T-Mobile

15 Commerce Way
Suite B
Norton, MA 02766

STRUCTURAL ANALYSIS
4PB0360A - HARBOR TERRACE

Address:
284 DANFORTH STREET
PORTLAND, ME 04102

Date:
25 NOVEMBER 2014

STATE OF MAINE
CLEMENT J. SALEK
No. 11793
LICENSED
PROFESSIONAL ENGINEER

CHAPPELL ENGINEERING ASSOCIATES, LLC
Civil • Structural • Land Surveying

R.K. Executive Centre ■ 201 Boston Post Road West ■ Suite 101 ■ Marlborough, MA 01752
t. 508.481.7400 ■ www.chappellengineering.com ■ f. 508.481.7406

CHAPPELL ENGINEERING ASSOCIATES, LLC
Civil • Structural • Land Surveying

November 26, 2014

T-Mobile
1 International Blvd
Suite 800
Mahwah, NJ 07495

RE:

Candidate Number	4PB0360A
Candidate Name	Harbor Terrace
Candidate Address	284 Danforth Street, 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 frames at the above-referenced location. T-Mobile proposes to install an elevated steel frame on the existing roof which will support the proposed T-Mobile equipment cabinets. Three (3) roof-mounted antenna frames are being proposed to support the proposed T-Mobile antennas.

