (for N.W	(.C) Vc	=	269.6	kip					n/i pl≥pt	= is	2.7024 OK	FALSE	
Note: If $Vu \le Acv^* \sqrt{(f'c)}$ can choose pt, pl accor		_	168.5	FALSE					hichr	15	UK.		
3. Required Horizontal Shear Reinforcement		-	100.5	I ALSE									
1/20Vc < Vu	1/2ΦVc	=	101.1	kip									
1/2000 < 00	1/2000	-	According t				T. HORIZ. REINF.		15	#6	@	12	" O.C.
Vs = Vu/(0.75) - Vc	Vs	=	2537.1	kip			ST. VERT. REINF.		30	#6	0		" O.C.
S = (Av*fy*d)/Vs	Ag	=	2663.9928			WALL DI	DI. VENI. KEINI.		50	#0	<sup>w</sup>	12	0.c.
3 = (AV IY U)/VS	0.0025*Ag	=	6.7	in^2									
<b>TRY</b> #6	Abar	-	0.44	in ^2									
Max. Spacing S ≤ L/3 = 49.3332	S	-	12.00	in	USE								
$S \le 3t = 54$	5	-	12.00		UJL								
$S \leq 18$ Governs	# bars required	=	15										
$\rho t = Av/(S^*t)$	pt	-		> 0.0025	ок								
5. Design for Flexture													
Assume Tension-controlled section, $\Phi = 0.9$													
$Mn = As^{fy^{(d-(a/2))}} = As^{fy^{j}} \qquad jd = 0.9^{d}$						TR	<b>Y</b> #6	A/bar	=	0.44	in^2		
C=T 0.85*f'c*a*b =As*fy	jd	=	106.56	in				# bars required	=	24			
$Mu = \Phi Mn = \Phi As^*fy^*j^*d$	As	=	11.41	in^2		Check Capacity:		а	=	10.57	in		
	а	=	11.19	in		C=T	0.85*f'c*a*b =As*f		=	12.43			
jd = d - (a/2)	jd	=	112.81	in			c = a/0.85	εt	=		> 0.0025	ок	
<u>.</u>	As	=	10.78	in^2		EU = 0.00	3 dt = L-3"						
						εt = εu*((			Wa	all 1			
CHECK MINIMUM REINFORCEMENT RATIO	S AND SPACING (ACI	318-0	8 14.3, 21.9	9.2)			T. HORIZ. REINF.		15	#6	0	12	" O.C.
ptprovd. = 0.0308 >	(pt ) <sub>min</sub> =		0.0025		ок		T. VERT. REINF.		32	#6	@	12	" O.C.
						WALL DI	DI. VENI. KEINI.		JZ	#0	<sup>w</sup>	12	0.c.
plprovd. = 0.0617 >	(pl ) <sub>min.</sub> =		0.0025		ок								
CHECK SHEAR CAPACITY (ACI 318-08 11.2								$\longrightarrow$					
$\Phi V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)$ $\alpha$	c = 2 (conservative) 116	688	kips	>	V <sub>u =</sub>	2105 OK		$\longrightarrow$					
CHECK FLEXURAL & AXIAL CAPACITY										e	_		
THE ALLOWABLE MOMENT AT AN AXIAL I	LOAD P <sub>u</sub> IS GIVEN BY									f			
Φ M <sub>n</sub> = 65,652 kip-ft	> M <sub>u</sub>	u =	5,471	ОК									
where $\Phi = 0.900$	(ACI 318-08 Fig. R9.3												
where $\Psi = 0.900$													
	F BOUNDARY ZONE CO	ONFIN	EMENT REIN	FORCEM	ENT IS GI	VEN BY ACI318-05 21.9	6.2,						
CHECK BOUNDARY ZONE REQUIREMENTS	IF BOUNDARY ZONE CO	ONFIN	EMENT REIN	IFORCEM	ent is gi	VEN BY ACI318-05 21.9	6.2,	、					
CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION O 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT			EMENT REIN 35.24										
CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION O	apply c <	<	35.24	in.	No Bou	VEN BY ACI318-05 21.9 <b>ndary Element Needec</b> utral axis at P <sub>u</sub> & M <sub>n</sub> lo							

INPUT DATA 8	& DESIGN SU	MMARY		Wall	12 f			X-Direction							
CONCRETE STR	ENGTH (ACI	318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E		Pu	=	703.0	k	
REBAR YIELD ST	TRESS		f <sub>v</sub>	=	60	ksi		FACTORED MOMEN	IT LOAD		M.	=	2887.0	ft-k	
HEIGHT OF WA	.u. —		Ĥ	=	462.0	in		FACTORED SHEAR	OAD		V.,	=	187.0	k	
LENGTH OF SH	EAR WALL		L	=	148.0	in					ŭ				
THICKNESS OF			t	=	18	in		THE WAI	L DESIGN IS ADEQU	ATE.					
			Acv	=	2663.9928	in^2									
ACI 318-08 § 21		: 2*Acv*V(f'c) :	need at least two curta	ains (rows) =	337.0	Need 1									
1. Check Permi								4. Required Vertica	I Shear Reiforcemen	t					
ACI 318-08 § 11		5							!5 +0.5 (2.5 - h/L)*(ρt-		ρΙ	=	0.06167	> 0.0025	ок
ΦVn ≩			Vu	=	187.0	kip		Max. Spacing	S ≤ L/3 =	49.3332	S	=	6	in	
Vn = '	Vc + Vs		d	=	118.4	in .			S ≤ 3t =	54					
Vn ≤	10*t*d*√(f'c)	d=0.8*L	Vn	=	1347.9	kip			S ≤ 18"	Governs					
			ΦVn	=	1010.9	kip			TR	<b>(</b> #11	A/bar	=	1.56	in^2	
ACI 318-08 § 21	L.9.4		Vn	≤	18048.5	kip					# bars required	=	4		
$V_n \leq A_{cv} (\alpha c^* \sqrt{f})$		c = 2 (conse			OK			ACI 318-08 § 21.9.4.	4, IF hw/lw≤2; need re	inf. In two directions (o					
2. Shear Streng									.,, ,		h/l	=	3.1216	FALSE	
	2*λ*t*d*√(f'c)		N.W.C) Vc	=	269.6	kip					ρl≥ρt	is	OK		
			according to Ch.14	=	168.5	FALSE					F.=P.		2		
3. Required Ho				-	100.5										
	/c < Vu		1/2ΦVc	=	101.1	kip									
_, _ + •			_, _ + + • c		According 1			WALL DI	ST. HORIZ. REINF.		8	#8	@	8	" O.C
Vs = V	Vu/(0.75) - Vo		Vs	=	-20.2	kip			ST. VERT. REINF.		4	#11	@		" O.C
	va/(0.75) va v*fy*d)/Vs		Ag	=	2663.9928								e	Ŭ	0.0
5 (/	., -//•5		0.0025*Ad		6.7	in^2									
	TRY	#8	A/bar	=	0.79	in^2									
Max. Spacing	S ≤ L/3 =		S	=	8.00	in	USE								
	S ≤ 3t =		Av	=	2663.99										
	S ≤ 18"	Governs	# bars requi		8										
ρt = A	Av/(S*t)		pt	=	0.0462	> 0.0025	ок							FALSE	
5. Design for F										A/bar	=	1.56	in^2		
Assume Tensio										# bars required	=	4			
Mn = As*fy*(d-		*j jd = 0.						TR	Y #11						
	c*a*b =As*fy		jd	=	106.56	in									
Mu = Φ Mn =	Φ As*fy*j*d		As	=	6.02	in^2		Check Capacity:		а	=	5.45			
			а	=	5.90	in		C=T	0.85*f'c*a*b =As*fy		=		in		
jd = d	l - (a/2)		jd	=	115.45	in			c = a/0.85	εt	=	0.07	>0.0025		
			As	=	5.56	in^2			3 dt = L-3"						
								εt = εu*((	dt-c)/c)		Wa	all 1			
CHECK BOUND															
				UNE CONFI	INEIVIEINT REIN	NFORCEM	EINT IS GIV	EN BY ACI318-05 21.9	.0.Z,						
	1 21.9.6.5(a) P				6.41		- ·	-							
	*H) / (600 d <sub>u</sub> )			c <	6.41		-	Element Needed							
whe								ral axis at P <sub>u</sub> & M <sub>n</sub> Io							
	d <sub>u</sub> =						nservative, s	ee ACI 318-08 21.9.6.2	2a. )						
CHECK MINIM	UM REINFOR	RCEMENT R	ATIOS AND SPACIN	G (ACI 318	-08 14.3, 21.9	9.2)									
ptprovd.	= 0.0462	>	(pt ) <sub>mi</sub>	n. =	0.0025		ок								
plprovd.	= 0.0617	>	(pl ) <sub>mi</sub>	n. =	0.0025		ок								
CHECK SHEAR	CADACITY	ACT 210 00	11 2 8 21 0 4												
				tive) 1010.0	king		V	1970 <b>OK</b>							
ΨV <sub>n</sub> ≤	≤ A <sub>cv</sub> (αc*√f <sub>c</sub> ' ·	+ p <sub>t*</sub> τ <sub>y</sub> )	αc = 2 (conservat	uve) 1010.9	kips	>	V <sub>u =</sub>	187.0 <b>OK</b>							
CHECK FLEXU	RAL & AXIAI	CAPACITY													
	ABLE MOME	NT AT AN A	ALL LOAD Pu IS GIV	'EN BY				WALL DI	ST. HORIZ. REINF.		8	#8	@	8.00	" O.C
THE ALLOW															
		kip-ft	>	M	2,887	ок		WALL DI	ST. VERT. REINF.		14	#11	@	6	" O.C
ΦM <sub>n</sub>	= 34,644		+ (e <sub>t</sub> - 0.002)(250/3)	M <sub>u =</sub>	2,887 0.900		8-08 Fig. R9		ST. VERT. REINF.		14	#11	@	6	" 0.0

