101 Eola

Technical Assignment 3

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101 Eola Drive, Orlando, FL

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Executive Summary

This report is an analysis and confirmation study of the lateral system of 101 Eola Dr, a 12 story, 130 foot tall precast concrete structure in Orlando, FL. Due to the unique complexity of the building's lateral force resisting system, this report consists of only a schematic analysis of a simplified frame. Additional calculations and analysis will be performed as needed for subsequent reports.

Gravity framing is provided by precast concrete hollowcore planks supported by unique concrete truss beams, which are placed on every other floor and also transfer shear forces between long direction shear walls and the central core as part of the lateral system. These trusses appear in the image to the right.

101 Eola Under Erection: *Courtesy of Finfrock Industries*

This report provides an in-depth lateral analysis of 101 Eola. Wind and seismic forces are calculated and applied to the building using two methods. The first method consists of member design verification and serviceability (drift) analysis through hand calculations. This method is then compared to a simplified computer analysis for drift.

101 Eola is determined to be controlled by wind along the long direction, and seismic along the short direction. The lateral system works for both strength and serviceability criteria, and is completely symmetric in both directions.

This structure was built in accordance to Florida Building Code 2004. I will be using IBC 2003 which references ASCE 7-02, and ACI 318-02 for my calculations unless otherwise noted. All calculations made herein are considered preliminary, "schematic" designs, and are not an exhaustive analysis of the building's lateral system. This report in no way makes any claims that the designer's methods, assumptions, calculations or resulting designs are incorrect or unsuitable.

I. Existing Structural System

Foundation System

According to the geotechnical report, footings needed to be placed on top of groups of vibro-reinforced stone columns, which are grouped and positioned according to the size of the footing they support. Footings are either a spread design (shear walls, core, and columns) or a continuous strip footing (east wall and retaining wall). Depths range from 12" to 39". Footings are designed with an allowable load of 8000 psf and f'c = 4000psi.

A slab on grade is used as the ground floor level with a minimum thickness of 5", typical, and a 8" slab on grade in the loading dock area, with expansion joints no more than 15 feet apart. Material strength is 4000psi.

Please refer to Technical Report 1 for more details on the existing foundation system.

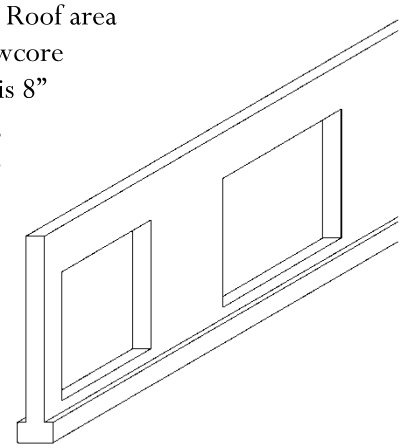
Typical Floor Framing

There are several variations in the floor framing of 101 Eola, depending on their location and usage characteristics. Most floor elements consist of precast double tee beams or hollowcore planks.

The ground floor consists of slab on grade construction and areas of exposed soil behind precast retaining walls. 6 inch equipment pads are placed beneath mechanical and electrical equipment on top of this slab. Additionally, ramping for the parking structure begins on this level with precast double tee beams supported by precast walls on either end.

Floors 2 thru 4 comprise an open air parking facility. Construction is precast double tee beams with single tee beams at edge spans and periodic intervals between double tees (See Figure I.2 of appendix on page 13). Corbels on precast walls are stepped and rotated to eliminate need for blocking in order to support sloped garage ramps. Average span is 62 feet E-W, but spans range from 20 feet to 68 feet in length depending on location. Floor 3 also adds precast flat slabs above the loading dock (used for storage) with span of around 23 feet. Floor 4 is typical of floor 2 with the exception of 12 foot wide precast single tee beams spanning 23 feet to support a pool and hot tub located on the 5th floor roof. These tees are also covered with a 6" structural concrete topping slab.

At floor 5, the entire building profile steps back an average 10 feet and the upper levels take shape. An accessible roof with pool and hot tub is located on the south side roof. Roof area construction is a mixture of precast flat slabs, single tee beams, and hollowcore planks with a C.I.P. topping sloped to roof drains. Interior floor makeup is 8" thick hollowcore slabs. These slabs span N-S with length 25'-5" to 26"-7", and are supported by unique precast trusses 45'-0" in length that attach to the lateral shear wall system to transfer shear loads (see illustration to right. These truss beams have a height of 12'-0", and are placed on every other floor. E-W edge planks are supported on the exterior wall by precast beams 2'-8" in width. Precast flat slabs form balconies on the exterior edges of the structure, spanning between the shear walls that run up the building's exterior. Please refer to the appendix for a floor plan illustrating the system layout.



For more information on the existing floor systems of 101 Eola, please refer to Technical Report 2.

Lateral Load Resisting System

101 Eola has an extensive lateral load resisting system that is comprised of 16" thick shear walls around all sides of the building. These walls also act as the principle gravity load carrying elements as well for the exterior of the building. The east and west sides of the structure contain 8 shear walls with spacing ranging from 21'-5" to 24'-7" apart. The north and south sides employ 3 shear walls 21'-2" apart. These shear walls run vertically up the side of the structure all the way to the roof and become an integral part of the building's architectural facade. From ground level to level 4, the walls are 18'-6" in width. As the structure steps back at level 5, the shear walls also step back, reducing to 9'-2 ½" in width. At penthouse and roof levels, east-west shear walls reduce further, in some spots to only 3'-0" in width. For further reference see page 11.

The precast concrete core of the building contains stair towers and elevator shafts, also provides some lateral resistance. Walls are 8" thick in this long narrow core that runs north-south, and 3 internal shear walls running east-west provide shear force transfer between the precast trusses that connect the exterior E-W shear walls on every other floor. Precast L beams handle tying the external edge of the east-west system to the north-south shear walls. Please refer to the appendix (pg 19 and 20) for a detail of these unique trusses and shear walls. A clarifying illustration of the lateral system can be found on page 11 (figure 4.1) of this report.

Gravity Load Carrying System

As stated above, the shear walls around the exterior of the building act as the main gravity load carrying system as well. The load is carried from the floor system (double tee beams or hollowcore/flat slabs) to the precast trusses and edge beams, and then into the shear wall system or the central core. In addition to these elements, there are miscellaneous columns in key places in the structure. One 24"x24" column in each corner of the west elevation runs vertically until reaching the 5th floor step-back in order to carry the corner load. Corner loads on the opposite side of the building is carried by precast walls that run monolithically up to the 5th story as part of the system that carries the pool and hot tub loads. 2 columns of the same size assist in carrying load surrounding a large garage door on this elevation also. The two main supporting columns in the structure are at either end of the central core strip. Each precast column is 36"x48". Details concerning the distribution of these loads can be found in the appendix on page 21.

II. Codes and Material Properties *(refer to Technical report 1 for complete list)*

Codes and Referenced Standards

Building Code:	Florida Building Code (2004 w/ 2005 revisions)
	I will be using International Building Code 2003 for calculations
Structural Concrete:	American Concrete Institute 2002 edition (ACI 318-02)
	Prestressed Concrete Institute (PCI MNL 116,120,129)
Fire Code:	Florida Fire Protection Code (2004)
	NFPA 1, Uniform Fire Code (2003)
Building Design Loads:	American Society of Civil Engineers (ASCE-7) 2002 edition
Materials Standards:	American Society for Testing and Materials (ASTM)

Material Strength Requirements

Cast in place concrete (normal weight 145pcf)		
Footings		4,000 psi
Column Piers		5,000 psi
Grade Beams		4,000 psi
Columns		5,000 psi
Walls		4,000 psi
Slab on grade (interior)		4,000 psi
Stairs, landings, lobbies		4,000 psi
Tee pour strips		5,000 psi
All other		4,000 psi
Precast Concrete (normal weight 145pcf)		
Shear walls and precast trusses		7,000 psi
All types		6,000 psi
Other Concrete		
Columns dry base pack		6,000 psi
N.S.N.S. grout		6,000 psi
Reinforcing and Connection Steel		
Welded bars	ASTM A706	60,000 psi
All bars u.n.o	ASTM A615	60,000 psi
Welded Wire Fabric (smooth)	ASTM A185	65,000 psi
Prestressing strand	ASTM A416	(f_{pu}) 270,000 psi
Coil bolts and coil rods u.n.o.		65,000 psi
Deformed bar anchors	ASTM A496	70,000 psi
Headed anchor studs	ASTM A108	50,000 psi
Structural Steel		
Structural Shapes	ASTM A36	36,000 psi
Bolts (1/2" Ø to 1"Ø) u.n.o	ASTM A325	92,000 psi
Bolts (1 1/8" Ø to 1 1/2"Ø) u.n.o	ASTM A325	81,000 psi

III. Design Load Requirements

Dead Loads

Supported parking and drive areas..40 psf
Concentrated wheel load (on 20 sq. inches).. 3,000 lb.
Bumper impact load, over 1 foot square, located 18” above finished floor, ultimate......... 10,000 lb.
Superimposed dead loads
Condominiums
Partitions.. 10 psf
Mechanical, Electrical, Misc... 10 psf
Roof
Superimposed.. 20 psf
Mechanical, Electrical, Misc... 20 psf
Slabs on grade.. 50 psf
Stairs, landings, lobbies.. 100 psf

Live Loads

Superimposed live loads
Condominiums
Condominiums.. 40 psf
Corridors.. 80 psf
Stairs, lobbies, balconies... 100 psf
Roof... 20 psf

Wind Loads

Listed below are the major assumptions made for determining 101 Eola’s wind loads. Wind loads were calculated in accordance with ASCE 7-02. 101 Eola is located in downtown Orlando, Florida, which is a *hurricane prone region*. My results for wind pressures were slightly higher than that of the engineer. This could be due to different code procedures or due to a more detailed wind analysis done by the engineer. Calculations and distribution tables can be found by referring to technical report 1. Further calculations will follow later in this report and in the appendix.