The existing roof deck consists of SIP decking laid over open web steel joists. 16H6's are located under the proposed alpha and gamma sector frames. 16H5's are located under the proposed beta sector frame. The steel joists span to and are supported by a steel beam and column grid as shown in the construction drawings. A stairwell enclosure exists under the proposed beta sector frame, further supporting the open web joists.

T-Mobile's antenna array will consist of 3 sectors, each supporting 3 panel antennas (3 per sector, total of 9). The proposed antenna frames are to be secured with solid concrete ballast blocks laid in the ballast trays of the antenna frames. The proposed ballast weights being considered are shown on our enclosed construction drawings.

The proposed T-Mobile equipment support frame tie into the existing roof support beams spanning between columns J3-K3 and from J2-K2. The connection details for the equipment frames are 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

STATE OF MAINE
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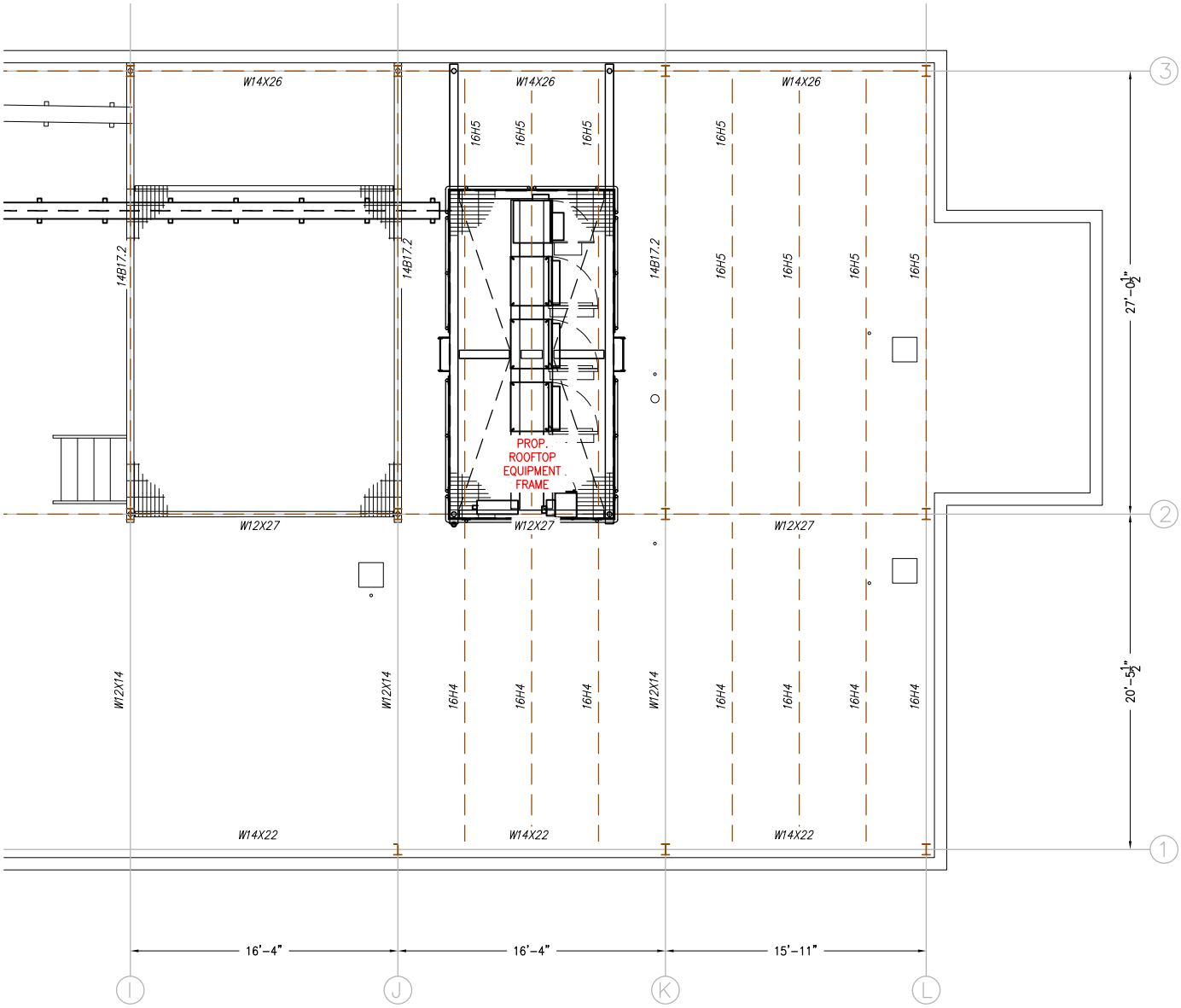
Clement J Salek, P.E.
CJS/cjs