		Speci	al Reinforc	ed Conc	rete Shea	r Wall Design Based	on ACI 318-0	8 Ch. 2	1.9					
INPUT DATA & DESIGN SUMMARY		Wall 3	13			X-Direction								
CONCRETE STRENGTH (ACI 318 5.1.1)	f <sub>c</sub> '	=	4	ksi		Load Combo: 1.2 D	+ 1.0L +1.0E			Pu	=	3716	k	at BAS
REBAR YIELD STRESS	f <sub>v</sub>	=	60	ksi		FACTORED BASE M	DMENT LOAD			Mu	=	41262	ft-k	
HEIGHT OF WALL	Ĥ	=	376.0	in		FACTORED BASE SI	IEAR LOAD			Vu	=	1317	k	
LENGTH OF SHEAR WALL	L	=	254.0	in										
THICKNESS OF WALL	t	=	24	in		THE WAL	L DESIGN IS AD	DEQUAT	E.					
	Acv	=	6096.96	in^2										
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*V(f'c) ; need	d at least two curtains (ro	ows) =	771.2	Need 2										
1. Check Permitted Shear Strength						4. Required Vertica								
ACI 318-08 § 11.9						$\rho I = Av/S*h \ge 0.002$		.)*(pt-0.0	1025)	ρΙ	=		>0.0025	ок
ΦVn ≥ Vu	Vu	=	1317.0	kip		Max. Spacing	S ≤ L/3 =		34.68	S	=	6	in	
Vn = Vc + Vs	d	=	203.2	in			S ≤ 3t =		72					
$Vn \le 10^{t} d^{t} \sqrt{f'c}$ $d=0.8^{t} L$	Vn	=	3084.8	kip			S ≤ 18"		Governs					
	ΦVn	=	2313.6	kip				TRY	#11	A/bar	=	1.56	in^2	
ACI 318-08 § 21.9.4	Vn	≤	53429.2	kip						# bars required	=	11		
$V_n \le A_{cv} (\alpha c^* \sqrt{f_c'} + \rho_{t^*} f_y)  \alpha c = 2 (conservative)$	ve)		OK			ACI 318-08 § 21.9.4.4	l, IF hw/lw≤2;ne	eed reinf	. In two directions (ρ					
2. Shear Strength Provided by Vc										h/l	=	1.4799	ρl≥ρt	
$Vc \le 2^{\lambda}t^{*}d^{*}\sqrt{f'c}$ $\lambda = 1.0$ (for N.W		=	617.0	kip						ρl≥ρt	is	ОК		
Note: If Vu≤Acv*√(f'c) can choose pt, pl accor		=	385.6	FALSE										
3. Required Horizontal Shear Reinforcement			004											
1/2ΦVc < Vu	1/2ΦVc	=	231.4	kip			-			10	"0	~	0	
V V (0.75) V			According				T. HORIZ. REIN			19 11	#8	@ @	8 6	" O.C.
Vs = Vu/(0.75) - Vc $S = (Av^*fy^*d)/Vs$	Vs	=	1139.0 6096.96	kip in^2		WALL DIS	T. VERT. REINF	•		11	#11	ŵ	6	" O.C.
$S = (AV^{-1}y^{-1}d)/VS$	Ag 0.0025*Ag	=	15.2	in^2										
<b>TRY</b> #8	Abar	=	0.79	in^2										
Max. Spacing S ≤ L/3 = 84.68	S	-	8.00	in	USE									
$S \le 2t = 72$	5	-	0.00		USL									
$S \le 18''$ Governs	# bars required	=	19											
$\rho t = Av/(S^*t)$	ρt	=	0.0794	> 0.0025	ок									
E Design for Eleviture														
5. Design for Flexture Assume Tension controlled section $\Phi = 0.9$														
Assume Tension-controlled section, $\Phi = 0.9$						TR	/ #11		A/bar		1 56	in^2		
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d	id	_	182 91	in		TR	4 #11		A/bar # bars required	=		in^2		
	jd As	=	182.91 50.13	in in^2			#11		# bars required	= = =	32			
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d	jd As a		50.13	in in^2 in		TR Check Capacity: C=T		As*fv	# bars required a	=		in		
$ \begin{array}{l} \mbox{Assume Tension-controlled section, } \Phi = 0.9 \\ \mbox{Mn} = As^{*} f y^{*} (d \cdot (a/2)) = As^{*} f y^{*} j & jd = 0.9^{*} d \\ \mbox{C=T} & 0.85^{*} f c^{*} a^{*} b = As^{*} f y \\ \mbox{Mu} = \Phi \mbox{ Mn} = \Phi \mbox{As}^{*} f y^{*} d \\ \end{array} $	As a	=		in^2		Check Capacity:	<pre>/ #11 0.85*f'c*a*b = c = a/0.85</pre>	As*fy	# bars required	=	32 36.48 42.92	in in	ок	
	As	= =	50.13 36.86	in^2 in		Check Capacity: C=T	0.85*f'c*a*b =	As*fy	# bars required a c	= = =	32 36.48 42.92	in	ок	
$ \begin{array}{l} \mbox{Assume Tension-controlled section, } \Phi = 0.9 \\ \mbox{Mn} = As^{+} f y^{*} (d \cdot (a/2)) = As^{+} f y^{+} j  jd = 0.9^{+} d \\ \mbox{C=T} \qquad 0.85^{+} f^{*} c^{+} a^{+} b = As^{+} f y \\ \mbox{Mu} = \Phi \mbox{ Mn} = \Phi \mbox{As}^{+} f y^{+} d \\ \end{array} $	As a jd	= = =	50.13 36.86 184.80	in^2 in in		Check Capacity: C=T	0.85*f'c*a*b = c = a/0.85 dt = L-3"	As*fy	# bars required a c	= = =	32 36.48 42.92	in in	ок	
$ \begin{array}{l} \mbox{Assume Tension-controlled section, } \Phi = 0.9 \\ \mbox{Mn} = As^{+} f y^{*} (d \cdot (a/2)) = As^{+} f y^{+} j  jd = 0.9^{+} d \\ \mbox{C=T} \qquad 0.85^{+} f^{*} c^{+} a^{+} b = As^{+} f y \\ \mbox{Mu} = \Phi \mbox{ Mn} = \Phi \mbox{As}^{+} f y^{+} d \\ \end{array} $	As a jd As	= = =	50.13 36.86 184.80 49.62	in^2 in in in^2		<b>Check Capacity:</b> C=T εu = 0.003	0.85*f'c*a*b = c = a/0.85 dt = L-3"	As*fy	# bars required a c	= = =	32 36.48 42.92 0.01	in in	ок	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As*fy*(d-(a/2)) = As*fy*j jd = 0.9*d C=T 0.85*fc*a*b = As*fy Mu = $\Phi$ Mn = $\Phi$ As*fy*j*d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO	As a jd As	= = =	50.13 36.86 184.80 49.62	in^2 in in in^2	ок	<b>Check Capacity:</b> C=T εu = 0.003	0.85*f'c*a*b = c = a/0.85 dt = L-3"	As*fy	# bars required a c	= = =	32 36.48 42.92 0.01	in in	ок	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As*fy*(d-(a/2)) = As*fy*j jd = 0.9*d C=T 0.85*fc*a*b = As*fy Mu = $\Phi$ Mn = $\Phi$ As*fy*j*d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptprod. = 0.0794 >	As a jd As <b>PS AND SPACING (AC</b> (pt ) <sub>min.</sub> =	= = =	50.13 36.86 184.80 49.62 08 14.3, 21.9 0.0025	in^2 in in in^2		<b>Check Capacity:</b> C=T εu = 0.002 εt = εu*((c	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c)		# bars required a c	= = = W	32 36.48 42.92 0.01 all 1	in in >0.0025		* 0.6
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As*fy*(d-(a/2)) = As*fy*j jd = 0.9*d C=T 0.85*fc*a*b = As*fy Mu = $\Phi$ Mn = $\Phi$ As*fy*j*d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO	As a jd As <b>S AND SPACING (AC</b>	= = =	50.13 36.86 184.80 49.62 08 14.3, 21.9	in^2 in in in^2	ОК	Check Capacity: C=T εu = 0.002 εt = εu*((α WALL DIS	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c) T. HORIZ. REIN	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	* O.C.
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As'ty*(d-(a/2)) = As*fy*j jd = 0.9*d C=T 0.85*fc*a*b = As*fy Mu = $\Phi$ Mn = $\Phi$ As*fy*j*d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptgrovd. = 0.0794 > ptgrovd. = 0.1186 >	As a jd As SAND SPACING (AC (ρt ) <sub>min.</sub> = (ρl ) <sub>min.</sub> =	= = =	50.13 36.86 184.80 49.62 08 14.3, 21.9 0.0025	in^2 in in in^2		Check Capacity: C=T εu = 0.002 εt = εu*((α WALL DIS	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c)	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1	in in >0.0025	8	" O.C. " O.C.
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As <sup>+</sup> fy <sup>+</sup> (d-(a/2)) = As <sup>+</sup> fy <sup>+</sup> j d = 0.9 <sup>+</sup> d C=T 0.85 <sup>+</sup> f'c <sup>+</sup> a <sup>+</sup> b = As <sup>+</sup> fy Mu = $\Phi$ Mn = $\Phi$ As <sup>+</sup> fy <sup>+</sup> j <sup>+</sup> d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptgroud. = 0.0794 > ptgroud. = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2	As a jd As S AND SPACING (AC (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = & 21.9.4)	= = = 21 318-0	50.13 36.86 184.80 49.62 08 14.3, 21.9 0.0025 0.0025	in^2 in in^2 9.2)	ок	Check Capacity: C=T εu = 0.003 εt = εu*((c WALL DIS WALL DIS	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c) T. HORIZ. REIN	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As <sup>+</sup> fy <sup>+</sup> (d-(a/2)) = As <sup>+</sup> fy <sup>+</sup> j d = 0.9 <sup>+</sup> d C=T 0.85 <sup>+</sup> f'c <sup>+</sup> a <sup>+</sup> b = As <sup>+</sup> fy Mu = $\Phi$ Mn = $\Phi$ As <sup>+</sup> fy <sup>+</sup> j <sup>+</sup> d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptgroud. = 0.0794 > ptgroud. = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2	As a jd As SAND SPACING (AC (ρt ) <sub>min.</sub> = (ρl ) <sub>min.</sub> =	= = = 21 318-0	50.13 36.86 184.80 49.62 08 14.3, 21.9 0.0025	in^2 in in in^2		Check Capacity: C=T εu = 0.002 εt = εu*((α WALL DIS	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c) T. HORIZ. REIN	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