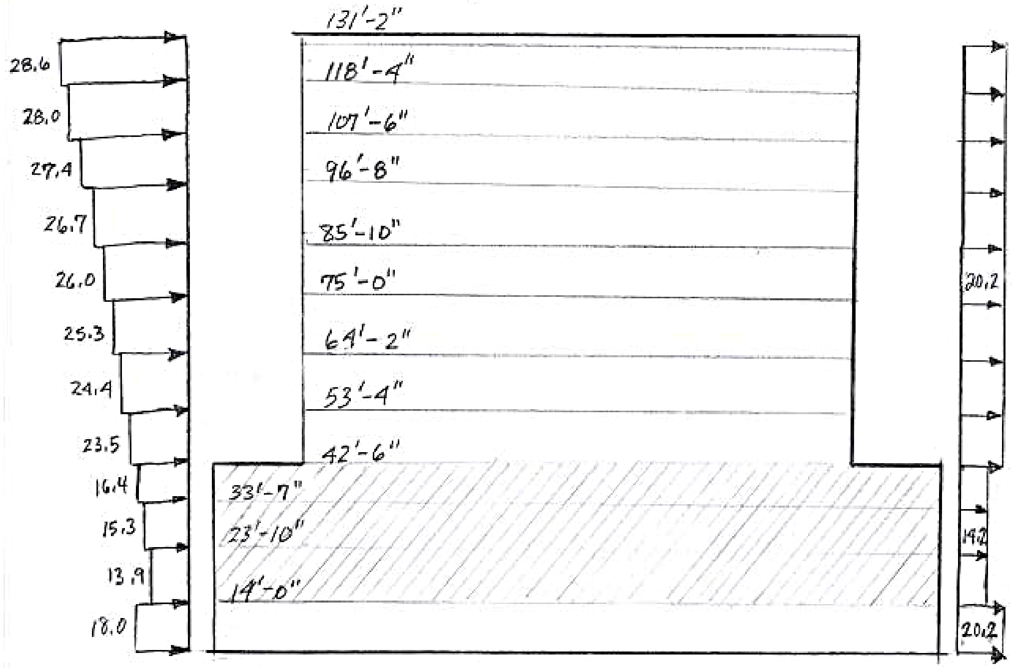
Basic Wind Speed......................... 110 mph
Exposure Category........................B
Enclosure Category....................... Enclosed
Occupancy Category......................III
Wind Directionality Factor (K_d)..........0.85
Importance Factor (I)..................... 1.15
Topographic Factor (K_{zt})..................1.0
Gust Effect Factor (G).................. 0.85
Internal Pressure Coefficient
Parking Garage.....................0
Enclosed rooms and elevator.........±0.18

Wind Load Summary				
Wind From N-S				
Windward		Leeward		Total (psf)
h (ft)	p (psf)	h (ft)	p (psf)	
14'-1"	17.96	14'-1"	-14.99	34.77
23'-10"	13.91	23'-10"	-10.81	24.72
33'-7"	15.34	33'-7"	-10.81	26.15
42'-6"	16.41	42'-6"	-10.81	27.22
53'-4"	23.50	53'-4"	-14.99	38.50
64'-2"	24.45	64'-2"	-14.99	39.45
75'-0"	25.30	75'-0"	-14.99	40.29
85'-10"	26.05	85'-10"	-14.99	41.05
96'-8"	26.75	96'-8"	-14.99	41.74
107'-6"	27.39	107'-6"	-14.99	42.38
118'-4"	27.98	118'-4"	-14.99	42.98
131'-2"	28.64	131'-2"	-14.99	43.63
Wind From E-W				
Windward		Leeward		Total (psf)
h (ft)	p (psf)	h (ft)	p (psf)	
14'-1"	17.96	14'-1"	-20.15	38.11
23'-10"	13.91	23'-10"	-14.15	28.06
33'-7"	15.34	33'-7"	-14.15	29.49
42'-6"	16.41	42'-6"	-14.15	30.56
53'-4"	23.50	53'-4"	-20.15	43.65
64'-2"	24.45	64'-2"	-20.15	44.60
75'-0"	25.30	75'-0"	-20.15	45.44
85'-10"	26.05	85'-10"	-20.15	46.20
96'-8"	26.75	96'-8"	-20.15	46.89
107'-6"	27.39	107'-6"	-20.15	47.53
118'-4"	27.98	118'-4"	-20.15	48.13
131'-2"	28.64	131'-2"	-20.15	48.78

Resulting Wind Loading Forces		
North – South Direction	Shear: 564.19 kips	Overturning Moment: 5993.14 ft-k
East – West Direction	Shear: 1100.86 kips	Overturning Moment: 11692.37 ft-k

Wind Loading Diagrams

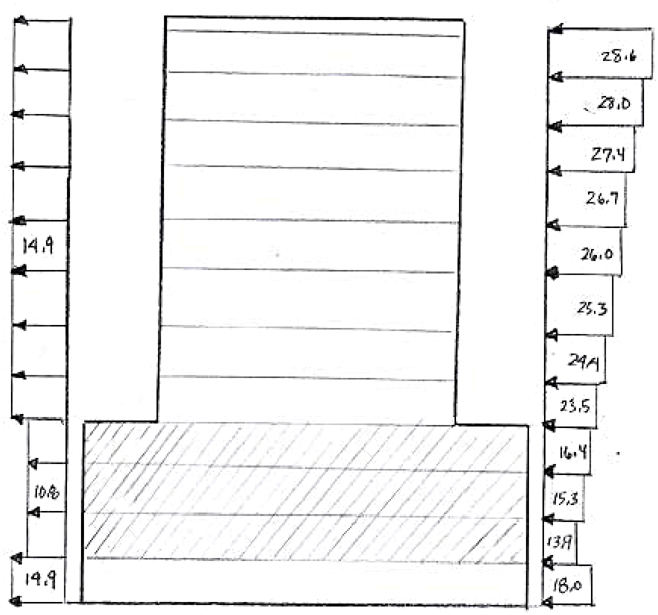
Wind E-W



Parking Deck - Internal Pressure Coefficient GCpi = 0

Enclosed Floors - Internal Pressure Coefficient GCpi = +/- 0.18

Wind N-S



Seismic Loads *(revised from Technical Report 1)*

Major assumptions used in the determination of 101 Eola's seismic response load are listed below. Seismic loads were calculated using the equivalent lateral force method in accordance with ASCE 7-02. Soil information came from the geotechnical report for the site, performed by ESC-Florida, LLC, dated February 2006. Because of the site location, and based on calculations, seismic loads will control only in the short direction. Calculations and distribution tables can be found by referring to technical report 1.

Seismic Use Group................................ II
Occupancy Importance Factor (I_E)................ 1.25
Site Class.. D
Soil Profile.. Stiff Soil Profile
Mapped Spectral Response Accelerations
$S_S = 0.10$
$S_1 = 0.04$
Site Class Factors
$F_A = 1.6$
$F_V = 2.4$
S_{MS} 0.160g
S_{M1} 0.096g
S_{DS} 0.107g
S_{D1} 0.064g
Seismic Design Category......................... A
Building Frame.................................. "Ordinary Reinforced Concrete Shear Walls"
Response Modification Factor (R)................. 5
Over-Strength Factor (W_O)...................... 2.5
Deflection Amplification Factor (C_D) 4.5
Period Coefficient (x)............................ 0.75
Seismic Response Coefficient (C_S)................ 0.01
Period Exponent (k)............................ 1.135

Resulting Seismic Forces	
Base Shear (V)	882.8 kips
Overturning Moment (M)	9572.5 ft-kips

Load Combinations

The following load combinations were considered. Snow loads were not included in this analysis due to building location (Orlando, Florida).

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

IV. Typical Framing Plans and Elevations

Typical Floor Framing Plans

See Appendix.. pg 16 - 17

Building Sections

See Appendix.. pg 18 - 19

Lateral System Elements

Lateral system elements are highlighted below. These consist of precast concrete shear walls around the exterior of the building, and a cast in place concrete core. These elements are tied together with precast truss girders (inverted "T" shape). The plan below is on one of the upper floors. Floors G-4 have much larger shear walls.

