Technical Report 1

The Residences
Anne Arundel County, Maryland

10/4/2010
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Ryan English - Structural Option

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Executive Summary

A structural analysis was performed on The Residences in Anne Arundel County, Maryland to better understand how the structural system functions. The Residences is a five to six story 300,000 s.f. mix use residential and retail apartment building. Included in this report are a study of the Structural Systems, a detail load analysis on the structure, and a series of spot checks to confirm the integrity of the structural.

From the load analysis it was determine that the seismic loads was the controlling lateral force on the structural, with a base shear of 1,355 kips and an overturning moment of 63,704.5 ft-kips. The wind load was confirmed to be much lower in comparison to the seismic lode, with a base shear of 62 kips and an overturning moment of 2,500 ft-kips. A further in-depth analysis of the lateral system will be presented in a future report.

After the completion of the load analysis, a series of spot checks were conducted on the structural to check the validity of the elements used. The spot check that was performed in this report was on a typical floor slap, floor joist, and bearing wall. From this analysis it was determined that these structural elements were more than sufficient to carry the gravity loads applied to them.

Introduction

Located in Anne Arundel County, Maryland the Residence is a new construction apartment and retail building part of the Arundel Preserve Town Center Phase I project (Figure 1). The Residence is a five to six story, 300,000 s.f., residential apartment building with 6,000 s.f. retail space surrounding a 5 story precast parking garage. This apartment building houses 242 upscale residential units consisting of studio, one, and two bedroom layouts and two level units. Along with the residential units the building also included a terrace level that contains a clubhouse, health center, and an outside pool. Construction of The Residence began in the fall of 2009 and should be completed in the beginning of 2011. It is own and managed by the Somerset Construction Company and was design by KTGY.

The structural of The Residence is comprises of the Hanbro floor system, this system uses a steel bar joist that supports a concrete slab (Figure 2). The floor systems are supported by 6" light gage metal studs bearing and shear walls located throughout the building. A more in-depth structural analysis and detail shall fallow in this report.

Figure 1: site plan, Light Broun-build, Gray-parking garage.

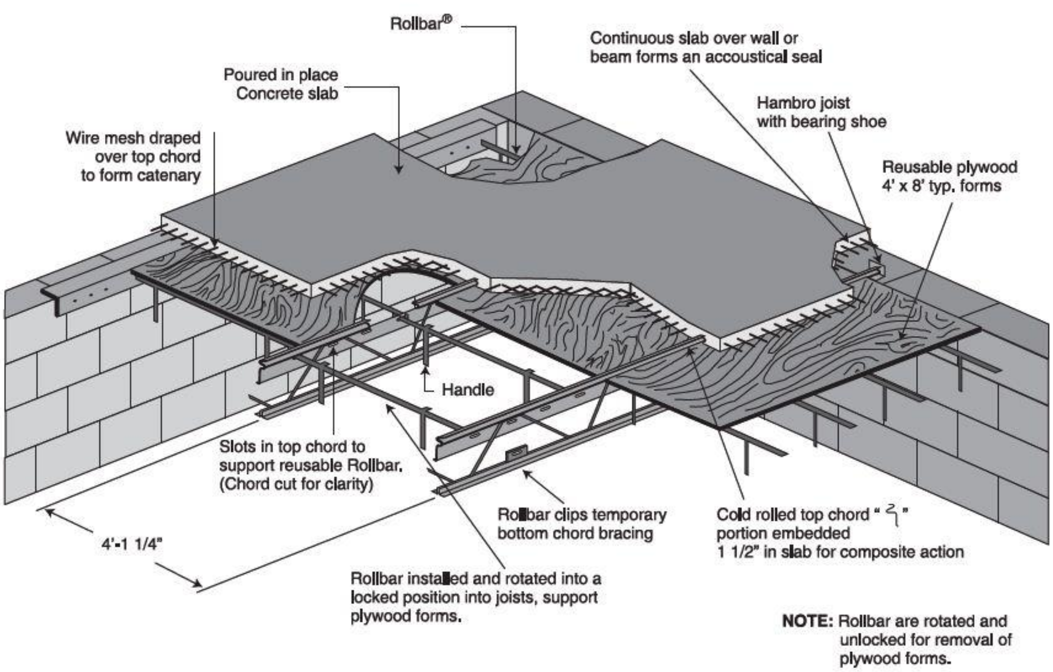


Figure 2: Hambro floor joist system.

Structural system

Foundation System

According to the geotechnical report the building rest on Silt-Clay Facies which is identified as clay, silt, and subordinate fine to medium grained muddy sand. The ground water table was located to be at min 24 feet below existing grade which is well below the foundation of the building. From the report it was determined that the structures can be supported on shallow spread footings with an allowable bearing pressure of 5,000 pounds per square foot.

The building foundation system uses a 3’-0” wide strip footing with 3’-0”x3’-0” to 15’-0”x15’-0” column footing pads located manly around the retail space and clubhouse area (Figure 3). The slab on grade was design to be 4” thick reinforced with 6 x 6 W1.4 xW1.4 welded wire fabric. All foundation concrete was to be a 3,000 psi at 28 day strength.

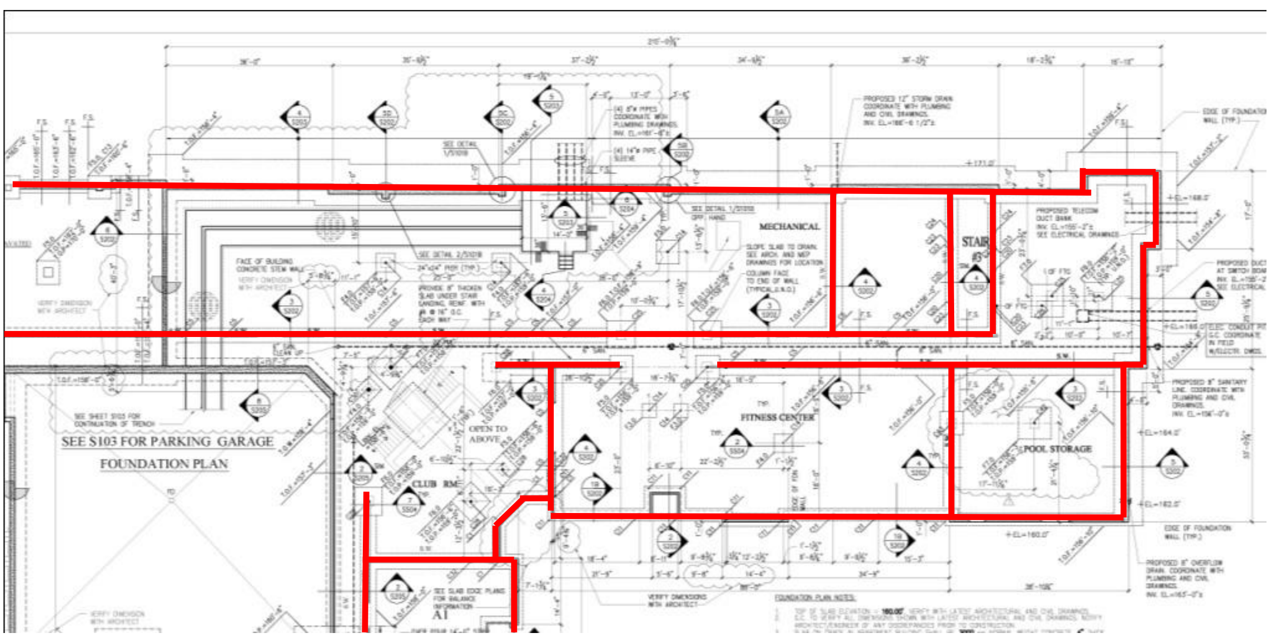
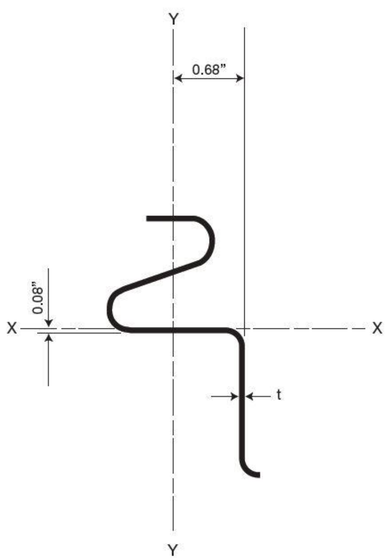


Figure 3: Foundation Plan, Part of the East wing. See Appendix A for more plans

Floor System

The Floor system that was used for the Residence was the Hambro floor joist flooring system (Figure 2). The Hambro floor system uses a spicily design steel bar joist with a "s" shape top compression cord which serves three functions in the system, a compression member in the non-composite joist during the construction stage, a chair for the welded wire fabric, and it becomes a continuous shear connection for the composite stage. Detail information of the "s" shape top cord can be seen in figure 4. The floor slab is a 3" thick 3,000psi concrete with 6 x 6 W2.9 x W2.9 welded wire fabric, this particular floor thinness was pick to give the system a 2 hours system. The slab is than supported by a 20" deep Hambro bar joist.

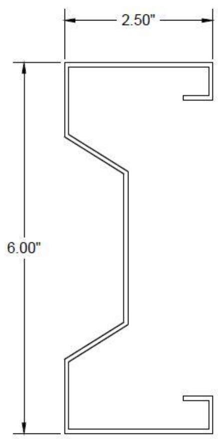


I_x = 0.66 in.^{4}
Top Chord S_x = 0.45 in.^{3}
Bottom Chord S_x = 0.287 in.^{3}
L_y = 0.187 in.^{4}
S_y = 0.167 in.^{3}
t = 13 ga. = 0.090 in.
A_{net} = 0.56 in.^{2}
= (deducting 3/8" deep slot = 6.25 x 0.090)
F_y = 50 ksi min.
F_a = 29.1 ksi

Figure 4: Top Card of the Hambro joist, "s" cord, with section properties.