$ \begin{array}{llllllllllllllllllllllllllllllllllll$	As a jd As S AND SPACING (AC (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = & 21.9.4)	= = = 21 318-0	50.13 36.86 184.80 49.62 08 14.3, 21.9 0.0025 0.0025	in^2 in in^2 9.2)	ок	Check Capacity: C=T εu = 0.003 εt = εu*((c WALL DIS WALL DIS	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c) T. HORIZ. REIN	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As fyr(d-(a/2)) = As fyr) jd = 0.9 d C=T 0.85 ffcatb As fyr) Mu = $\Phi$ Mn = $\Phi$ As fyr) d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptprovd. = 0.0794 > ptprovd. = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2 $\Phi V_n \le A_{cr} (\alpha c^{-1} f_c + \rho_n f_y)$ o CHECK FLEXURAL & AXIAL CAPACITY	As a jd As <b>SAND SPACING (AC</b> (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = & 21.9.4) ac = 2 (conservative) 4	= = = CI 318-0	50.13 36.86 184.80 49.62 08 14.3, 21.9 0.0025 0.0025	in^2 in in^2 9.2)	ок	Check Capacity: C=T εu = 0.003 εt = εu*((c WALL DIS WALL DIS	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c) T. HORIZ. REIN	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As fyr(d-(a/2)) = As fyr j d = 0.9 d C=T 0.85 ffc*a*b = As fyr Mu = $\Phi$ Mn = $\Phi$ As fyf j*d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO plgrowd. = 0.0794 > plgrowd. = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2 $\Phi V_n \le A_{cv} (\alpha c^* h_c^* + \rho_r f_y)$ o CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL	As a jd As <b>DS AND SPACING (ACC</b> (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = <b>&amp; 21.9.4</b> ) KC = 2 (conservative) <b>4</b> LOAD P <sub>u</sub> IS GIVEN B <sup>1</sup>	= = = CI 318-C	50.13 36.86 184.80 49.62 <b>18 14.3, 21.</b> 0.0025 0.0025 0.0025	in^2 in in^2 <b>9.2)</b>	ок	Check Capacity: C=T εu = 0.003 εt = εu*((c WALL DIS WALL DIS	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c) T. HORIZ. REIN	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As 'fy'(d-(a/2)) = As 'fy'j jd = 0.9 'd C=T 0.85 'f'c'a'b = As 'fy Mu = $\Phi$ Mn = $\Phi$ As 'fy'j'd jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptprovd. = 0.0794 > ptprovd. = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2 $\Phi V_n \le A_{cv} (\alpha c^* h_c' + \rho_n f_v)$ o CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL $\Phi M_n = 495,139$ kip-ft	As a jd As <b>PS AND SPACING (AC</b> (pt ) <sub>min.</sub> = (pl ) <sub>min.</sub> = & 21.9.4) KC = 2 (conservative) 4 LOAD P <sub>u</sub> IS GIVEN B <sup>1</sup> > M	= = = 1 318-0 10072 Y M <sub>u</sub> =	50.13 36.86 184.80 49.62 08 14.3, 21.9 0.0025 0.0025	in^2 in in^2 9.2)	ок	Check Capacity: C=T εu = 0.003 εt = εu*((c WALL DIS WALL DIS	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c) T. HORIZ. REIN	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As'ty'(d-(a/2)) = As'ty' jd = 0.9'd C=T 0.85'tf'(d-a') = As'ty' Mu = $\Phi$ Mn = $\Phi$ As'tfy'i'd jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO plaroid. = 0.0794 > plaroid. = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2 $\Phi$ Vn = $A_{cv} (\alpha c^{*} fc' + \rho_{r} fy)$ 0 CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL $\Phi$ Mn = 495.139 kip-ft where $\Phi$ = 0.900	As a jd As <b>DS AND SPACING (ACC</b> (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = <b>&amp; 21.9.4</b> ) KC = 2 (conservative) <b>4</b> LOAD P <sub>u</sub> IS GIVEN B <sup>1</sup>	= = = 1 318-0 10072 Y M <sub>u</sub> =	50.13 36.86 184.80 49.62 <b>18 14.3, 21.</b> 0.0025 0.0025 0.0025	in^2 in in^2 <b>9.2)</b>	ок	Check Capacity: C=T εu = 0.003 εt = εu*((c WALL DIS WALL DIS	0.85*f'c*a*b = c = a/0.85 dt = L-3" it-c)/c) T. HORIZ. REIN	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As fyr(d-(a/2)) = As fyr j d = 0.9 d C=T 0.85 ffc+a+b = As fy Mu = $\Phi$ Mn = $\Phi$ As fy'j+d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptgrovd. = 0.0794 > ptgrovd. = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2 $\Phi V_n \le A_{cv} (\alpha c^+ \sqrt{f_c} + \rho_v f_y)$ o CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL $\Phi M_n = \Phi S_{1.23}$ kip-ft where $\Phi S_{1.00}$ Not	As a jd As SAND SPACING (ACC (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = & 21.9.4) to C = 2 (conservative) LOAD P <sub>u</sub> IS GIVEN B <sup>b</sup> > N (ACI 318-08 Fig. R <sup>5</sup>	= = = CI 318-0 00072 Y M <sub>u</sub> = 9.3.2)	50.13 36.86 184.80 49.62 88 14.3, 21.9 0.0025 0.0025 kips 41,262	in^2 in in^2 9.2)	OK V <sub>u =</sub>	Check Capacity: C=T εu = 0.003 εt = εu*(( WALL DIS WALL DIS 1317 ΟΚ	0.85*fc*a*b = c = a/0.85 dt = L-3" tt-c)/c) T. HORIZ. REIN T. VERT. REINF	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As fyr(d-(a/2)) = As fyr) jd = 0.9 d C=T 0.85 ffc+a+b = As fy Mu = $\Phi$ Mn = $\Phi$ As fyr) d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptoroid = 0.0794 > ptoroid = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2 $\Phi V_n \le A_{cv} (\alpha c^* \sqrt{f_c} + \rho_v f_y) = 0$ CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL $\Phi M_n = 495.139$ kip-ft where $\Phi = 0.900$ CHECK BROMDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION C	As a jd As SAND SPACING (ACC (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = & 21.9.4) to C = 2 (conservative) LOAD P <sub>u</sub> IS GIVEN B <sup>b</sup> > N (ACI 318-08 Fig. R <sup>5</sup>	= = = CI 318-0 00072 Y M <sub>u</sub> = 9.3.2)	50.13 36.86 184.80 49.62 88 14.3, 21.9 0.0025 0.0025 kips 41,262	in^2 in in^2 9.2)	OK V <sub>u =</sub>	Check Capacity: C=T εu = 0.003 εt = εu*(( WALL DIS WALL DIS 1317 ΟΚ	0.85*fc*a*b = c = a/0.85 dt = L-3" tt-c)/c) T. HORIZ. REIN T. VERT. REINF	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As 'fy'(d-(a/2)) = As 'fy'j jd = 0.9'd C=T 0.85 'f'c'a'b = As 'fy' Mu = $\Phi$ Mn = $\Phi$ As 'fy'j'd jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptprovd = 0.0794 > ptprovd = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2 $\Phi V_n \le A_{cv} (\alpha c^* h_c' + \rho_r f_v) = 0$ CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL $\Phi M_n = 495,139$ kip-ft where $\Phi = 0.900$ CHECK BOUNDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION C 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT	As a jd As <b>PS AND SPACING (AC</b> (pt) <sub>min.</sub> = (pl) <sub>min.</sub> = <b>&amp; 21.9.4</b> ) ac = 2 (conservative) 4 LOAD P <sub>u</sub> IS GIVEN B <sup>1</sup> ACI 318-08 Fig. R <sup>2</sup> OF BOUNDARY ZONE C	= = = CI 318-0 00072 Y M <sub>u</sub> = 9.3.2) CONFIN	50.13 36.86 184.80 49.62 0.0025 0.0025 0.0025 kips 41,262 EMENT REIN	in^2 in in/2 <b>9.2)</b>	OK V <sub>u =</sub> ENT IS GIV	Check Capacity: C=T εu = 0.002 εt = εu*((r WALL DIS WALL DIS 1317 OK //EN BY ACI318-05 21.9.	0.85*fc*a*b = c = a/0.85 d = L-3" it-c)/c) T. HORIZ. REIN T. VERT. REINF	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	
Assume Tension-controlled section, $\Phi = 0.9$ Mn = As fyr(d-(a/2)) = As fyr) $jd = 0.9$ d C=T 0.85 ffc <sup>+</sup> a <sup>+</sup> b = As fy Mu = $\Phi$ Mn = $\Phi$ As ffy <sup>+</sup> j <sup>+</sup> d jd = d - (a/2) CHECK MINIMUM REINFORCEMENT RATIO ptoroid = 0.0794 > ptoroid = 0.1186 > CHECK SHEAR CAPACITY (ACI 318-08 11.2 $\Phi V_n \le A_{cv} (\alpha c^* Nf_c^+ + \rho_r f_y) = 0$ CHECK FLEXURAL & AXIAL CAPACITY THE ALLOWABLE MOMENT AT AN AXIAL $\Phi M_n = 495.139$ kip-ft where $\Phi = 0.900$ CHECK BROMDARY ZONE REQUIREMENTS AN EXEMPTION FROM THE PROVISION C	As         a           jd         As           SAND SPACING (ACC         (pt )min. =           (pl )min. =         (pl )min. =           & 21.9.4)         xc = 2 (conservative) 4           LOAD Pu IS GIVEN B'         > M           (ACI 318-08 Fig. R:         > M	= = = CI 318-C 00072 Y M <sub>u</sub> = 9.3.2) CONFIN : <	50.13 36.86 184.80 49.62 0.0025 0.0025 kips 41,262 EMENT REIN 60.49	in^2 in in^2 э.2) > ОК иFORCEM in.	OK Vu = ENT IS GIV No Bou	Check Capacity: C=T εu = 0.003 εt = εu*(( WALL DIS WALL DIS 1317 ΟΚ	0.85*fc*a*b = c = a/0.85 dt = L-3" dt-c)/c) T. HORIZ. REIN T. VERT. REINF	VF.	# bars required a c	= = = W	32 36.48 42.92 0.01 all 1 #8	in in >0.0025	8	

