

• • **T** • • Mobile •

15 Commerce Way
Suite B
Norton, MA 02766

STRUCTURAL ANALYSIS
44PB1204B - DEERING AVE LAW LIBRARY



Address:
232 DEERING AVE
PORTLAND, ME 04102

Date:
28 NOVEMBER 2014

A handwritten signature in blue ink is written over a circular professional engineer seal. The seal contains the text: STATE OF MAINE, CLEMENT J. SALEK, No. 11793, LICENSED PROFESSIONAL ENGINEER.

**CHAPPELL
ENGINEERING
ASSOCIATES, LLC**
Civil • Structural • Land Surveying

December 01, 2014

••T••Mobile•

1 International Blvd
Suite 800
Mahwah, NJ 07495

RE:

Candidate Number	4PB1204B
Candidate Name	Deering Ave Law Library
Candidate Address	232 Deering Ave, Portland, ME 04102

To whom it may concern:

Chappell Engineering Associates, LLC has performed a structural analysis of the proposed roof-mounted equipment frame and antenna frame at the above-referenced location. T-Mobile proposes to install an elevated steel frame on the existing roof curbs which will support the proposed T-Mobile equipment cabinets. A single three-sided antenna frame is being proposed to support the proposed T-Mobile antennas.


The existing roof deck consists of a post-tensioned concrete deck laid over open a column and beam-strip arrangement. The proposed equipment frame is to be located on the existing concrete roof curbs. T-Mobile's antenna array will be located on a proposed roof mounted triangular antenna frame anchored to the existing mechanical equipment roof framing members. The antenna frame will support a total of 9 antennas (3 per sectors, total of 3 sectors). The connection details for the antenna frame and the equipment frame is included in our construction drawings.

Based upon our site walk on 06-18-2014, our review of the loads and of the existing available building plans, and our analysis of the existing roof members under the proposed loading, Chappell Engineering Associates, LLC has determined that the existing structure **has adequate capacity** to support the proposed antenna configuration as shown on the construction drawings. Our analysis and results are enclosed in this report.

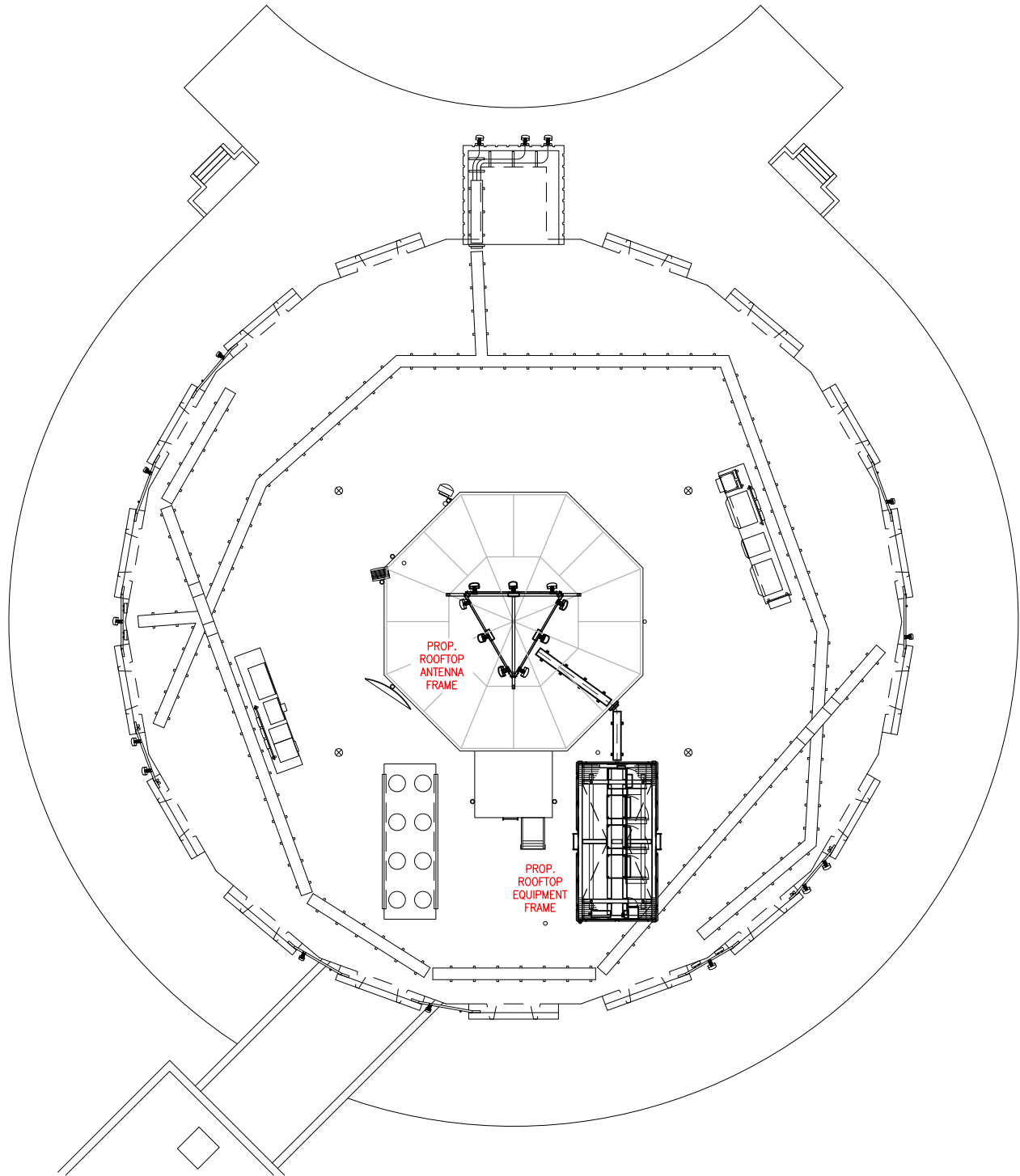
If you have any questions regarding this matter, please do not hesitate to call.

Very truly yours,

CHAPPELL ENGINEERING ASSOCIATES, LLC


Clement J Salek, P.E.
CJS/cjs



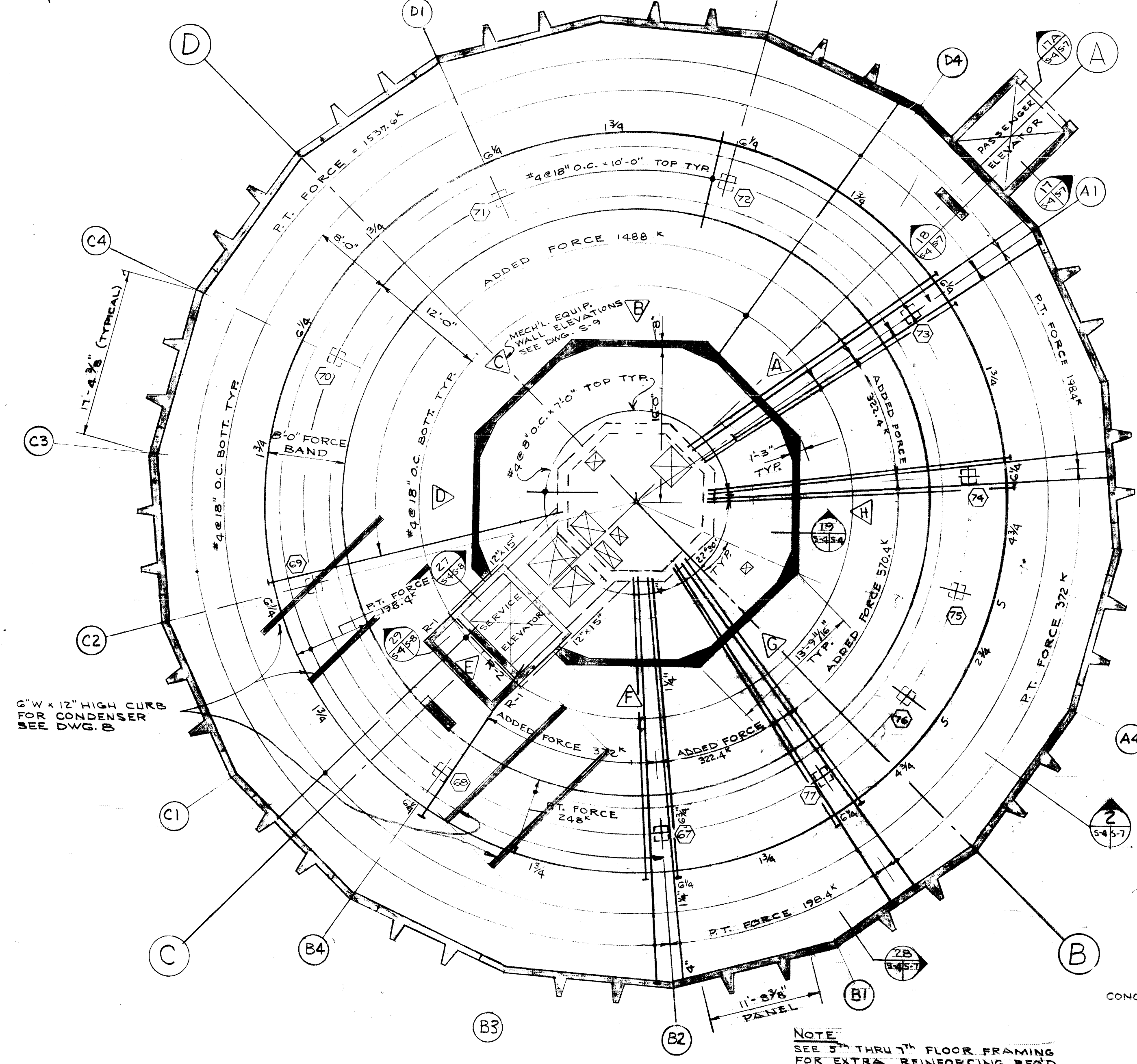


ROOF PLAN

SCALE: 1"=20'



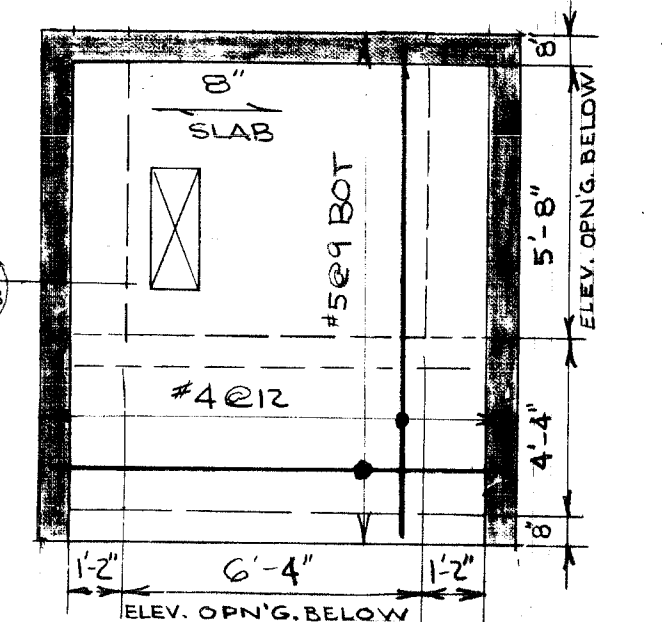
SECTION 19
SCALE 3/4" = 1'-0"



ROOF FRAMING PLAN
SCALE 1/8" = 1'-0"
DESIGN L = 40 P.S.F. OUTSIDE MECH. EQUIP. RM.
150 P.S.F. INSIDE MECH. EQUIP. RM.
ROOF ELEV. 138.00'

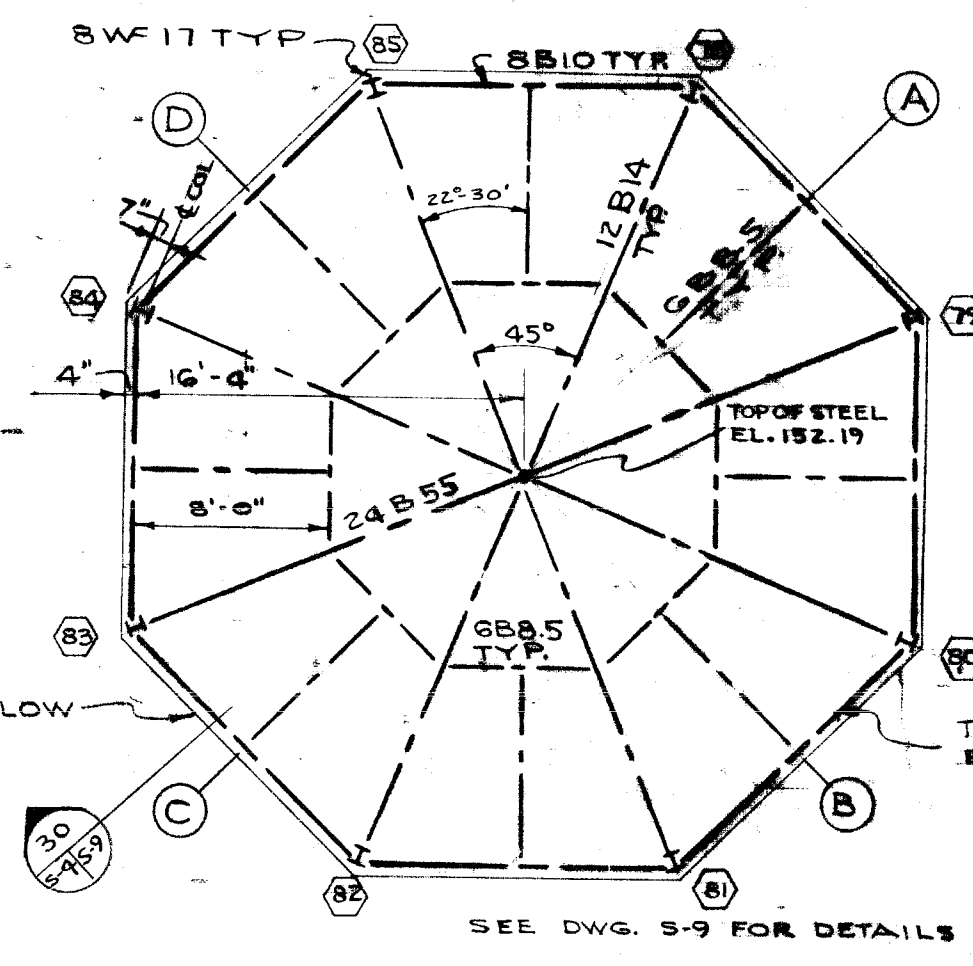
NOTE:
SEE 5TH THRU 7TH FLOOR FRAMING
FOR EXTRA REINFORCING REQ'D.
OVER EACH COLUMN.
SEE ARCH'L. FOR SLOPING
LIGHT WEIGHT CONC. FILL.

ROOF FRAMING
DESIGN L = 40 P.S.F.
TOP OF SLAB EL. 150.5'

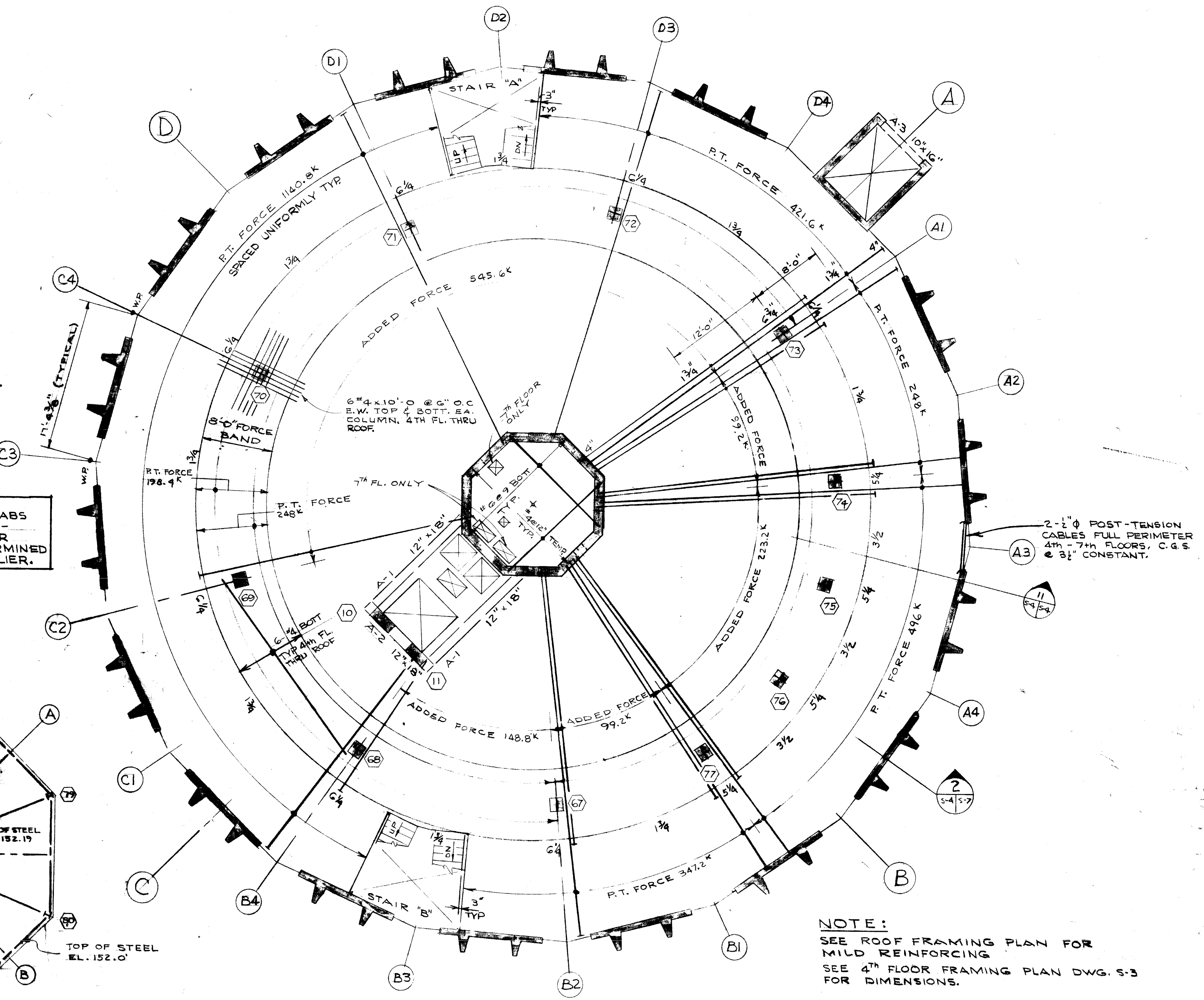


MACHINE ROOM FRAMING
DESIGN L = 150 P.S.F.
SERVICE ELEVATOR
SCALE 1/4" = 1'-0"

NOTE:
STRESSING POCKETS IN SLABS
WILL BE REQUIRED FOR POST-
TENSIONED STRANDS, NUMBER
AND LOCATION TO BE DETERMINED
BY POST-TENSIONING SUPPLIER.

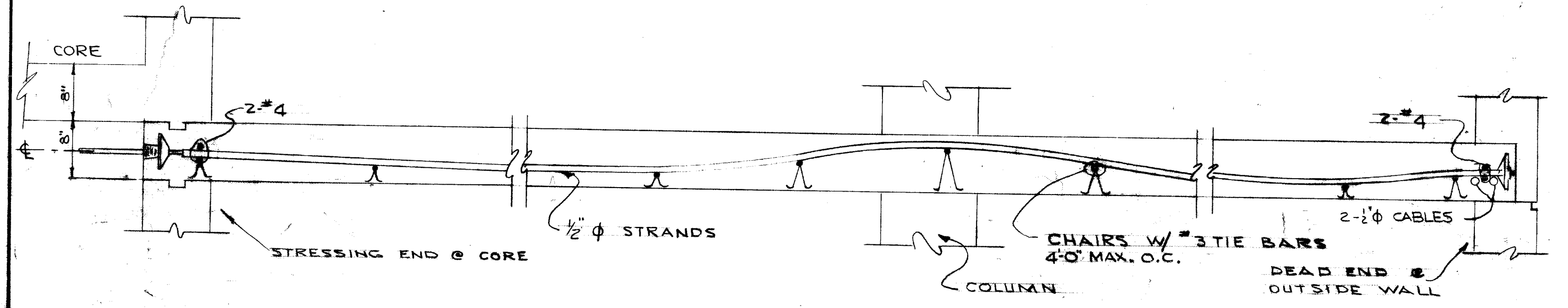


MECH. EQUIP. ROOM
ROOF FRAMING PLAN
SCALE 1/8" = 1'-0"
DESIGN L = 40 P.S.F.