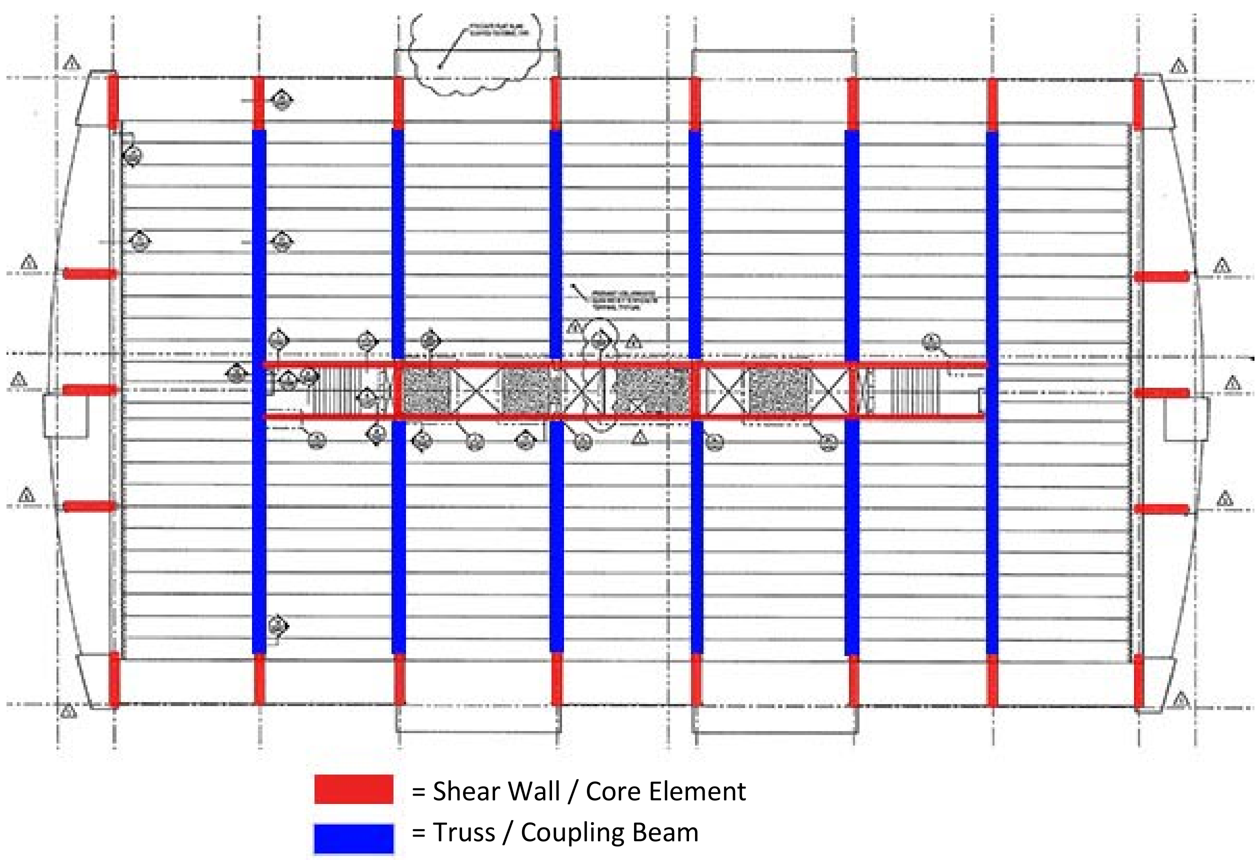


Figure 4.1

V. Lateral System Analysis and Design

Load Analysis

After calculations and analysis it was determined that 101 Eola's lateral force resisting system is controlled by wind forces in the short (E-W) direction (long side perpendicular to wind), and by seismic forces in the long (N-S) direction. The lateral system is illustrated on the previous page of this report (figure 4.1). Wind forces were calculated by the tributary area method with forces determined from ASCE 7-02 methodology, and distributed to shear walls according to calculated relative stiffness. Seismic forces were calculated using the equivalent lateral force method in accordance with ASCE 7-02 and distributed according to relative stiffness and center of mass of the shear wall system.

For the gravity load analysis, calculations were made based on the worst case scenario: the largest bay size on either side of the shear wall. Gravity loads from the floor system were calculated by the tributary area method, and distributed to the precast concrete trusses which support the hollowcore planks making up the floor construction. Each precast truss is responsible for carrying the loads of two floors of hollowcore planks and superimposed loads, one floor resting on the top chord, the other on the bottom flange. These trusses in turn transfer a shear load at either end to the shear walls on either side of the building, and to the central core. These trusses have simple connections (modeled as pin connections) to notches in the upper shear walls. The welded plate connections are meant to only transfer shear forces, therefore the trusses are very large, capable of resisting the shear and moment of 2 floors over its 45 foot span without transfer. A detail of this loading and of the unique trusses can be found in the appendix (pages 19 - 23). Loads from the flat slab balconies were also considered in the gravity loading calculations. These balconies are supported on steel angles embedded into either side of the shear walls. Loads were distributed based on tributary area.

Lateral loads were distributed according to relative stiffness of the shear wall systems and the assumption that the floor system is considered to be a rigid diaphram. The lateral system for 101 Eola is exactly symmetric in both directions, as well as the building itself. The precast shear walls were formed using a technique called match casting, where forms are placed to form every other shear wall with a space between them equal to the dimensions of the wall that will join them. After they are poured and cured, the forms on either end are removed, and the other pieces are poured directly up against the previously poured pieces (forming a cold joint). This ensures the matched walls will have the most uniform connection possible, as if they were cast in place, allowing complete shear and moment force transfer to occur. This technique is illustrated below (figure 5.1). Refer to the appendix for detailed calculations and load summaries.

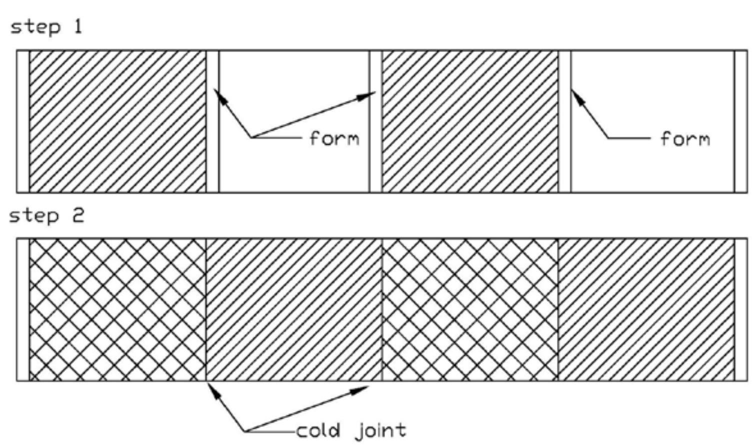


Figure 5.1

Torsion Analysis

Due to the fact that 101 Eola is two way symmetric, both when referring to the overall building construction and to the lateral system, torsion will have a negligent effect on the building's structural system. Because of the symmetry, both the center of mass and the center of rigidity for calculation purposes are located directly in the center of the structure. The only torsion that occurs within the building's structural system is at the truss beams that carry the floor load. Because of the difference in bay sizes between these trusses, a minimal torsional force will be created on the truss. However the truss construction is more than sufficient to resist these effects. The embed plate at the top and bottom of the connection of these trusses to the shear walls is also sufficient to provide this torsional resistance provided a proper weld is applied. The torsional effect is 318plf for the largest difference in bay size. This load is applied only 8" from the center line of the truss, creating a 17.7 ft-lb torsional moment per foot, which has virtually no effect on the truss because of its size.

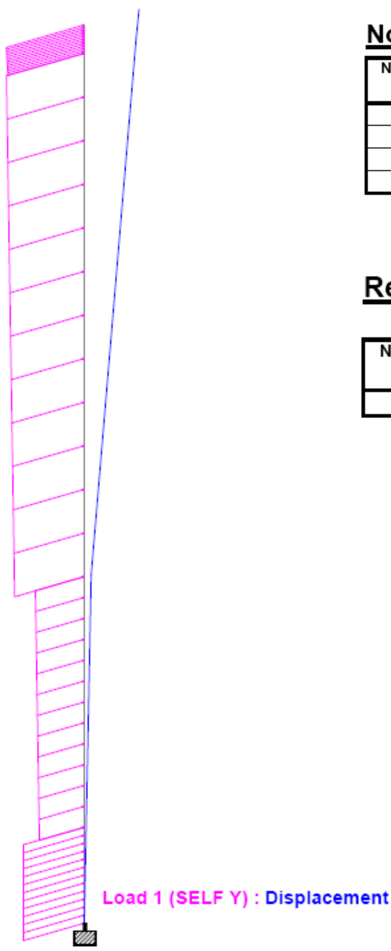
Serviceability (drift) Analysis

In my drift analysis a maximum deflection of H/400 (allowable code drift) was used. In the case of 101 Eola, the maximum permissible drift is 3.90" over the 130 foot height. Hand calculations yielded a maximum drift value of 0.21" in the E-W (short) direction (wind controlled), and 0.33" in the long direction (seismic controlled). These values seem small but the lateral system is very massive and rigid in this building's construction, allowing for little movement from lateral forces. Calculations can be found in the appendix of this report (pg. 22 to 25).

A simple computer model was also used as a schematic analysis only, focusing on only one frame of the structure due to the complexity of modeling the precast truss beams. The program yielded a drift of 2.43", which is still within the 3.9" drift limit. The large difference in these drift numbers is most likely attributed to the extremely simplified data entry into the computer model, which modeled the lateral system as a single fixed beam with varying gross properties to simulate the lateral system. Based on this assumption, the hand calculations will be taken as the more accurate determination of building drift. Additional information concerning the computer model appears on the following page.

VI. Computer Model

Due to the complexity of modeling my entire lateral system, particularly the precast truss beams, and my lack of familiarity with the programs, I was given permission to perform a schematic model of 101 Eola's lateral system using a simple beam model with gross section properties. Due to its schematic nature however, the values I received (2.43") were not similar to the values I obtained though hand calculations, which will be considered far more accurate. STAAD Pro was used for the purpose of modeling, and the output is below.



Node Displacements

Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
1	1:LOAD CASE	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	1:LOAD CASE	0.305	-0.010	0.000	0.306	0.000	0.000	-0.001
3	1:LOAD CASE	2.295	-0.019	0.000	2.295	0.000	0.000	-0.003
4	1:LOAD CASE	2.426	-0.019	0.000	2.426	0.000	0.000	-0.003

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
1	1:LOAD CASE	-145.093	339.966	0.000	0.000	0.000	125E 3

VII. Spot Checks

A strength and serviceability spot check was performed on a base shear wall, a hollowcore plank, and on the foundation beneath the shear wall. Worst case scenario was considered in all cases due to the uniform design of all members in the building. Hand calculations confirmed that all members were capable of handling applied loads and forces based on section and reinforcement properties, as well as soil characteristics pertaining to the foundation design. Strength, drift, and overturning were all considered in these design checks. Applicable calculations can be located in the appendix of this report on pages 26 through 30.