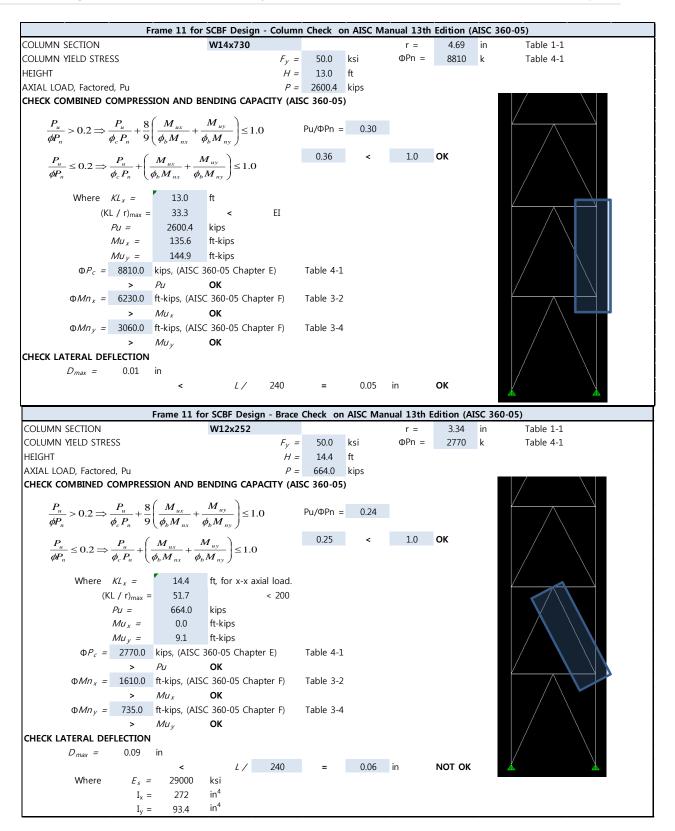
# Appendix G – Lateral Force Resisting System Design Checks-System #2