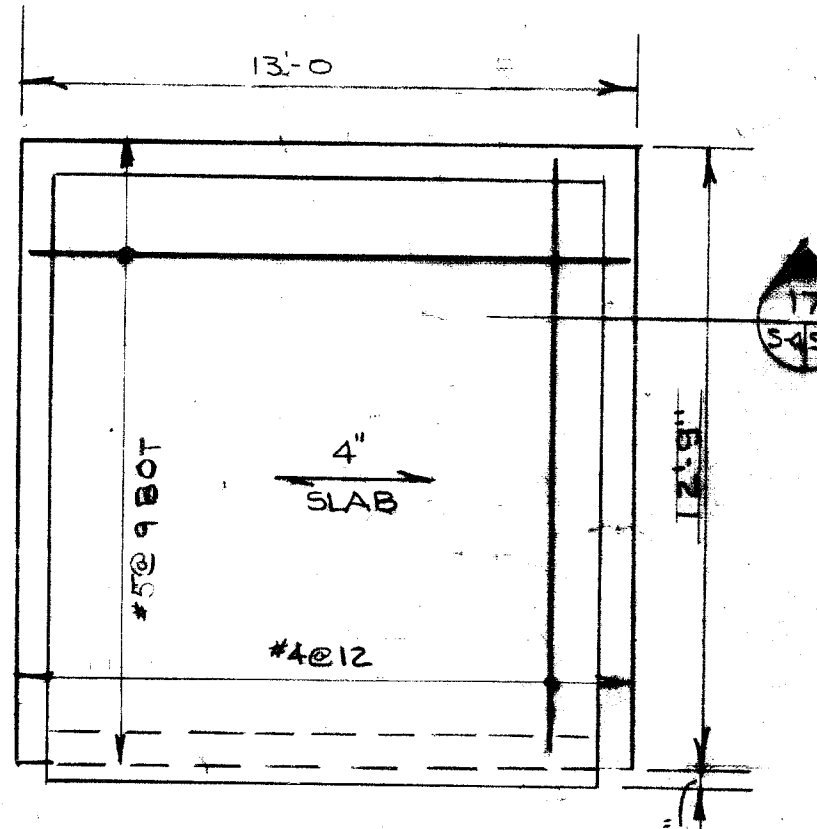


5TH THRU 7TH FLOOR FRAMING PLAN
SCALE 1/8" = 1'-0"
FIN. FIFTH FLOOR ELEV. 106.00'
FIN. CORE FLOOR ELEV. 106.67'
FIN. SIXTH FLOOR ELEV. 116.00'
FIN. CORE FLOOR ELEV. 116.67'
FIN. SEVENTH FLOOR ELEV. 126.00'
FIN. CORE FLOOR ELEV. 126.67'
DESIGN L = 80 P.S.F.
PARTITIONS 20 P.S.F.

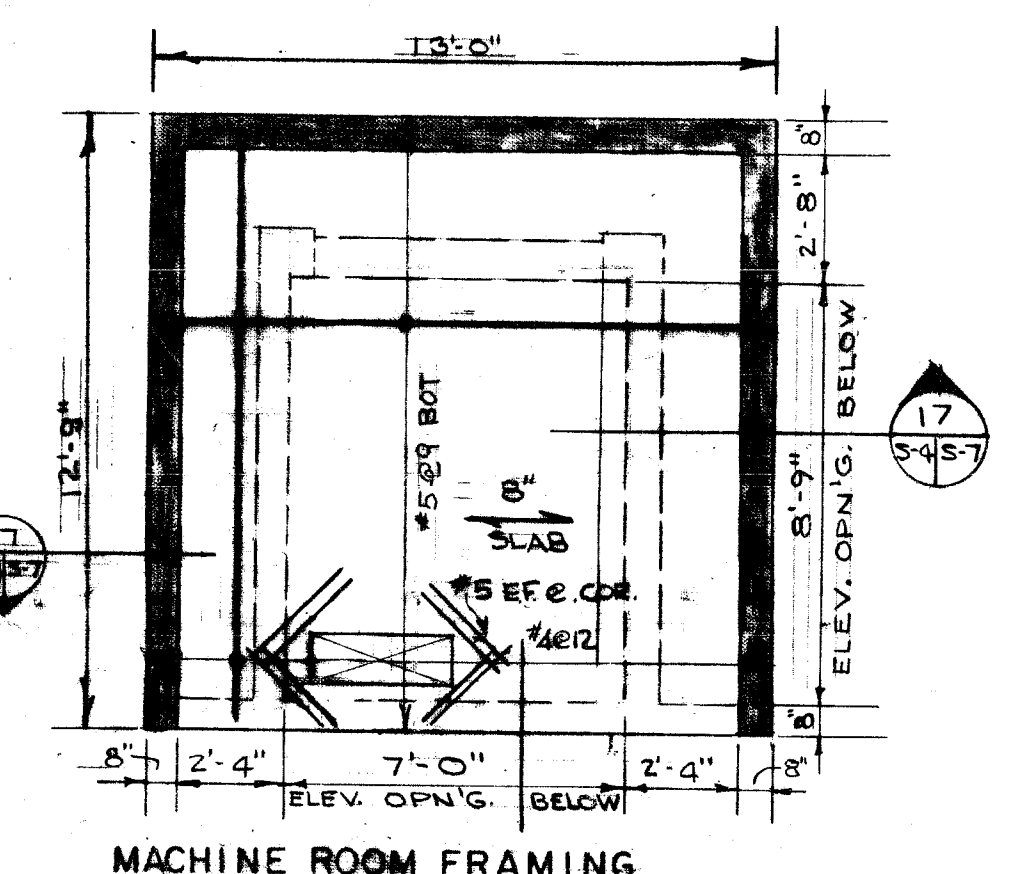
NOTE:
SEE ROOF FRAMING PLAN FOR
MILD REINFORCING
SEE 4TH FLOOR FRAMING PLAN DWG. S-3
FOR DIMENSIONS.



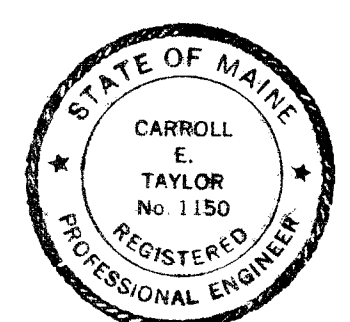
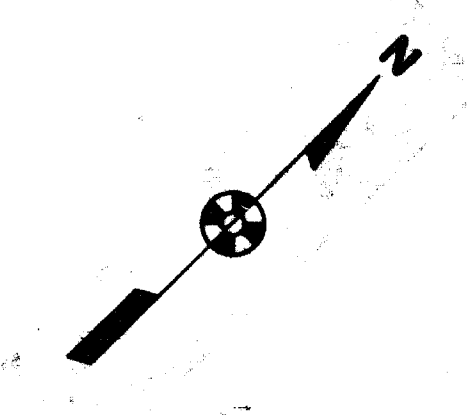
SECTION 11
NO SCALE



ROOF FRAMING
DESIGN L = 40 P.S.F.
TOP OF SLAB EL. 150.5'



MACHINE ROOM FRAMING
DESIGN L = 150 P.S.F.
PASSENGER ELEVATOR
SCALE 1/4" = 1'-0"
TOP OF SLAB EL. 142.0'



Signed and sealed these plans by the Director of the Bureau of Public Improvements, and under a guarantee as to the accuracy of the design and design details are the responsibility of the designer.

CARROLL E. TAYLOR & ASSOCIATES
CONSULTING ENGINEERS
SUMMER STATION

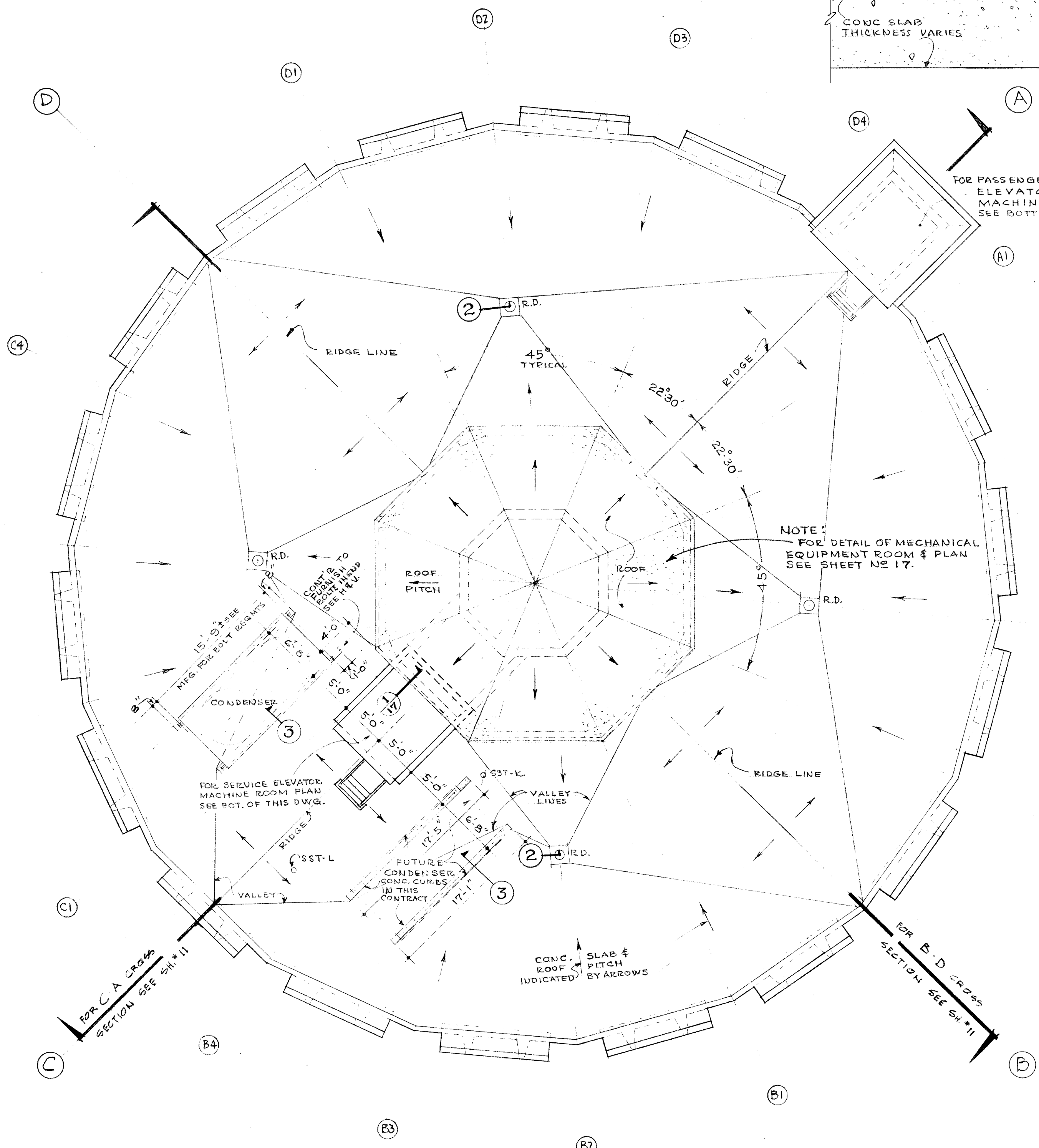
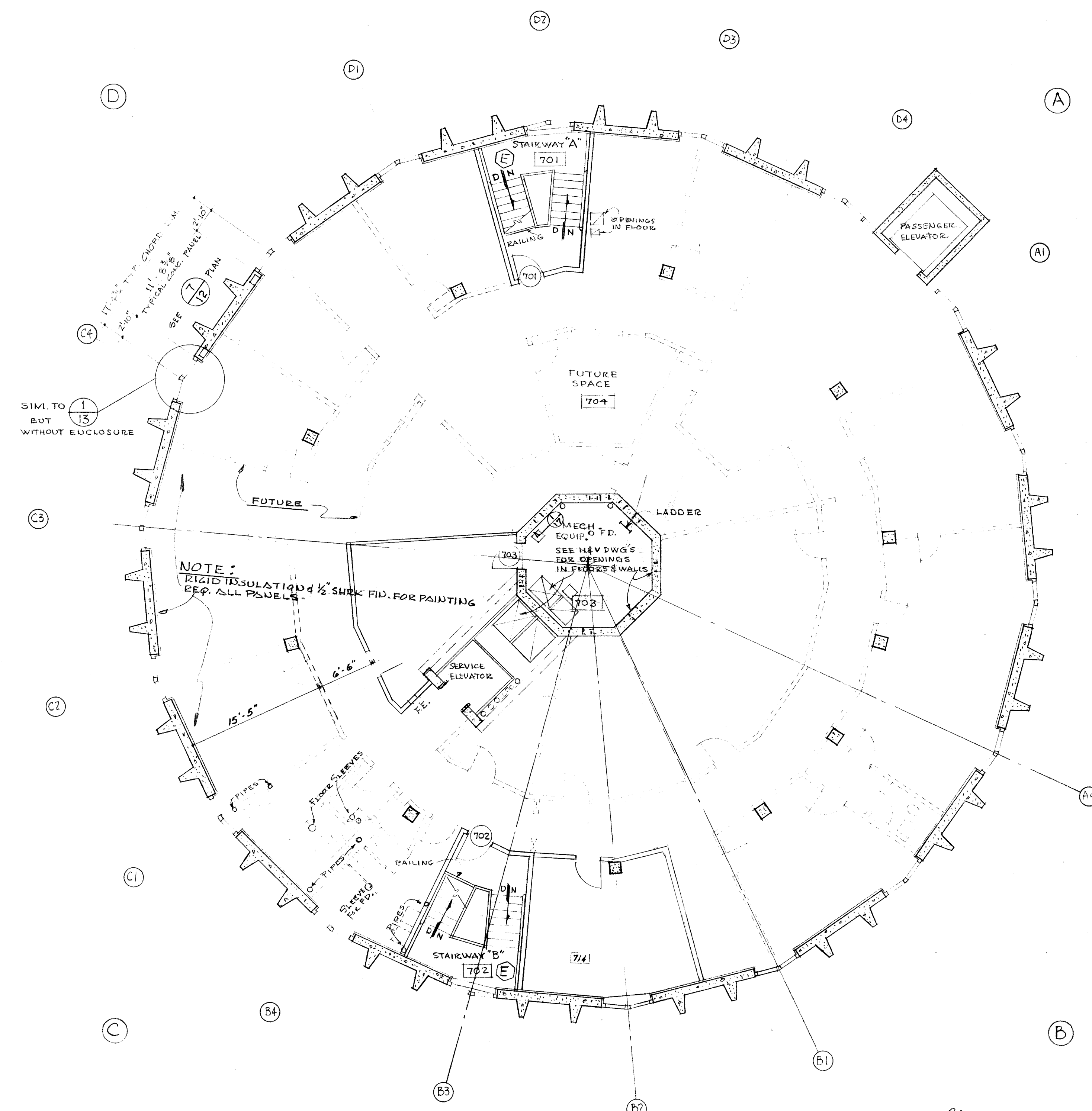
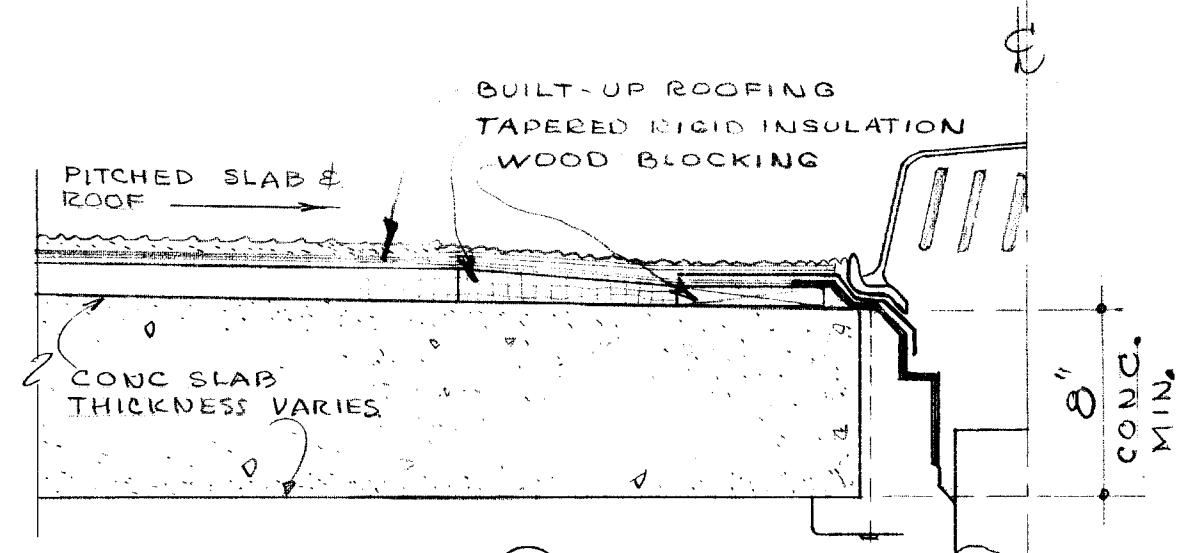
AS BUILT DRAWING