Framing System

The design framing system used in the Residence was a light gage steel load bearing walls which is used to support the Hambro floor system and gravity loads in the build. The particular system that was used in the construction of the build was the SigmaStud® load bearing light gage steel stud which is a product of The Steel Network Company. The stud design is engineered to have a significant increase in load capacity when compared to the conventional “C” shaped studs. The Residence uses a 6” wide 18 gage stud with a flange length of 2.5”, See figure 5 for detail information of stud used for the building. The exterior wall and interior corridor walls of the Residence are primarily the bearing walls in the building; figure 6 shows the location off the bearing walls in the building.



$A=0.772\ in^2$
$I_x=4.183\ in^4$
$I_y=0.513\ im^4$
$F_y=50$ ksi
$r_x=2.328$ in
$r_y=0.815$ in
$E=29{,}000$ ksi

Figure 5: Section of light gage steel stud, with section properties.

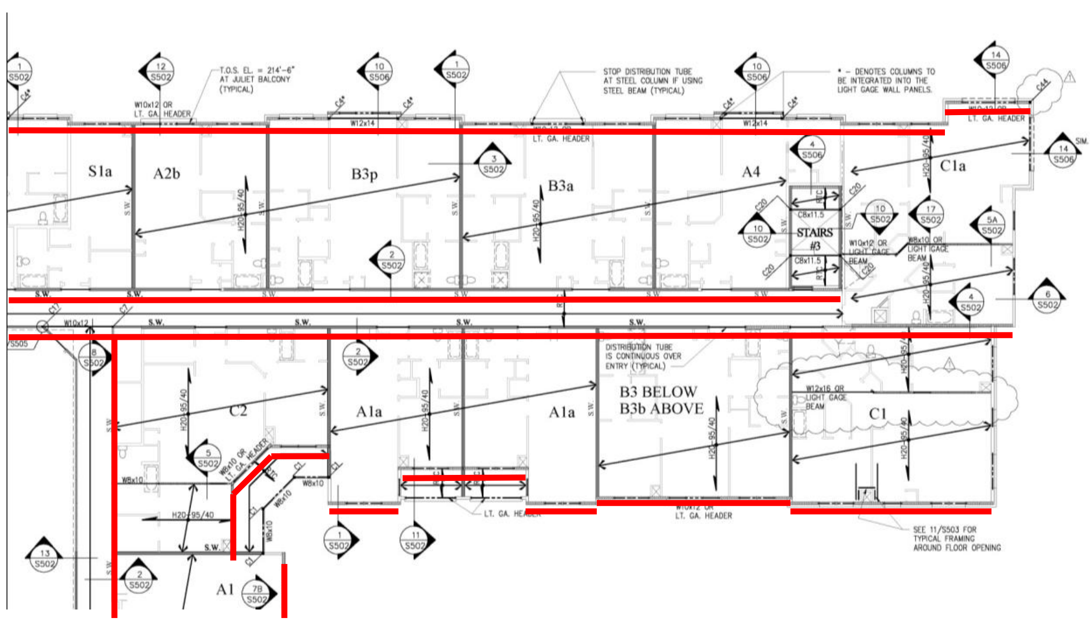


Figure 6: Location of bearing walls, See Appendix A for more plans.

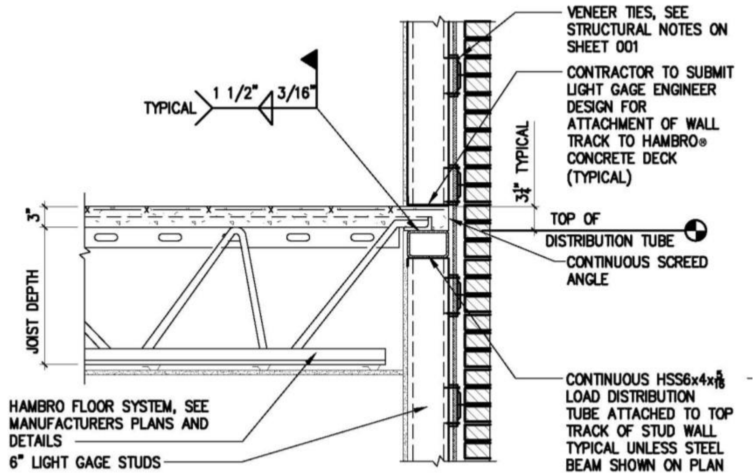


Figure 7: Exterior wall framing details

Lateral System

The lateral system used in the Residence was a light gage shear wall system design and engineered by The Steel Network Company. The system utilizes light gage 50 ksi steel hot dipped galvanized coated straps on both sides of the wall for shear resistance. A 6" wide flat strap was used in lateral system of the Residence. See figure 8 for a simple framing detail. The shear walls are located all throughout the build, figure 9, with most of the shear wall located in the corridors walls and the walls separating the apartment.

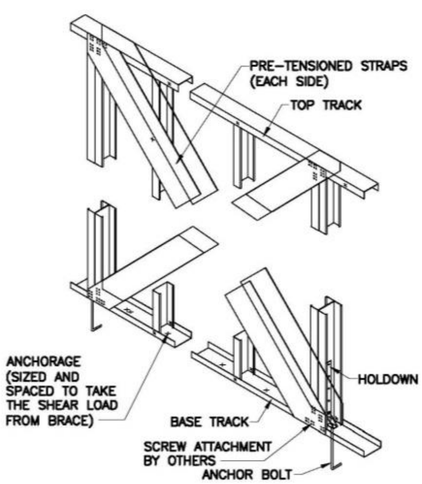


Figure 8: Lateral resistance system.

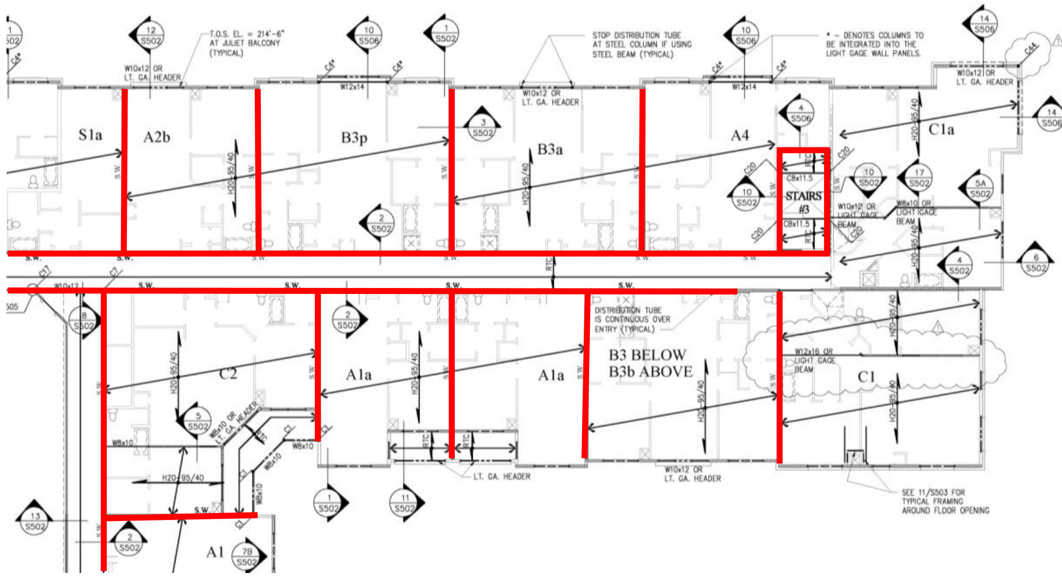


Figure 9: Location of the shear walls, Appendix A for more plans.

Roof System

The roof system was the same system, Hambro flooring system, which was used for the floor throughout the building. The roof slab is 3" thick 3,000psi concrete with 6 x 6 W2.9 x W2.9 welded wire fabric which is support by a 20" deep Hambro joist.

Materials Used

Concrete

Floor Slab	Norman Weight	f'c=3,000 psi
Roof Slab	Norman Weight	f'c=3,000 psi
Slab on grade	Norman Weight	f'c=3,000 psi
Footings	Norman Weight	f'c=3,000 psi

Steel

W shapes	ASTM A992	Grade 50
Square and Rectangular HSS	ASTM 500	Grade B
Channels	ASTM A36	
Angles shapes	ASTM A36	
Steel Plates	ASTM A36	

Reinforcement

Deformed bars	ASTM A-615	Grade 60
Welded wire Fabric	ASTM A-185	

Codes and References

Design Codes

National Model Code:

2006 International Building Code

Design Codes:

Steel construction Manual 13th edition, AISC

American Iron and Steel Institute (AISI) Design of Cold Formed Steel Structural members

American Concrete Institute (ACI) ACI 530-05, Building Code Requirements for Masonry Structures

American Concrete Institute (ACI) ACI 318-08, Building Code Requirements for Structural Concrete

Structural Standards:

American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design loads for Buildings and other Structures

Thesis Codes

National Model Code:

2006 International Building Code

Design Codes:

Steel construction Manual 13th edition, AISC

ACI 318-08, Building Code Requirements for Structural Concrete

Structural Standards:

American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design loads for Buildings and other Structures

Load Analysis

Gravity Load

For this report and all further report the use of the ASCE7-05 minimal design loads will be used. When comparing the design live loads to the minimal ASCE7-05 loads it was found that all the load with the exception of the roof live load were identical to the ASCE7-05. Table 1.1 shows the design and ASCE7-05 live loads on the build. The roof live load was design to be 30 psf witches is slightly higher than what is stated in ASCE7-05, 20 psf. It is likely that this value was higher to support some of the MEP system on the roof as well as experience of the designers.