CHAPPELL ENGINEERING ASSOCIATES, LLC Civil · Structural · Land Surveying	R.K. EXECUTIVE CENTRE 201 BOSTON POST RD WEST MARLBOROUGH MA MA 01742 P. (508) 481-7400 F. (508) 481-7406 www.chappellengineering.com	TITLE: Harbor Terrace, 284 Danforth Street, Portland, ME 04102			PAGE:
		PROJECT #: 4PB0360A (1424.004)			
		DATE: NOVEMBER 26, 2014	BY: CJS	CHK: JMF	



PARTIAL ROOF PLAN (1 / SDC)

SCALE: 1"=10'

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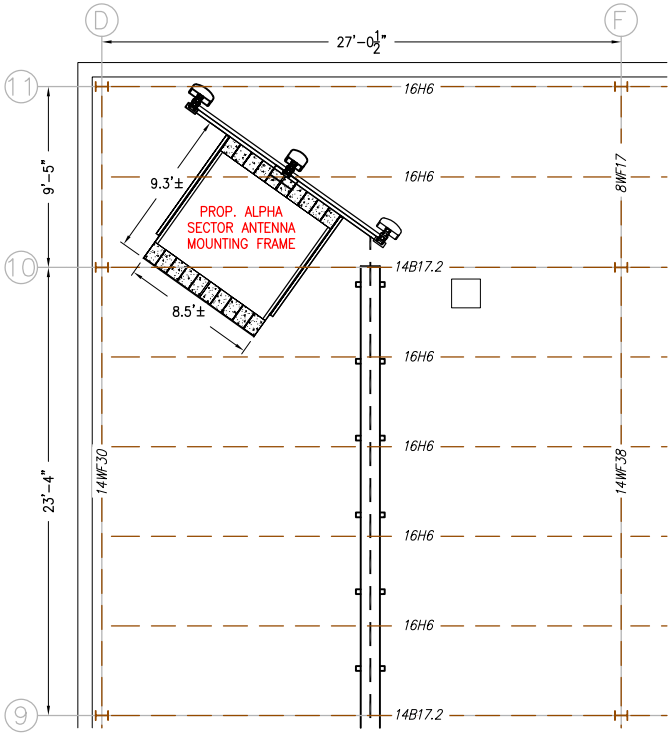
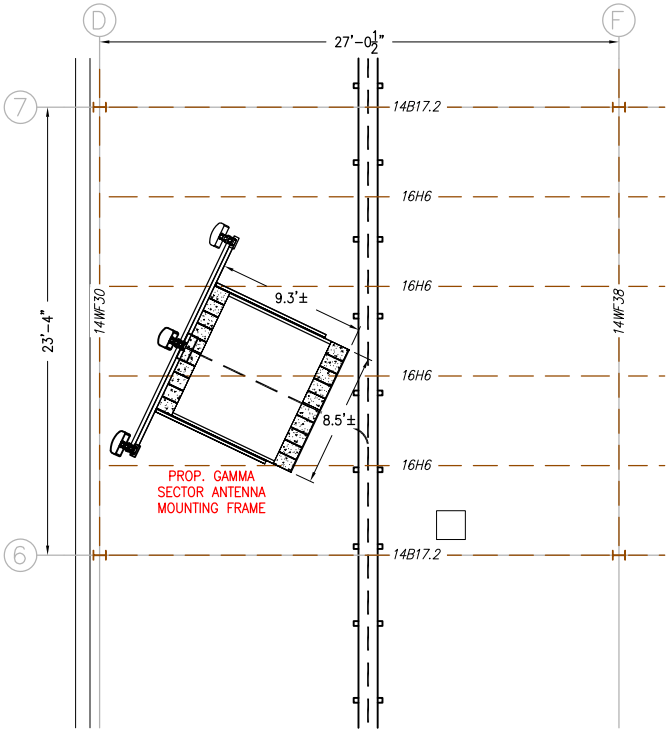
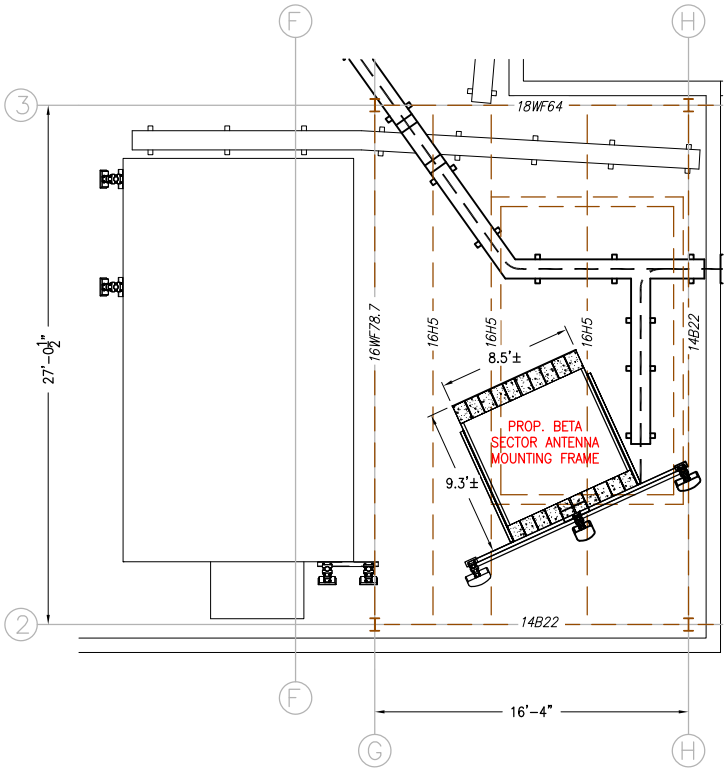


ALPHA SECTOR FRAME (1 / SDC)
SCALE: 1"=10'



GAMMA SECTOR FRAME (2 / SDC)
SCALE: 1"=10'



BETA SECTOR FRAME (3 / SDC)
SCALE: 1"=10'