REGISTERED ARCHITECT

STATE OF MAINE

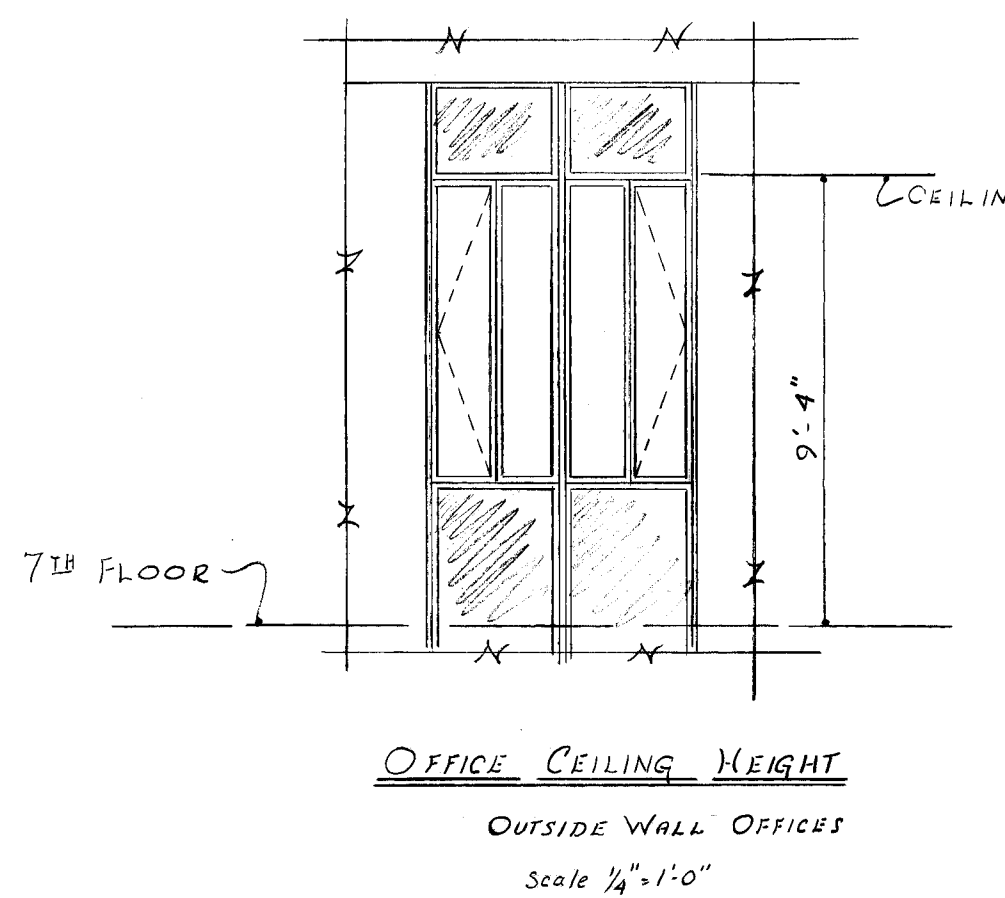
DEPT. OF HEALTH & WELFARE

REGISTERED PROFESSIONAL ENGINEER

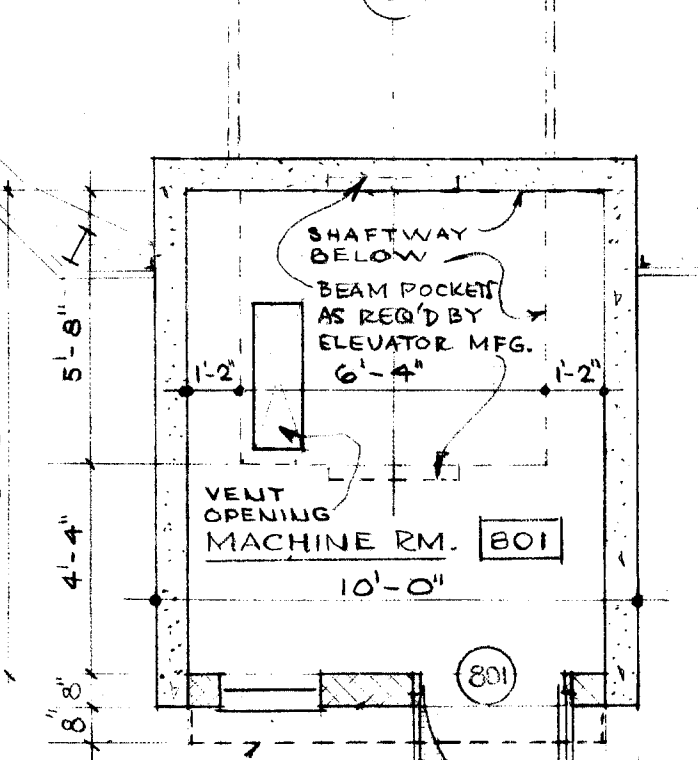
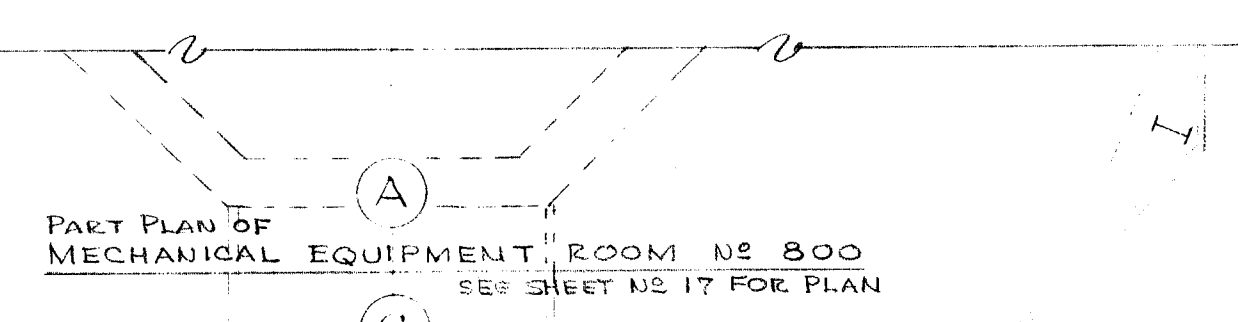
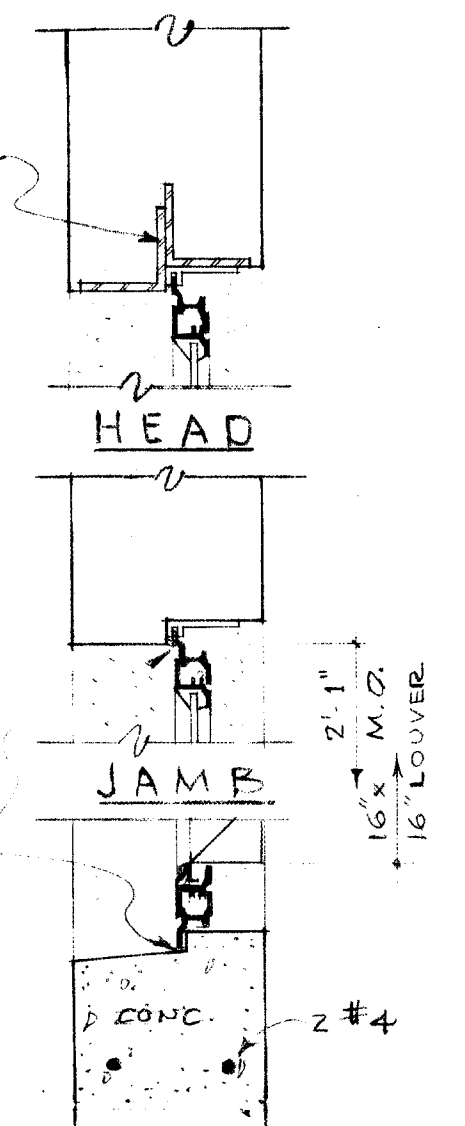
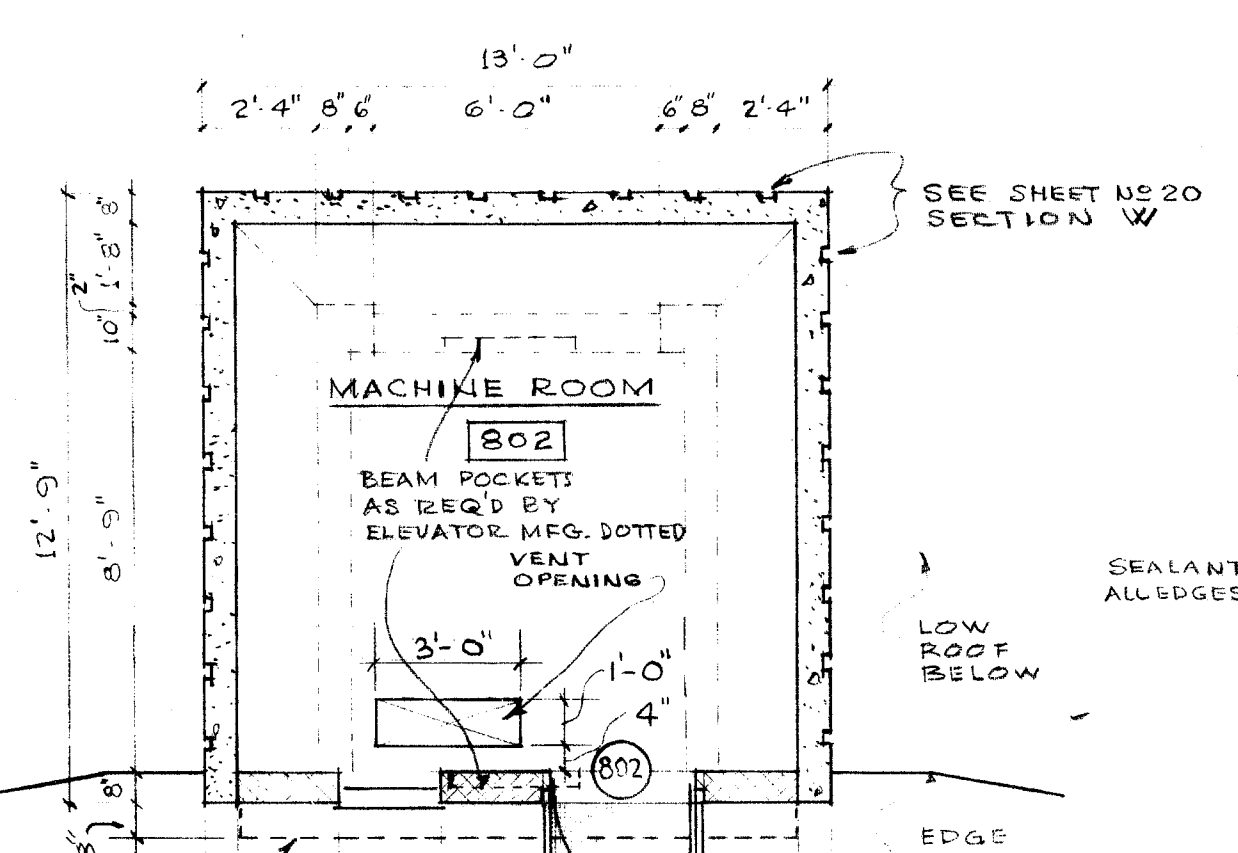


Roof Drain Sump Detail
 SCALE: 1/2" = 1'-0"

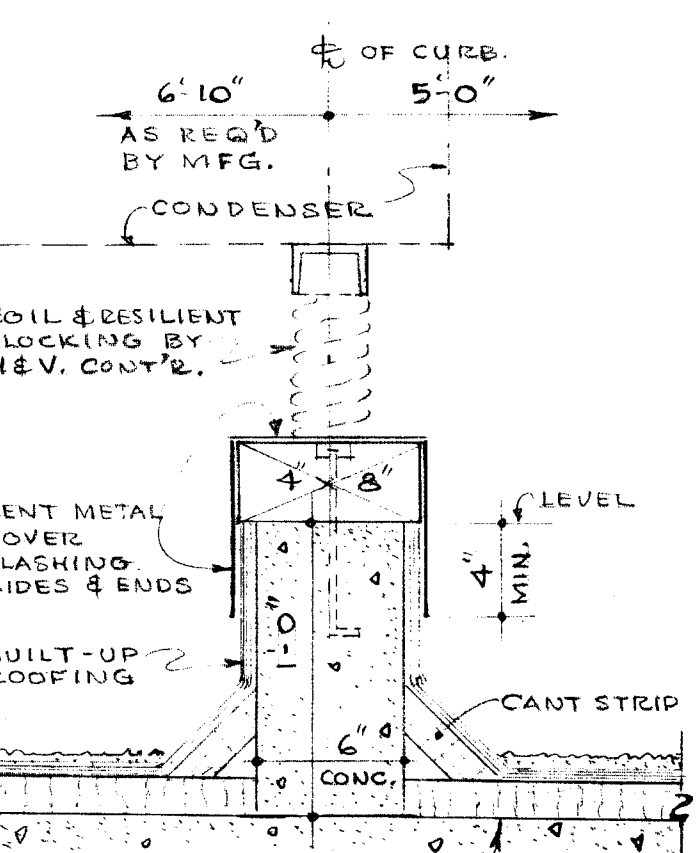
4 CURB DETAIL FOR GOOSENECK
 SCALE: 1/2" = 1'-0"



SEVENTH FLOOR



NOTE:
 SEE SHEET NO 20 FOR ELEVATOR SECTIONS & DETAILS



ROOF

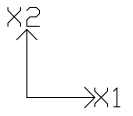
AS-BUILT DRAWING

APPROVED FOR DEPT. OF HEALTH & WELFARE	APPROVED FOR DEPT. OF INSURANCE
BY BUREAU OF PUBLIC IMPROVEMENTS	BY APPROVED FOR
DATE	DATE
BY DIRECTOR	BY DIRECTOR
DATE	DATE

REGISTERED ARCHITECT
 DONALD L. BISHOP
 No. 320

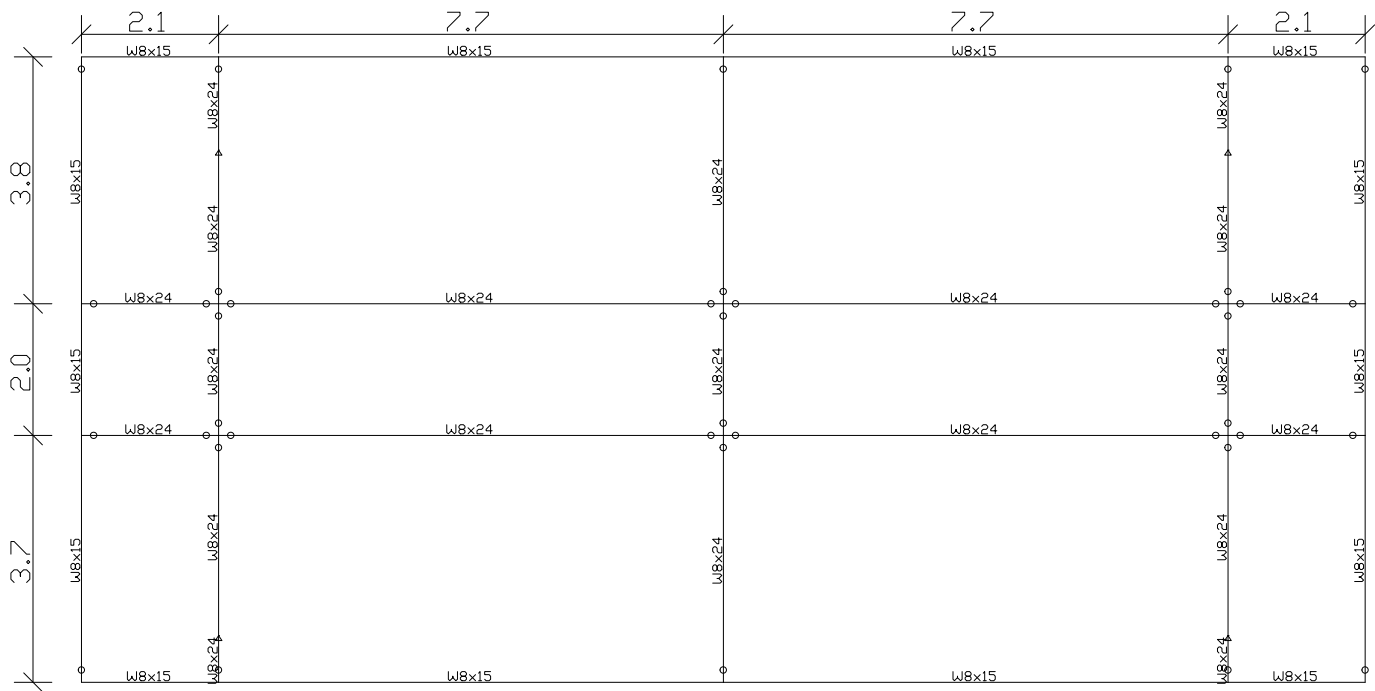
STATE OF MAINE
 LAW SCHOOL, RESEARCH AND ADVANCED STUDY BUILDING
 UNIV. OF MAINE PORTLAND CAMPUS

Portland Law Library Equipment Frame



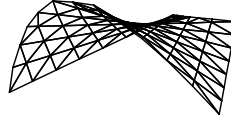
SCALE = 1:35

DATE: 11/28/14



Prepared by:

STRAP



USA AGENT
ATIR
ENGINEERING SOFTWARE
3314 WEST RANCE TERRACE
CHICAGO, IL 60645-3831
PHONE: 847-677-1945
FAX: 847-677-3456
E-MAIL: strap@atir.com

Load no. 1: Selfweight (units - kips ft.)

/ BEAM LOADS
SELF X3 -1. B 1 TO 39
/ END

FORCE SUMMATION

FX1=0.
FX2=0.
FX3=-2.5322

Load no. 2: Grating Load (units - kips ft.)

/ GLOBAL LOADS
DIST FX3 -0.01 PLANE -2.0833 -0.7083 1. -2.0833 8.7917 1. 17.417
8.7917 1. PT 0. 19.5 BEAMS
/ END

FORCE SUMMATION

FX1=0.
FX2=0.
FX3=-1.8525

Load no. 3: Cabinet Loads (units - kips ft.)

/ GLOBAL LOADS
DIST FX3 -0.1 PLANE 8.7167 2.7917 1. 5.7167 2.7917 1. 5.7167 5.2917
1. PT 0. 2.5 BEAMS
DIST FX3 -0.1 PLANE 9.3167 2.7917 1. 9.3167 5.2917 1. 12.317 5.2917
1. PT 0. 3. BEAMS
DIST FX3 -0.1 PLANE 5.1167 2.7917 1. 5.1167 5.2917 1. 2.1167 5.2917
1. PT 0. 3. BEAMS
DIST FX3 -0.35 PLANE 1.4167 2.7917 1. 1.4167 5.2917 1. -1.5833
5.2917 1. PT 0. 3. BEAMS
/ END STATIC

Portland Law Library Equipment Frame

Page:
Date: 11/28/14

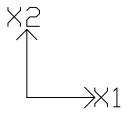
Prepared by:

Load no. 3: Cabinet Loads (units - kips ft.)

FORCE SUMMATION

FX1=0.
FX2=0.
FX3=-4.875

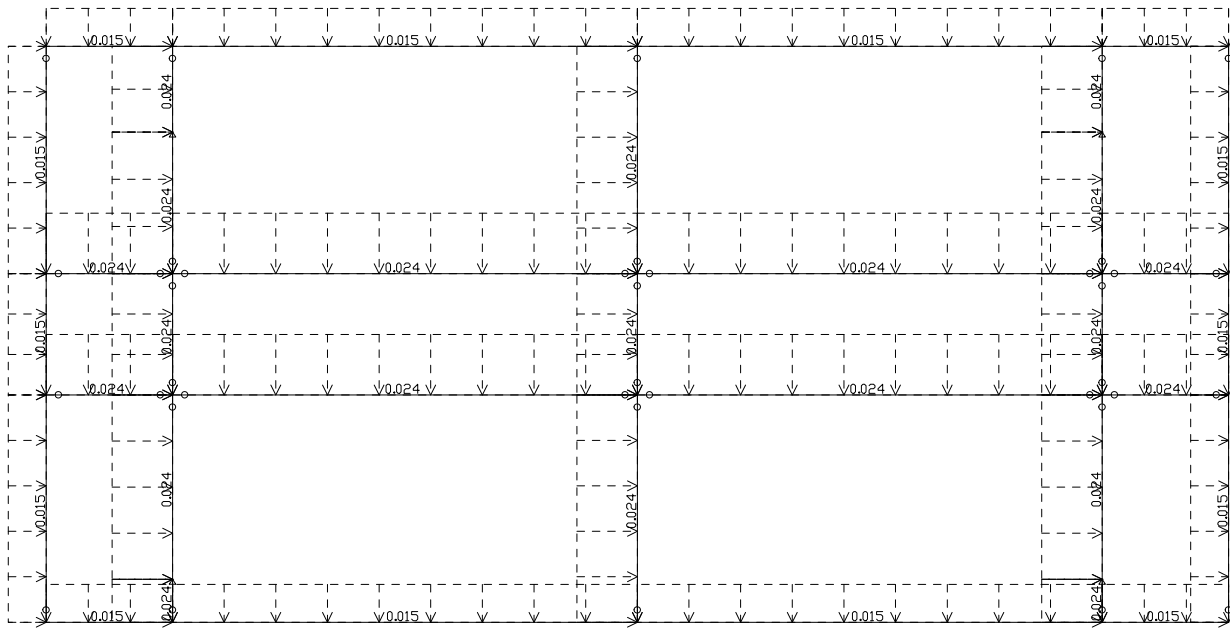
Load 1: Selfweight



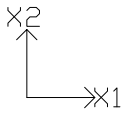
SCALE = 1:38

UNITS: kip ft

DATE: 11/28/14



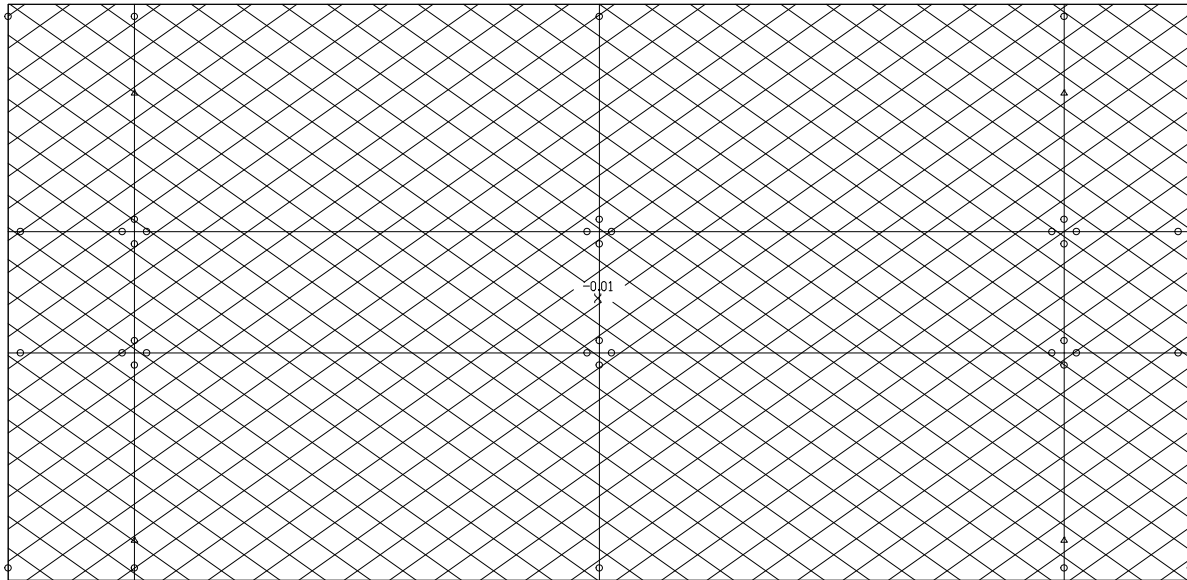
Load 2: Grating Load



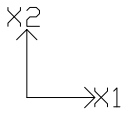
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UNITS: kip ft