Table 1.1: Live Loads

Location	Design (psf)	ASCE7-06 (psf)
Roof	30	20
Living	40	40
Private Decks/Balconies	60	60
Corridors Exit stairs	100	100
Light Storage	125	125

Dead loads values we found form a series of sources including but not limited to ASCE7-05 and manufacturer specification. Design dead load on the build can be found in table 1.2. Also a listing of assumed dead loads can be found id table 1.3, and these lodes will be used throughout this report.

Table 1.2: Design Dead Loads

Location	Design (psf)
Roof	40
Living	55
Private Decks/Balconies	45
Corridors Exit stairs	45
Light Storage	45

Table 1.3: Assumed Dead Load

	Assumed load (psf)
Slab	36*
Joist	5
Supper impose Dead load	15
wall	15

* Slab dead load was calculated using a 3" think slab and 145 pcf for concrete

Snow Load

Due to the location of this build being a snow region, snow loads were calculated in accordance to ASCET-07 section 7. The results of the load calculation can be seen in table 2, with detail calculation and notes can be found in Appendix B.

Table 2: Snow loads

Ground snow load	Pg= 30 psf
Flat roof snow load	Pf= 21 psf
Slop roof snow load	Ps= 21 psf

Wind Load

The wind loads analysis on the build was determined using method 2 of the ASCE7-05 Section 6. The assumption that the building act rigidly was assumed and was confirmed later to be an accurate assumption. The results of the analysis can be seen in the following tables with detail calculations can be found in Appendix C. Figure 10 shows the build base plan and the designated N-S and E-W direction that will be used in the analysis.

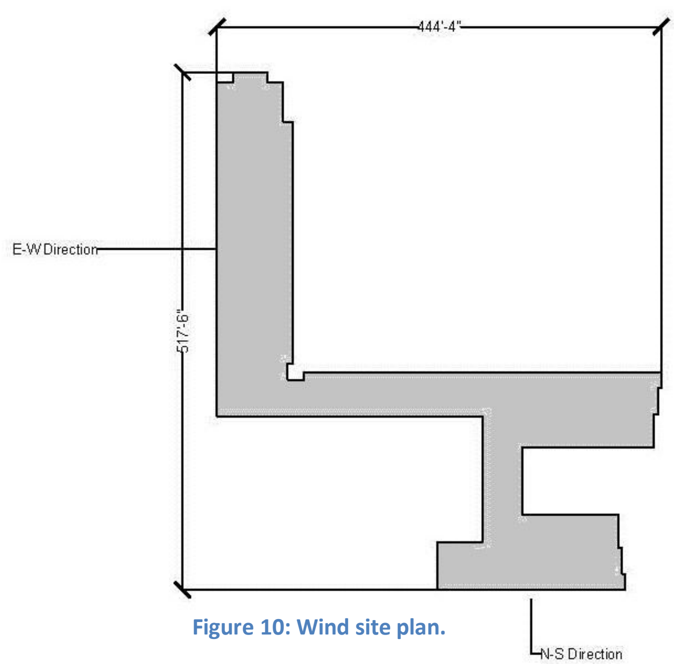


Figure 10: Wind site plan.

Table 3.1a: E-W wind load pressures.

E-W Direction	z	K_z	q_z	Cp	P w/+GC_{pi} (psf)	P w/-GC_{pi} (psf)
Windward	0.00	0.57	10.05	0.80	3.46	8.91
	15.00	0.57	10.05	0.80	3.46	8.91
	20.00	0.62	10.93	0.80	4.01	9.46
	25.00	0.67	11.81	0.80	4.55	10.00
	30.00	0.70	12.34	0.80	4.88	10.32
	35.00	0.73	12.87	0.80	5.20	10.65
	40.00	0.76	13.40	0.80	5.53	10.98
	45.00	0.79	13.92	0.80	5.85	11.30
	50.00	0.81	14.28	0.80	6.07	11.52
	55.00	0.83	14.63	0.80	6.29	11.74
	60.00	0.85	14.98	0.80	6.50	11.95
	62.17	0.86	15.14	0.8	6.60	12.05
Leeward	-	-	15.14	-0.50	-8.52	-3.13
Side	-	-	15.14	-0.70	-10.85	-5.46
Roof						
zone 1	-	-	15.14	-0.90	-13.19	-7.79
	-	-	15.14	-0.18	-4.79	0.60
zone 2	-	-	15.14	-0.90	-13.19	-7.79
	-	-	15.14	-0.18	-4.79	0.60
zone 3	-	-	15.14	-0.50	-8.52	-3.13
	-	-	15.14	-0.18	-4.79	0.60
zone 4	-	-	15.14	-0.30	-6.19	-0.80
	-	-	15.14	-0.18	-4.79	0.60
Parapets	-	0.886	15.62		23.42	-15.62

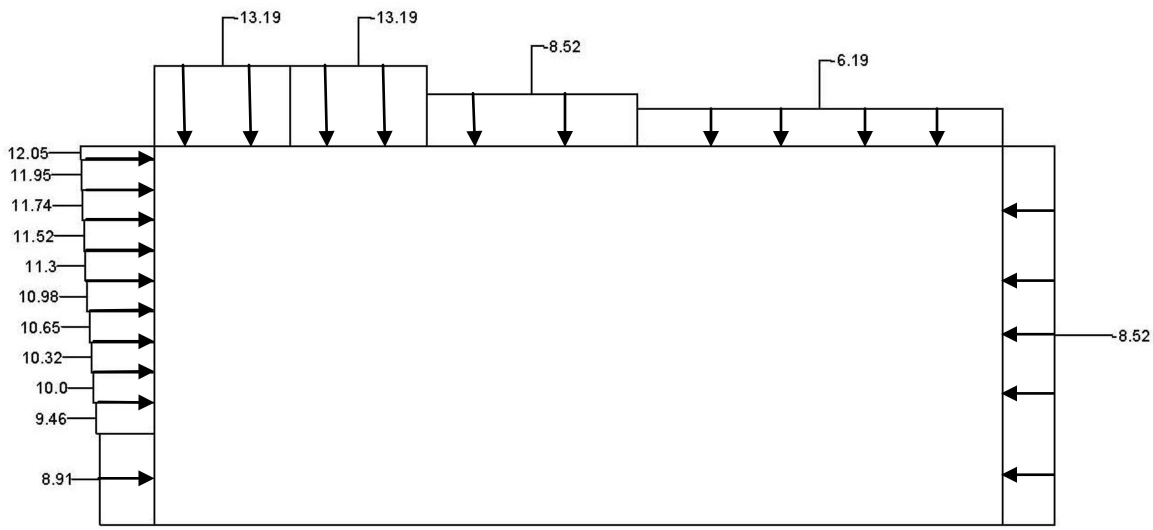


Figure 11.a E-W Pressures in psf.

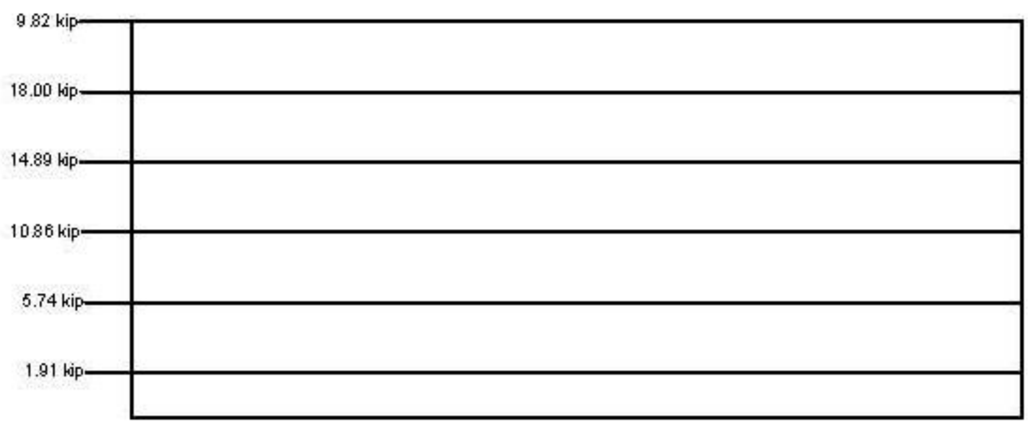


Figure 11.b: E-W Wind Load Story Shear.

Base Shear:

V=61.9 Kip

Over turning moment:

M=2,563.1 Kip-ft

Table 3.2a: N-W Wind load pressers.