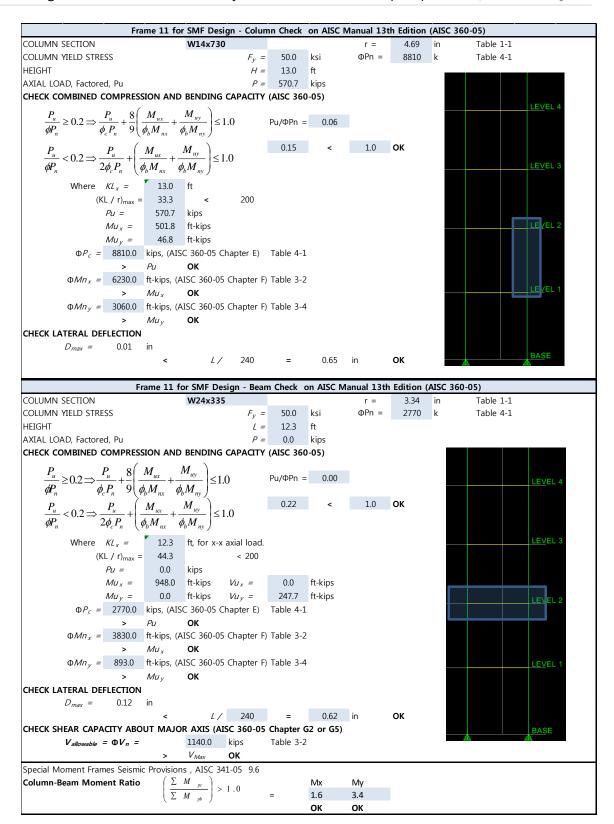
							Special Mon	er of Rigidity nent Frame - Leve	2
						X Direction	<b>k</b> ix <b>(k/ft)</b>	xi (ft)	kix Xi
						SW3	870.83	31.50	27431.06
	Special	Moment F	rame - Level	2		SW14	521.74	224.00	116869.57
	Special	Woment	ky	2		SW5	1200.00	21.67	26000.00
Moment	kx cantilever	D(in)**	cantilever	Ix	I	SW6	1200.00	32.00	38400.00
Frame #	(k/in)	D(III)	(k/in)	=Ri/∑Ri	y=Ri/∑Ri	SW11	857.14	224.00	192000.00
1	125.00	0.0	(,	17.54%		SW12	857.14	256.00	219428.57
2	72.57	0.0		10.18%		SW15	500.00	247.67	123835.00
3		0.0	72.57		14.44%	Σ	6006.85		743964.20
5		0.0	100.00		19.91%	L		k (ft) = ∑k <sub>ix</sub> x <sub>i</sub> /k <sub>ix</sub> =	123.85
6		0.0	100.00		19.91%	Y Direction	kiy (k/ft)	y; (ft)	kiy yi
7	125.00	0.0		17.54%		SW1	1500.00	97.00	145500.00
8	125.00	0.0		17.54%		SW2	870.83	64.00	55732.95
9	41.67	0.0		5.85%		SW2	1500.00	24.00	36000.00
10	41.67	0.0		5.85%		SW8	1500.00	44.00	66000.00
11		0.0	71.43		14.22%	SW16	1090.91	64.00	69818.18
12		0.0	71.43		14.22%	SW10	1090.91	94.00	102545.45
14		0.0	43.48		8.65%				
15		0.0	43.48		8.65%	SW9	500.00	124.00	62000.00
16	90.91	0.0		12.76%		SW10	500.00	111.67	55835.00
17	90.91	0.0		12.76%		Σ		· (ft) ΣΙ. · · /Ι.	593431.58
∑Ri =	712.72		502.38	100.00%	100.00%			$y$ (ft) = $\sum k_{iy} y_i / k_{iy}$ =	69.39
								A	
								er of Rigidity	
							ial Concentr	ic Braced Frame -	
						X Direction	ial Concentr kıx (k/ft)	ic Braced Frame - x: (ft)	<b>k</b> ix <b>X</b> i
						X Direction SW3	ial Concentr k <sub>ix</sub> (k/ft) 1333.33	<b>ic Braced Frame</b> - 1 <b>x</b> i (ft) 31.50	<b>k</b> ix <b>x</b> i 42000.00
						X Direction SW3 SW4	ial Concentr k <sub>ix</sub> (k/ft) 1333.33 1333.33	ic Braced Frame - 1 x: (ft) 31.50 65.50	<b>k</b> ix <b>x</b> i 42000.00 87333.33
						X Direction SW3 SW4 SW5	ial Concentr k₁x (k/ft) 1333.33 1333.33 1714.29	ic Braced Frame - xi (ft) 31.50 65.50 21.67	<b>k</b> ix <b>x</b> i 42000.00 87333.33 37142.86
						X Direction SW3 SW4 SW5 SW6	ial Concentr k <sub>i∗</sub> (k/ft) 1333.33 1333.33 1714.29 1714.29	ic Braced Frame - x <sub>i</sub> (ft) 31.50 65.50 21.67 32.00	<b>k</b> ix <b>x</b> i 42000.00 87333.33 37142.86 54857.14
						X Direction SW3 SW4 SW5 SW6 SW11	ial Concentr k <sub>ix</sub> (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00	ic Braced Frame - xi (ft) 31.50 65.50 21.67 32.00 224.00	<b>k</b> ix <b>x</b> i 42000.00 87333.33 37142.86 54857.14 448000.00
	Special Conc	entric Brad	ced Frame -	Level 2		X Direction SW3 SW4 SW5 SW6 SW11 SW12	ial Concentr k₁x (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00	ic Braced Frame - xi (ft) 31.50 65.50 21.67 32.00 224.00 256.00	kix xi           42000.00           87333.33           37142.86           54857.14           448000.00           512000.00
Moment			ky		Iv	X Direction SW3 SW4 SW5 SW6 SW11 SW12 SW13	ial Concentr k₁x (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1500.00	ic Braced Frame - xi (ft) 31.50 65.50 21.67 32.00 224.00 256.00 65.50	k <sub>ix</sub> x <sub>i</sub> 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00
Moment Frame #	kx cantilever	entric Brad	ky cantilever	Ix	Iy =Ri/ΣRi	X Direction SW3 SW4 SW5 SW6 SW11 SW12 SW13 SW14	ial Concentr k₁x (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00	ic Braced Frame - xi (ft) 31.50 65.50 21.67 32.00 224.00 256.00	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00
Frame #	kx cantilever (k/in)	∆(in)	ky	Ix =Ri/∑Ri	Iy =Ri/∑Ri	X Direction SW3 SW4 SW5 SW6 SW11 SW12 SW13 SW14 SW15	ial Concentr k <sub>ix</sub> (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1500.00 1575082.14 1407353.53	ic Braced Frame - xi (ft) 31.50 65.50 21.67 32.00 224.00 256.00 65.50	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00
<b>Frame #</b>	kx cantilever (k/in) 53.56	<b>∆(in)</b> 0.02	ky cantilever	Ix =Ri/∑Ri 13.30%	-	X Direction SW3 SW4 SW5 SW6 SW11 SW12 SW13 SW14 SW15	ial Concentr k₀ (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1500.00 1575082.14 1407353.53 2994030.90	ic Braced Frame            xi (ft)         31.50           31.50         65.50           21.67         32.00           224.00         226.00           65.50         0.00           256.00         256.00	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87
<b>Frame #</b> 1 2	kx cantilever (k/in)	Δ <b>(in)</b> 0.02 0.02	ky cantilever (k/in)	Ix =Ri/∑Ri	=Ri/∑Ri	X Direction SW3 SW4 SW5 SW6 SW11 SW12 SW13 SW14 SW15 Σ	ial Concentr k₀ (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1500.00 1575082.14 1407353.53 2994030.90	ic Braced Frame - xi (ft) 31.50 65.50 21.67 32.00 224.00 256.00 65.50 0.00	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54
Frame # 1 2 3	kx cantilever (k/in) 53.56	Δ(in) 0.02 0.02 0.02	ky cantilever (k/in) 55.56	Ix =Ri/∑Ri 13.30%	= <b>Ri/∑Ri</b> 11.78%	<b>X Direction</b> SW3 SW4 SW5 SW6 SW11 SW12 SW13 SW14 SW15 Σ <b>Y Direction</b>	ial Concentr k <sub>ix</sub> (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1500.00 1575082.14 1407353.53 2994030.90 x k <sub>iy</sub> (k/ft)	ic Braced Frame - xi (ft) 31.50 65.50 21.67 32.00 224.00 256.00 65.50 0.00 256.00 (ft) = ∑kix xi/kix = yi (ft)	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 120.76 kiy yi
Frame # 1 2 3 4	kx cantilever (k/in) 53.56	Δ(in) 0.02 0.02 0.02 0.02	ky cantilever (k/in) 55.56 55.56	Ix =Ri/∑Ri 13.30%	= <b>Ri/∑Ri</b> 11.78% 11.78%	X Direction           SW3           SW4           SW5           SW6           SW11           SW12           SW13           SW14           SW15           ∑           Y Direction           SW1	ial Concentr k s k s k s l l l l l l l l l l l l l l	ic Braced Frame - x₁ (ft) 31.50 65.50 21.67 32.00 224.00 256.00 65.50 0.00 256.00 (ft) = ∑kix x₁/kix = y₁ (ft) 97.00	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 <b>120.76</b> k <sub>i</sub> y yi 123961.66
Frame # 1 2 3 4 5	kx cantilever (k/in) 53.56 55.56	Δ(in) 0.02 0.02 0.02 0.02 0.02 0.01	ky cantilever (k/in) 55.56	Ix =Ri/∑Ri 13.30% 13.79%	= <b>Ri/∑Ri</b> 11.78%	<b>X Direction</b> SW3 SW4 SW5 SW6 SW11 SW12 SW13 SW14 SW15 Σ <b>Y Direction</b>	ial Concentr k <sub>ix</sub> (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1500.00 1575082.14 1407353.53 2994030.90 x k <sub>iy</sub> (k/ft)	ic Braced Frame - xi (ft) 31.50 65.50 21.67 32.00 224.00 256.00 65.50 0.00 256.00 (ft) = ∑kix xi/kix = yi (ft)	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 120.76 kiy yi
Frame # 1 2 3 4 5 6	kx cantilever (k/in) 53.56 55.56 71.43	Δ(in) 0.02 0.02 0.02 0.02 0.02 0.01 0.01	ky cantilever (k/in) 55.56 55.56	Ix =Ri/∑Ri 13.30% 13.79%	= <b>Ri/∑Ri</b> 11.78% 11.78%	X Direction           SW3           SW4           SW5           SW6           SW11           SW12           SW13           SW14           SW15           ∑           Y Direction           SW1	ial Concentr k s k s k s l l l l l l l l l l l l l l	ic Braced Frame - x₁ (ft) 31.50 65.50 21.67 32.00 224.00 256.00 65.50 0.00 256.00 (ft) = ∑kix x₁/kix = y₁ (ft) 97.00	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 <b>120.76</b> kiy yi 123961.66
Frame # 1 2 3 4 5 6 7	kx cantilever (k/in) 53.56 55.56 71.43 83.33	Δ(in) 0.02 0.02 0.02 0.02 0.01 0.01 0.01	ky cantilever (k/in) 55.56 55.56	Ix =Ri/∑Ri 13.30% 13.79% 	= <b>Ri/∑Ri</b> 11.78% 11.78%	X Direction SW3 SW4 SW5 SW6 SW11 SW12 SW13 SW14 SW15 Σ Y Direction SW1 SW1 SW2	ial Concentr k₀ (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1500.00 1575082.14 1407353.53 2994030.90 x k₀y (k/ft) 1277.96 1333.33	ic Braced Frame - x₁ (ft) 31.50 65.50 21.67 32.00 224.00 256.00 65.50 0.00 256.00 c5.50 0.00 256.00 c5.60 c5.6	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 <b>120.76</b> kiy yi 123961.66 85333.33
Frame # 1 2 3 4 4 5 6 6 7 7 8	kx cantilever (k/in) 53.56 55.56 71.43 83.33 83.33	Δ(in) 0.02 0.02 0.02 0.02 0.01 0.01 0.01 0.01	ky cantilever (k/in) 55.56 55.56	Ix =Ri/∑Ri 13.30% 13.79% 17.73% 20.69% 20.69%	= <b>Ri/∑Ri</b> 11.78% 11.78%	X Direction SW3 SW4 SW5 SW6 SW11 SW12 SW13 SW14 SW15 Σ Y Direction SW1 SW2 SW7	ial Concentr k <sub>u</sub> (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1500.00 1575082.14 1407353.53 2994030.90 x k <sub>u</sub> (k/ft) 1277.96 1333.33 1333.33	ic Braced Frame - x₁ (ft) 31.50 65.50 21.67 32.00 224.00 224.00 65.50 0.00 256.00 65.50 0.00 256.00 35.60 55.0	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 <b>120.76</b> kiy yi 123961.66 85333.33 32000.00
Frame # 1 2 3 4 5 6 7 7 8 9	kx cantilever (k/in) 53.56 55.56 71.43 83.33	Δ(in) 0.02 0.02 0.02 0.02 0.01 0.01 0.01 0.01 0.02	ky cantilever (k/in) 55.56 55.56 71.43	Ix =Ri/∑Ri 13.30% 13.79% 	= <b>Ri/∑Ri</b> 11.78% 11.78% 15.15%	X Direction           SW3           SW4           SW5           SW6           SW11           SW12           SW13           SW14           SW15           ∑           Y Direction           SW1           SW2           SW7           SW8	ial Concentr k. (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1500.00 1500.00 1575082.14 1407353.53 2994030.90 x k. (k/ft) 1277.96 1333.33 1333.33	ic Braced Frame - xi (ft) 31.50 65.50 21.67 32.00 224.00 2256.00 65.50 0.00 256.00 35.60 if(t) = ∑kix xi/kix = yi (ft) 97.00 64.00 24.00 44.00	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 <b>120.76</b> kiy yi 123961.66 85333.33 32000.00 58666.67
Frame # 1 2 3 4 5 6 6 7 7 8 8 9 9 10	kx cantilever (k/in) 53.56 55.56 71.43 83.33 83.33	Δ(in) 0.02 0.02 0.02 0.02 0.01 0.01 0.01 0.01 0.02 0.02	ky cantilever (k/in) 55.56 55.56 71.43 55.56	Ix =Ri/∑Ri 13.30% 13.79% 17.73% 20.69% 20.69%	= <b>Ri∕∑Ri</b> 11.78% 11.78% 15.15% 11.78%	X Direction           SW3           SW4           SW5           SW6           SW11           SW12           SW13           SW14           SW15           ∑           Y Direction           SW1           SW2           SW7           SW8           SW9	ial Concentr k₀ (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 1575082.14 1407353.53 2994030.90 x k₀ (k/ft) 1277.96 1333.33 1333.33 1333.33 1333.33 1714.29	ic Braced Frame - x₁ (ft) 31.50 65.50 21.67 32.00 224.00 2256.00 65.50 0.00 256.00 (ft) = ∑kix x₁/kix = y₁ (ft) 97.00 64.00 24.00 124.00	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 <b>120.76</b> kiy yi 123961.66 85333.33 32000.00 58666.67 212571.43
Frame # 1 2 3 4 5 6 7 8 8 9 9 10 11	kx cantilever (k/in) 53.56 55.56 71.43 83.33 83.33	Δ(in) 0.02 0.02 0.02 0.02 0.01 0.01 0.01 0.01 0.02	ky cantilever (k/in) 55.56 55.56 71.43 55.56 83.33	Ix =Ri/∑Ri 13.30% 13.79% 17.73% 20.69% 20.69%	= <b>Ri/∑Ri</b> 11.78% 11.78% 15.15% 11.78% 11.78% 17.68%	X Direction           SW3           SW4           SW5           SW6           SW11           SW12           SW13           SW14           SW15           ∑           Y Direction           SW1           SW2           SW7           SW8           SW9           SW10	ial Concentr k <sub>∞</sub> (k/ft) 1333.33 1333.33 1714.29 1714.29 2000.00 2000.00 15700.00 15700.01 15700.01 1575082.14 1407353.53 2994030.90 x k <sub>☉</sub> (k/ft) 1277.96 1333.33 1333.33 1333.33 1714.29 1714.29	ic Braced Frame - x₁ (ft) 31.50 65.50 21.67 32.00 224.00 256.00 65.50 0.00 256.00 (ft) = Σkix xi/kix = y₁ (ft) 97.00 64.00 24.00 124.00 124.00 111.67	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 <b>120.76</b> kiy yi 123961.66 85333.33 32000.00 58666.67 212571.43 191434.29
Frame # 1 2 3 4 5 6 6 7 7 8 8 9 9 10	kx cantilever (k/in) 53.56 55.56 71.43 83.33 83.33	Δ(in) 0.02 0.02 0.02 0.02 0.02 0.01 0.01 0.01 0.01 0.02 0.02 0.02 0.02 0.01	ky cantilever (k/in) 55.56 55.56 71.43 55.56	Ix =Ri/∑Ri 13.30% 13.79% 17.73% 20.69% 20.69%	= <b>Ri∕∑Ri</b> 11.78% 11.78% 15.15% 11.78%	X Direction           SW3           SW4           SW5           SW6           SW11           SW12           SW13           SW14           SW15           ∑           Y Direction           SW1           SW2           SW7           SW8           SW9           SW10           SW16           SW17	ial Concentr       kx       kx       1333.33       1333.33       1714.29       1714.29       2000.00       1500.00       1575082.14       1407353.53       2994030.90       1277.96       1333.33	ic Braced Frame - $x_i$ (ft)         31.50         65.50         21.67         32.00         224.00         256.00         65.50         0.00         256.00         65.50         97.00         64.00         24.00         124.00         124.00         111.67         4.00	kix xi 42000.00 87333.33 37142.86 54857.14 448000.00 512000.00 98250.00 0.00 360282502.54 361562085.87 <b>120.76</b> kiy yi 123961.66 85333.33 32000.00 58666.67 212571.43 191434.29 1135367.03