DATE: 11/28/14



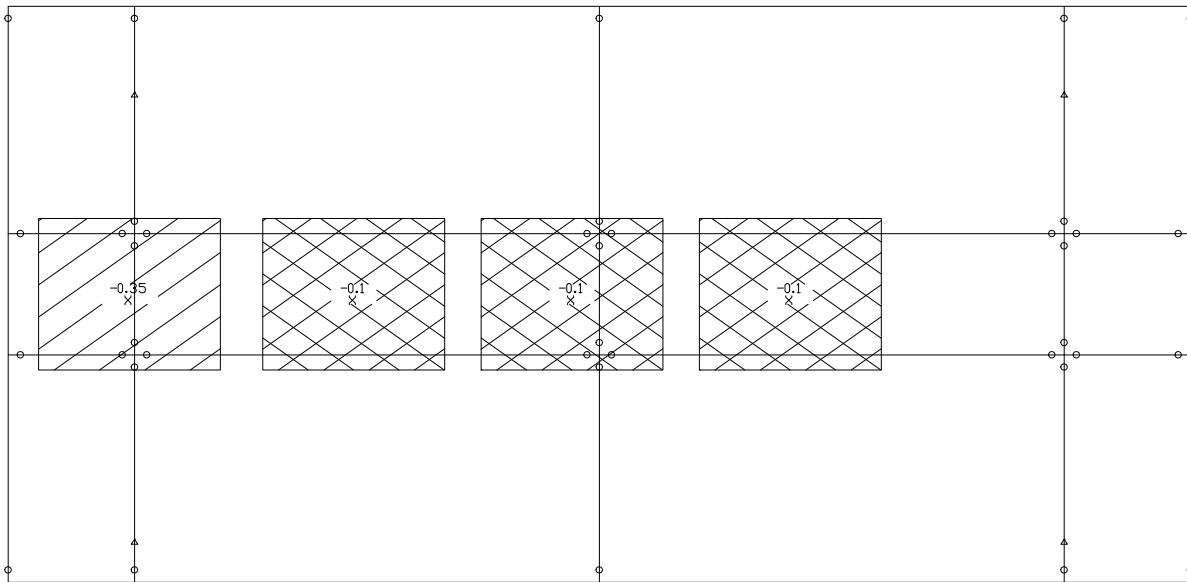
Load 3: Cabinet Loads

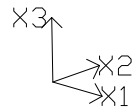


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UNITS: kip ft

DATE: 11/28/14

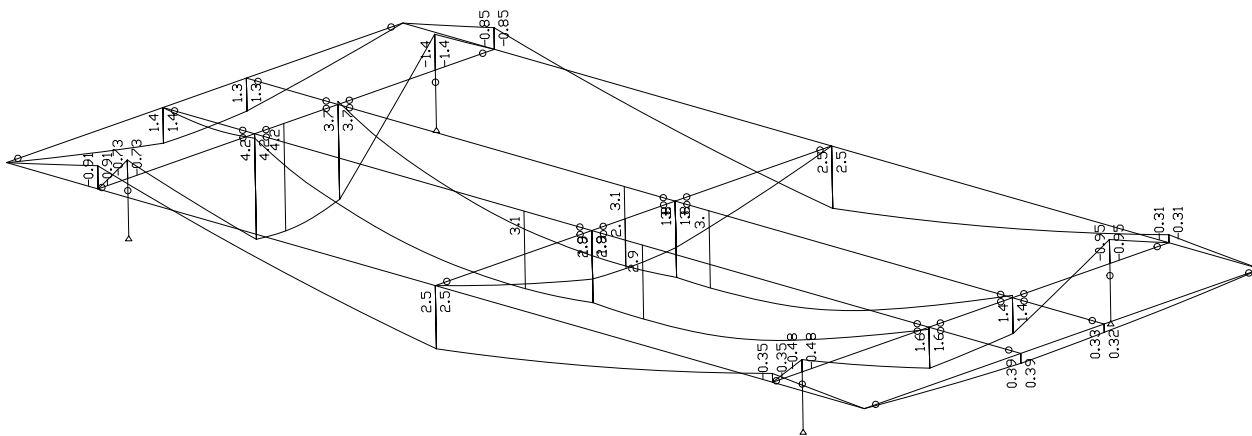




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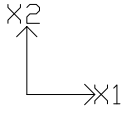
UNITS: kip*ft

DATE: 11/28/14



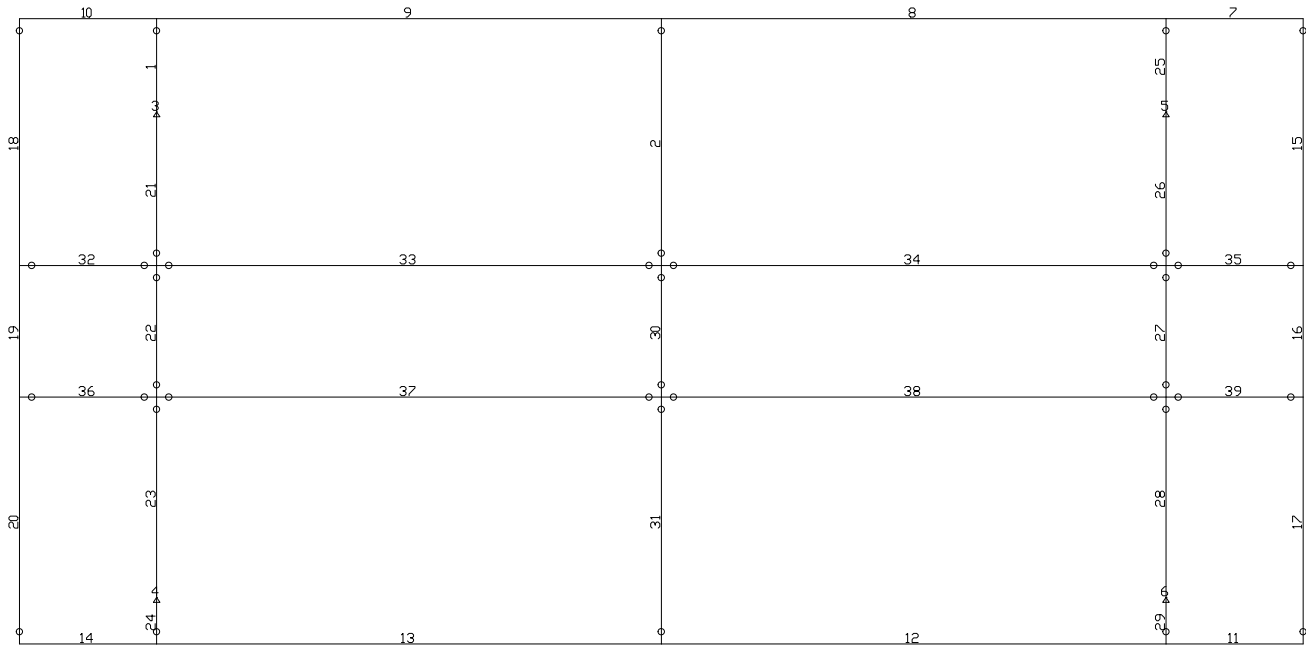
M2 MOMENT COMB. NO. 1 Total Load

Portland Law Library Equipment Frame



SCALE = 1:35

DATE: 11/28/14



Portland Law Library Equipment Frame_

Code: AISC-ASD

Prepared by:

Page:

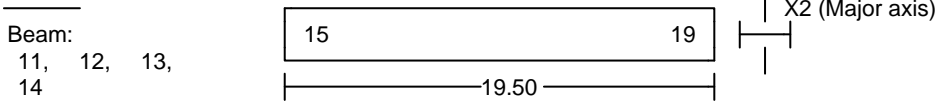
Date: 11/28/14

Results Summary Table

Bea	Section	Co	Defl L/	Slen	CAPACITY					Combined Axial+Mom
					Axial	Dir	Shea	Mom	LTB	
1	W 8x24	1		11	0.00	MJ	0.04	0.03	0.03	0.03
2	W 8x24	1	8342	71	0.00	MJ	0.02	0.05	0.05	0.05
3	PIPE 3	1	9999	10	-0.07	MI	0.00	0.00	0.00	0.07
4	PIPE 3	1	9999	10	-0.06	MI	0.00	0.00	0.00	0.06
5	PIPE 3	1	9999	10	-0.04	MI	0.00	0.00	0.00	0.04
6	PIPE 3	1	9999	10	-0.03	MI	0.00	0.00	0.00	0.03
7	W 8x15	1	2905	267	0.00	MJ	0.02	0.10	0.18	0.18
11	W 8x15	1	2905	267	0.00	MJ	0.02	0.10	0.18	0.18
	W 8x15	1	9999	130	0.00	MJ	0.01	0.02	0.02	0.02
18	W 8x15	1	7041	130	0.00	MJ	0.01	0.06	0.06	0.06
21	W 8x24	1	9999	17	0.00	MJ	0.08	0.09	0.09	0.09
22	W 8x24	1	9999	15	0.00	MJ	0.02	0.10	0.10	0.10
23	W 8x24	1	9999	23	0.00	MJ	0.06	0.10	0.10	0.10
24	W 8x24	1	9999	5	0.00	MJ	0.04	0.02	0.02	0.02
25	W 8x24	1	9999	11	0.00	MJ	0.03	0.02	0.02	0.02
26	W 8x24	1		17	0.00	MJ	0.04	0.03	0.03	0.03
27	W 8x24	1	9999	15	0.00	MJ	0.00	0.04	0.04	0.04
28	W 8x24	1	9999	23	0.00	MJ	0.03	0.04	0.04	0.04
29	W 8x24	1	9999	5	0.00	MJ	0.02	0.01	0.01	0.01
32	W 8x24	1	2794	146	0.00	MJ	0.04	0.07	0.08	0.08
36	W 8x24	1	2796	146	0.00	MJ	0.04	0.07	0.08	0.08

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

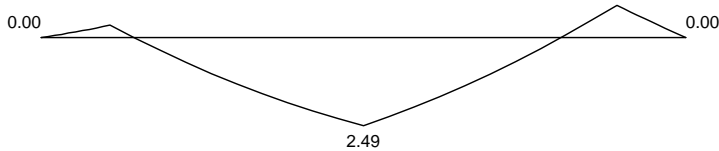
- Kx = 1.00 - Ky = 1.00
- Allow. Slend. : 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

Section: W 8x15

ix = 48.00 ly = 3.41in4 Zx = 13.60 Zy = 2.67in3 Area = 4.43
hw = 8.11 bf = 4.02in tw = 0.24 tf = 0.31in
J = 0.14 Cw = 48.41in6

DESIGN COMBINATION = 1

M2 Moment Diagram



Max. AXIAL Force = 0.00 (tens.) Max. SHEAR Force = 0.56

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios: Compac Non-Compact
d/t= 28.21 < 106.7 161.7 (Fy= 36.0 R= 0.000)
b/t= 6.38 < 10.8 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (G2.1.a)	$\frac{V_u}{V_n} / 1.5 < 1.00$ $V_n = 0.6 \cdot F_y \cdot A_w$	$A_v = 1.98$	$V_u = 0.56$ $V_n = 42.81$	0.02
M2 Moment (F2-1) without LTB	$\frac{M}{0.6M_n} < 1.00$	$Z = 13.60$	$M = 2.49$ $M_n = 40.85$	0.10
Deflection	$\frac{\text{defl.}}{L / 240} < 1.00$		$\text{defl} = 0.08055$	0.08
Lateral Torsional Buckling (F2-3)	$\frac{M}{0.6M_n} < 1.00$	$L_b = 19.50$ $L_p = 3.65$ $L_r = 12.83$ $C_b = 1.52$	$M = 2.49$ $M_n = 22.79$ $M_r = 24.89$ $F_{cr} = 23.12$	0.18
Critical Segment from 0.00 to 19.50 on +z flange Segment End Moments: 0.00 and 0.00				

Portland Law Library Equipment Frame_

Code: AISC-ASD

Prepared by:

Page: 3

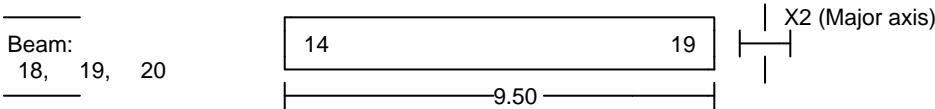
Date: 11/28/14

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

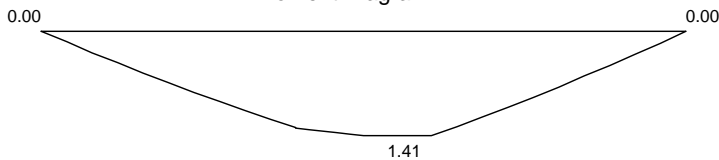
- Kx = 1.00 - Ky = 1.00
- Allow. Slend.: 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

Section: W 8x15

Ix = 48.00 Iy = 3.41in4 Zx = 13.60 Zy = 2.67in3 Area = 4.43
hw = 8.11 bf = 4.02in tw = 0.24 tf = 0.31in
J = 0.14 Cw = 48.41in6

DESIGN COMBINATION = 1

M2 Moment Diagram



Max. AXIAL Force = 0.00 (tens.) Max. SHEAR Force = 0.42

SECTION CLASSIFICATION: * COMPACT *****

Limiting Ratios: Compac Non-Compact
d/t= 28.21 < 106.7 161.7 (Fy= 36.0 R = 0.000)
b/t= 6.38 < 10.8 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (G2.1.a)	$V_u/V_n < 1.5 < 1.00$ $V_n = 0.6 \cdot F_y \cdot A_w$	$A_v = 1.98$	$V_u = 0.42$ $V_n = 42.81$	0.01
M2 Moment (F2-1) without LTB	$M < 1.00$ $0.6M_n$	$Z = 13.60$	$M = 1.41$ $M_n = 40.85$	0.06

Portland Law Library Equipment Frame_

Code: AISC-ASD

Prepared by:

Page: 4

Date: 11/28/14

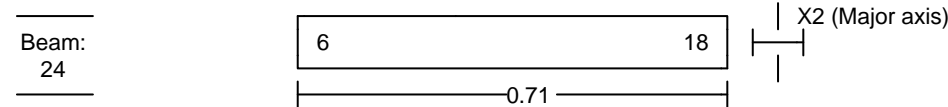
Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

DESIGN	EQUATION	FACTORS	VALUES	RESUL
Deflection	$\frac{\text{defl.}}{L / 240} < 1.00$		defl = 0.01619	0.03
Lateral Torsional Buckling (F2-2)	$\frac{M}{0.6M_n} < 1.00$	Lb = 9.50 Lp = 3.65 Lr = 12.83 Cb = 1.22	M = 1.41 Mn = 37.24 Mr = 24.89 Mp = 40.85	0.06
Critical Segment from 0.00 to 9.50 on +z flange Segment End Moments: 0.00 and 0.00				

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

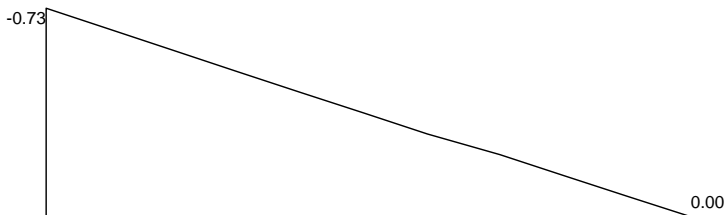
- Kx = 1.00 - Ky = 1.00
- Allow. Slend.: 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

Section: W 8x24

Ix = 82.80 Iy = 18.30in⁴ Zx = 23.20 Zy = 8.57in³ Area = 7.08
hw = 7.93 bf = 6.50in tw = 0.24 tf = 0.40in
J = 0.35 Cw = 256.95in⁶

DESIGN COMBINATION = 1

M2 Moment Diagram



Max. AXIAL Force = 0.00 (tens.) Max. SHEAR Force = 1.04

Portland Law Library Equipment Frame_

Code: AISC-ASD

Prepared by:

Page: 5

Date: 11/28/14

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

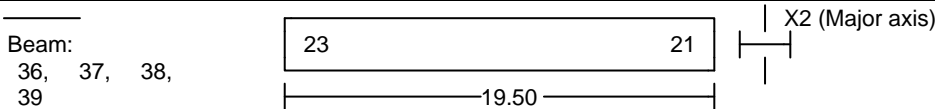
SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios: Compac Non-Compact
 $d/t = 25.90 < 106.7$ 161.7 ($F_y = 36.0$ $R = 0.000$)
 $b/t = 8.09 < 10.8$ 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (G2.1.a)	$V_u/V_n < 1.00$ $V_n = 0.6 F_y A_w$	$A_v = 1.94$	$V_u = 1.04$ $V_n = 41.86$	0.04
M2 Moment (F2-1) without LTB	$\frac{M}{0.6 M_n} < 1.00$	$Z = 23.20$	$M = 0.73$ $M_n = 69.69$	0.02
Lateral Torsional Buckling	$\frac{M}{0.6 M_n} < 1.00$ Critical Segment from Segment End Moments:	$L_b = 0.71$ $L_p = 6.70$ 0.00 to 0.71 on -z flange -0.73 and 0.00	$M = 0.73$ $M_n = 69.55$	0.02

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
 - Steel Grade: A36

DESIGN DATA

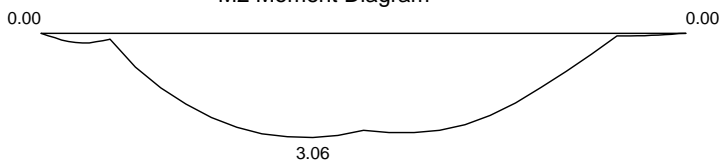
- $K_x = 1.00$ - $K_y = 1.00$
 - Allow. Slend. : 200 (compr.) 300 (tens.)
 - Allowable Deflection : 1/240
 - Tension Area Reduction Factor : 1.00
 - Building type : Unbraced

Section: W 8x24

$I_x = 82.80$ $I_y = 18.30 \text{ in}^4$ $Z_x = 23.20$ $Z_y = 8.57 \text{ in}^3$ Area = 7.08
 $h_w = 7.93$ $bf = 6.50 \text{ in}$ $tw = 0.24$ $tf = 0.40 \text{ in}$
 $J = 0.35$ $C_w = 256.95 \text{ in}^6$

DESIGN COMBINATION = 1

M2 Moment Diagram



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Code: AISC-ASD

Prepared by:

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Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