N-S Direction	z	K_z	q_z	Cp	P w/+GC_{pi} (psf)	P w/-GC_{pi} (psf)
Windward	0.00	0.57	10.05	0.80	3.46	8.91
	15.00	0.57	10.05	0.80	3.46	8.91
	20.00	0.62	10.93	0.80	4.01	9.46
	25.00	0.67	11.81	0.80	4.55	10.00
	30.00	0.70	12.34	0.80	4.88	10.32
	35.00	0.73	12.87	0.80	5.20	10.65
	40.00	0.76	13.40	0.80	5.53	10.98
	45.00	0.79	13.92	0.80	5.85	11.30
	50.00	0.81	14.28	0.80	6.07	11.52
	55.00	0.83	14.63	0.80	6.29	11.74
	60.00	0.85	14.98	0.80	6.50	11.95
	62.17	0.86	15.14	0.80	6.60	12.05
Leeward	-	-	15.14	-0.47	-8.17	-2.78
Side	-	-	15.14	-0.70	-10.85	-5.46
Roof						
zone 1	-	-	15.14	-0.90	-13.19	-7.79
	-	-	15.14	-0.18	-4.79	0.60
zone 2	-	-	15.14	-0.90	-13.19	-7.79
	-	-	15.14	-0.18	-4.79	0.60
zone 3	-	-	15.14	-0.50	-8.52	-3.13
	-	-	15.14	-0.18	-4.79	0.60
zone 4	-	-	15.14	-0.30	-6.19	-0.80
	-	-	15.14	-0.18	-4.79	0.60
Parapets	-	0.886	15.62		23.42	-15.62

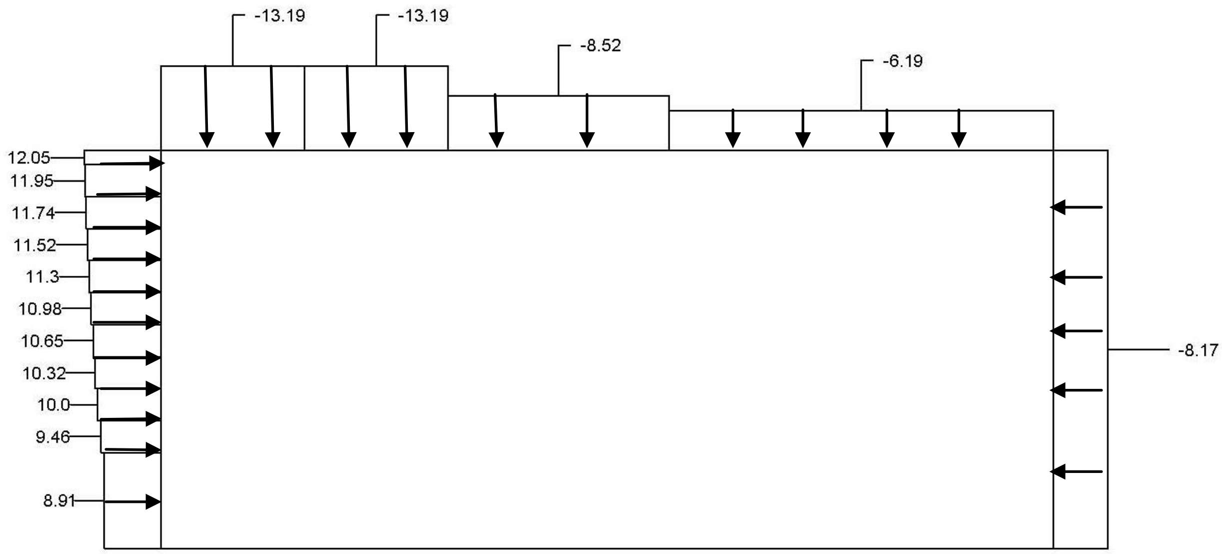


Figure 13.a: N-S Wind load Pressure in psf.

Figure 12.b: N-S Wind Load Story Shear.

Base Shear:

V=62.87Kip

Over turning moment:

M=2,501.0 Kip-ft

Seismic Load

Seismic load was determined using the Equivalent Lateral Force method as described in ASCE7-05 section 11 and 12. A sit class D was recommended from the Geo Technical report and will be used in this analysis. The building weight were assumed and calculated, the values used and the final building weight can be seen in Table 4.1. The results of the analysis can be seen in the following tables and figures with detail information in Appendix D.

Table 4.1: Story weight

	High	Area (sf)	Kips
Gourd	11’	61,709	4391.3
Second	22’	61,709	4391.3
Third	33’	61,709	4391.3
Fourth	44’	61,709	4391.3
Fifth	55’	61,709	4391.3
Roof	67’-8”	61,709	3456.0

Table 4.2: Assumed dead load.

Load	
slab	36 psf
Joist	5 psf
SDL	15 psf
Wall	15 psf
Total	71 psf

Table 4.3: Seismic load.

	Story Height (ft)	h_x	w_x	w_xh_x	C_{vx}	Lateral Force F_x	Story Shear Vx	Moments Mx
Ground	11	11	4391.3	48304.3	0.0504	68.3	1355.3	751.4
Second	11	22	4391.3	96608.6	0.1008	136.6	1287.0	3005.6
Third	11	33	4391.3	144912.9	0.1512	204.9	1150.4	6762.6
Fourth	11	44	4391.3	193217.2	0.2016	273.2	945.5	12022.3
Fifth	11	55	4391.3	241521.5	0.2520	341.5	672.2	18784.9
Roof	12.667	67.667	3456	233857.2	0.2440	330.7	330.7	22377.8
			25412.5	958421.7		1355.3		63704.5

Base Shear:

V=1,355.5 Kip

Over turning moment:

M=63,704.5 Kip-ft

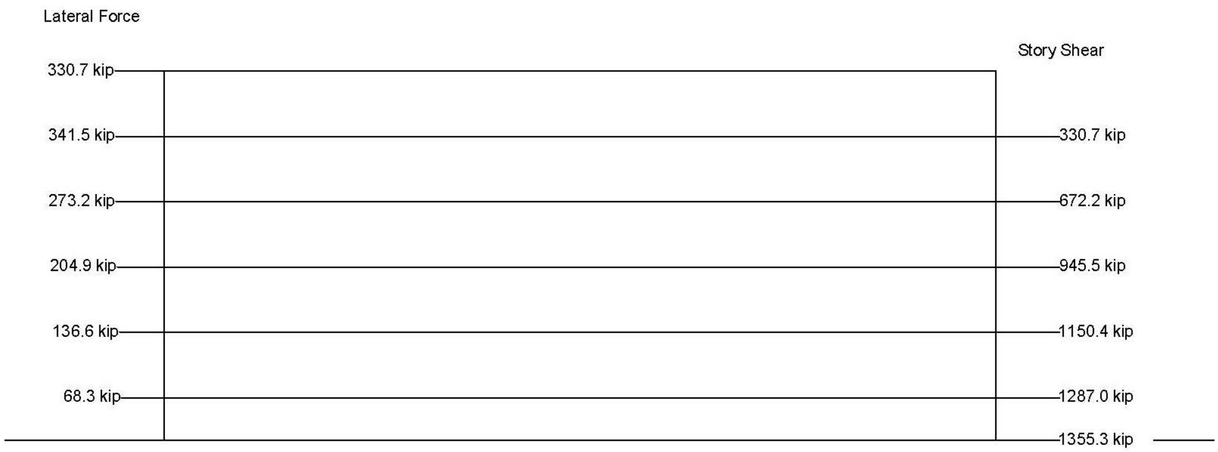


Figure 13: Seismic load diagram.

Spot Checks

A series of spot checks were preformed to check the accuracy of the gravity loads in this report on the structural of the building. Spot checks were preformed on the floor slab, the floor joist, and the load bearing walls. Detail calculations on these spot checks can be located in Appendix E. Figure 14 shows a simple floor plan that will be used for the spot checks.

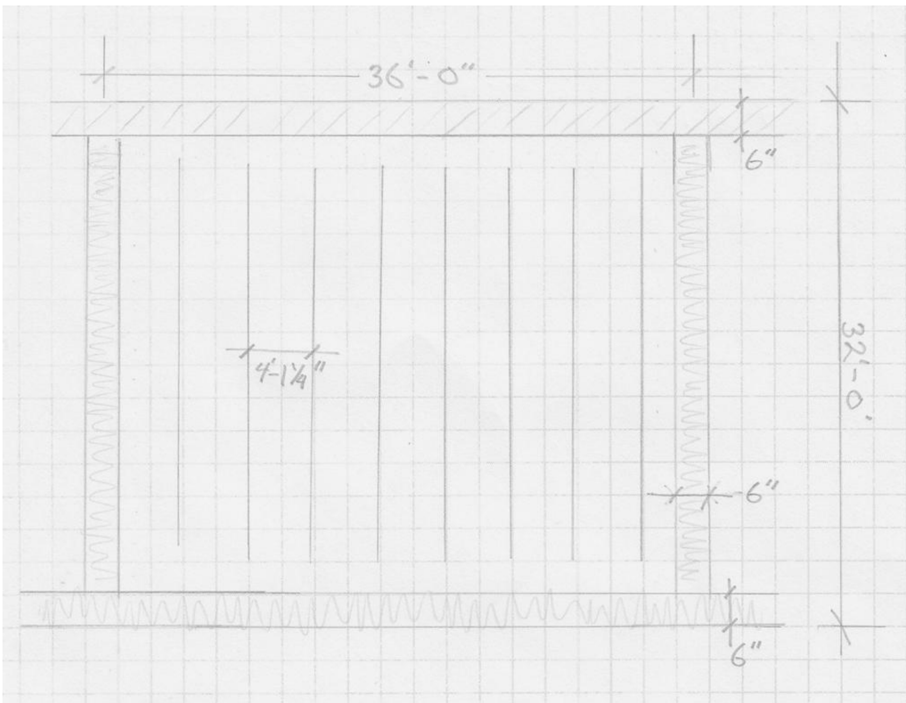


Figure 14: Simple Floor Plan used for spot checks.

The first spot check was conductive on the one way floor slab that is sued in the floor system. The slab strength, shear, and deflection were check and were found to be more than sufficient to carry the gravity load applied on it. However it was found that the slab did not meet the recommended shrinkage and temperature reinforcement in accordance to ACI 318-08 (7.12.2.1), which was 0.0018 reinforcement area to gross concrete area and the slab had a ratio of 0.0016. One reason for this could be the designer experience or possibility an exception in the code.