VIII. Summary and Conclusions

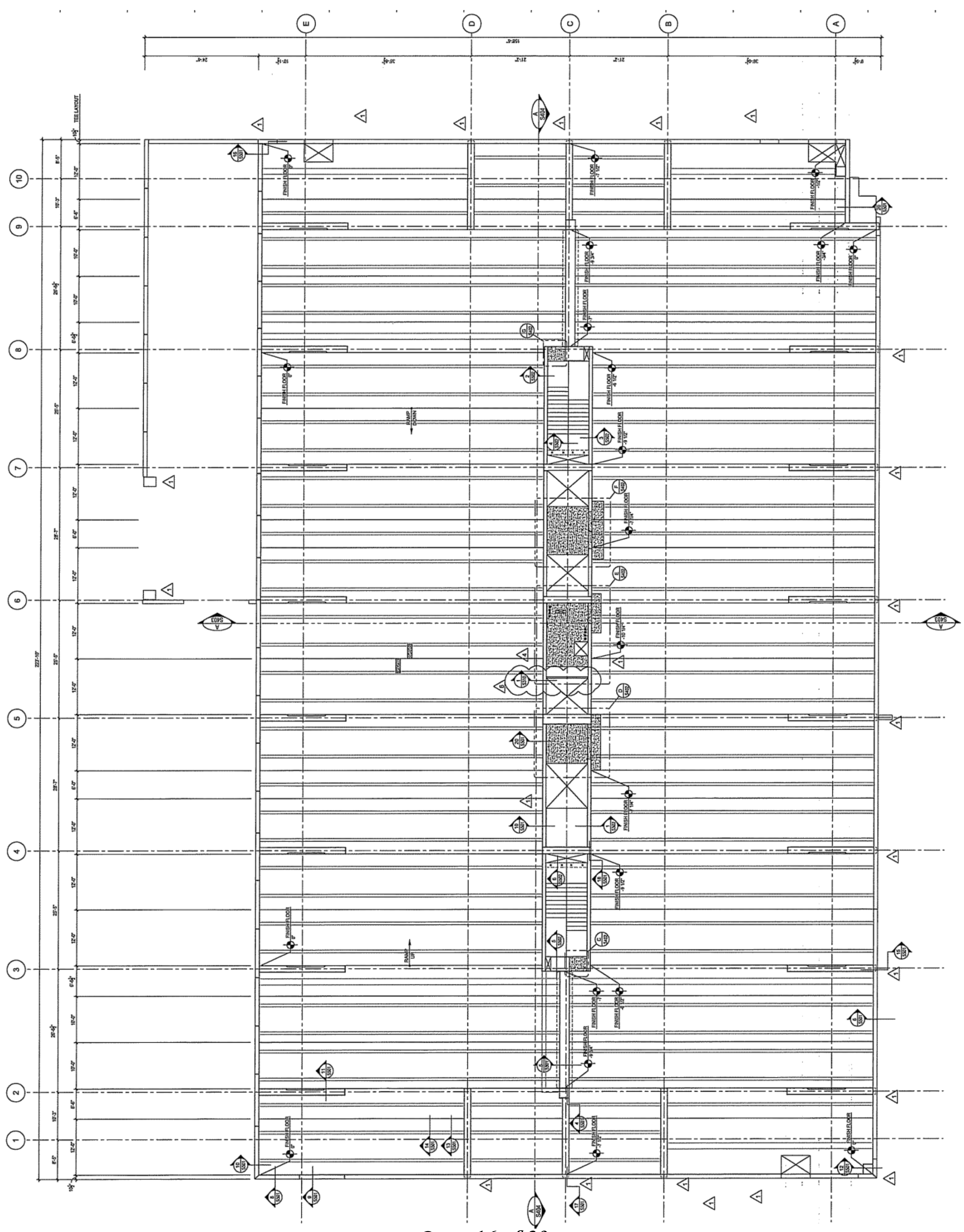
This report used hand calculations and computer modeling to understand the lateral system and lateral loads 101 Eola. The building was analyzed for strength and serviceability and my overall conclusion from my studies in this report is that all inspected members are sufficient (or in some cases more than sufficient) to carry and resist all loads and forces applied to the structure as designed. Internal forces and drift are all within code assigned values when compared to ASCE 7-02, IBC 2003 and ACI 318-02.

I have learned that I needed to be more specific with my computer programs in order to model what I intended to model. For example, the overturning moment that the computer calculated is slightly more than the one calculated by hand (by around 1000kip-ft), but base shear is far less (by 420kip) than the one that I calculated by hand.

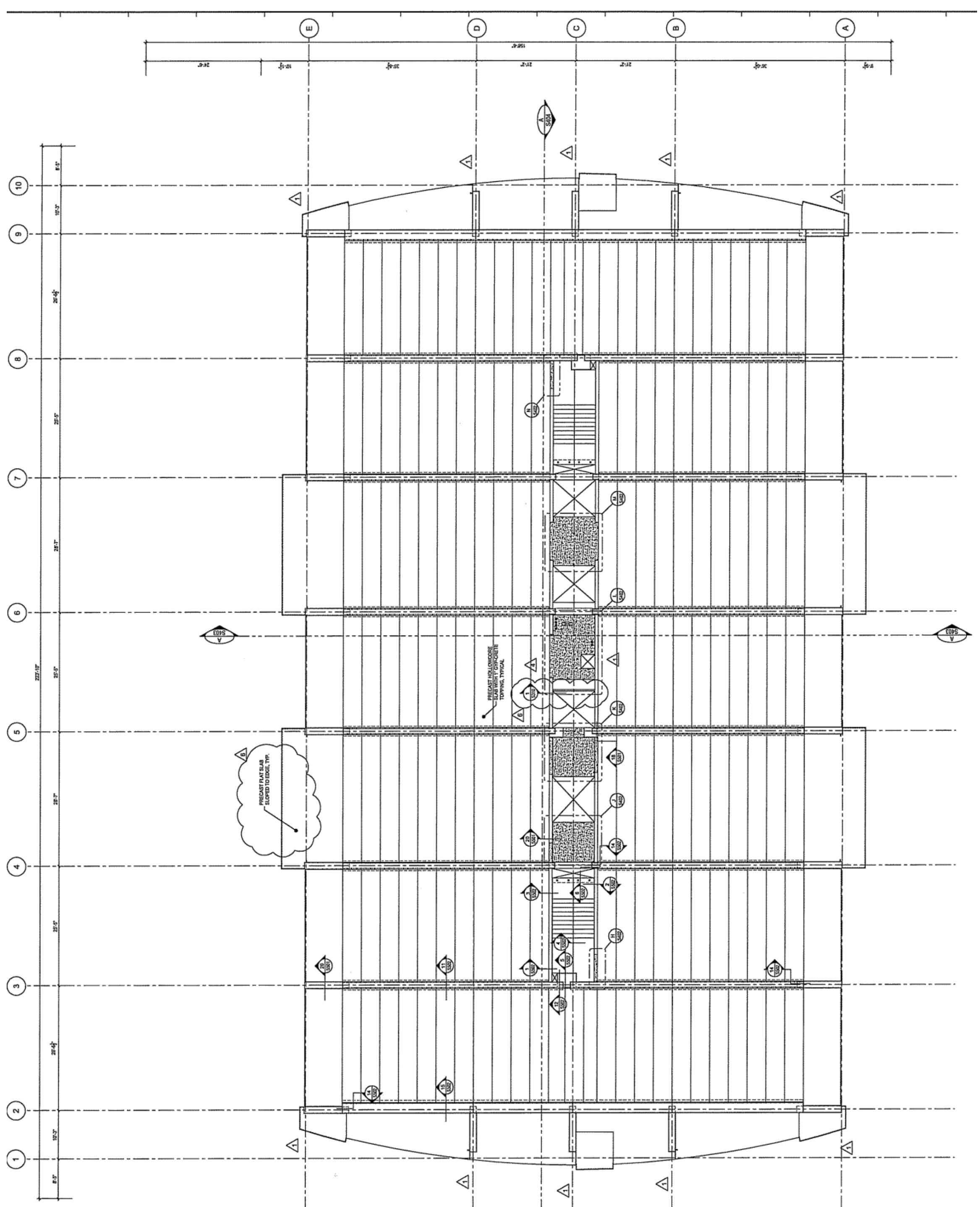
I am satisfied that I understand the existing lateral system well enough to make informed decisions regarding the lateral system for the purpose of my thesis proposal.