Max. AXIAL Force = 0.00 (tens.) Max. SHEAR Force = 1.14

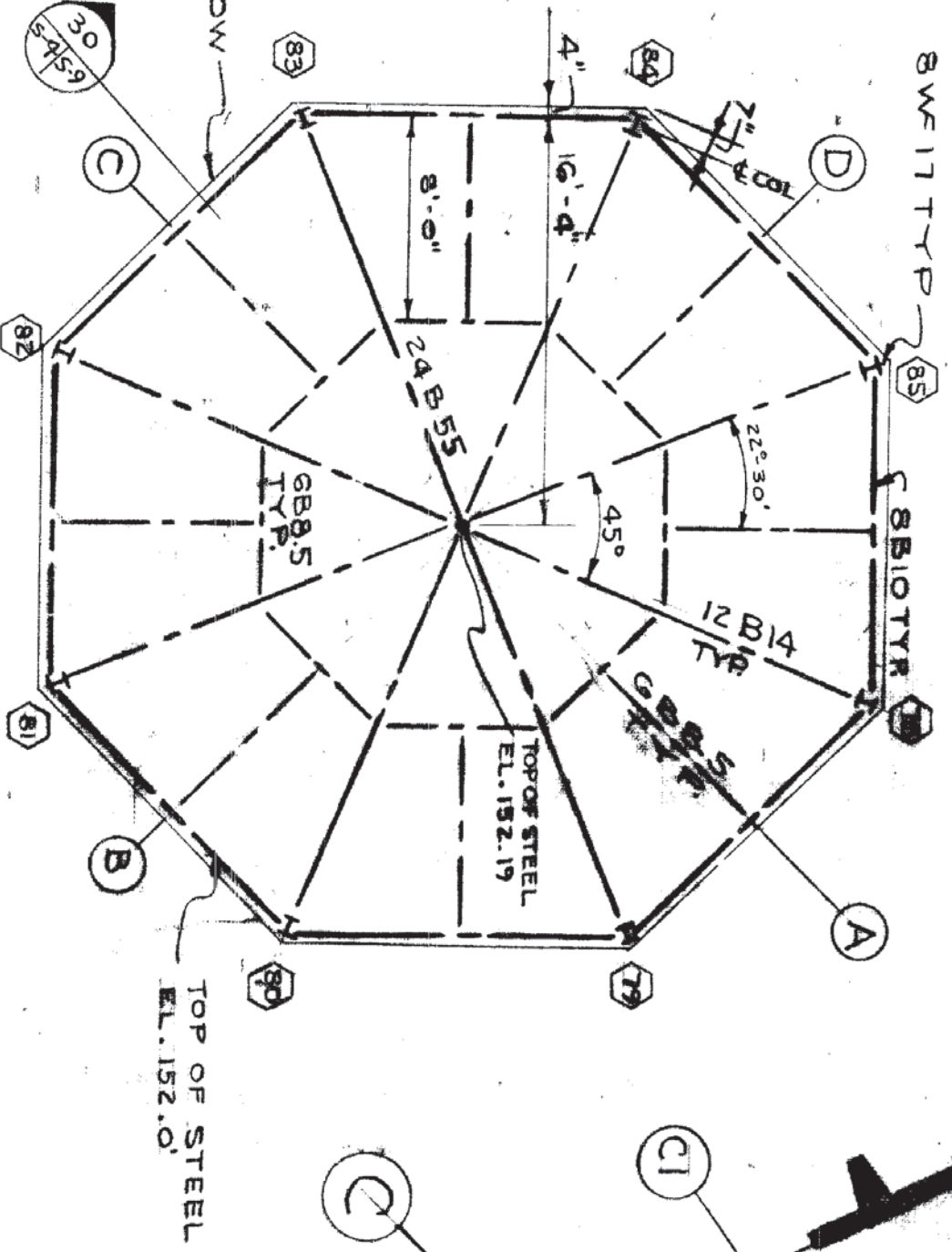
SECTION CLASSIFICATION: *** COMPACT ***

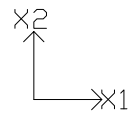
Limiting Ratios: Compac Non-Compact
d/t= 25.90 < 106.7 161.7 (Fy= 36.0 R= 0.000)
b/t= 8.09 < 10.8 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (G2.1.a)	$\frac{V_u}{V_n} / 1.5 < 1.00$ $V_n = 0.6 * F_y * A_w$	$A_v = 1.94$	$V_u = 1.14$ $V_n = 41.86$	0.04
M2 Moment (F2-1) without LTB	$\frac{M}{0.6 M_n} < 1.00$	$Z = 23.20$	$M = 3.06$ $M_n = 69.69$	0.07
Deflection	$\frac{\text{defl.}}{L / 240} < 1.00$		$\text{defl} = 0.08369$	0.09
Lateral Torsional Buckling (F2-2)	$\frac{M}{0.6 M_n} < 1.00$ Critical Segment from Segment End Moments:	$L_b = 19.50$ $L_p = 6.70$ $L_r = 24.99$ $C_b = 1.21$ 0.00 to 19.50 on +z flange 0.00 and 0.00	$M = 3.06$ $M_n = 62.27$ $M_r = 43.91$ $M_p = 69.69$	0.08

**MECH. EQUIP. ROOM,
ROOF FRAMING PLAN**

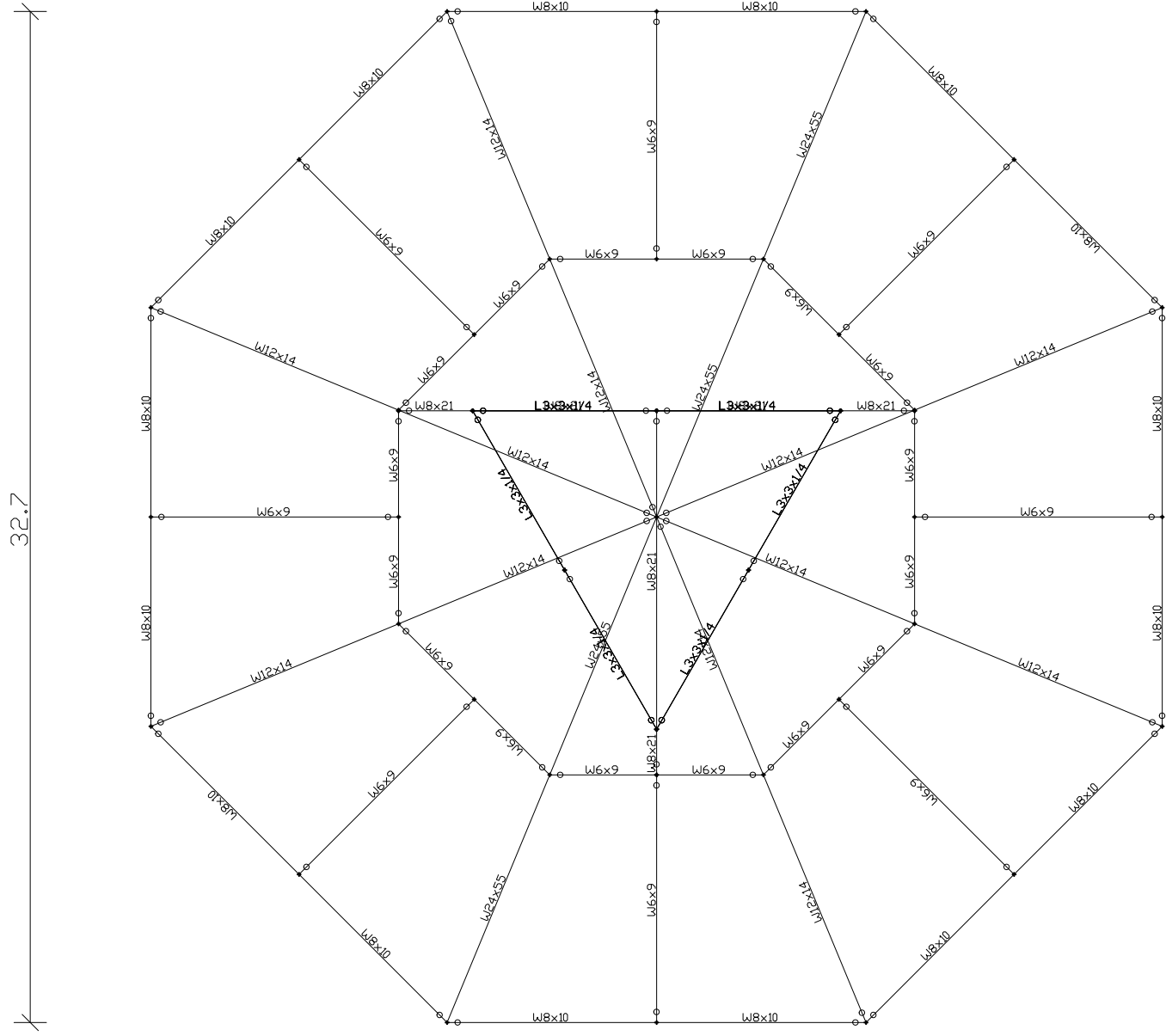
SEE DWG. S-9 FOR DETAILS

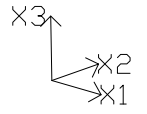




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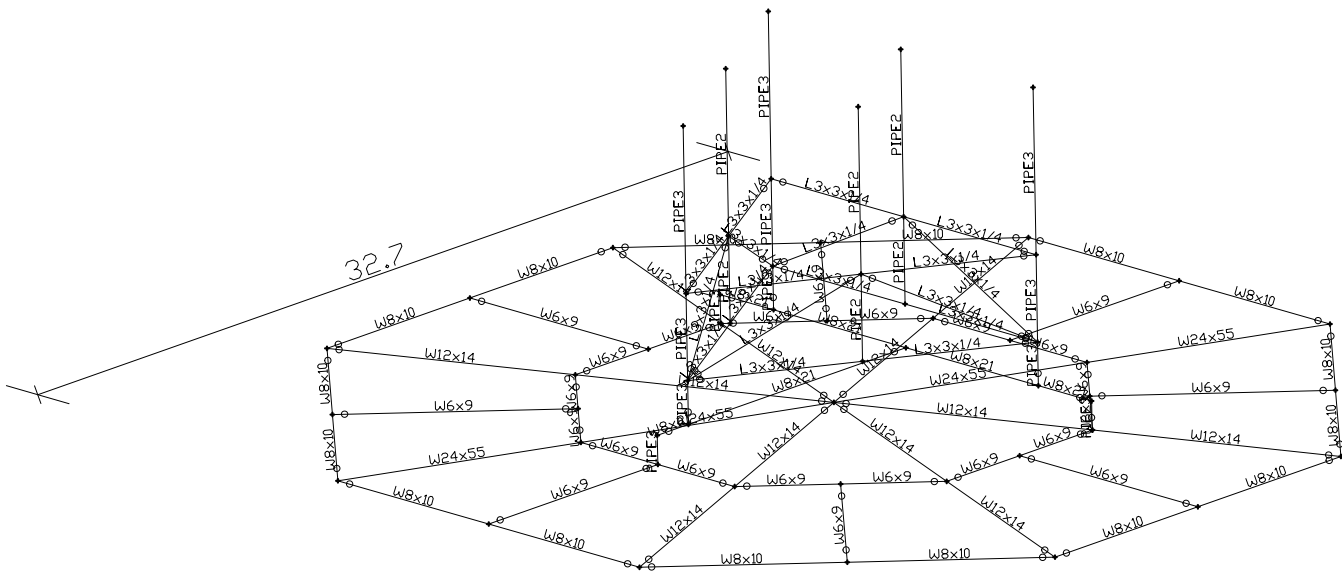
DATE: 11/28/14



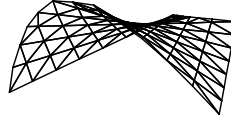


SCALE = 1:75

DATE: 11/28/14



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Load no. 1: Selfweight (units - kips ft.)

/ BEAM LOADS
/ BEAM LOADS
SELF X3 -1. B 1 TO 8 51 TO 100
/ END

FORCE SUMMATION

FX1=0.
FX2=0.
FX3=-5.6265

Load no. 2: Roof Deck Load (units - kips ft.)

/ GLOBAL LOADS
/ GLOBAL LOADS
DIST FX3 -0.005 PLANE -6.7654 -16.333 0. -16.333 -6.7654 0. -16.333
6.7654 0. PT 23.099 23.099 13.531 32.667 0. 32.667 -9.5677 23.099
-9.5677 9.5677 BEAMS
/ END

FORCE SUMMATION

FX1=0.
FX2=0.
FX3=-4.4196

Load no. 3: Snow Load (units - kips ft.)

/ GLOBAL LOADS
/ GLOBAL LOADS
DIST FX3 -0.032 PLANE -16.333 -6.7654 0. -16.333 6.7654 0. -6.7654
16.333 0. PT 23.099 23.099 13.531 32.667 0. 32.667 -9.5679 23.099
-9.5679 9.5679 BEAMS
/ END

Load no. 3: Snow Load (units - kips ft.)

FORCE SUMMATION

FX1=0.
FX2=0.
FX3=-28.286

Load no. 4: -X2 Wind Load (units - kips ft.)

/ BEAM LOADS
DIST GL FX2 -0.012 B 9 TO 50
CONC GL FX2 -0.2 FR 0.5 B 11 19 14 23 21 17
/ END

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Date: 11/28/14

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Load no. 1: Selfweight (units - kips ft.)

FORCE SUMMATION

FX1=0.
FX2=-3.596
FX3=0.

Load no. 5: -X1 Wind Loads (units - kips ft.)

/ BEAM LOADS
DIST GL FX1 -0.012 B 9 TO 50
CONC GL FX1 -0.2 FR 0.5 B 11 19 14 23 21 17
/ END

FORCE SUMMATION

FX1=-3.596
FX2=0.
FX3=0.

Load no. 6: X2 Wind Load (units - kips ft.)

/ JOINT LOADS
/ BEAM LOADS
DIST GL FX2 0.012 B 9 TO 50
CONC GL FX2 0.2 FR 0.5 B 11 19 14 23 21 17
/ END

FORCE SUMMATION

FX1=0.
FX2=3.596
FX3=0.

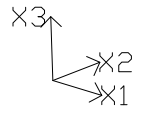
Load no. 7: Antenna Frame Selfweight (units - kips ft.)