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Dr. Richard Behr Structural Option





### Appendix H –Cost Analysis for Lateral Force Resisting Systems

						RS	Veans	2007								
			Fxisit	ing Special	Reinforce				Materi	ial and	Labor Tak	e-Off	:			
Item	ı Lengt	h (ft) Width (ft)		Volume (cf)	Add 10% for waste	Material Unit Cost*	Mate	erial Cost cf per cy]	Labor Cost (\$/	Unit	Labor Cost	Wall Cost	l Finish Unit t (Mataterial abor. \$/SF)*	vva	ll 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	\$	0.3		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	\$	0.3	3 \$	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	\$	0.3	3 \$	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	\$	0.3	3 \$	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	\$	0.3	3 \$	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	\$	0.3	3 \$	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	\$	0.3	3 \$	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	\$	0.3	3 \$	476.23	\$ 204,907.06
Wall		.00 1.00	71.83	2298.56	2528.42	\$108/CY	\$	9,194.24			\$ 660,422.26		0.3	3 \$	1.517.05	\$ 671,133.88
Wall 1			71.83	2298.56	2528.42	\$108/CY	\$	9,194.24			\$ 660,422.26		0.3			\$ 671,133.88
Wall 1			71.83	1328.85	1461.74	\$108/CY	\$	5,315.41			\$ 381,805.59			3 \$		\$ 387,706.02
Wall 1			71.83	1328.85	1461.74	\$108/CY	\$	5,315.41			\$ 381,805.59			3 \$		\$ 387,706.02
Wall 1		.00 2.00	69.83	2793.20	3072.52	\$108/CY	\$	11,172.80			\$ 780,196.62		0.3			\$ 507,700.02 \$ 792,291.51
vvali 1	15 20	.00 2.00	09.05	2795.20	5072.52	\$100/C1	Þ	11,172.00	\$	0.92	\$ 760,190.02	¢				\$ 792,291.51 \$5,726,922
** Placin *** Wall		ete (Walls, pumped)	for Labor an		s 2007 COST	S - EXISTING	SPECIA	L REINFORC	ED SHEA	AR WALLS	5 REBAR					
			Vertical Bar	Vertical Bar	Vertical Bar W	eight Horizonta	Spacing		Ho	orizontal B	ar Bar Weight	1				
Type a	Thickness (in) 12	Vertical Spacing (in) 12	Size 6	Diameter (in) 0.75	(plf) 1.502	(ir 1		Horizontal B	ar Size D	Diameter (in 0.75	1) (plf) 1.502					
b	12	6	11	1.41	5.313	8		8		1	2.67					
c d	12	12	6 11	0.75	1.502	1		6		0.75	1.502					
e	18	12	6	0.75	1.502	1:	2	6		0.75	1.502					
f g	18 18	6	11 11	1.41 1.41	5.313 5.313	8		8		1	2.67					
h	18	6	11	1.41	5.313	8		8		1	2.67	1				
WALL	Height (ft)	# Veritical bar spaces	# bars	Bar Length (ft) 3" cover	Total bar len (ft)	gth Total v (pou		Add 10% for and la		Length (ft)	# Horizontal bar spaces	# bars	Bar Length (ft) 3" cover	Total ba length	r Total weigh (pounds)	t Add 10% for waste and lap
Wall 1_a Wall 1_b	24.33 38.50	24.33 77.00	25 78	23.83 38.00	603.61 2964.00	906 1574		997.29 17322.5		34.00 34.00	34.00 51.00	35 52	33.50 33.50	1172.50 1742.00	1761.10 4651.14	1937.20 5116.25
Wall 2_a	31.33	31.33	32	30.83	996.73	1497		1646.8		34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 2_b Wall 3 a	38.50 31.33	77.00 31.33	78 32	38.00 30.83	2964.00 996.73	4179		4597.10 1646.80		34.00 34.00	51.00 34.00	52 35	33.50 33.50	1742.00 1172.50	4651.14 1761.10	5116.25 1937.20
Wall 3_b	38.50	77.00	78	38.00	2964.00	1497		17322.5		34.00	51.00	52	33.50	1742.00		5116.25
Wall 4_a Wall 4_b	24.33	24.33 77.00	25 78	23.83	603.61	906		997.29		34.00	34.00 51.00	35	33.50	1172.50	1761.10	1937.20
Wall 5_a	38.50 31.33	31.33	32	38.00 30.83	2964.00 996.73	1574 1497		17322.5		34.00 20.00	20.00	52 21	33.50 19.50	1742.00 409.50	4651.14 615.07	5116.25 676.58
Wall 5_b	38.50	77.00	78	38.00	2964.00	1574		17322.5		20.00	30.00	31	19.50	604.50	1614.02	1775.42
Wall 6_a Wall 6_b	31.33 38.50	31.33 77.00	32 78	30.83 38.00	996.73 2964.00	1497 1574		1646.8/ 17322.5		20.00 20.00	20.00 30.00	21 31	19.50 19.50	409.50 604.50	615.07 1614.02	676.58 1775.42
Wall 7_c	31.33	31.33	32	30.83	996.73	1497		1646.8		10.33	10.33	11	9.83	111.44	167.38	184.12
Wall 7_d Wall 8_c	38.50 31.33	57.75 31.33	59 32	38.00 30.83	2232.50 996.73	1186 1497		13047.4 1646.8		10.33 10.33	15.50 10.33	16 11	9.83 9.83	162.24 111.44	433.18 167.38	476.50 184.12
Wall 8_d	38.50	57.75	59	38.00	2232.50	1186	1.27	13047.4	40	10.33	15.50	16	9.83	162.15	432.93	476.22
Wall 9_c Wall 9_d	33.33 38.50	33.33 57.75	34 59	32.83 38.00	1127.05 2232.50	1692		1862.12		34.00 34.00	34.00 51.00	35 52	33.50 33.50	1172.50 1742.00	1761.10 4651.14	1937.20 5116.25
Wall 10_c	33.33	33.33	34	32.83	1127.05	1692	.83	1862.12	2	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 10_d	38.50 33.33	57.75 33.33	59 34	38.00 32.83	2232.50 1127.05	1186 1692		13047.4		34.00 12.33	51.00 12.33	52 13	33.50 11.83	1742.00 157.77	4651.14 236.97	5116.25 260.67
Wall 11		77.00	78	38.00	2964.00	1574		17322.5		12.33	12.55	19	11.83	230.74	616.07	677.68
Wall 11_e Wall 11_f	38.50		34	32.83	1127.05	1692	.83	1862.12	2	12.33	12.33	13	11.83	157.78	236.98	260.68
Wall 12_e	33.33	33.33				1574	7.73	17322.5	51	12.33	18.50	19	11.83	230.75	616.10	677.71
Wall 11_f Wall 12_e Wall 12_f	33.33 38.50	77.00	78	38.00	2964.00											
Wall 11_f	33.33			38.00 30.83 38.00	2964.00 1962.64 2964.00	1042	7.49	11470.2	24	21.17	31.76 31.76	33 33	20.67 20.67	677.05 677.05	1016.92 1807.71	1118.62 1988.48
Wall 11_f Wall 12_e Wall 12_f Wall 13_g	33.33 38.50 31.33	77.00 62.66 77.00	78 64 78	30.83	1962.64 2964.00	1042 1574	7.49 7.73	11470.2 17322.5	51	21.17	31.76	33	20.67	677.05	1016.92	1118.62
Wall 11_f Wall 12_e Wall 12_f Wall 13_g Wall 13_h	33.33 38.50 31.33 38.50	77.00 62.66 77.00	78 64 78 pecial Reinfo	30.83 38.00	1962.64 2964.00	1042 1574	7.49 7.73 erial an	11470.2 17322.5	24 51 St	21.17 21.17 Material Cos	31.76 31.76	33	20.67	677.05	1016.92	1118.62
Wall 11_f Wall 12_e Wall 12_f Wall 13_g Wall 13_h	33.33 38.50 31.33 38.50 TTEM ebar #3 to #7	77.00 62.66 77.00 Existing S Daily Output (tons) 3	78 64 78 pecial Reinfo Labor (Hours) 10.67	30.83 38.00 rced Shear Wa Material Cost \$850/ton	1962.64 2964.00 Alls - Reinford Labor Cos \$440/ton	1042 1574 Cing Bars Mat t Work Ho 3.5	7.49 7.73 erial an urs/ton 6	11470.2 17322.5 d Labor Cos Material (t 16.59	24 51 st tons) M \$	21.17 21.17 Material Cos 14,105.6	31.76 31.76 t Labor Cost 8 \$ 7,301.76	33	20.67	677.05	1016.92	1118.62
Wall 11_f Vall 12_e Vall 12_f Vall 13_g Vall 13_h Vall 13_h	33.33 38.50 31.33 38.50	77.00 62.66 77.00 Existing S	78 64 78 pecial Reinfo Labor (Hours)	30.83 38.00 rced Shear Wa Material Cost	1962.64 2964.00 alls - Reinford Labor Cos	1042 1574 Cing Bars Mat t Work Ho	7.49 7.73 erial an urs/ton 6	11470.2 17322.5 d Labor Cos Material (1	24 51 tons) M \$ 0 <u>\$</u>	21.17 21.17 Material Cos 14,105.6 91,205.3	31.76 31.76	33	20.67	677.05	1016.92	1118.62