VIII. Appendix

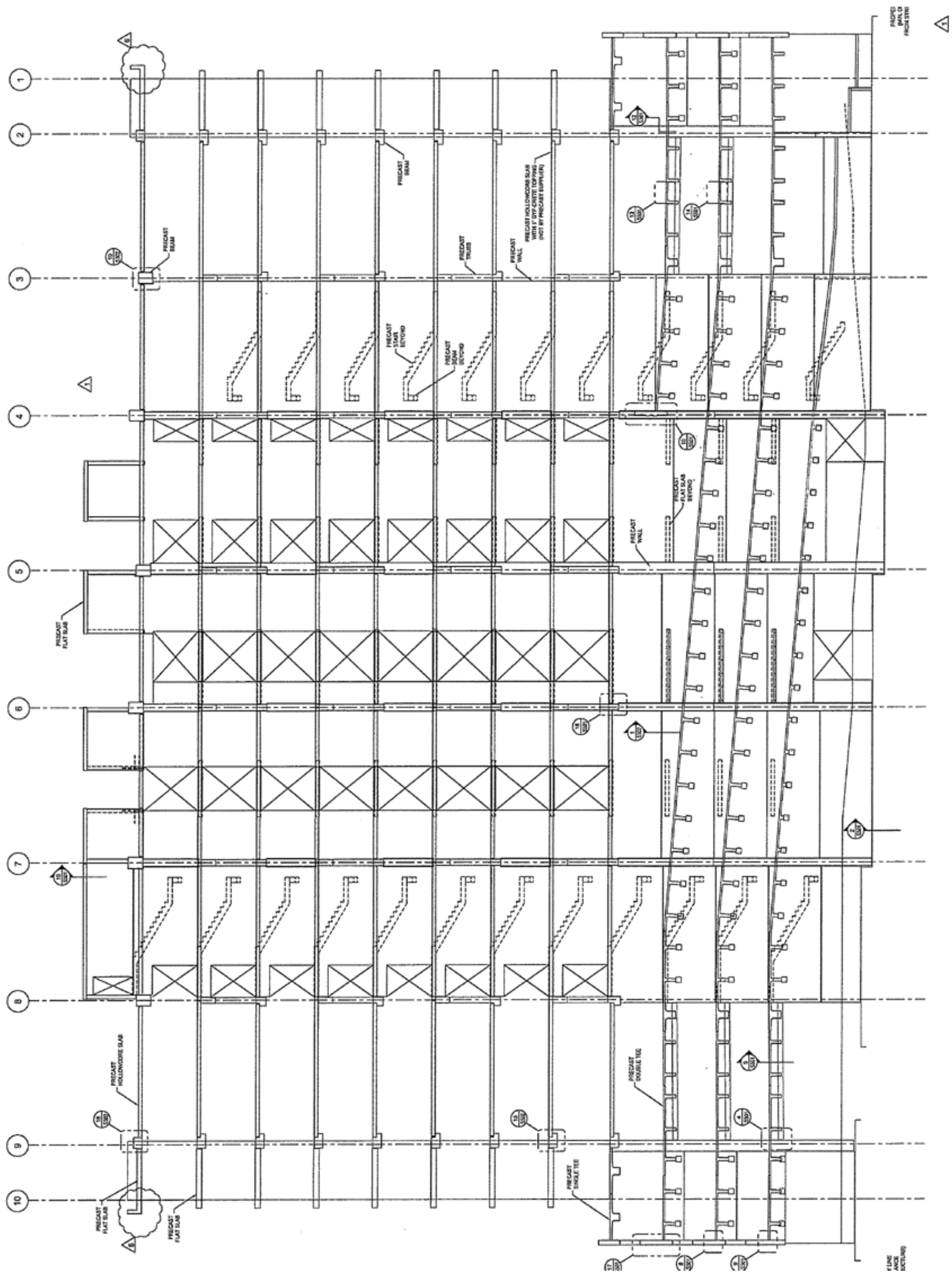
Floor Plan (levels 2-4)



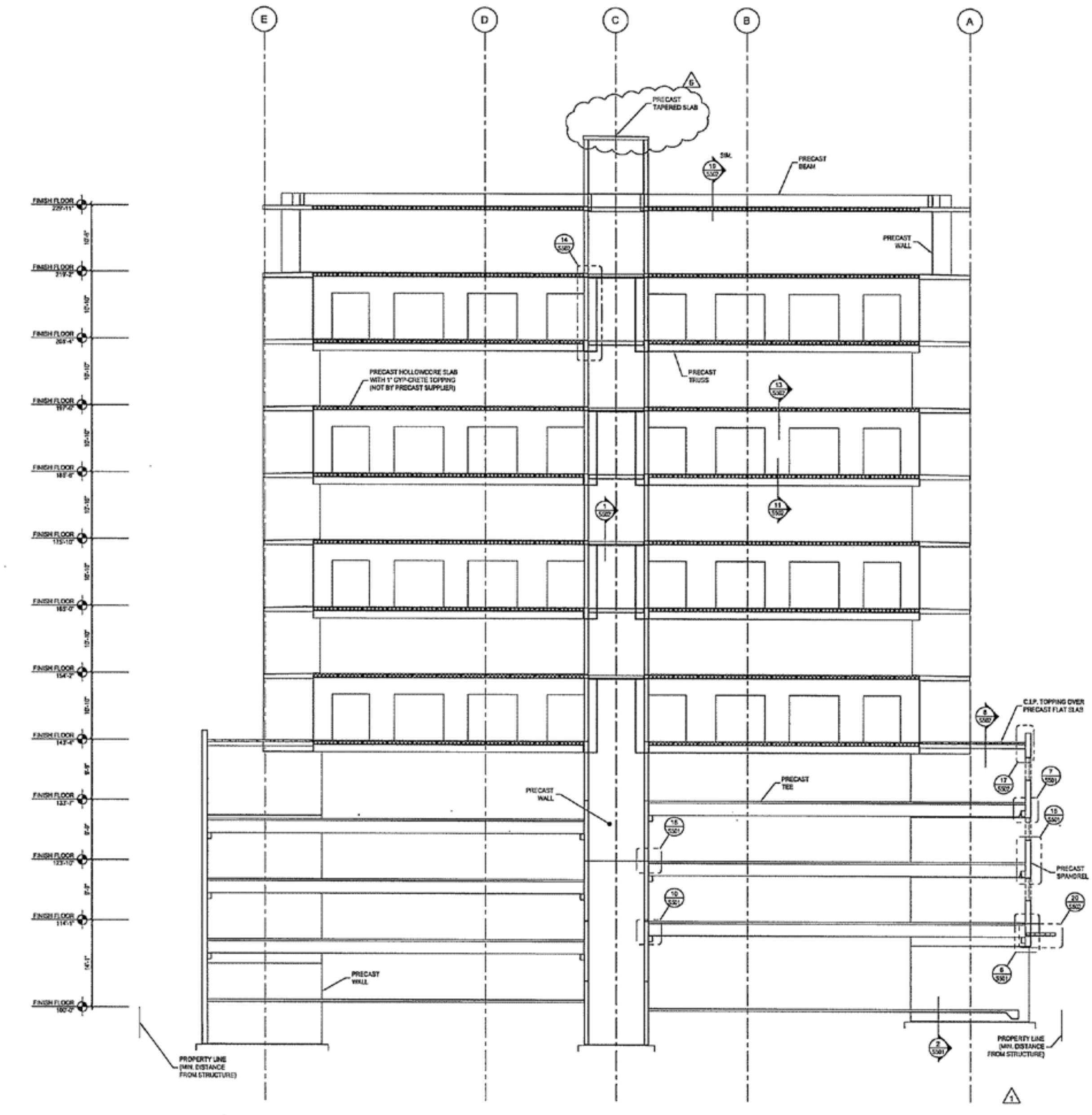
Floor Plan (levels 5-12)



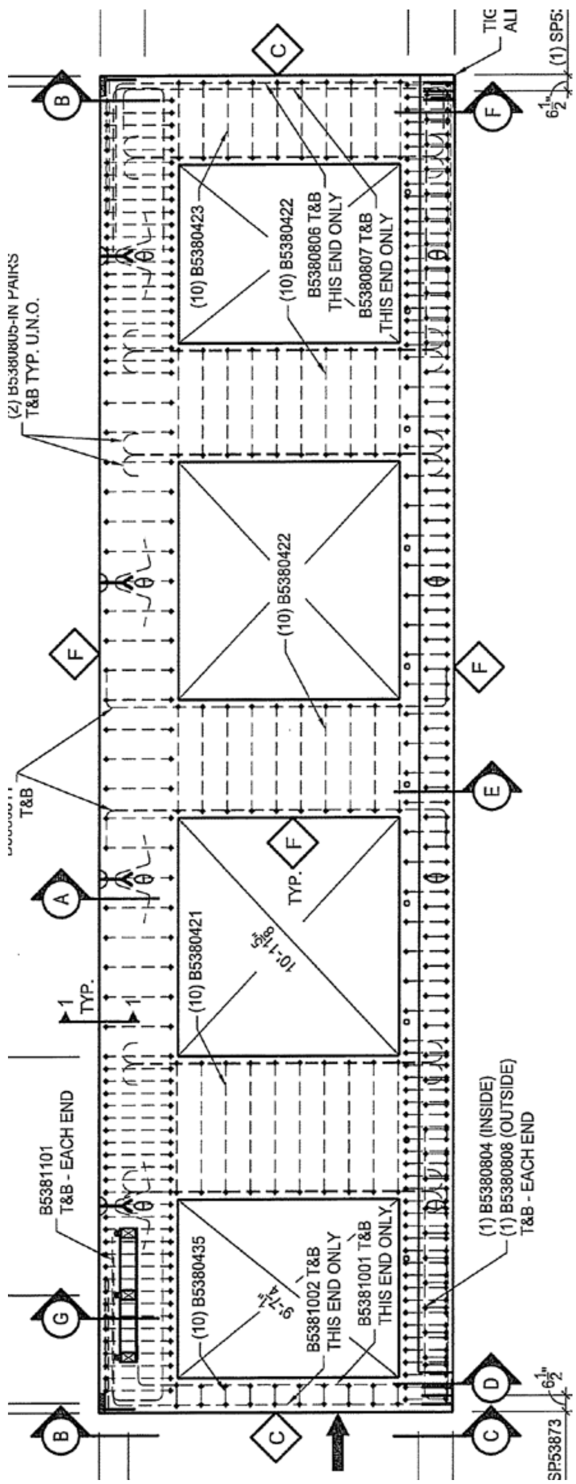
N-S Building Section

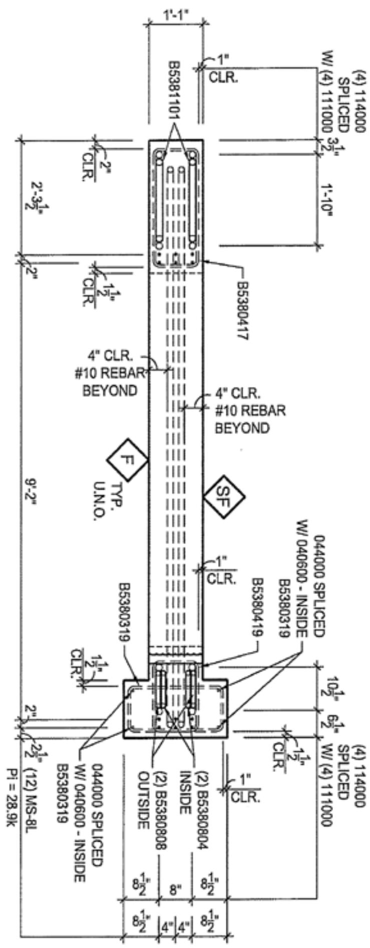


E-W Building Section (lateral system)