/ BEAM LOADS
SELF X3 -1. B 9 TO 50
/ END STATIC

FORCE SUMMATION

FX1=0.
FX2=0.
FX3=-1.4922

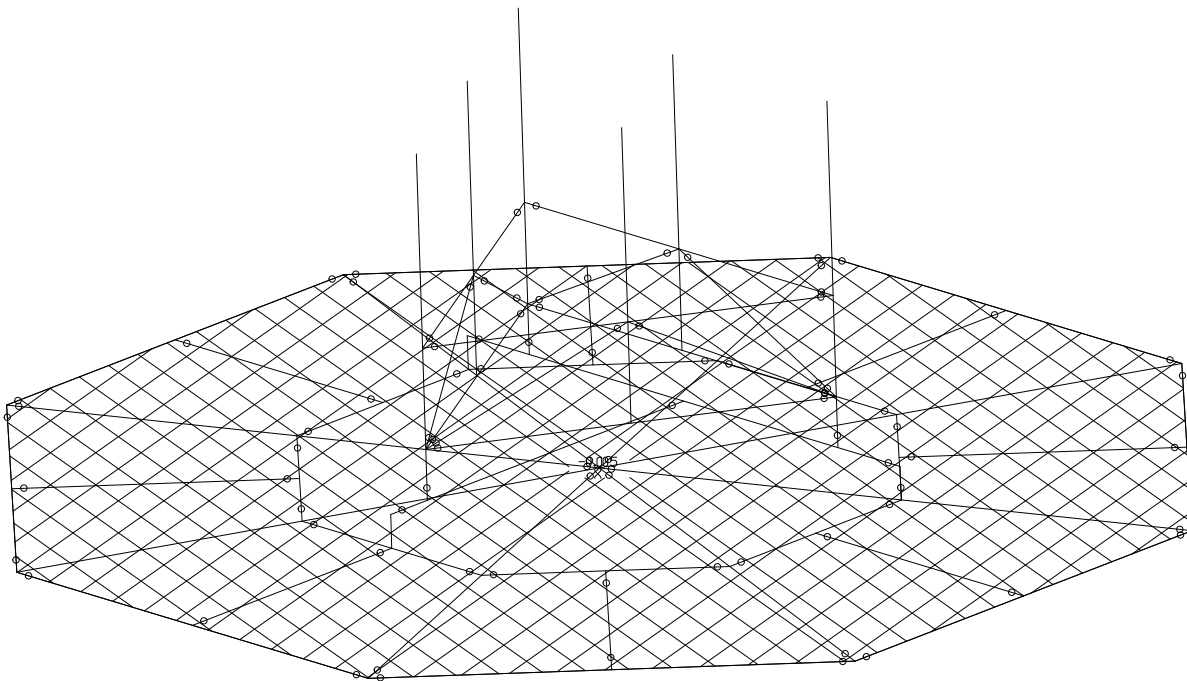
Load 2: Roof Deck Load



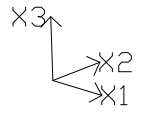
SCALE = 1:64

UNITS: kip ft

DATE: 11/28/14



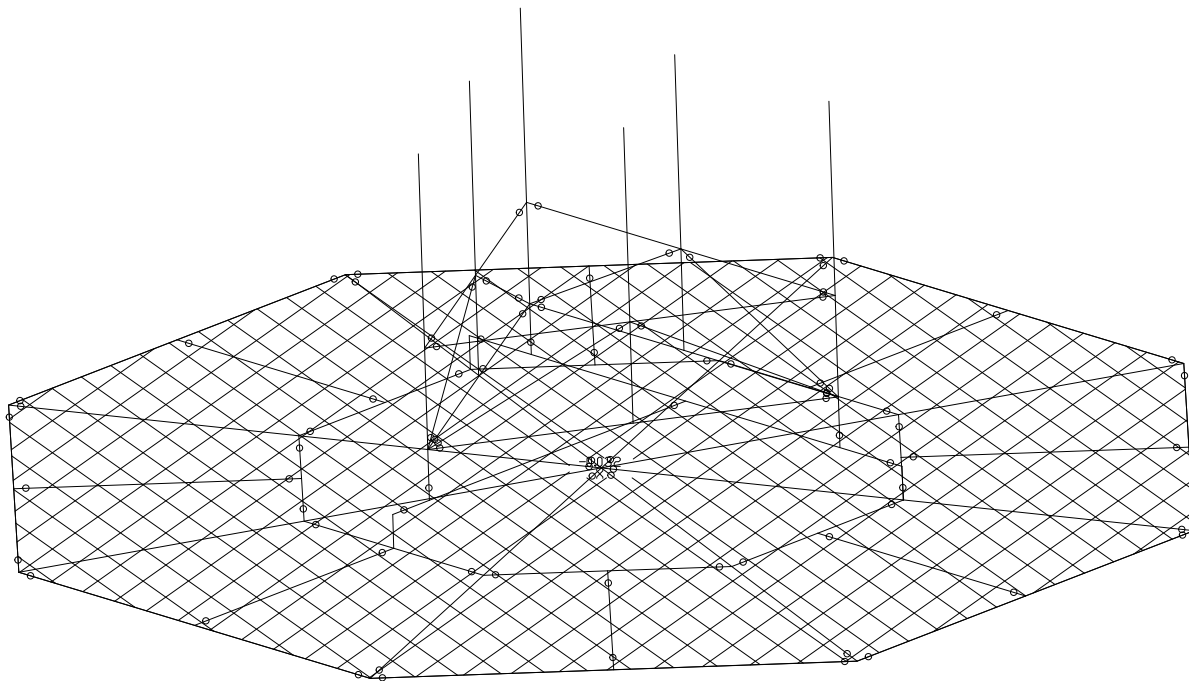
Load 3: Snow Load

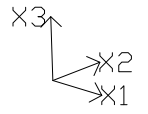


SCALE = 1:64

UNITS: kip ft

DATE: 11/28/14

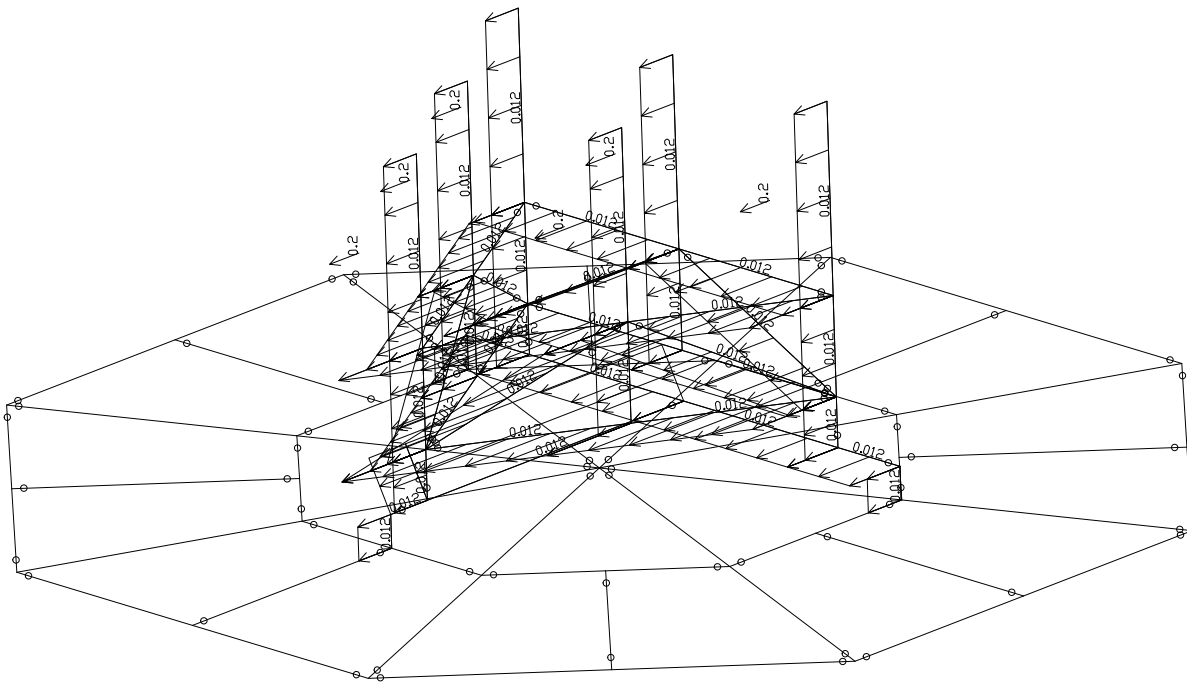




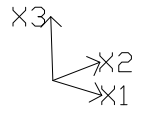
SCALE = 1:64

UNITS: kip ft

DATE: 11/28/14



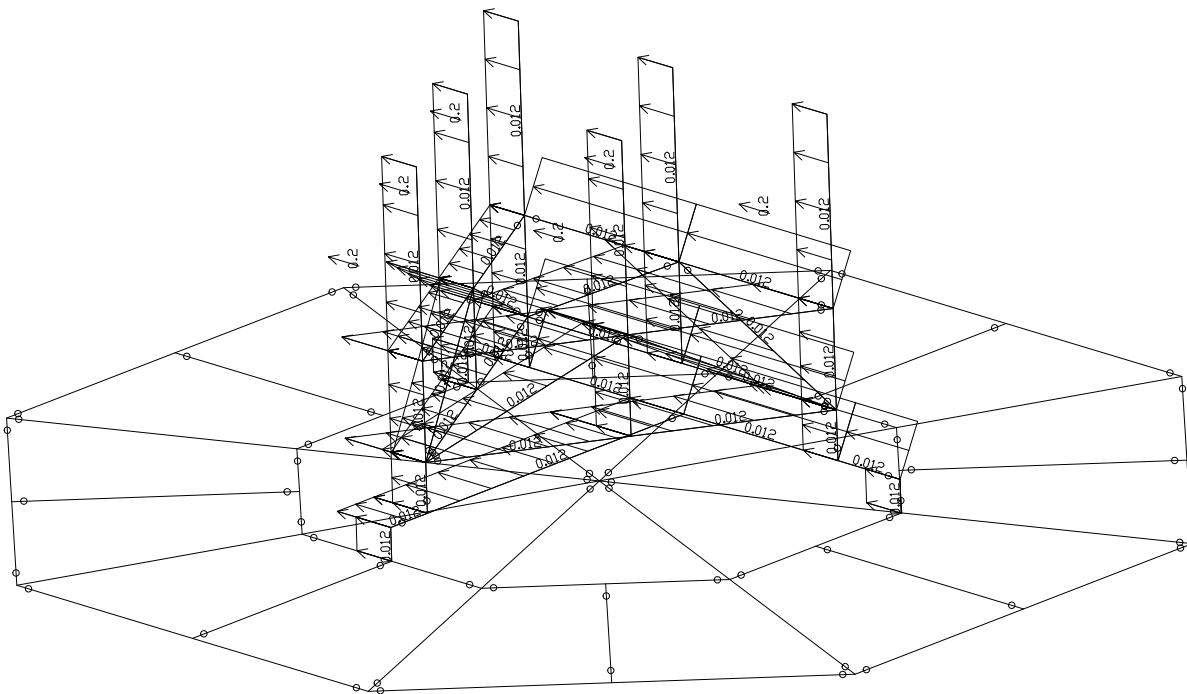
Load 5: -X1 Wind Loads

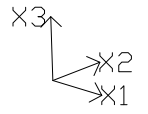


SCALE = 1:64

UNITS: kip ft

DATE: 11/28/14

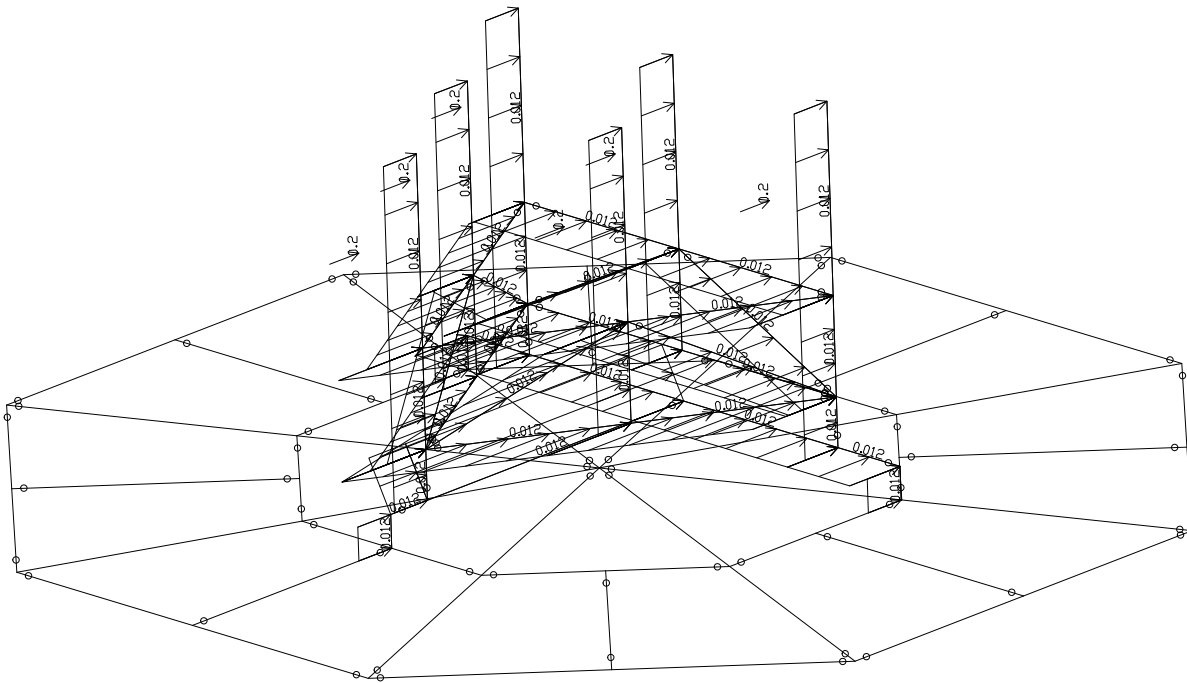




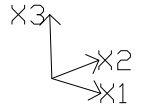
SCALE = 1:64

UNITS: kip ft

DATE: 11/28/14



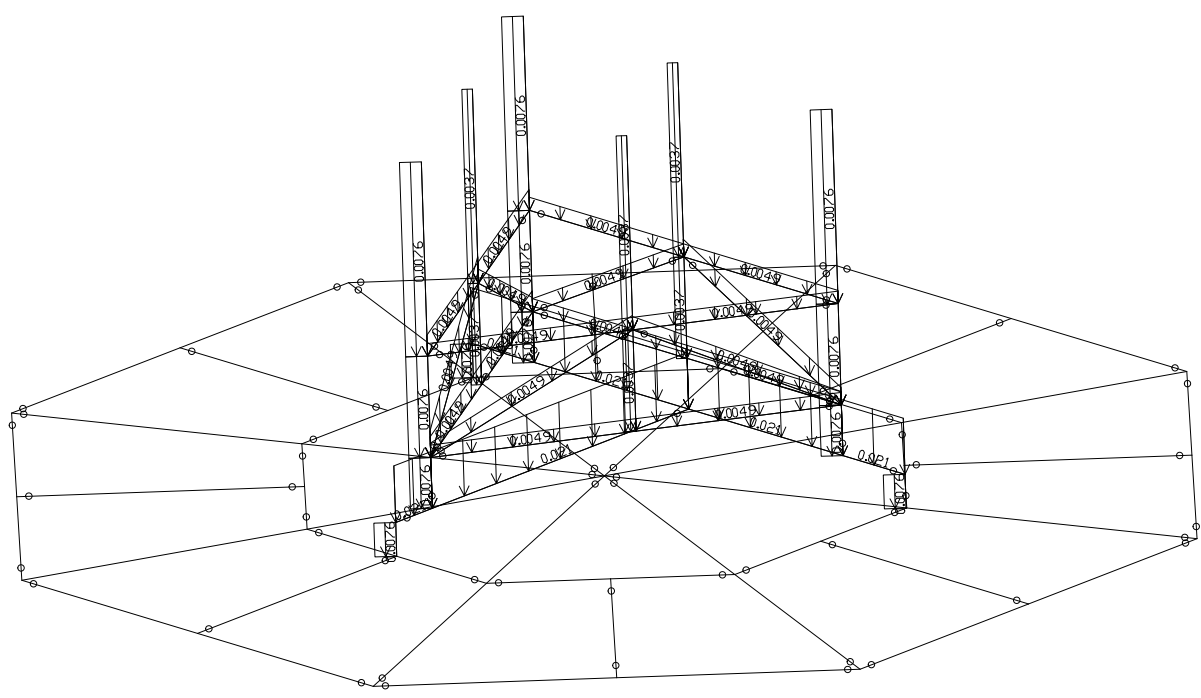
Load 7: Antenna Frame Selfweight



SCALE = 1:64

UNITS: kip ft

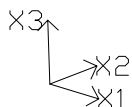
DATE: 11/28/14



Prepared by:

COMBINATIONS TABLE

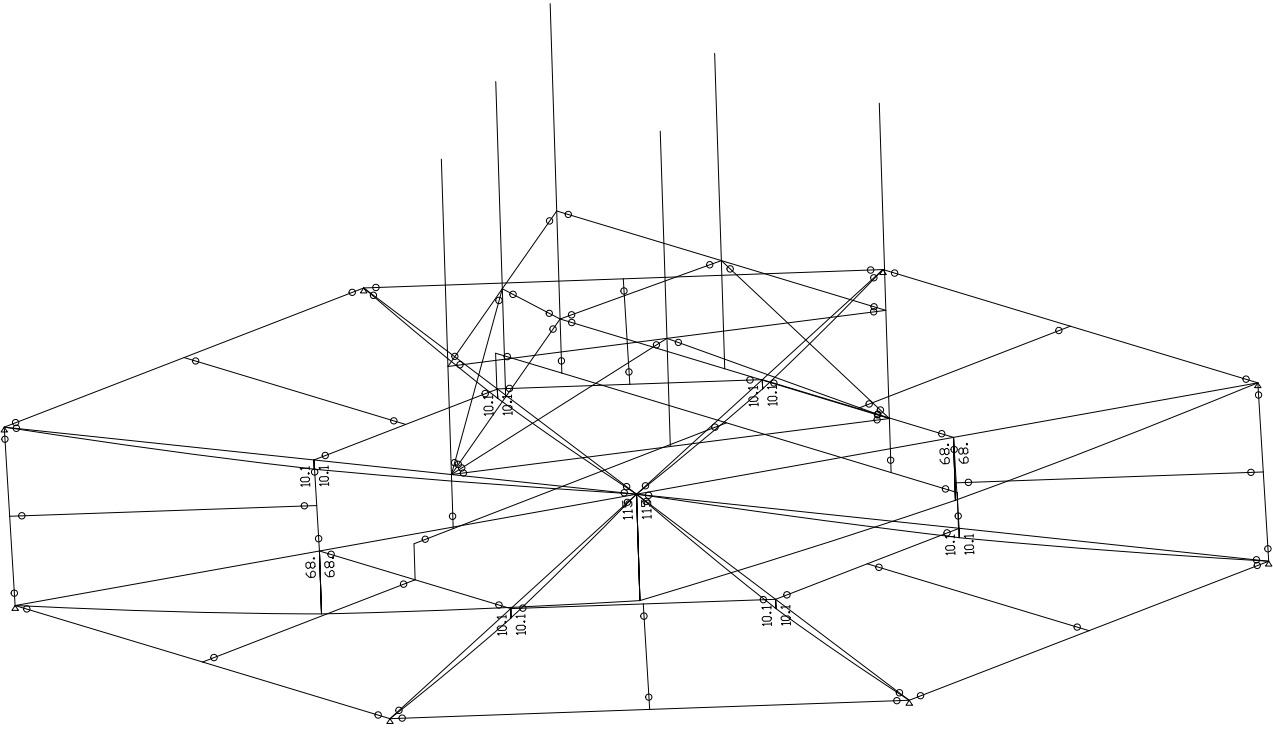
<i>Comb.</i>	
Existing	
1	1 * 1.00 + 2 * 1.00 + 3 * 1.00
Prop X2	
2	1 * 1.00 + 2 * 1.00 + 3 * 1.00 + 6 * 1.00
Prop -X2	
3	1 * 1.00 + 2 * 1.00 + 3 * 1.00 + 4 * 1.00
Prop X1	
4	1 * 1.00 + 2 * 1.00 + 3 * 1.00 + 5 * 1.00