			-			RS N	<b>/</b> lea	ns 2007								
			Modi	fied Special	Reinforce	d Shear W	alls	- Concrete	M	aterial an	d Labor Tak	e-Of	f			
Item	Length (ft)	Width (ft)	Height (ft)	Volume (cf)	Add 10% for waste	Material Unit Cost*		aterial Cost 7 cf per cy]		abor Unit st (\$/CF) **	Labor Cost	Cos	ll Finish Unit t (Mataterial .abor, \$/SF)***	w	all 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	Cor	crete Cost	\$5,292,936
* Normal W	eight Concrete	Ready Mix	4000psi (Ag	ilia, Self Conso	lidating Con	crete)										
** Placing o	f concrete (Wa	lls, pumped)	for Labor ar	nd Equipment												
*** Wall finis	sh															
****Agilia, S	Self-Consolida	ting Concret	e from Lafar	ge Conrete												

				RS Means	2007 COSTS - N	ODIFIED SPECIA	L REINFORCED SH	IEAR WALLS	REBAR					
Type	Thickness (in)	Vertical Spacing (in)	Vertical Bar	Vertical Bar	Vertical Bar Weight	Horizontal Spacing	Horizontal Bar Size	Horizontal Bar	Horizontal	I				
Type		vertical spacing (III)	Size	Diameter (in)	(plf)	(in)	Homzontal bar Size	Diameter (in)	Bar Weight					
а	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	l				
с	12	12	6	0.75	1.502	12	6	0.75	1.502	l				
d	12	8	11	1.41	5.313	8	8	1	2.67	l				
e	18	12	6	0.75	1.502	12	6	0.75	1.502	1				
f	18	6	11	1.41	5.313	8	8	1	2.67	1				
g	24	6	11	1.41	5.313	8	8	1	2.67	1				
h	24	6	11	1.41	5.313	8	8	1	2.67	l				
				Bar Length (ft)	Total bar length	Total weight	Add 10% for waste		# Horizontal		Bar Length (ft)	Total bar	Total weight	Add 10% for
WALL	Height (ft)	# Veritical bar spaces	# bars	3" cover	(ft)	(pounds)	and lap	Length (ft)	bar spaces	# bars	3" cover	length	(pounds)	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
1		Modified 9	necial Peinfo	rced Shear Wa	alls - Reinforcing	Bars Material an	d Labor Cost							
	ITEM	Daily Output (tons)	Labor (Hours)	Material Cost	Labor Cost	Work Hours/ton	Material (tons)	Material Cost	Labor Cost					
	Rebar #3 to #7	Jally Output (tons)	10.67	\$850/ton	\$440/ton	3.56	15.55	\$ 13,220.46						
		3				2.00	84.86							
vvalls, Re	bar #8 to #18	4	8.00	\$850/ton	\$330/ton	2.00			\$ 28,004.34					
1							Total Reinforcing Ba	r Cost	\$ 120,200.71					

	RS MEANS 2007	COSTS - SPECIAL N	OMENT 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

KS WILAI	N3 2007 CO.	SIS - SPECIAL I	VIOIVILINT FRAIVILS
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

				Spe	cial Mor	ment Fram	ie					
Iteam	I	Tonnage	of Steel	Material(\$/ton) Labor (\$/tor				n) Equipment (\$/ton)				Total
Beams	5	29	294		2050		225		115		\$	702,660
Column	าร	40	406		2050		225		115		\$	970,297
			•	Ċc	nnection	Fabricatio	n					
	#N	/IF's	# o Conne	Cost			Cost (\$/Fa	br.hr)	То	tal		
	-	15	9!	5	4.8	Ea.	45		\$	20,52	20	
_				Co	nnection	Installatio	n					
			ion Time ays)	Installatio (hr		Cost (\$/L	abor hr)	Тс	otal			
		1	.5	95	5	4.8 I	Ea. S	3,240				

# Appendix I – Mechanical Breadth Cost Analysis

Electricity Costs (PNM)				Summer	Winter	1								
Utility				June-Aug	Sept-May									
Electricity consumption/	٢W			\$9.56	\$8.19									
Electricity demand per m	ont	:h/KWh		\$0.0821025	\$0.064170									
Gas Costs (New Mexico		Jan	Feb	Mar	April	Mav	Jun	Jul		Aua	Sept	Oct	Nov	
Gas Company)		Jan	reb	IVICI	April	iviay	Jun	Jui	,	lug	Sept	011	NOV	
Gas distribution/ therm	\$	0.49100	\$ 0.53090	\$ 0.47790	\$0.50080	\$ 0.46870	\$ 0.52350	\$ 0.56990	\$ (	).53470	\$ 0.49170	\$ 0.53750	\$ 0.49010	

#### VRE 3-54 Glazing

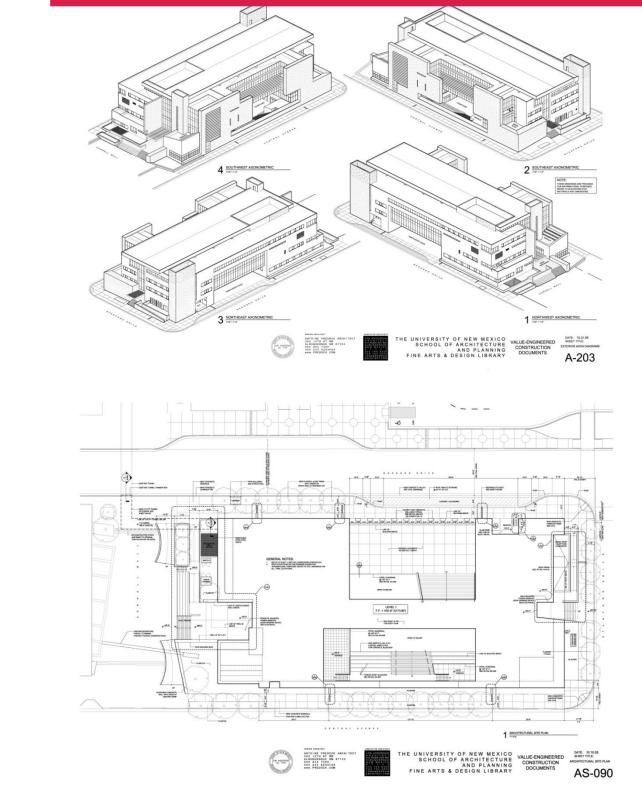
Monthly Costs	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	
Elec(KWh)	2,051	1,819	2,170	1,955	3,576	4,815	5,946	4,755	3,458	2,271	2,022	1,981	
Consumption (\$)	\$ 131.61	\$ 116.72	\$ 139.25	\$ 125.45	\$ 229.47	\$ 395.32	\$ 488.18	\$ 390.40	\$ 221.90	\$ 145.73	\$ 129.75	\$ 127.12	
Peak(KW)	10	11	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 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	\$
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	\$
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	\$