Truss Details





DESIGN SECTION

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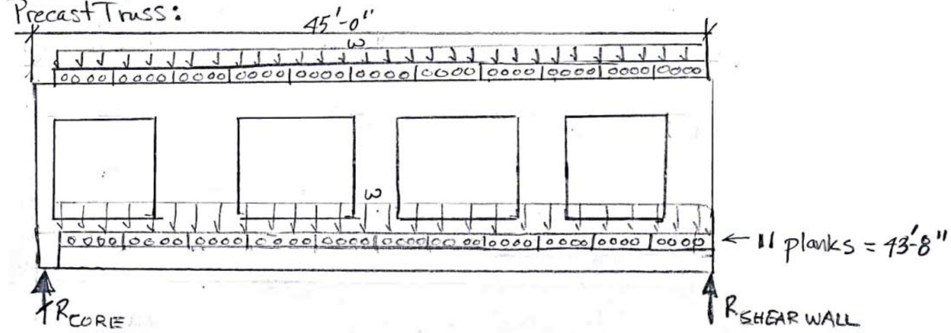
Gravity Loading - Transfer to shear walls

Floor is precast hollowcore planks - simple connection (no moment transfer)

Precast Truss:



Floor: DL = 20 psf
LL = 40 psf
self wt = 54.25 psf

$1.2D + 1.6L = 153.1\,psf$

$\omega = 153.1\left[\frac{1}{2}(27'\text{-}7'' + 25'\text{-}5'') - (1'\text{-}2'')\right]$

$= 153.1\,[25'\text{-}10'']$

$= 3955\,plf$

Truss wt = 59.21^k

$\Sigma F_y = 0 = 59.21^k + 2(3955^{plf}(43'\text{-}8'')) - 2R$

$R = 202.3^k$

Flat Slab Balcony:

Estimate self wt: $\omega_D = \frac{4''}{12}(8')(150)(25'\text{-}10'') = 10.3^k$

LL = 100 psf $\omega_L = 100(8')(25'\text{-}10'') = 20.7^k$

$1.2D + 1.6L = 45.5^k$

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Dist of Forces to shear walls

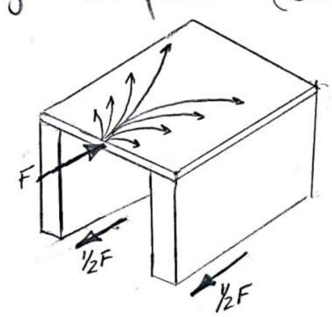
Force (plf)	psf		
1317.6	48.8	P	12'-10"
1298.7	48.1	11	10'-10"
1282.5	47.5	10	10'-10"
1266.3	46.9	9	10'-10"
1247.4	46.2	8	10'-10"
1225.8	45.4	7	10'-10"
1204.2	44.6	6	10'-10"
1179.9	43.7	5	10'-10"
826.2	30.6	4	9'-9"
796.5	29.5	3	9'-9"
758.7	28.1	2	9'-9"
1028.7	38.1	G	14'-1"

$$\frac{28'-7" + 25'-5"}{2} = 27'-0"$$ effective width for largest tributary effect

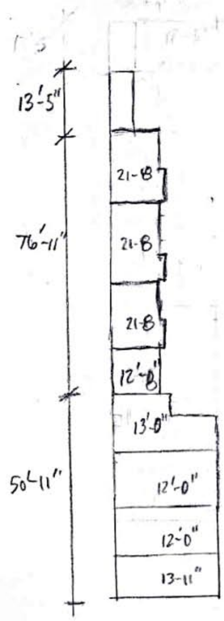
Sample calc for effective force on shear wall (Bottom floor)

38.1 psf(27'-0") = 1028.7 plf

- Each section of shear walls is match casted, therefore will function like a single shear wall tower.
- Lateral System is completely symmetric
  center of mass = center of rigidity = center of structure
- Rigid Diaphram (concrete)



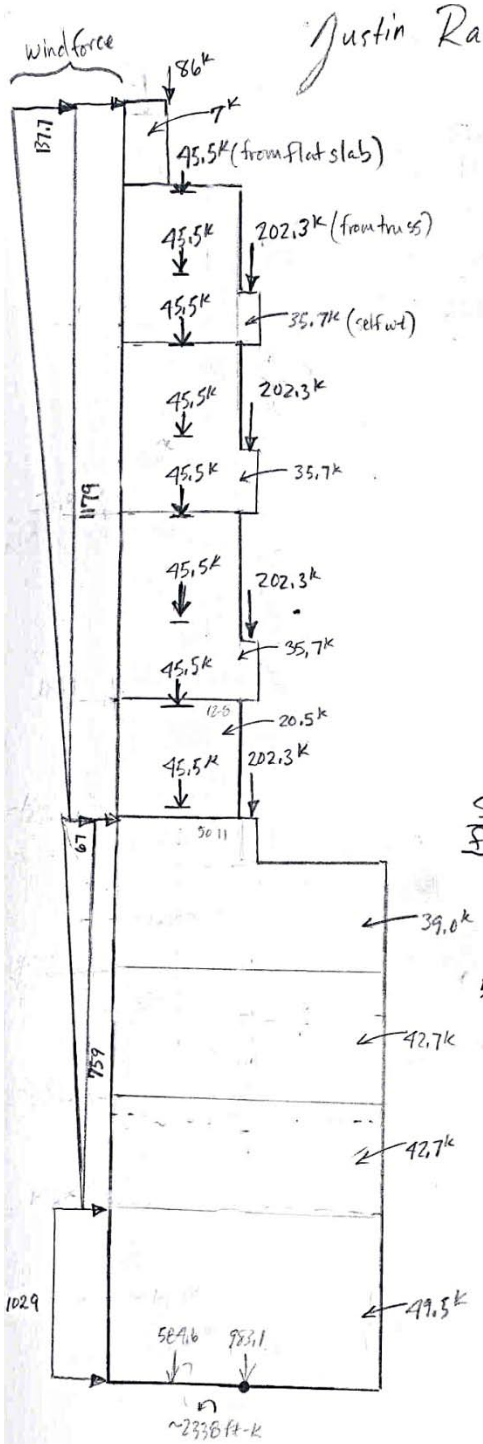
SHEAR WALL DIAGRAM:



Justin Raducha

Shear Wall Loading Diagram

Relative Stiffness Calcs.



$\Sigma F_{LAT} = 156.3^k$

$\Sigma M_{base\ center} = 11088$ ft-k

$P_u = 1613.2$

$$K = \frac{E}{\left(\frac{h}{b}\right)^3 + 3\left(\frac{h}{b}\right)}$$

thickness is constant

Precast

$$E_c = \left(40{,}000\sqrt{f'c} + 1000000\right)\left(\frac{w_c}{145}\right)^{1.5}$$

$w_c = 145$ pcf $\quad f'c = 7000$ psi

$= (40000\sqrt{7000} + 1000000)\,1$

$= 4{,}346{,}640$ psi

$$I_g = \frac{1}{12} b h^3$$

Drift Limit $= H/400 = 130'(12)/400 = \underline{3.90\text{ in.}}$

Stiffness Factors

floor	stiffness MF
12	K = 423
5-11	K = 842.7
4	K = 2502.8
3	K = 2502.8
2	K = 2502.8
G	$K = \dfrac{E}{\left(\frac{14.083}{18.416}\right)^3 + 3\left(\frac{14.083}{18.416}\right)} = 1585.6$

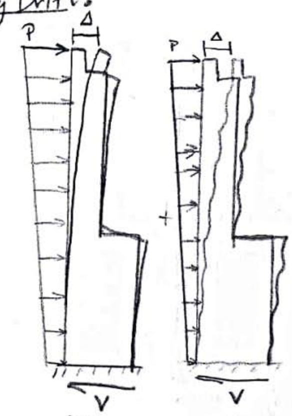
$\Sigma K = 15035.2$

$$\text{rel. stiffness} = \frac{K_i}{\Sigma K}$$

$$\text{Story force} = \frac{K_i}{\Sigma K} V = f_x$$

$$\text{Story shear} = \sum_{i=0}^{n} F_i$$

Story Drift:



$\Delta = \Delta_F + \Delta_S$

Drift Calculations

WIND (E-W)

story	story ht (ft)	wall width (ft)	stiffness	relative stiffness	story force (kips)	story shear (kips)	I (in^4)	Δf	Δs	Δ
P	12.8333	3	42.3	0.0028	1.55	1.55	62208	0.0070	0.0003	0.0073
11	10.8333	9.208	842.7	0.0560	30.85	32.40	1798781	0.0030	0.0016	0.0047
10	10.8333	9.208	842.7	0.0560	30.85	63.26	1798781	0.0059	0.0032	0.0091
9	10.8333	9.208	842.7	0.0560	30.85	94.11	1798781	0.0088	0.0048	0.0136
8	10.8333	9.208	842.7	0.0560	30.85	124.97	1798781	0.0117	0.0063	0.0180
7	10.8333	9.208	842.7	0.0560	30.85	155.82	1798781	0.0146	0.0079	0.0225
6	10.8333	9.208	842.7	0.0560	30.85	186.68	1798781	0.0175	0.0095	0.0270
5	10.8333	9.208	842.7	0.0560	30.85	217.53	1798781	0.0204	0.0110	0.0314
4	9.75	18.4166	2502.8	0.1665	91.64	309.17	14391658	0.0026	0.0071	0.0097
3	9.75	18.4166	2502.8	0.1665	91.64	400.81	14391658	0.0034	0.0092	0.0126
2	9.75	18.4166	2502.8	0.1665	91.64	492.44	14391658	0.0042	0.0112	0.0154
G	14.08333	18.4166	1585.6	0.1055	58.06	550.50	14391658	0.0142	0.0182	0.0323
SUM	132.0		15035.2	1	550.5			0.1133	0.0903	0.2036