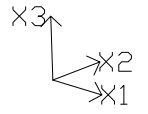
SCALE = 1:60

UNITS: kip*ft

DATE: 11/28/14



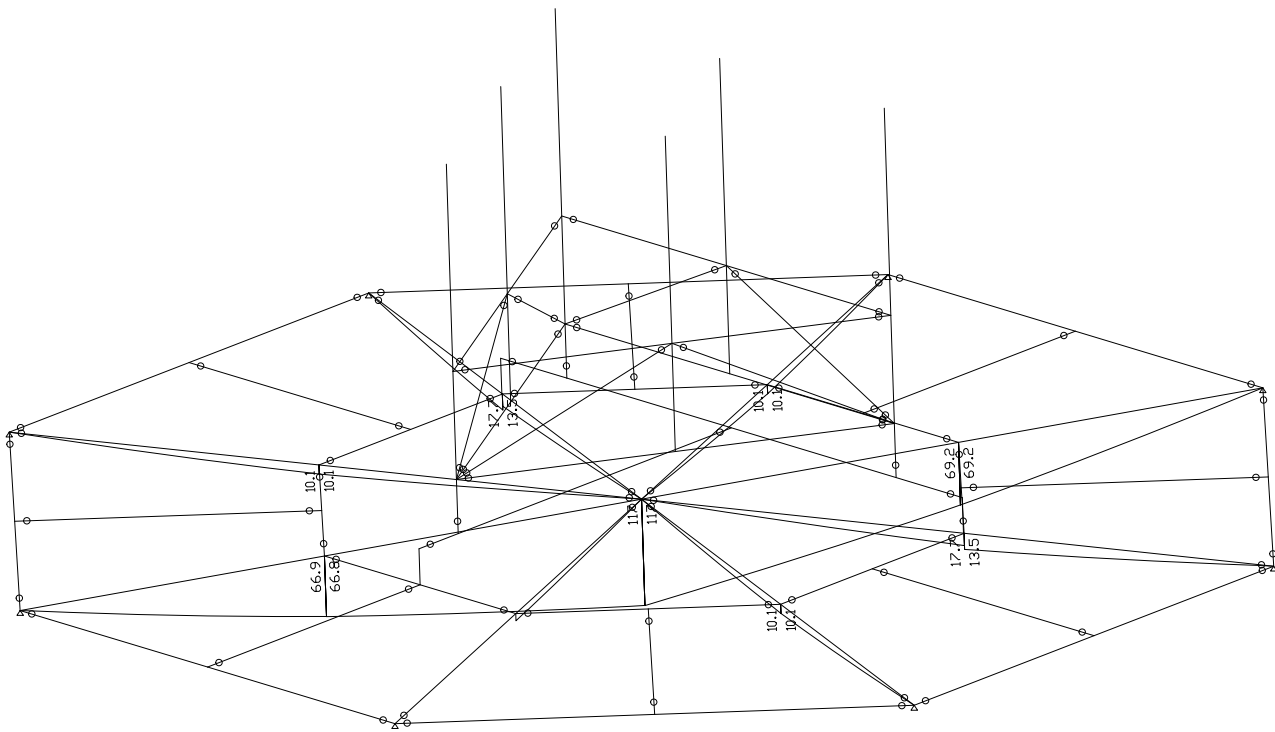
M2 MOMENT COMB. NO. 1 Existing Load



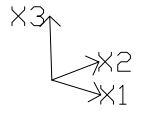
SCALE = 1:60

UNITS: kip*ft

DATE:11/28/14



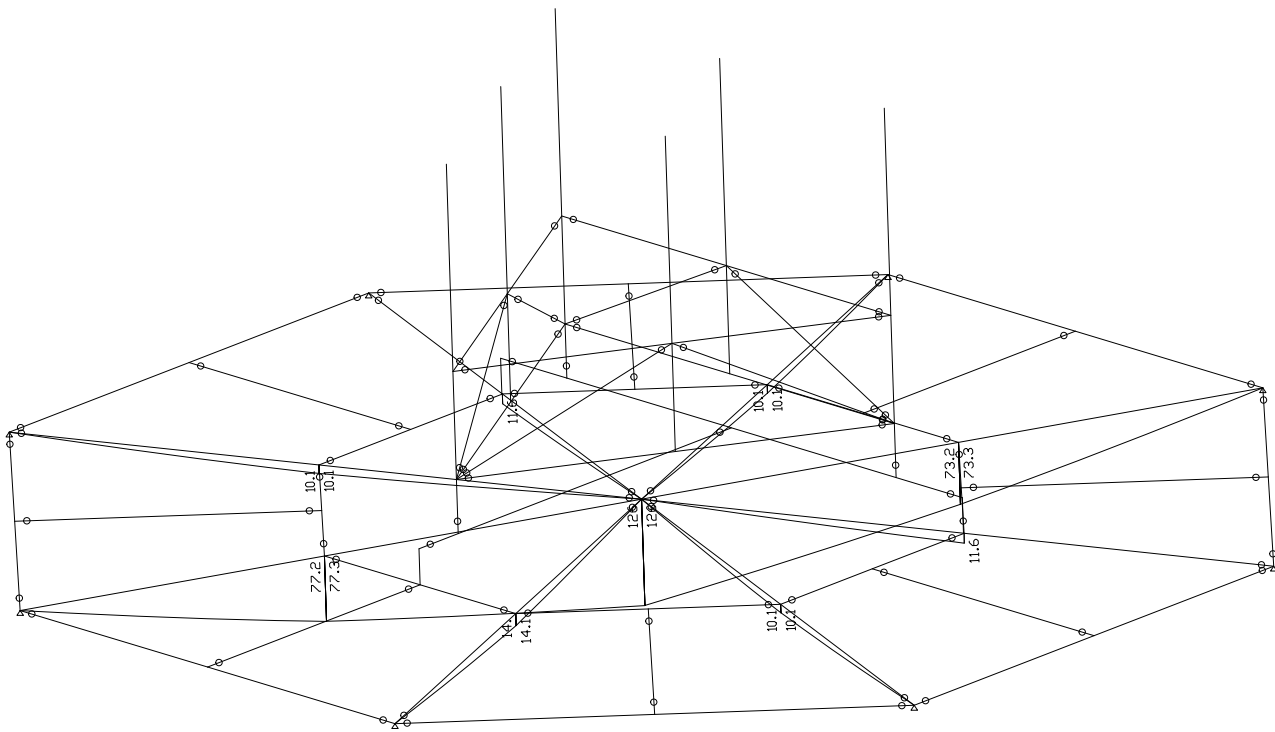
M2 MOMENT COMB. NO. 2 op X2 Wind



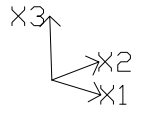
SCALE = 1:60

UNITS: kip*ft

DATE:11/28/14



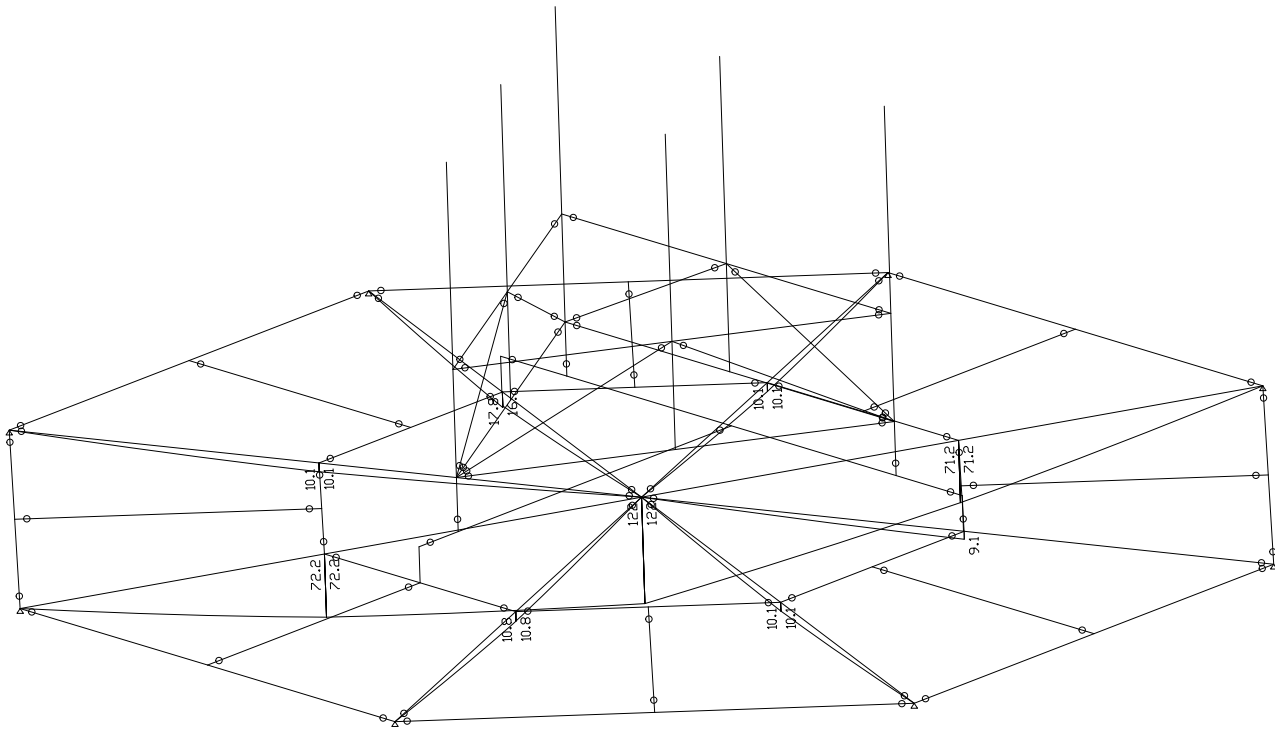
M2 MOMENT COMB. NO. 3 Prop -X2 Wind



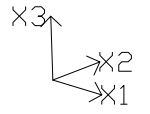
SCALE = 1:60

UNITS: kip*ft

DATE:11/28/14



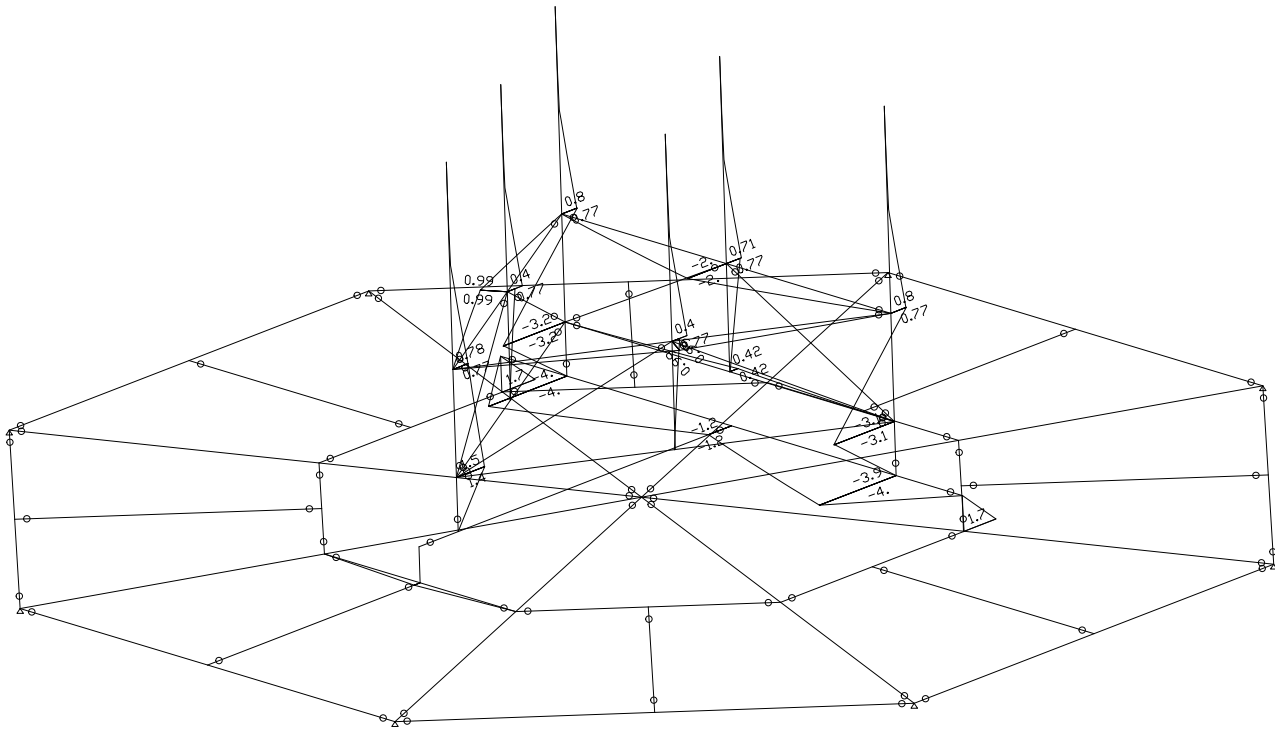
M2 MOMENT COMB. NO. 4 op X1 Wind



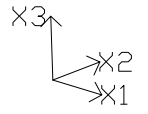
SCALE = 1:60

UNITS: kip*ft

DATE: 11/28/14



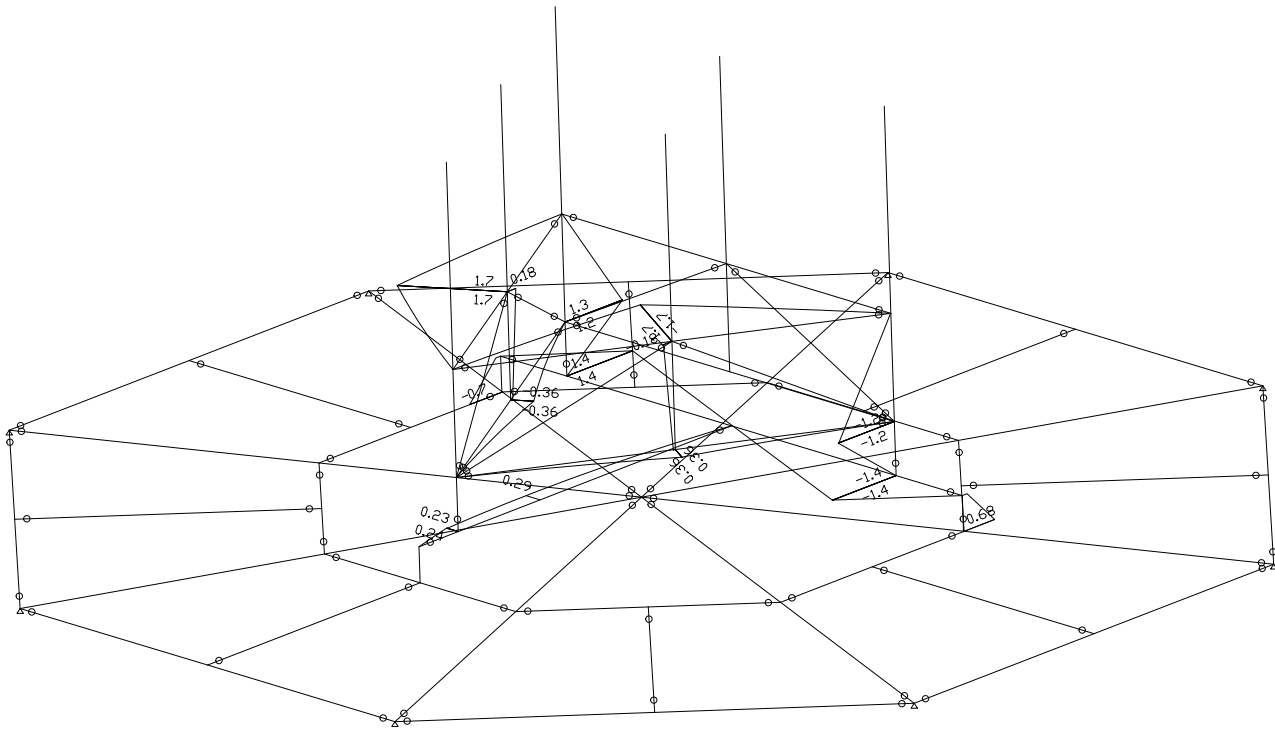
M3 MOMENT COMB. NO. 3 op -X2 Wind



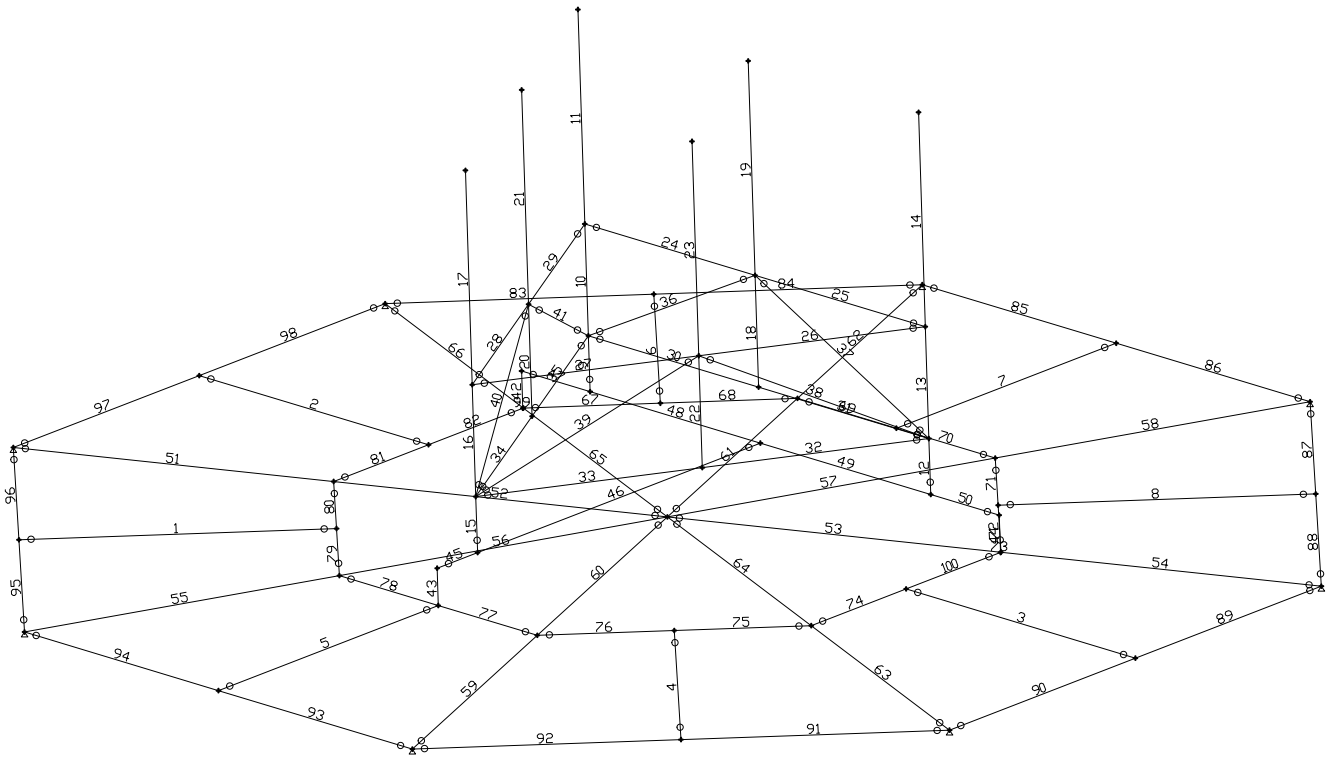
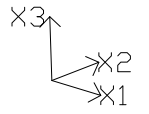
SCALE = 1:60

UNITS: kip*ft

DATE: 11/28/14



M3 MOMENT COMB. NO. 4 op X1 Wind



Prepared by:

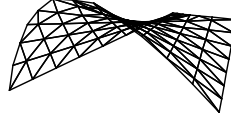
Results Summary Table

Bea	Section	Co	Defl L/	Slen	CAPACITY					Combined Axial+Mom
					Axial	Dir	Shea	Mom	LTB	
1	W 6x9	4		106	0.00	MJ	0.04	0.12	0.13	0.13
2	W 6x9	2	3035	106	0.00	MJ	0.04	0.12	0.13	0.13
3	W 6x9	2	3035	106	0.00	MJ	0.04	0.12	0.13	0.13
4	W 6x9	3	3035	106	0.00	MJ	0.04	0.12	0.13	0.13
5	W 6x9	3	3035	106	0.00	MJ	0.04	0.12	0.13	0.13
6	W 6x9	2	3035	106	0.00	MJ	0.04	0.12	0.13	0.13
7	W 6x9	4	3035	106	0.00	MJ	0.04	0.12	0.13	0.13
8	W 6x9	2	3035	106	0.00	MJ	0.04	0.12	0.13	0.13
45	W 8x21	3	5122	112	-0.02	MJ	0.06	0.07	0.07	0.08
						MI	0.00	0.03	0.00	
47	W 8x21	2	385	159	-0.11	MJ	0.05	0.10	0.12	0.61
						MI	0.03	0.40	0.00	
51	W 12x14	4	1029	144	-0.01	MJ	0.05	0.32	0.36	0.36
	W 12x14	2	696	144	-0.07	MJ	0.08	0.57	0.58	0.63
						MI	0.00	0.01	0.00	

Results Summary Table

Bea	Section	Co	Defl L/	Slen	CAPACITY					Combined Axial+Mom
					Axial	Dir	Shea	Mom	LTB	
55	W 24x55	3	655	81	0.00	MJ	0.07	0.52	0.52	0.52
59	W 12x14	3	765	144	-0.01	MJ	0.06	0.45	0.48	0.49
62	W 12x14	4	1028	144	-0.01	MJ	0.05	0.32	0.36	0.36
	W 12x14	3	1028	144	-0.01	MJ	0.05	0.32	0.36	0.36
66	W 12x14	2	645	144	-0.07	MJ	0.08	0.57	0.61	0.66
						MI	0.00	0.01	0.00	
68	W 6x9	3	3166	46	0.00	MJ	0.04	0.14	0.14	0.14
70	W 6x9	3	3176	46	0.00	MJ	0.04	0.14	0.14	0.14
72	W 6x9	3	3172	46	0.00	MJ	0.04	0.14	0.14	0.14
	W 8x10	3	9999	0	-0.02	MJ	0.10	0.11	0.11	0.15
						MI	0.19	0.03	0.00	
74	W 6x9	3	3199	46	-0.01	MJ	0.04	0.14	0.14	0.14
76	W 6x9	3	3173	46	-0.01	MJ	0.04	0.14	0.14	0.14
77	W 6x9	3	2785	46	-0.01	MJ	0.10	0.41	0.41	0.51
						MI	0.00	0.10	0.00	
78	W 6x9	3		46	-0.01	MJ	0.10	0.41	0.41	0.52
						MI	0.00	0.10	0.00	
80	W 6x9	3	3172	46	-0.01	MJ	0.04	0.14	0.14	0.14
82	W 6x9	3	3199	46	-0.01	MJ	0.04	0.14	0.14	0.14
85	W 8x10	3	9999	98	0.00	MJ	0.04	0.21	0.21	0.23
						MI	0.00	0.03	0.00	
99	W 8x10	3	9999	0	-0.02	MJ	0.11	0.11	0.11	0.15
						MI	0.19	0.03	0.00	

STRAP

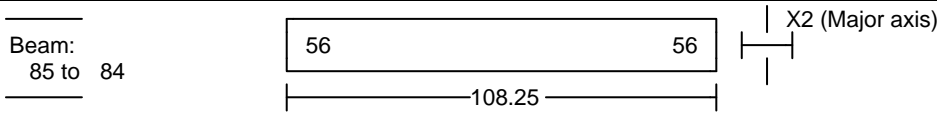


STRUCTURAL ANALYSIS PROGRAMS

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E-MAIL: strap@atir.com

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

- Kx = 1.00 - Ky = 1.00
- Allow. Slend.: 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

INTERMEDIATE SUPPORTS

L =	6.75	13.50	20.29	27.04	33.79	40.54	47.29	54.08	60.83	67.5
Lat.-Tors.	+ -	+ -	+ -	+ -	+ -	+ -	+ -	+ -	+ -	+ -
Compress.	X Y	X Y	X Y	X Y	X Y	X Y	X Y	X Y	X Y	X Y

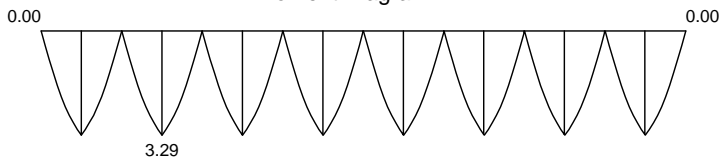
L =	74.33	81.08	87.88	94.62	101.38
Lat.-Tors.	+ -	+ -	+ -	+ -	+ -
Compress.	X Y	X Y	X Y	X Y	X Y

Section: W 8x10

$I_x = 30.80$ $I_y = 2.09in^4$ $Z_x = 8.87$ $Z_y = 1.66in^3$ $Area = 2.96$
 $hw = 7.89$ $bf = 3.94in$ $tw = 0.17$ $tf = 0.20in$
 $J = 0.04$ $C_w = 29.79in^6$

DESIGN COMBINATION = 3

M2 Moment Diagram



Moments at Intermediate Supports:

3.28 3.28 3.28 3.27 3.27 3.26 3.26 3.26
0.02 0.01 0.03 0.03 0.05 0.07 0.06

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Code: AISC-ASD

Prepared by:

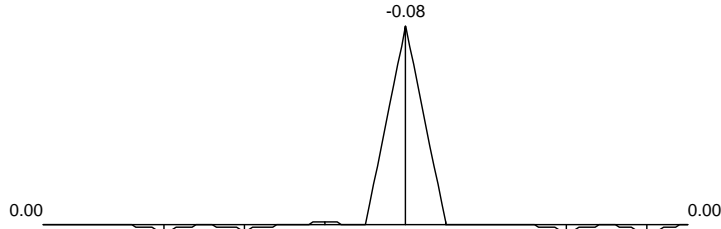
Date: 11/28/14

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

Max. AXIAL Force = 0.00 (compr.) Max. SHEAR Force = 0.68

M3 Moment Diagram



Moments at Intermediate Supports:

0.00 0.00 0.00 0.00 0.00 0.00 -0.08 0.00 0.00 0.00 0.00

Max. AXIAL Force = 0.00 (compr.) Max. SHEAR Force = 0.01

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios: Compac Non-Compact
 $d/t = 40.67 < 106.7$ 161.7 (Fy= 36.0 R = 0.000)
 $b/t = 9.62 < 10.8$ 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
M3 Moment (F6-1) without LTB	$\frac{M}{0.6M_n} < 1.00$	Z = 1.66	M = 0.08 Mn = 4.99	0.03
V3 Shear (G2.1.a)	$\frac{V_u}{V_n} < 1.00$ $V_n = 0.6 F_y A_w$	Av = 1.34	Vu = 0.68 Vn = 28.88	0.04
M2 Moment (F2-1) without LTB	$\frac{M}{0.6M_n} < 1.00$	Z = 8.87	M = 3.29 Mn = 26.65	0.21
Lateral Torsional Buckling (F2-2)	$\frac{M}{0.6M_n} < 1.00$	Lb = 6.75 Lp = 3.50 Lr = 10.37 Cb = 1.45	M = 3.29 Mn = 26.60 Mr = 16.42 Mp = 26.65	0.21
Critical Segment from 20.29 to 27.04 on +z flange Segment End Moments: 3.28 and 0.01				
Combined Forces (compress.) (H1-1b)	$\frac{P_r}{2\phi P_n} + \frac{M_{rx}}{\phi M_n x} + \frac{M_{ry}}{\phi M_n y} < 1.00$	Cmx = 1.00 Cmy = 1.00 Pex = 1259.14 Pey = 88.63 Mnx = 26.60 (0.00 + 0.21 + 0.03)	Mrx = 3.29 Mry = 0.08 B1x = 1.00 B1y = 1.00 Mny = 4.99	0.23
Critical Segment from 60.83 to 67.58				