#### VNE 1-30 Glazing

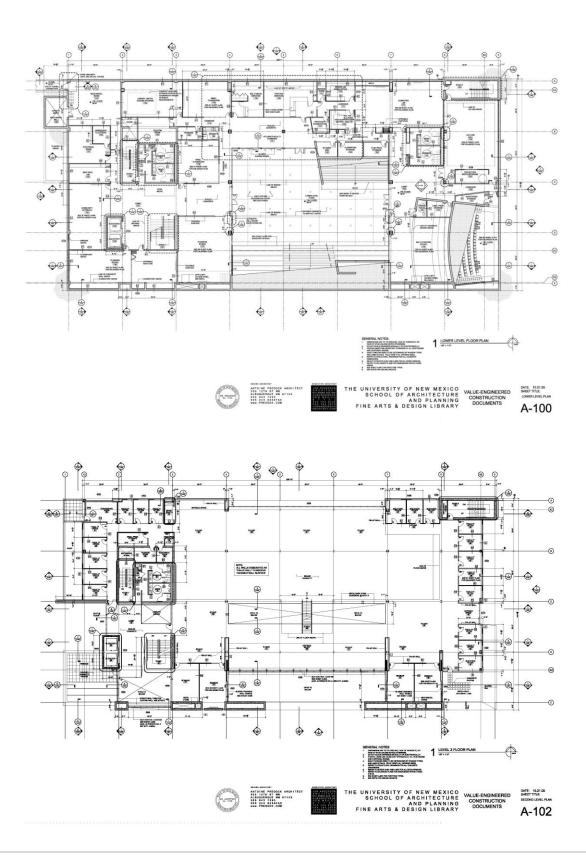
Monthly Costs	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	
Elec(KWh)	2,037	1,805	2,152	1,973	3,618	4,776	5,936	4,751	3,476	2,280	2,011	1,963	
Consumption (\$)	\$ 130.71	\$ 115.83	\$ 138.09	\$ 126.61	\$ 232.17	\$ 392.12	\$ 487.36	\$ 390.07	\$ 223.05	\$ 146.31	\$ 129.05	\$ 125.96	
Peak(KW)	10	11	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	т
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
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
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

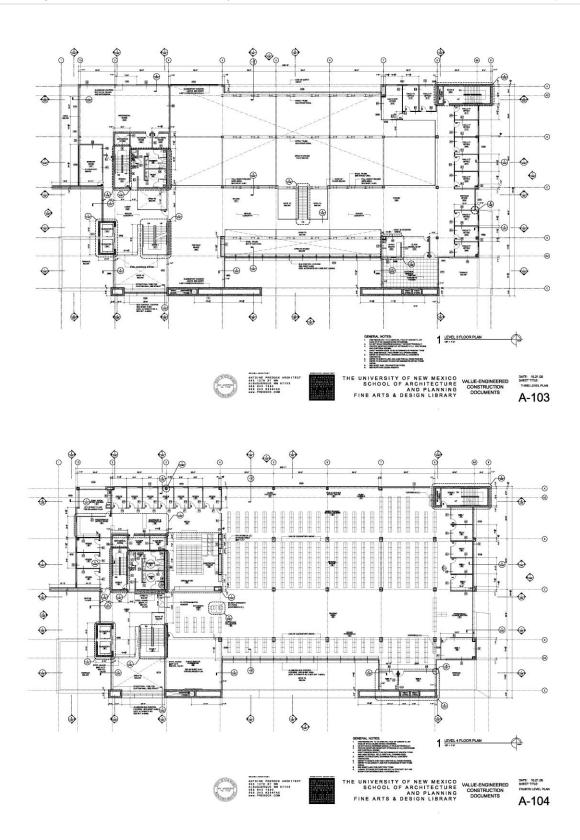
VRE1-63 Glazing													_
Monthly Costs	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	1
Elec(KWh)	2,014	1,799	2,143	1,970	3,643	4,774	5,906	4,749	3,500	2,292	1,999	1,940	1
Consumption (\$)	\$ 129.24	\$ 115.44	\$ 137.52	\$ 126.41	\$ 233.77	\$ 391.96	\$ 484.90	\$ 389.90	\$ 224.59	\$ 147.08	\$ 128.28	\$ 124.49	
Peak(KW)	10	11	11	11	17	19	18	17	16	12	11	10	1
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	1
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	1
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	1
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.
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.
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.

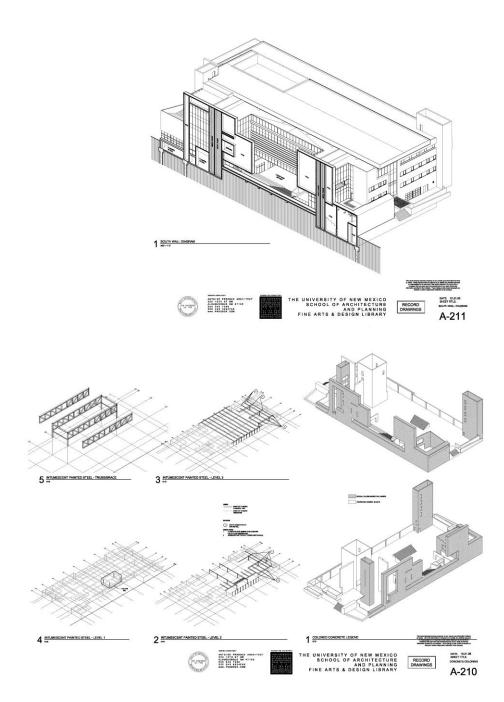


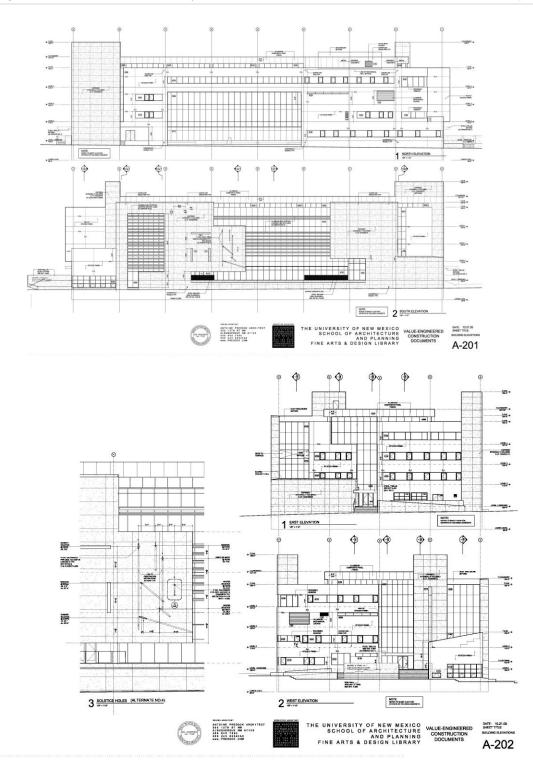


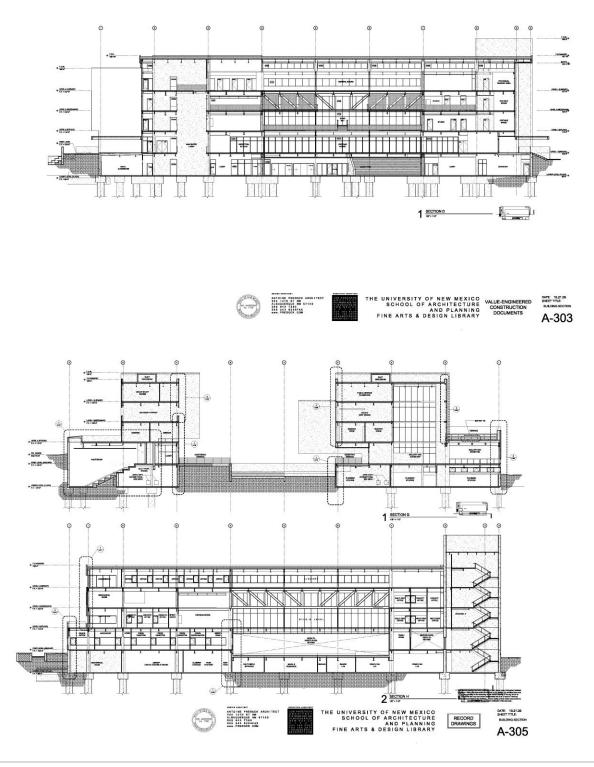
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