SIESMIC (E-W)

story	story ht (ft)	wall width (ft)	stiffness	relative stiffness	story force (kips)	story shear (kips)	I (in^4)	Δf	Δs	Δ
P	12.8333	3	42.3	0.0028	1.24	1.24	62208	0.0056	0.0002	0.0058
11	10.8333	9.208	842.7	0.0560	24.74	25.98	1798781	0.0024	0.0013	0.0038
10	10.8333	9.208	842.7	0.0560	24.74	50.72	1798781	0.0048	0.0026	0.0073
9	10.8333	9.208	842.7	0.0560	24.74	75.46	1798781	0.0071	0.0038	0.0109
8	10.8333	9.208	842.7	0.0560	24.74	100.20	1798781	0.0094	0.0051	0.0145
7	10.8333	9.208	842.7	0.0560	24.74	124.94	1798781	0.0117	0.0063	0.0180
6	10.8333	9.208	842.7	0.0560	24.74	149.68	1798781	0.0140	0.0076	0.0216
5	10.8333	9.208	842.7	0.0560	24.74	174.42	1798781	0.0163	0.0089	0.0252
4	9.75	18.4166	2502.8	0.1665	73.48	247.90	14391658	0.0021	0.0057	0.0078
3	9.75	18.4166	2502.8	0.1665	73.48	321.37	14391658	0.0027	0.0073	0.0101
2	9.75	18.4166	2502.8	0.1665	73.48	394.85	14391658	0.0034	0.0090	0.0124
G	14.08333	18.4166	1585.6	0.1055	46.55	441.40	14391658	0.0114	0.0146	0.0259
SUM	132.0		15035.2	1	441.4			0.0909	0.0724	0.1633

WIND (N-S)

story	story ht (ft)	wall width (ft)	stiffness	relative stiffness	story force (kips)	story shear (kips)	I (in^4)	Δf	Δs	Δ
P	12.8333	3	42.3	0.0028	1.59	1.59	62208	0.0071	0.0003	0.0074
11	10.8333	9.208	842.7	0.0560	31.62	33.21	1798781	0.0031	0.0017	0.0048
10	10.8333	9.208	842.7	0.0560	31.62	64.83	1798781	0.0061	0.0033	0.0094
9	10.8333	9.208	842.7	0.0560	31.62	96.45	1798781	0.0090	0.0049	0.0139
8	10.8333	9.208	842.7	0.0560	31.62	128.08	1798781	0.0120	0.0065	0.0185
7	10.8333	9.208	842.7	0.0560	31.62	159.70	1798781	0.0150	0.0081	0.0231
6	10.8333	9.208	842.7	0.0560	31.62	191.32	1798781	0.0179	0.0097	0.0276
5	10.8333	9.208	842.7	0.0560	31.62	222.95	1798781	0.0209	0.0113	0.0322
4	9.75	18.4166	2502.8	0.1665	93.92	316.86	14391658	0.0027	0.0072	0.0099
3	9.75	18.4166	2502.8	0.1665	93.92	410.78	14391658	0.0035	0.0094	0.0129
2	9.75	18.4166	2502.8	0.1665	93.92	504.70	14391658	0.0043	0.0115	0.0158
G	14.08333	18.4166	1585.6	0.1055	59.50	564.20	14391658	0.0145	0.0186	0.0331
SUM	132.0		15035.2	1	564.2			0.1161	0.0925	**0.2087**

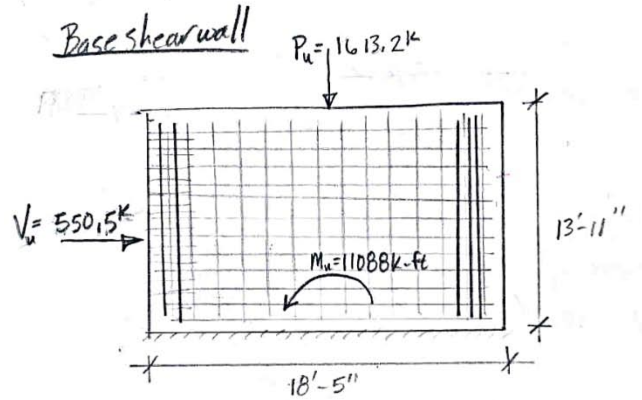
SIESMIC (E-W)

story	story ht (ft)	wall width (ft)	stiffness	relative stiffness	story force (kips)	story shear (kips)	I (in^4)	Δf	Δs	Δ
P	12.8333	3	42.3	0.0028	2.48	2.48	62208	0.0112	0.0005	0.0116
11	10.8333	9.208	842.7	0.0560	49.48	51.96	1798781	0.0049	0.0026	0.0075
10	10.8333	9.208	842.7	0.0560	49.48	101.44	1798781	0.0095	0.0051	0.0146
9	10.8333	9.208	842.7	0.0560	49.48	150.92	1798781	0.0141	0.0077	0.0218
8	10.8333	9.208	842.7	0.0560	49.48	200.40	1798781	0.0188	0.0102	0.0289
7	10.8333	9.208	842.7	0.0560	49.48	249.88	1798781	0.0234	0.0127	0.0361
6	10.8333	9.208	842.7	0.0560	49.48	299.36	1798781	0.0280	0.0152	0.0432
5	10.8333	9.208	842.7	0.0560	49.48	348.84	1798781	0.0327	0.0177	0.0504
4	9.75	18.4166	2502.8	0.1665	146.95	495.79	14391658	0.0042	0.0113	0.0156
3	9.75	18.4166	2502.8	0.1665	146.95	642.75	14391658	0.0055	0.0147	0.0202
2	9.75	18.4166	2502.8	0.1665	146.95	789.70	14391658	0.0067	0.0180	0.0248
G	14.08333	18.4166	1585.6	0.1055	93.10	882.80	14391658	0.0227	0.0291	0.0518
SUM	132.0		15035.2	1	882.8			0.1817	0.1448	**0.3265**

Spot Checks

Justin Raducha Shear Wall Spot Check

Base shearwall

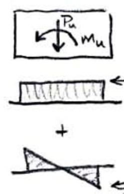


$f'_c = 7000\text{ psi}$
$f_y = 60{,}000\text{ psi}$
$t = 16''$

$$P_{u,BE} = \frac{P_{u,wall}}{2} + \frac{M_{u,wall}}{z} = \frac{1613.2}{2} + \frac{11088}{18.41\bar{6}} = 1408.7^k$$

$A_g = \frac{16''}{12}(18.41\bar{6}') = 24.56\text{ ft}^2$

$I_g = \frac{(\frac{16''}{12})(18.41\bar{6}')^3}{12} = 694.0\text{ ft}^4$



$\frac{P_u}{A_g} = \frac{1613.2}{24.56} = 65.7\text{ ksf}$

$\frac{M_u(\frac{h_w}{2})}{I_g} = \frac{11088(\frac{18.41\bar{6}}{2})}{694.0} = 147.1\text{ ksf}$

$f_c = 65.7 + 147.1 = 212.8\text{ ksf} = 1.478\text{ ksi}$

$0.2f'_c = 0.2(7) = 1.4\text{ ksi}$

$1.4 \approx 1.478$ ok? No B.E. needed (error in moment/Gravity?)

$A_{cv} = (16'')(18.41\bar{6})(12) = 3536\text{ in}^2$

$2A_{cv}\sqrt{f'_c} = 2(3536)(\sqrt{7000}) = 591.7^k$ > $550.5^k \not< 591.7^k$ ∴ may not need 2 curtains but test design that uses 2 curtains

req'd ρ_l, ρ_t:

try $\rho_l, \rho_t \geq 0.0025$

$A_{s,long,req'd} = 0.0025(16'')(12) = 0.48\text{ in}^2/\text{ft}$

#5 bars → 2 curtains of #5 bars $= 0.62\text{ in}^2/\text{spacing}$

spacing: $\frac{0.48}{12''} = \frac{0.62}{s}$ $\quad s_{req'd} = 15.5'' > \underset{\text{design spacing}}{15''} < \underset{\text{ACI}}{18''}$ ∴ ok ✓

Shear Capacity:

use $\alpha_c = 2.0$

$\rho_t = \frac{2(0.31)}{12(16)} = 0.0032$

$$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_t f_y) = (3536)\left(\frac{2\sqrt{7000} + 0.0032(60000)}{1000}\right)$$

$= 1270.6^k \rightarrow \phi V_n = 762^k > 550.5^k$ ∴ ok ✓

Check B.E. Capacity:

$A_{ST} = (6)\#11 = 6(1.56 in^2) = 17.16 in^2$

$\rho_{ST} = \frac{17.16}{16(\approx 30)} = 0.0357$ $\quad \overset{\rho_{min}}{0.01} < 0.0357 < \overset{\rho_{max}}{0.06}$ ∴ OK ✓

Axial load capacity

$\phi P_{n,max} = 0.8\phi\left[0.85 f'_c(A_g - A_{ST}) + f_y A_{ST}\right]$

$= 0.8(0.7)\left[0.85(7)(480 - 17.16) + 60(17.16)\right]$

$= 2118.8^k > 1408.7^k$ ∴ OK ✓

∴ This shear wall is sufficient as designed.