The next spot check was conductive on the Hambro Joist that is used in the floor system of the building. The 20’ deep bar joist was check for strength and deflection at the construction, non-composite, phase and at the full composite phase, full load case with the slab. There was some difficulty in acquiring the needed information to complete the spot check, but after contacting the manufacture the information was acquired. From the analysis it was found that the bar joist that was pick for the building was more than sufficient to carry the gravity load applied doing the non-composite and composite phase of construction.

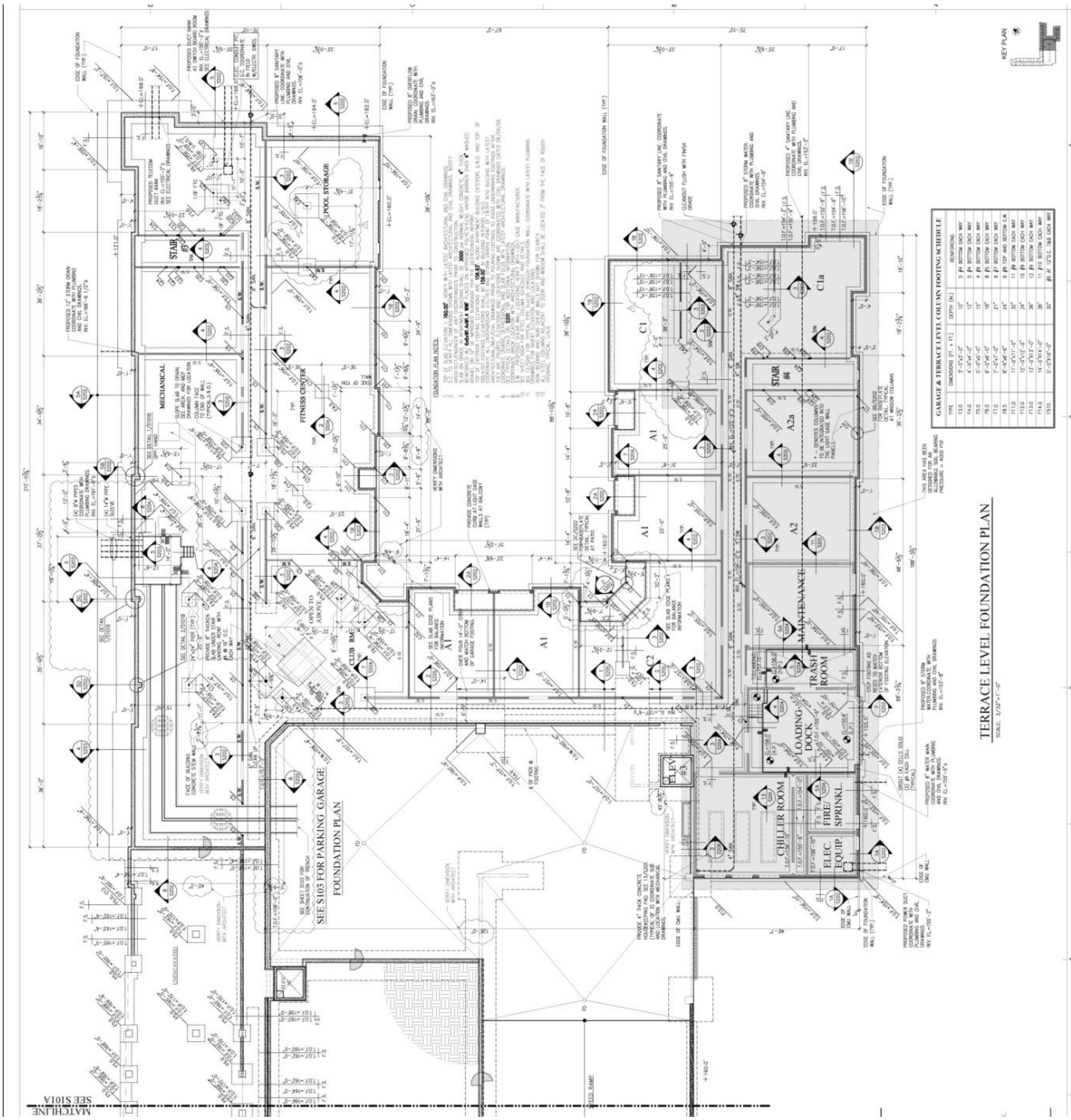
The final spot check was completed on the exterior load bearing 6” light gage steel stud wall for bearing strength. The analysis was performed on a stud located on the terrace level; this location was picked to give it the highest load that the stud would have to carry. From the analysis it was determined that the light gage steel stud have sufficient strength to carry the load applied to it.

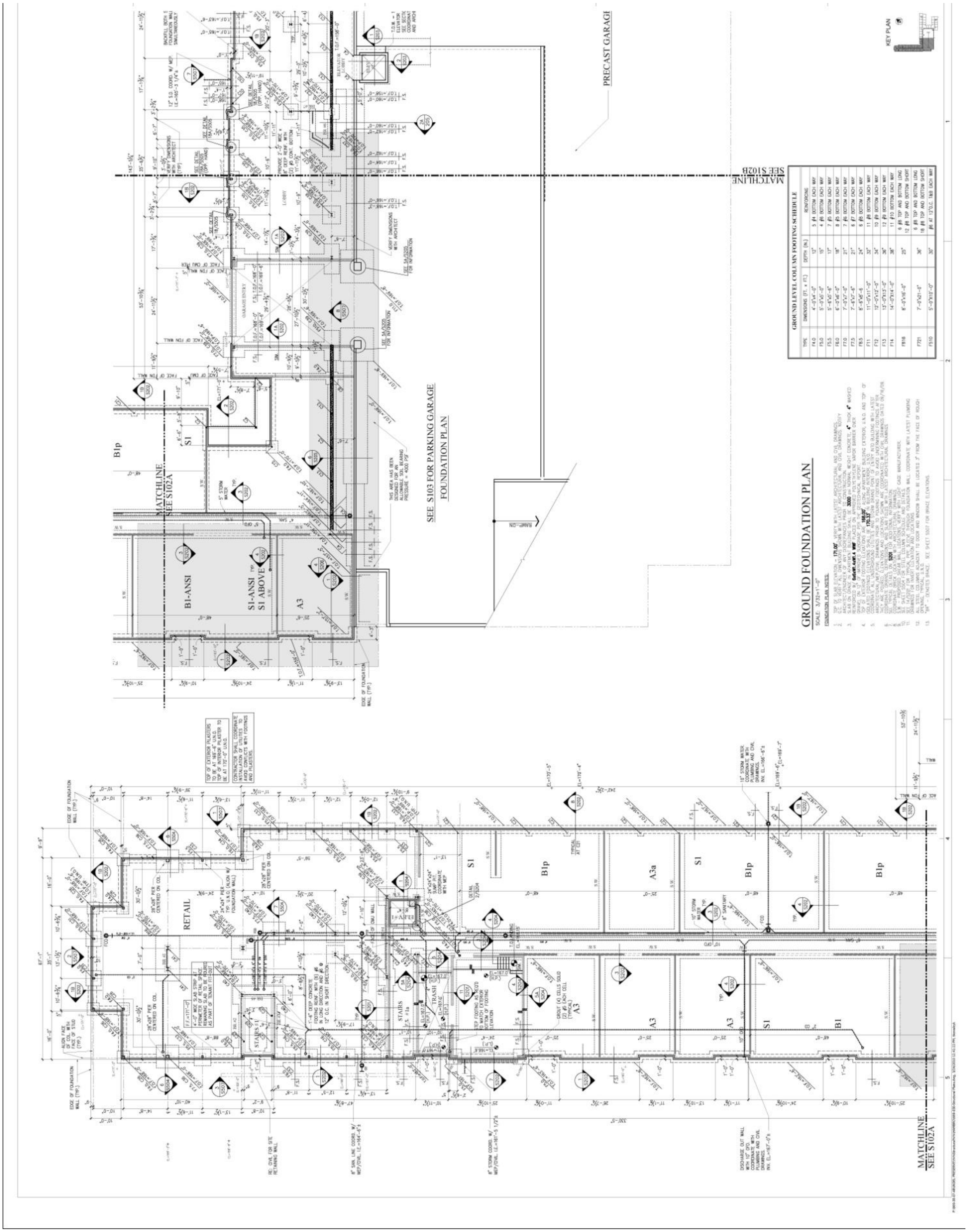
Conclusion

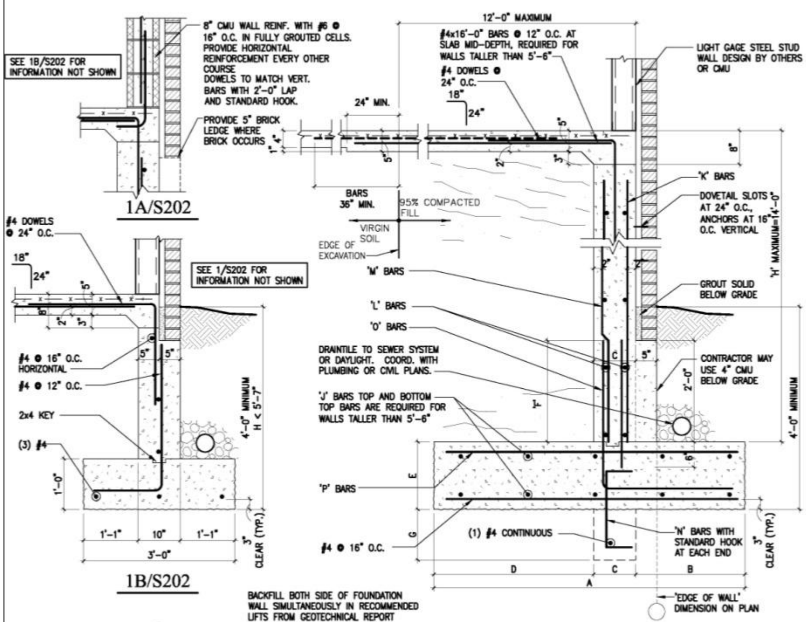
Analysis of the structural system of The Residence and various gravity spot checks confirmed that the structure of the Residence can adequately carry the loads applied on the system. The lateral loads were determined in accordance to ASCE7-05, the lateral wind forces were found by ASCE7-05 section 6 Method 2 analytical procedure and the lateral seismic forces were determined by ASCE7-05 section 11 and 12 equivalent lateral force procedure. It was determined that the seismic forces were the controlling lateral forces on the build with a base shear of 1,355 Kips and an overturning moment of 63,704.5 ft-k. Do to the location of the building a calculation of the snow load was also conductive according to ASCE7-05 section 7. All the loads were compared to the design loads for the build and were found that the loads were almost identical to the minimal load requirements of ASCE7-05. The slight differences could be attributive to the experience of the designer.