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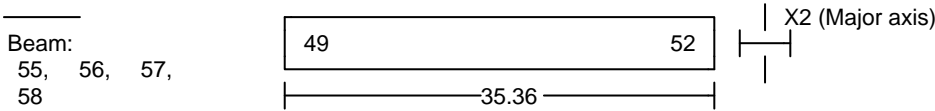
Code: AISC-ASD

Prepared by:

Date: 11/28/14

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

- Kx = 1.00 - Ky = 1.00
- Allow. Slend. : 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

INTERMEDIATE SUPPORTS

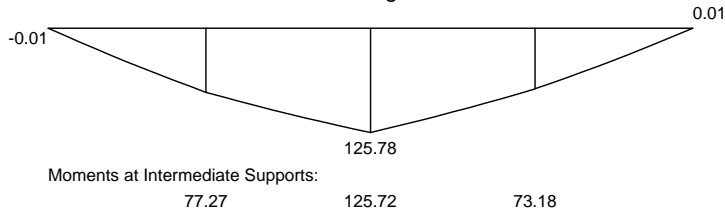
L =	8.67	17.67	26.71
Lat.-Tors.	+ -	+ -	+ -
	X Y	X Y	X Y

Section: W 24x55

Ix = 1350.00 Iy = 29.10in4 Zx = 134.0 Zy = 13.30in3 Area = 16.20
hw = 23.57 bf = 7.00in tw = 0.39 tf = 0.50in
J = 1.18 Cw = 3869.13in6

DESIGN COMBINATION = 3

M2 Moment Diagram



Max. AXIAL Force = 0.73 (tens.), -0.81 (compr.) Max. SHEAR Force = ***

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios: Compac Non-Compact
d/t= 54.78 < 106.2 161.5 (Fy= 36.0 R= 0.001)
b/t= 6.95 < 10.8 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (G2.1.a)	$V_u/V_n/1.5 < 1.00$ $V_n = 0.6 \cdot F_y \cdot A_w$	Av = 9.28	Vu = 9.66 Vn = 200.68	0.07
M2 Moment (F2-1) without LTB	$\frac{M}{0.6M_n} < 1.00$	Z = 134.00	M = 125.78 Mn = 402.48	0.52
Deflection	$\frac{\text{defl.}}{L/240} < 1.00$		defl = 0.64788	0.37

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Date: 11/28/14

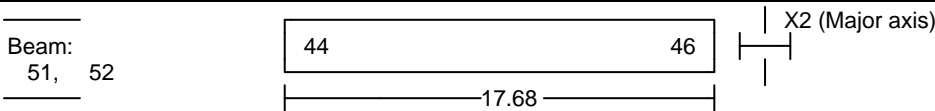
Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

DESIGN	EQUATION	FACTORS	VALUES	RESUL
Axial Force (E3-1)	$\frac{P_u}{0.6A_g F_{cr}} < 1.00$	(kL/r) _x =12 (kL/r) _y =81	P _u = 0.81 A _g = 16.20 F _{cr} = 25.52	0.00
Lateral Torsional Buckling (F2-2)	$\frac{M}{0.6M_n} < 1.00$	L _b = 9.04 L _p = 5.58 L _r = 16.80 C _b = 1.19	M = 125.78 M _n = 401.68 M _r = 240.84 M _p = 402.48	0.52
Critical Segment from 17.67 to 26.71 on +z flange Segment End Moments: 125.72 and 73.18				
Combined Forces (compress.) (H1-1b)	$\frac{P_r}{2\phi P_n} + \frac{M_{rx}}{\phi M_{nx}} + \frac{M_{ry}}{\phi M_{ny}} < 1.00$	C _{mx} = 1.00 C _{my} = 1.00 P _{ex} = 32340.17 P _{ey} = 709.80	M _{rx} = 125.79 M _{ry} = 0.00 B _{1x} = 1.00 B _{1y} = 1.00	0.52

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

- K_x = 1.00 - K_y = 1.00
- Allow. Slend. : 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

INTERMEDIATE SUPPORTS

L =	8.67
Lat.-Tors.	+ -
Compress.	X Y

Section: W 12x14

I_x = 88.60 I_y = 2.36in⁴ Z_x = 17.40 Z_y = 1.90in³ Area = 4.15
 h_w = 11.91 b_f = 3.97in t_w = 0.20 t_f = 0.22in
 J = 0.07 C_w = 78.20in⁶

DESIGN COMBINATION = 4

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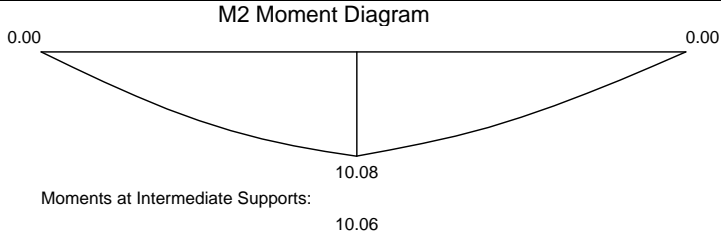
Code: AISC-ASD

Prepared by:

Date: 11/28/14

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



Max. AXIAL Force = -0.38 (compr.) Max. SHEAR Force = 1.72

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios: Compac Non-Compact
 $d/t = 54.08 < 105.8$ 161.3 ($F_y = 36.0$ $R = 0.003$)
 $b/t = 8.84 < 10.8$ 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (G2.1.a)	$\frac{V_u}{V_n} / 1.5 < 1.00$ $V_n = 0.6 * F_y * A_w$	$A_v = 2.39$	$V_u = 1.72$ $V_n = 51.71$	0.05
M2 Moment (F2-1) without LTB	$\frac{M}{0.6 M_n} < 1.00$	$Z = 17.40$	$M = 10.08$ $M_n = 52.26$	0.32
Deflection	$\frac{\text{defl.}}{L / 240} < 1.00$		$\text{defl} = 0.20626$	0.23
Axial Force (E3-1)	$\frac{P_u}{0.6 A_g F_{cr}} < 1.00$	$(kL/r)_x = 23$ $(kL/r)_y = 144$	$P_u = 0.38$ $A_g = 4.15$ $F_{cr} = 12.15$	0.01
Lateral Torsional Buckling (F2-2)	$\frac{M}{0.6 M_n} < 1.00$ Critical Segment from 8.67 to 17.68 on +z flange Segment End Moments: 10.06 and 0.00	$L_b = 9.01$ $L_p = 3.14$ $L_r = 9.30$ $C_b = 1.46$	$M = 10.08$ $M_n = 47.17$ $M_r = 31.28$ $M_p = 52.26$	0.36
Combined Forces (compress.) (H1-1b)	$\frac{P_r}{2\phi P_n} + \frac{M_{rx}}{\phi M_n x} + \frac{M_{ry}}{\phi M_n y} < 1.00$	$C_{mx} = 1.00$ $C_{my} = 1.00$ $P_{ex} = 2257.71$ $P_{ey} = 57.60$	$M_{rx} = 10.09$ $M_{ry} = 0.01$ $B_{1x} = 1.00$ $B_{1y} = 1.01$	0.36

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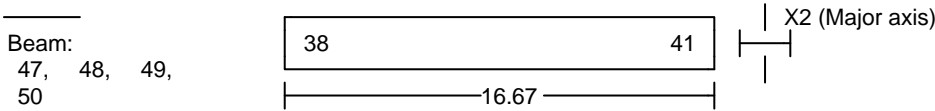
Code: AISC-ASD

Prepared by:

Date: 11/28/14

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

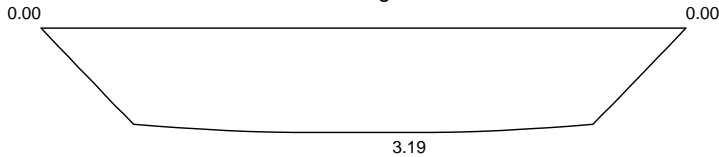
- Kx = 1.00 - Ky = 1.00
- Allow. Slend.: 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

Section: W 8x21

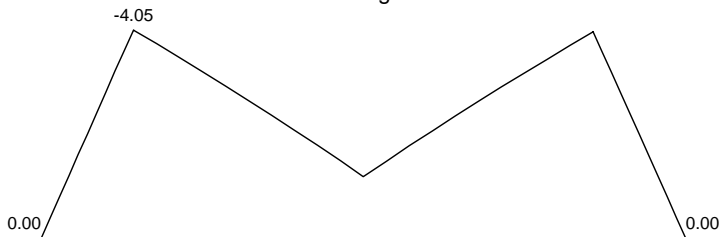
$I_x = 75.30$ $I_y = 9.77 \text{ in}^4$ $Z_x = 20.40$ $Z_y = 5.69 \text{ in}^3$ Area = 6.15
 $h_w = 8.28$ $b_f = 5.27 \text{ in}$ $t_w = 0.25$ $t_f = 0.40 \text{ in}$
 $J = 0.28$ $C_w = 148.96 \text{ in}^6$

DESIGN COMBINATION = 2

M2 Moment Diagram



Max. AXIAL Force = -3.93 (compr.) Max. SHEAR Force = 1.25
M3 Moment Diagram



Max. AXIAL Force = -3.93 (compr.) Max. SHEAR Force = 1.71

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios: Compac Non-Compact
 $d/t = 27.71 < 100.9$ 159.3 (Fy= 36.0 R= 0.018)
 $b/t = 6.56 < 10.8$ 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V2 Shear (G2.1.b-i)	$V_u/0.6V_n < 1.00$ $V_n = 0.6 \cdot F_y \cdot A_w$	$A_v = 4.23$	$V_u = 1.71$ $V_n = 91.74$	0.03
M3 Moment (F6-1) without LTB	$\frac{M}{0.6M_n} < 1.00$	$Z = 5.69$	$M = 4.05$ $M_n = 17.08$	0.40

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Code: AISC-ASD

Prepared by:

Date: 11/28/14

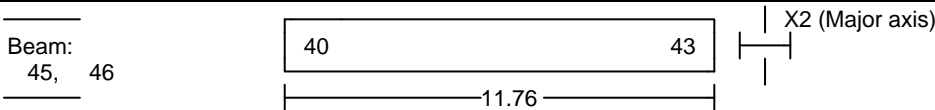
Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (G2.1.a)	$V_u/V_n < 1.00$ $V_n = 0.6 F_y A_w$	$A_v = 2.05$	$V_u = 1.25$ $V_n = 44.41$	0.04
M2 Moment (F2-1) without LTB	$\frac{M}{0.6 M_n} < 1.00$	$Z = 20.40$	$M = 3.19$ $M_n = 61.27$	0.09
Deflection	$\frac{\text{defl.}}{L / 240} < 1.00$		$\text{defl} = 0.51888$	0.62
Axial Force (E3-1)	$\frac{P_u}{0.6 A_g F_{cr}} < 1.00$	$(kL/r)_x = 57$ $(kL/r)_y = 159$	$P_u = 3.93$ $A_g = 6.15$ $F_{cr} = 9.96$	0.11
Lateral Torsional Buckling (F2-2)	$\frac{M}{0.6 M_n} < 1.00$ Critical Segment from 0.00 to 16.67 on +z flange Segment End Moments: 0.00 and 0.00	$L_b = 16.67$ $L_p = 5.25$ $L_r = 19.13$ $C_b = 1.02$	$M = 3.19$ $M_n = 42.96$ $M_r = 38.24$ $M_p = 61.27$	0.12
Combined Forces (compress.) (H1-1b)	$\frac{P_r}{2 \phi P_n} + \frac{M_{rx}}{\phi M_{nx}} + \frac{M_{ry}}{\phi M_{ny}} < 1.00$	$C_{mx} = 1.00$ $C_{my} = 1.00$ $P_{ex} = 544.54$ $P_{ey} = 69.98$	$M_{rx} = 3.22$ $M_{ry} = 4.45$ $B_{1x} = 1.01$ $B_{1y} = 1.10$	0.61

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

- $K_x = 1.00$ - $K_y = 1.00$
- Allow. Slend. : 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

Section: W 8x21

$I_x = 75.30$ $I_y = 9.77 \text{ in}^4$ $Z_x = 20.40$ $Z_y = 5.69 \text{ in}^3$ Area = 6.15
 $h_w = 8.28$ $h_f = 5.27 \text{ in}$ $t_w = 0.25$ $t_f = 0.40 \text{ in}$
 $J = 0.28$ $C_w = 148.96 \text{ in}^6$

DESIGN COMBINATION = 3

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Code: AISC-ASD

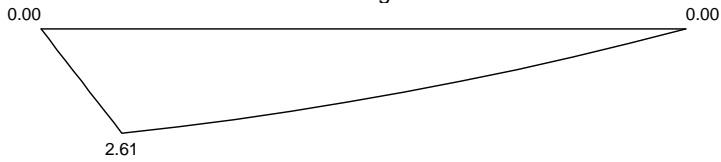
Prepared by:

Date: 11/28/14

Detailed Results Table

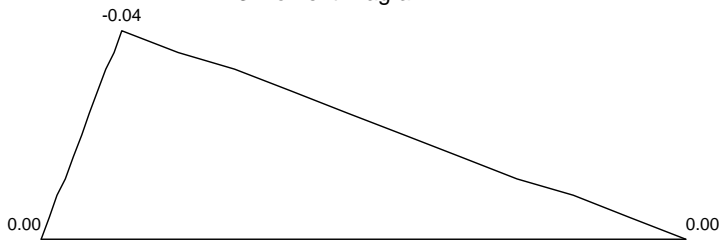
Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

M2 Moment Diagram



Max. AXIAL Force = -1.14 (compr.) Max. SHEAR Force = 1.79

M3 Moment Diagram



Max. AXIAL Force = -1.14 (compr.) Max. SHEAR Force = 0.03

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios: Compac Non-Compact
 d/t= 27.71 < 105.0 161.0 (Fy= 36.0 R = 0.005)
 b/t= 6.56 < 10.8 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
M3 Moment (F6-1) without LTB	$\frac{M}{0.6M_n} < 1.00$	Z = 5.69	M = 0.04 Mn = 17.08	0.00
V3 Shear (G2.1.a)	$\frac{V_u}{V_n} < 1.00$ Vn=0.6*Fy*Aw	Av = 2.05	Vu = 1.79 Vn = 44.41	0.06
M2 Moment (F2-1) without LTB	$\frac{M}{0.6M_n} < 1.00$	Z = 20.40	M = 2.61 Mn = 61.27	0.07
Deflection	$\frac{\text{defl.}}{L / 240} < 1.00$		defl = 0.02416	0.04
Axial Force (E3-1)	$\frac{P_u}{0.6A_g F_{cr}} < 1.00$	(kL/r)x =40 (kL/r)y =112	Pu = 1.14 Ag = 6.15 Fcr = 18.64	0.02
Lateral Torsional Buckling (F2-2)	$\frac{M}{0.6M_n} < 1.00$ Critical Segment from 0.00 to 11.76 on +z flange Segment End Moments: 0.00 and 0.00	Lb = 11.76 Lp = 5.25 Lr = 19.13 Cb = 1.38	M = 2.61 Mn = 61.15 Mr = 38.24 Mp = 61.27	0.07

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Prepared by:

Date: 11/28/14

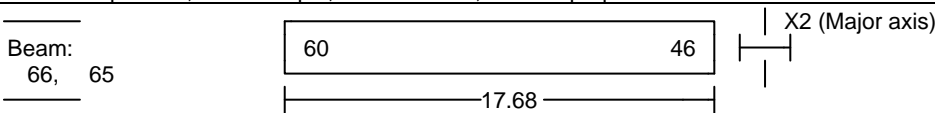
Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

DESIGN	EQUATION	FACTORS	VALUES	RESUL
Combined Forces (compress.) (H1-1b)	$\frac{Pr}{2\phi P_n} + \frac{M_{rx}}{\phi M_{nx}} + \frac{M_{ry}}{\phi M_{ny}} < 1.00$	Cmx = 1.00 Cmy = 1.00 Pex = 1105.76 Pey = 141.04	Mrx = 2.62 Mry = 0.04 B1x = 1.00 B1y = 1.01	0.08

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

- Kx = 1.00 - Ky = 1.00
- Allow. Slend. : 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

INTERMEDIATE SUPPORTS

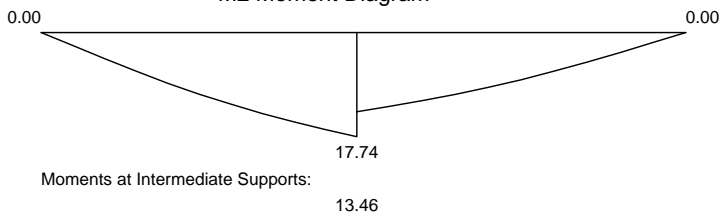
L =	8.67
Lat.-Tors.	+ -
Compress.	X Y

Section: W 12x14

Ix = 88.60 Iy = 2.36in⁴ Zx = 17.40 Zy = 1.90in³ Area = 4.15
hw = 11.91 bf = 3.97in tw = 0.20 tf = 0.22in
J = 0.07 Cw = 78.20in⁶

DESIGN COMBINATION = 2

M2 Moment Diagram



Max. AXIAL Force = 1.83 (tens.), -2.25 (compr.) Max. SHEAR Force = ***

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Code: AISC-ASD

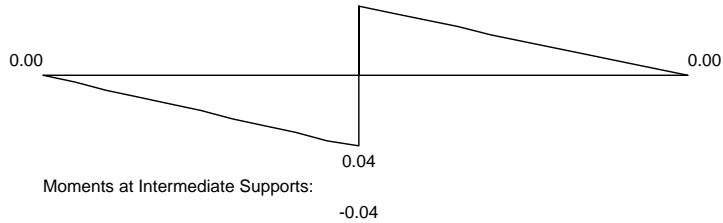
Prepared by:

Date: 11/28/14

Detailed Results Table

Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

M3 Moment Diagram



Max. AXIAL Force = 1.83 (tens.), -2.25 (compr.) Max. SHEAR Force = ***

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios: Compac Non-Compact
 $d/t = 54.08 < 101.8$ 159.7 ($F_y = 36.0$ $R = 0.015$)
 $b/t = 8.84 < 10.8$ 15.8

DESIGN	EQUATION	FACTORS	VALUES	RESUL
M3 Moment (F6-1) without LTB	$\frac{M}{0.6M_n} < 1.00$	$Z = 1.90$	$M = 0.04$ $M_n = 5.70$	0.01
V3 Shear (G2.1.a)	$\frac{V_u}{V_n} / 1.5 < 1.00$ $V_n = 0.6 F_y A_w$	$A_v = 2.39$	$V_u = 2.60$ $V_n = 51.71$	0.08
M2 Moment (F2-1) without LTB	$\frac{M}{0.6M_n} < 1.00$	$Z = 17.40$	$M = 17.74$ $M_n = 52.26$	0.57
Deflection	$\frac{\text{defl.}}{L / 240} < 1.00$		$\text{defl} = 0.30489$	0.34
Axial Force (E3-1)	$\frac{P_u}{0.6 A_g F_{cr}} < 1.00$	$(kL/r)_x = 23$ $(kL/r)_y = 144$	$P_u = 2.25$ $A_g = 4.15$ $F_{cr} = 12.15$	0.07
Lateral Torsional Buckling (F2-2)	$\frac{M}{0.6M_n} < 1.00$ Critical Segment from 8.67 to 17.68 on +z flange Segment End Moments: 13.46 and 0.00	$L_b = 9.01$ $L_p = 3.14$ $L_r = 9.30$ $C_b = 1.51$	$M = 17.74$ $M_n = 48.66$ $M_r = 31.28$ $M_p = 52.26$	0.61
Combined Forces (compress.) (H1-1b)	$\frac{P_r}{2\phi P_n} + \frac{M_{rx}}{\phi M_{nx}} + \frac{M_{ry}}{\phi M_{ny}} < 1.00$	$C_{mx} = 1.00$ $C_{my} = 1.00$ $P_{ex} = 2257.71$ $P_{ey} = 57.60$	$M_{rx} = 17.76$ $M_{ry} = 0.05$ $B_{1x} = 1.00$ $B_{1y} = 1.07$	0.66