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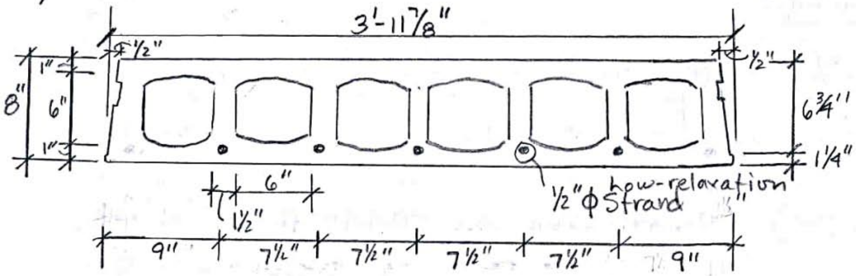
Hollowcore Design Check



$f'_c = 6000$ psi
$f'_{ci} = 4000$ psi
$w_o = 54.25$ psf
WT/FT = 145 lbs
$P_i = 28.9^k$ ea
$f_{pu} = 270$ ksi
$w_D = 20$ psf
$w_L = 40$ psf
$L = 27'-8''$

$A_{ps} = 5(0.153 in^2) = 0.765 in^2$

$\gamma_p = 0.28 \quad \frac{f_{py}}{f_{pu}} \geq 0.9 \quad f_{py} \geq 243$ ksi

Loads

$w_D = (w_o + w_D)(3.99') = (54.25 + 20)(3.99') = 296.2$ plf

$w_L = w_L(3.99') = 40(3.99') = 159.6$ plf

$M_D = \frac{w_D l^2}{8} = \frac{296.2(27.667)^2}{8} = 28340.6$ ft-lb

$M_L = \frac{w_L l^2}{8} = \frac{159.6(27.667)^2}{8} = 15270.6$ ft-lb or $= 20{,}711.3$ before loading

$M_u = 1.2(28340.6) + 1.6(15270.6) = 58{,}441.7$ ft-lb

$A_c \approx (3.99')(8'') - 6(6^2)$
$\approx 167 in^2$

$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p f_{pu} \rho_p}{\beta_1 f'_c}\right]$

$\beta_1 = 0.85 - 0.05\left(\frac{6000-4000}{1000}\right) = 0.75$

$\rho_p = \frac{A_p}{bd_p} = \frac{0.765}{6.75(3.99)_{\times 12}} = 0.0024$

$= 270\left[1 - \frac{.28(270)(0.0024)}{.75(6)}\right] = 259.3$ ksi

$a = \frac{A_{ps} f_{ps}}{.85 f'_c b} = \frac{0.765(259.3)}{.85(6)(3.99\times 12)} = 0.81'' < 1''$ ∴ ✓ok

$c = a/\beta_1 = 1.083''$

unknown (estimated) $\begin{cases} I \approx 1581 in^4 & S_t = 392.3 in^3 \quad S_b = 398.2 in^3 \\ y \approx 3.97'' & C_1 = 4.03 \quad C_2 = 3.97 \quad e = 2.97'' \end{cases}$

prestressing force @ transfer

$f_p \leq \begin{cases} .74 f_{pu} = 200 \text{ ksi} \leftarrow \text{use} \\ .82 f_{py} \geq 200 \text{ ksi} \end{cases}$

$P_i = 28.9^k(5) = 144.5^k$

use R=0.82 $\quad P_e = P_i R = 144.5(.82) = 118.5^k$

	Top	Bottom (psi)
$P_i/A = 144.5^k / 167 in^2 * 1000$	−865	−865
$P_i e/S = 144.5(2.97)/S * 1000$	+1094	−1078
Initial Stress @ ends	+229	−1943
$M_D/S = 20711 \times 12/S$	−634	+624
Initial Stress @ midspan	−405	−1319
$P_e/A = 118.5/167 * 1000$	−710	−710
$P_e e/S = 118.5(2.97)/S \times 1000$	+897	−884
Final Stress @ ends	+187	−1594
$M_D/S = 28340.6 \times 12/S$	−867	+854
Long term stresses @ midspan	−680	−740
$M_L/S = 15270.6 \times 12/S$	−467	+460
Max Service Stress	−1147	−280

$f_{ti} = 3\sqrt{f'_{ci}} = {}^{+}190$ psi
$f_{ci} = .6 f'_{ci} = {}^{-}2400$ psi
$f_{ts} = 7.5\sqrt{f'_c} = {}^{+}581$ psi
$f_{cs} = .45 f'_c = -2700$ psi

Check vs ACI Allowable:

f_{ci}: −2400 psi > −1943 ∴ ✓ok $\qquad f_{cs}$: −2700 psi > −1594 ∴ ok ✓

f_{ti}: +190 $\not>$ +392 × not ok $\qquad f_{ts}$: +581 psi > +187 ∴ ok ✓

↳ need 1 #4 bar (OMIT IN PRACTICE)

Check flexural strength

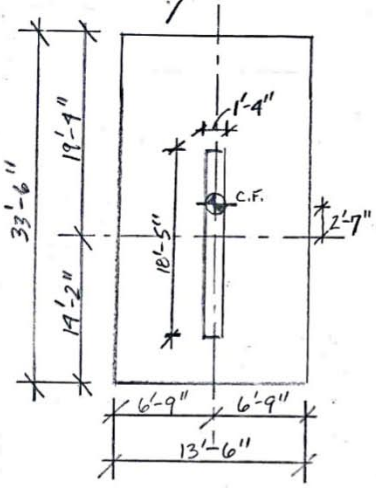
$M_N = A_{ps} f_{ps} (d - a/2) = 0.765(259.3)(6.75 - \frac{.81}{2})(\frac{1000}{12}) = 104885$ ft-lb

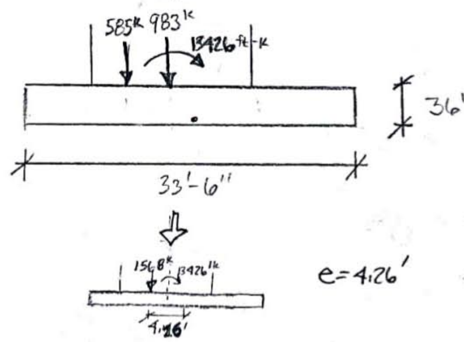
$\frac{c}{d_t} = \frac{1.083''}{6.75''} = 0.16 < 0.375 \therefore \phi = 0.9$

$\phi M_N = 0.9(104885) = 94397$ ft-lb $\gg 58442$ ft-lb $\therefore$ OK ✓

★ This member is sufficient to carry the applied service loads at the longest span recorded using (5) ½φ 270K Low Relaxation tendons.

Justin Raducha . Footing Spot Check





$f'c = 4000\,psi$

$q_a = 8000\,psf$

$e = 4.26'$

$\frac{B}{6} = \frac{33.5}{6} = 5.58' > 4.26'$

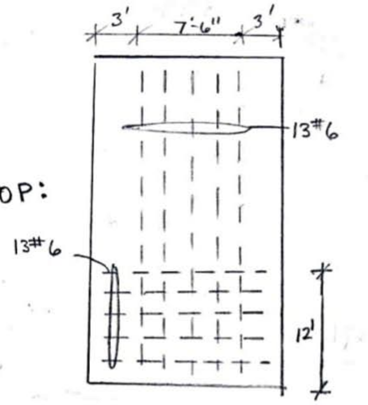
inside kern
∴ stable

$B' = B = 13.5'$

$L' = L - 2e = 33.5' - 2(4.26') = 24.98'$

$A' = B'L' = 337.2\,ft^2$

TOP:



$q_u = \frac{P_u}{A'} = \frac{1568^k}{337.2\,ft^2} = 4.65\,ksf < 8000\,psf$ ∴ ok ✓

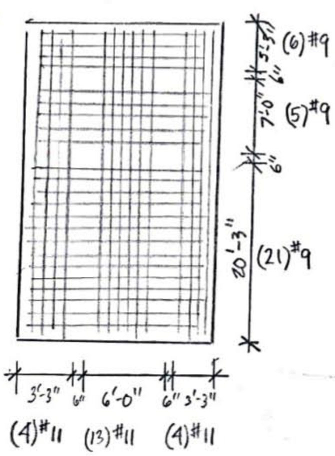
→ 32.3 psi

$V_c = \phi 4\sqrt{f'c} = 0.75(4)(\sqrt{4000}) = 190\,psi$

$d^2\left(190 + \left(\frac{32.3}{4}\right)\right) + d\left(190 + \frac{32.3}{2}\right)(16+221) = \frac{32.3}{4}\left[452.25 - 16(221)\right]$

$d \geq 9.79'' < 36'' - 3'' - .705 = 32.3$ ∴ ok ✓

BOT:



long: $l = \frac{13.5' - \frac{16}{12}}{2} = 6.08'$

$M_u = \frac{4.65(6.08)^2}{2} = 86.04'^k$

$V_u = 4.65(6.08) = 28.3^k$

$\phi V_N = .75(2)\sqrt{4000}(32.3)($
$= 36.77^k$

$\phi V_n > V_u$ ∴ ok ✓

$A_s = 21(1.56) + 13(0.44) = 38.48\,in^2$

$a = \frac{38.48(60)}{.85(4)(36)} = 18.86$

$\rho = \frac{A_s}{bd} = \frac{38.48}{13.5(12)(36)} = 0.0066 > 0.0018$ ∴ ok ✓

Check M_u: $86.04(12) \leq 0.9(32.76)(60)\left(32.3 - \frac{18.86}{2}\right)$

$1032.5 \leq 40455$ ∴ ok

$\phi B_N = 0.65(0.85)(4)(16(221))$

$= 7815^k > 1568^k$ ∴ ok ✓