Spot checks were performed on the floor slab, the floor bar joist, and the exterior light gage steel load bearing walls to verify that the member sizes were adequate to carry the gravity loads placed on them. The strength and deflections of the structural member were compared and determined to be well under the design criteria. There was no lateral force analysis conducted on the structural in this report but will be address later in technical report 3.

Appendix A: Plans




GROUND FOUNDATION PLAN
SEE S103 FOR PARKING GARAGE FOUNDATION PLAN
MATCHLINE SEE S102B
MATCHLINE SEE S102A
PRECAST GARAGE
GROUND LEVEL COLUMN FOOTING SCHEDULE
RETAIL
B1-ANSI
S1-ANSI S1 ABOVE
A3
B1p
S1
A3a
B1
STAIRS

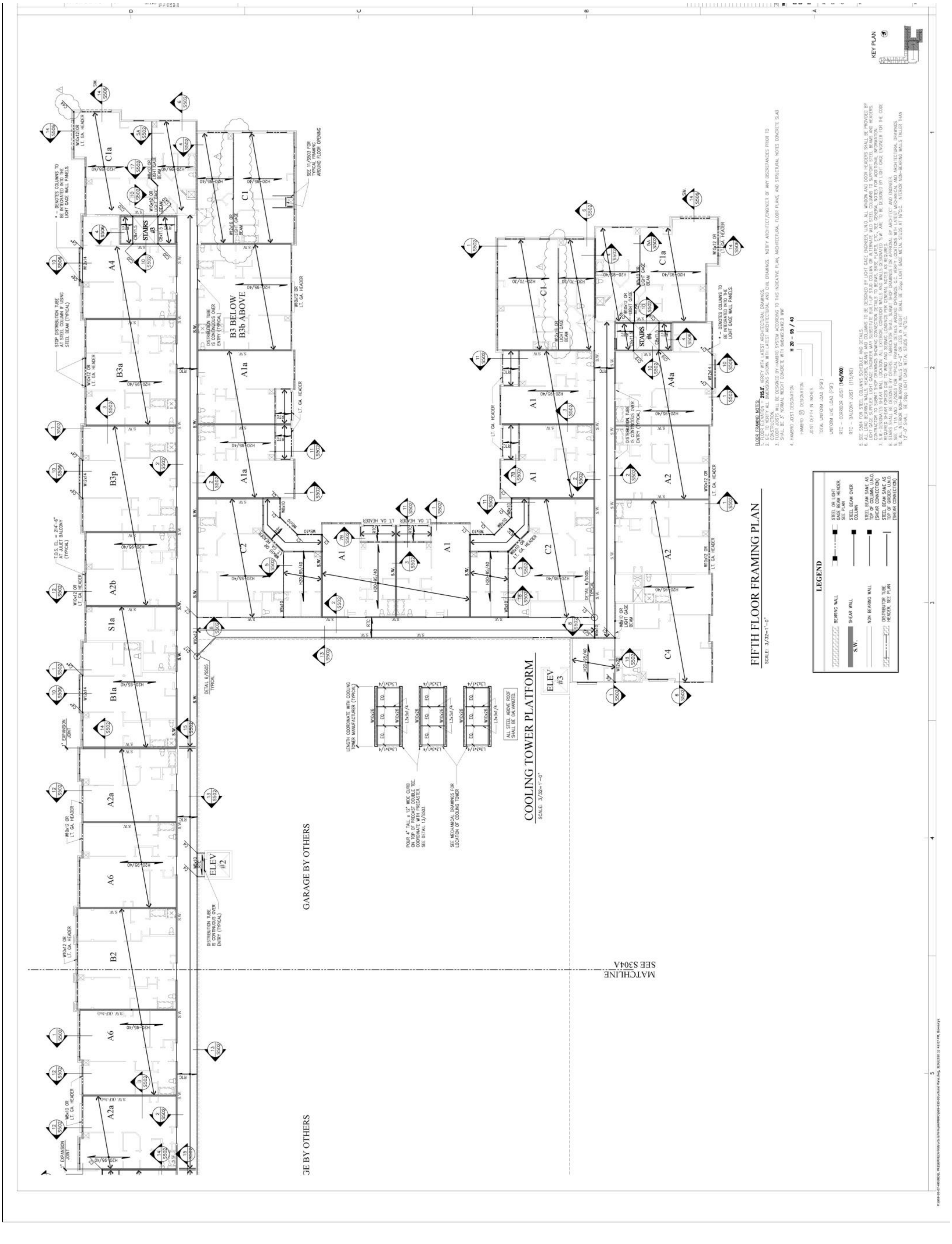


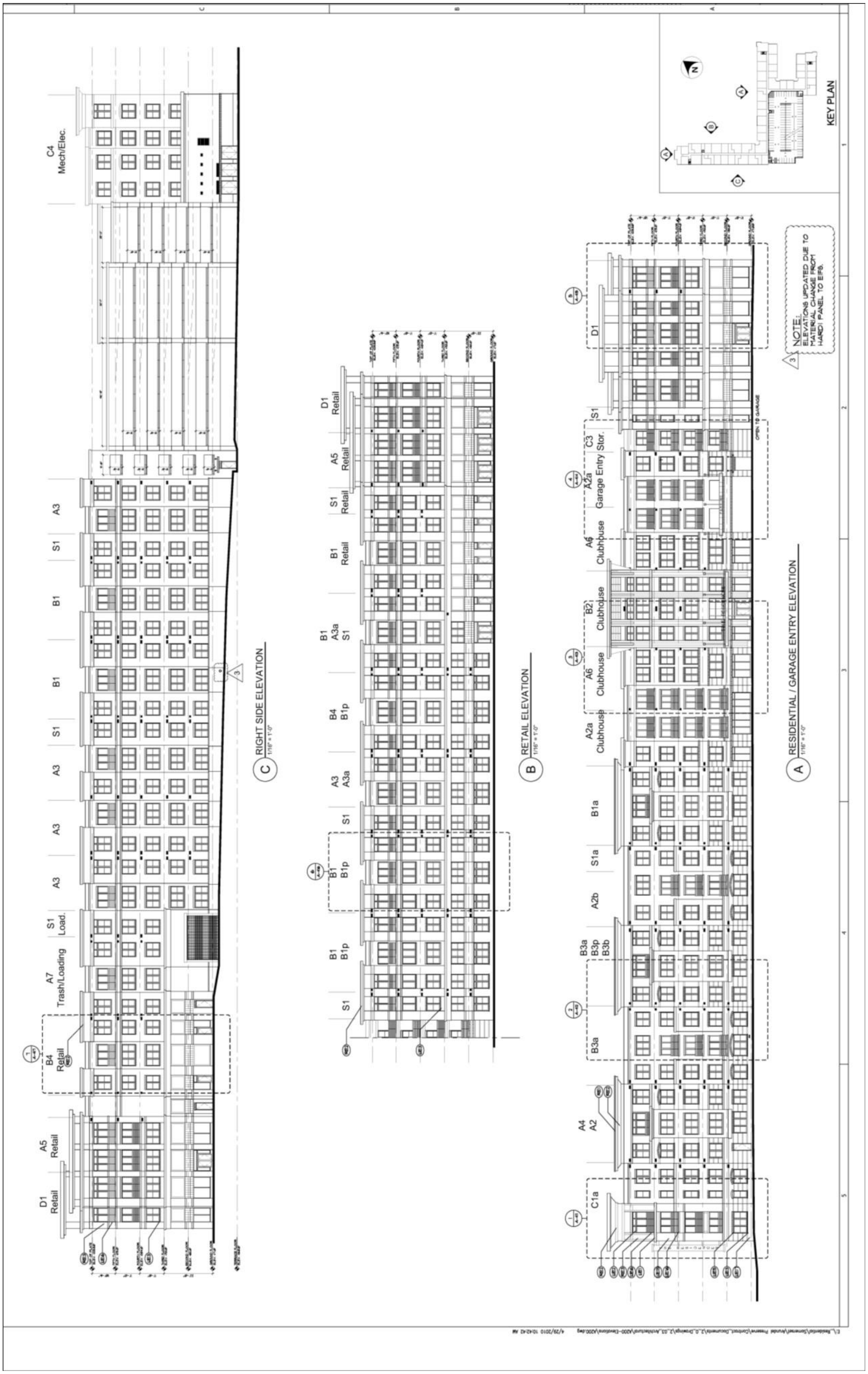
1,1A,1B / S202 TYPICAL EXTERIOR FOOTING

SCALE: 3/4"=1'-0"

EXTERIOR CONCRETE RETAINING WALL (AT-REST PRESSURE = 65 PCF) DETAIL 1,1A/S202

WALL HEIGHT	DIMENSIONS						
H	A	B	C	D	E	F	G
5'-7" TO 9'-0"	6'-0"	2'-0"	10"	3'-2"	1'-0"	3'-6"	–
9'-1" TO 14'-0"	8'-3"	2'-6"	1'-2"	4'-7"	1'-2"	4'-9"	1'-6"
WALL HEIGHT	**REINFORCING**						
H	J	K	L	M	N	O	P
5'-7" TO 9'-0"	4 @ 16"	#4 @ 9"	#4 @ 16"	#4 @ 18"	–	#5 @ 9"	#5 @ 14"
9'-1" TO 14'-0"	4 @ 16"	#5 @ 8"	#4 @ 16"	#5 @ 12"	#4 @ 12"	#6 @ 6"	#6 @ 12"


FIFTH FLOOR FRAMING PLAN
SCALE: 3/32"=1'-0"
COOLING TOWER PLATFORM
SCALE: 3/32"=1'-0"
GARAGE BY OTHERS
LEGEND
KEY PLAN
MATCHLINE
SEE S304A
ELEV #2
ELEV #3
STAIRS #3
STAIRS #4


RIGHT SIDE ELEVATION
RETAIL ELEVATION
RESIDENTIAL / GARAGE ENTRY ELEVATION
KEY PLAN
C4 Mech/Elec.
A3
S1
B1
A7 Trash/Loading
B4 Retail
A5 Retail
D1 Retail
B1 Retail
S1 Retail
B1 B1p
B4 B1p
A3 A3a
B1 A3a S1
C1a
A4 A2
B3a
B3a B3p B3b
A2b
S1a
B1a
A2a Clubhouse
A6 Clubhouse
B2 Clubhouse
A2a Garage Entry
C3 Stor.
NOTE:
ELEVATIONS UPDATED DUE TO MATERIAL CHANGE FROM HARDI PANEL TO EIFS.

Appendix B: Snow Load Analysis

ASCE7-05 Section 7

(7.2) Ground snow load

P_g= 30 psf

(7.3) Flat Roof

P_f= 0.7 C_e C_t I P_g

(7.3.1) Exposure Factor

Table 7-2

C_e = 0.9

(7.3.2) Thermal Factor

C_t = 1.1

(7.3.3) Importance Factor

I = 1.0

P_f= 0.7(0.9)(1.1)(1.0)(30)= 20.79 -> 21 psf

(7.4) Slop Roof

P_s=C_s P_f

C_s =1.0

P_s = 21 psf

Appendix C: Wind Load Analysis

ASCE7-05 section 6: method 2

(6.5.4) Basic wind speed

Fig 6-1, V=90 mph

(6.5.5) Importance factor

Occupancy category: II

I=1.00

(6.5.6) Exposure category

Exposure category: B

(Table 6-3) Kz (B case 2)

z	K_z
0.00	0.57
15.00	0.57
20.00	0.62
25.00	0.67
30.00	0.70
35.00	0.73
40.00	0.76
45.00	0.79
50.00	0.81
55.00	0.83
60.00	0.85
62.17	0.86

(6.5.7) Top Factor

Kzt = 1.0

(6.5.8) Gust effect

n_1=75/h=75/62'-2"=1.2 > 1

n_1=100/h=100/62'-2"= 1.61>1

- Structure is ridge.

$$G=0.925\left(\frac{(1+1.7\,g_Q I_z Q)}{1+1.7\,g_v I_z}\right)$$

$$I_z = c\left(\frac{33}{z}\right)^{1/6}$$

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

$$L_z = l\left(\frac{z}{33}\right)^{\epsilon}$$

Z	37.32	
z min	30	
c	0.3	
Ɛ	0.333333	
l	320	
gq	3.4	
gv	3.4	
Iz	0.293912	
Lz	333.3951	
	E-W	N-S
Q	0.727009	0.741373
G	0.77	0.77

(6.5.9) Enclosure classification

Enclosed building

(6.5.11.1) Internal pressure coefficient

$GC_{pi} = \pm 0.18$

(6.5.11.2) External Pressure coefficients Cp

	E-W	N-S
L/B	0.8586	1.1647
H/L	0.135	0.1159

See table for values.

(6.5.10)

$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I$

$K_d = 0.85$

(6.5.12)

$P = q\ G\ C - q_i\ (GC_{pi})$

(6.5.12.2.4) Parapets

$Pp = q_p\ GC_{pn}$

h=69 ft

GC_{pn}= +1.5 Windward

-1.0 leeward

EW	
k_{zt}	1.00
K_d	0.85
v	90
I	1.00
GCpi	0.18
G	0.77

	z	K_z	q_z	Cp	P w/+GC_{pi}	P w/-GC_{pi}
Windward	0.00	0.57	10.05	0.80	3.46	8.91
	15.00	0.57	10.05	0.80	3.46	8.91
	20.00	0.62	10.93	0.80	4.01	9.46
	25.00	0.67	11.81	0.80	4.55	10.00
	30.00	0.70	12.34	0.80	4.88	10.32
	35.00	0.73	12.87	0.80	5.20	10.65
	40.00	0.76	13.40	0.80	5.53	10.98
	45.00	0.79	13.92	0.80	5.85	11.30
	50.00	0.81	14.28	0.80	6.07	11.52
	55.00	0.83	14.63	0.80	6.29	11.74
	60.00	0.85	14.98	0.80	6.50	11.95
	62.17	0.86	15.14	0.8	6.60	12.05
Leeward	-	-	15.14	-0.50	-8.52	-3.13
Side	-	-	15.14	-0.70	-10.85	-5.46

Roof						
zone 1	-	-	15.14	-0.90	-13.19	-7.79
	-	-	15.14	-0.18	-4.79	0.60
zone 2	-	-	15.14	-0.90	-13.19	-7.79
	-	-	15.14	-0.18	-4.79	0.60
zone 3	-	-	15.14	-0.50	-8.52	-3.13
	-	-	15.14	-0.18	-4.79	0.60
zone 4	-	-	15.14	-0.30	-6.19	-0.80
	-	-	15.14	-0.18	-4.79	0.60

Parapets		
Kz	0.886	
q	15.62	
GCpn	1.50	
	-1.00	
P wind	23.42	(psf)
P Lee	-15.62	(psf)

NS	
k_{zt}	1.00
K_d	0.85
v	90
I	1.00
GCpi	0.18
G	0.77

	z	K_z	q_z	Cp	P w/+GC_{pi}	P w/-GC_{pi}
Windward	0.00	0.57	10.05	0.80	3.46	8.91
	15.00	0.57	10.05	0.80	3.46	8.91
	20.00	0.62	10.93	0.80	4.01	9.46
	25.00	0.67	11.81	0.80	4.55	10.00
	30.00	0.70	12.34	0.80	4.88	10.32
	35.00	0.73	12.87	0.80	5.20	10.65
	40.00	0.76	13.40	0.80	5.53	10.98
	45.00	0.79	13.92	0.80	5.85	11.30
	50.00	0.81	14.28	0.80	6.07	11.52
	55.00	0.83	14.63	0.80	6.29	11.74
	60.00	0.85	14.98	0.80	6.50	11.95
	62.17	0.86	15.14	0.80	6.60	12.05
Leeward	-	-	15.14	-0.47	-8.17	-2.78
Side	-	-	15.14	-0.70	-10.85	-5.46

Roof						
zone 1	-	-	15.14	-0.90	-13.19	-7.79
	-	-	15.14	-0.18	-4.79	0.60
zone 2	-	-	15.14	-0.90	-13.19	-7.79
	-	-	15.14	-0.18	-4.79	0.60
zone 3	-	-	15.14	-0.50	-8.52	-3.13
	-	-	15.14	-0.18	-4.79	0.60
zone 4	-	-	15.14	-0.30	-6.19	-0.80
	-	-	15.14	-0.18	-4.79	0.60

Parapets		
Kz	0.886	
q	15.62	
GCpn	1.50	
	-1.00	
P wind	23.42	(psf)
P Lee	-15.62	(psf)

Appendix D: Seismic Load Analysis

ASCE7-05 Seismic Equivalent lateral force procedure

Seismic importance factor: 1.0

Seismic Occupancy Category: II

Site Class D (from Geotechnical Report)

(11.4) Seismic Ground Motion Values

$Ss= 0.2g$ $S_{ms}= F_a\ S_s$ $S_{ds}=(2/3)S_{ms}$

$S1=0.06g$ $S_{m1}=F_v\ S_1$ $S_{d1}=(2/3)S_{m1}$

Table 11.4-1

Fa=1.6

Table 11.4-2

Fv=2.4

(12.8)

Base shear V=Cs W

$Cs=S_{ds}/(R/I) = S_{ds}/R$

R for Light Framed walls with system using flat straps bracing: R=4

Cs= 0.0533

(12.8.2) Fundamental period

$Ta=C_t\ h_n^{\ x} = 0.4312$

$Ct=0.02$ $x=.75$ $h_n=67.67$

(12.8.3) Distribution of seismic forces

$F_x=C_{vx}\ V$

$Cvx=\frac{w_x h_x{}^k}{\sum w_i h_i{}^k}$

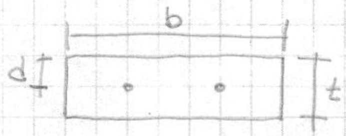
Ta=0.4312 < 0.5: K=1.0

SIF	1
SOC	II
Site Class	D
Ss	0.200
S1	0.060
Sms	0.320
Sma	0.144
Sds	0.213
Sd1	0.096
SDC	B
R	4
Cs	0.053
Ct	0.02
hn	67.667
x	0.75
Ta	0.472
V	1355.333

	Story High (ft)	h_x	w_x	$w_x h_x$	C_{vx}	Lateral Force F_x	Story Shear Vx	Moments Mx
Ground	11	11	4391.3	48304.3	0.0504	68.3	1355.3	751.4
Second	11	22	4391.3	96608.6	0.1008	136.6	1287.0	3005.6
Third	11	33	4391.3	144912.9	0.1512	204.9	1150.4	6762.6
Fourth	11	44	4391.3	193217.2	0.2016	273.2	945.5	12022.3
Fifth	11	55	4391.3	241521.5	0.2520	341.5	672.2	18784.9
Roof	12.667	67.667	3456	233857.2	0.2440	330.7	330.7	22377.8
Total			25412.5	958421.7		1355.3		63704.5

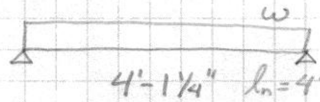
Appendix E: Spot Checks

Floor slab spot check



$b = 12''$
$t = 3''$
$f'_c = 3{,}000\,psi$
$d = 2.1''$

$f_s = 60{,}000\,psi$
6×6 W2.9×W2.9 WWF
$A_{1wire} = .029\,in^2$
$A = .058\,in^2/ft$ $D = .192$
Cover 3/4"



Moment

$\phi M_n = A_s f_y (d - a/2)\,\phi$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{.058\,(60{,}000)}{0.85(3{,}000)(12'')} = .1137$

$d - a/2 = 2.1 - \frac{.1137}{2} = 2.043$

$\phi M_n = 0.9(.058)(60{,}000)(2.043) = 6399\ lb\text{-}in$

$533\ lb\text{-}ft/ft \geq 250.9$ ok

$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c)$

$c = a/\beta_1 \quad \beta_1 = 0.85$

$c = .134$

$\varepsilon_u = .003 \quad \varepsilon_s = 0.0441 \geq .005$ ok.

$\rho = A_s/bd = .0023 \geq .0018$ ok

Shear

$V_c = 2\lambda\sqrt{f'_c}\,b_w d$

$\lambda = 1.0$ NW

$2\sqrt{3{,}000}(12'')(2.1'') = 2760\ lb$

$\phi V_n = .75 V_c = 2070\ lb \geq 250.8\ lb$

M: $w = $ SW = 36.2 lb/ft; SDL = 15 psf → 15 lb/ft; LL = 40 psf → 40 lb/ft

125.4 plf

Ass Simply Suporting.

$M_u = \frac{wL^2}{8} = \frac{(125.4\,plf)(4)^2}{8}$

$= 250.9\ lb/ft$

S: $V_{max} = \frac{wL}{2} \quad \frac{125.4(4.\ldots)}{2} = 250.8\ lb$

$\Delta_{max} \quad \frac{5wL^4}{384 E_c I_c}$

$I_c = \frac{bh^3}{12} = \frac{12(3)^3}{12} = 27\,in^4$

$E_c = 57\sqrt{f'_c} = 3122\ ksi$

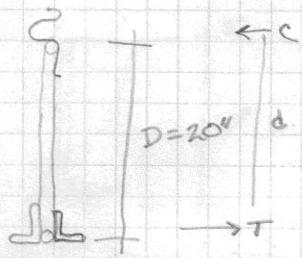
$\frac{5\,(125.4)(4)^4\,\frac{1728}{1000}}{384\,E_c I_e} = .0086$

$L/360 = \frac{(4)(12)}{360} = .1333 > .0086$ ok

ME-01

Floor joist spot check

Construction phase (non-composite)



$M_n = A_s f_y d$

$A_s = .56 \text{ in}^2$

$f_y = 60 \text{ ksi}$

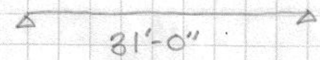
$d = D + .08 - .6970$

19.38

651.17 k-in

54.27 k-ft

$\phi M_n = 0.9(54.27) = 48.8 \geq 40.4 \text{ k-ft}$ ok



$w = \begin{cases} LL & 20 \text{ psf} & & 20 \\ DL & 36 \text{ psf} & \text{slab} & \\ & 5 \text{ psf} & \text{sw} & 41 \end{cases}$

$1.2D + 1.6L = 81.2 \text{ psf}$

$w = 81.2(4.1458)$

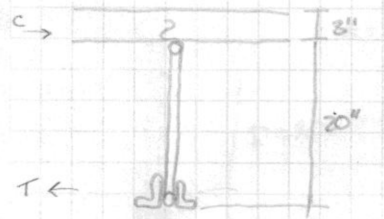
$= 336.6 \text{ plf}$

$M_u = \frac{wL^2}{8} = \frac{336.6(31)^2}{8}$

$= 40.4 \text{ k-ft}$

ME-01

Composite



$A_s = 1.33\ in^2$

$F_y = 50{,}000$ psi

$f'_c = 3{,}000$ psi

$b_{eff} = \begin{cases} 31'(12)/8 = 46.5" \\ 1/2(4.1958)(12) = \boxed{24.9"} \end{cases}$

$d = 20 - .697 = 19.3$

31'

$w = \begin{cases} LL = 40\ psf \\ DL = 36\ psf\ Slab \\ \quad 4\ psf\ Joist \\ \quad 15\ psf\ SDL \end{cases}$

$1.2D + 1.6L = 130\ psf$

$w = 130\ psf\ (4.1458)$

$w = 539\ PLF$

$M_u = \frac{wL^2}{8} = \frac{(539)(31)^2}{8} = 64.7\ k\text{-}ft$

$L/360 = \frac{31(12)}{360} = 1.033$

$a = \frac{A_s f_y}{0.85 f'_c b_{eff}}$

$M_n = A_s f_y (d + t - a/2)$

$a = \frac{1.33(50)}{0.85(3)(24.9)} = 1.0473 \leq 3$

$M_n = 1.33(50)\left(19.3 + 3 - \frac{1.047}{2}\right)$

$= 1448\ k\text{-}in \rightarrow 120.6\ k\text{-}ft$

$\phi M_n = 0.9(120.6) = 108.5 \geq 64.7\ k\text{-}ft$

$I = \frac{bh^3}{12} + bh(a/2 - \bar{y})^2 + (n-1)A_s(d - \bar{y})^2 \qquad n = \frac{29000}{57\sqrt{3{,}000}} = \frac{29000}{3122} = 9.29$

$\frac{(24.9)(3)^3}{12} + 24.9(3)\left(\frac{1.047}{2} - 4.06\right)^2 + (9.29-1)1.33(3.37) = 1027$

$\frac{5(539)(31)^4(1728)}{384(29000)(1027)(1000)} = .3764$

ME-01

Load bearing wall spot check

Roof

w = DL 36 Slap, 5 Joist; LL 20; Snow 21

$1.2(41) + 1.6(21) + 20 = 101.6$ psf

$16'(101.6) = 1625.6$ plf

floor

w = DL 36 Slab, 5 Joist, 15 SDL; LL 40

$1.2(56) + 1.6(40) = 131.2$ psf

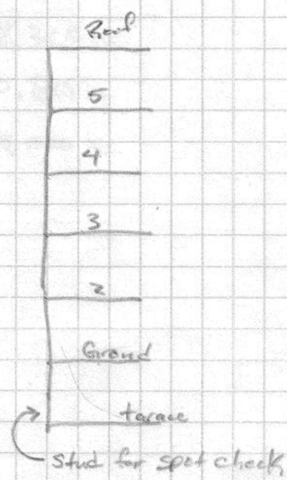
$16(131.6) = 2105.6$ plf

Wall

Ass 15 psf $1.2(15) = 18.0$ psf

$11'(18) = 198$ plf

floor + wall 2303.6 plf



1625.6 plf	Roof
2303.6 plf	5
2303.6	4
2303.6	3
2303.6	2
2303.6	Grand
198	trace

13341.6 plf → 13.34 klf

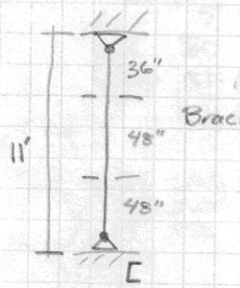
Ass stud are place 16" oc

$13.34 \left(\frac{16}{12}\right) = 17.79^{k}/16"$

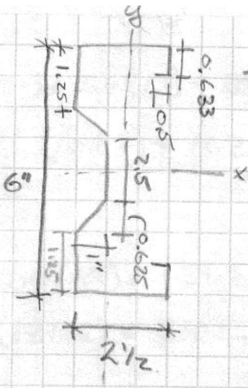
$P_u = 17.79^{k}$

ME-01

6" × 2½" stud



Bracing @ 48" oc



$t = 0.0713$ in (54 mils)

600S250-54

$A = 0.772\ in^2$

$I_x = 4.183\ in^4$

$I_y = 0.513\ in^4$

$F_y = 50$ Ksi

$r_x = 2.328$ in

$r_y = 0.815$ in

$E = 29{,}000$ ksi

$L = 11' \rightarrow 132$ in

$\phi P_n = F_{cr} A_g (0.9)$

$K_y = \frac{48}{132} = .3636 \quad K_x = 1.0$

$\frac{KL}{r} = \frac{.3636(132)}{0.815} = 58.9 \qquad \frac{1(132)}{2.328} = 56.7$ ← Controlling

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad \frac{\pi^2(29000)}{56.7^2} = 89.0 > 0.44 F_y = 22$

$F_{cr} = \left[0.658^{F_y/F_e}\right] F_y$

$\left[0.658^{50/89}\right] 50 = 39.5$

$\phi P_n = 39.5(0.772)(0.9) = 27.4^K \geq 17.79^K$

ME-01