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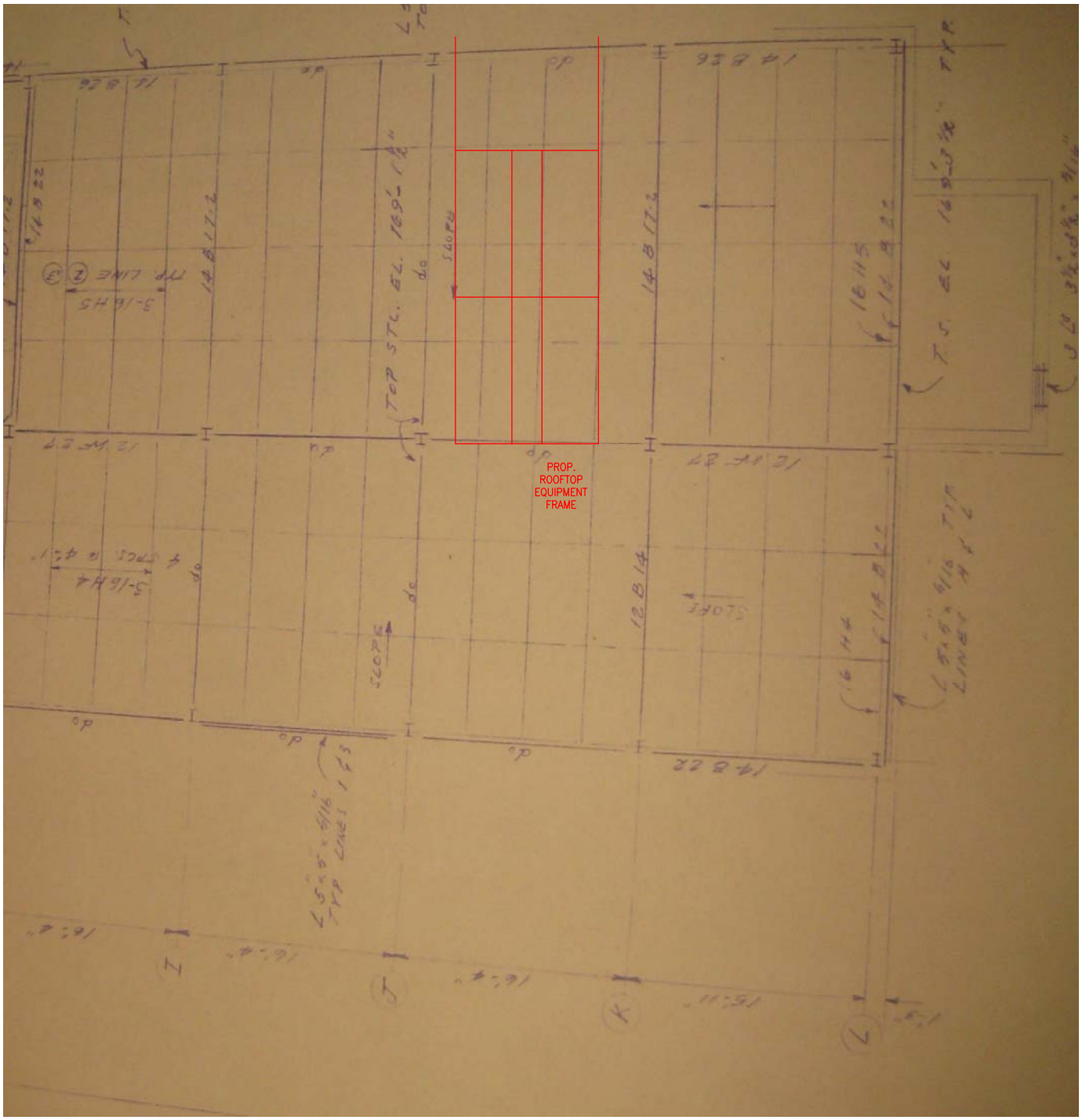

PARTIAL ROOF PLAN (1 / SDC)
SCALE: 1"=10'

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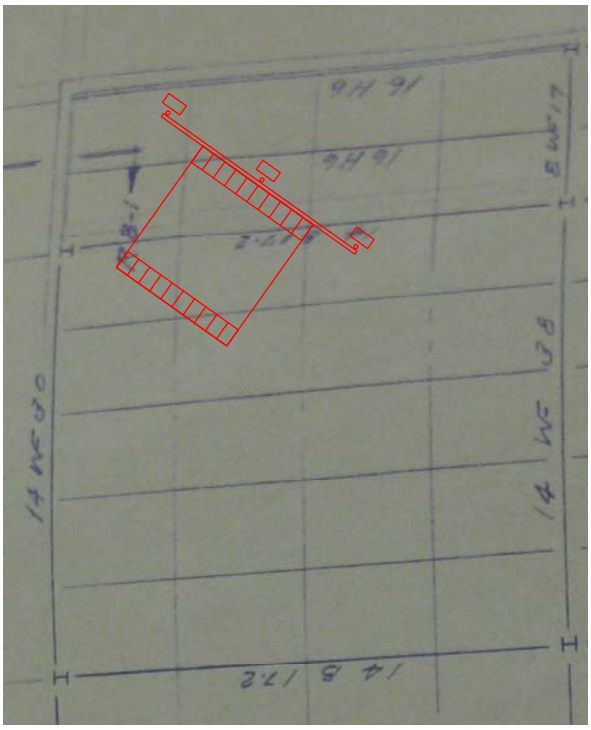
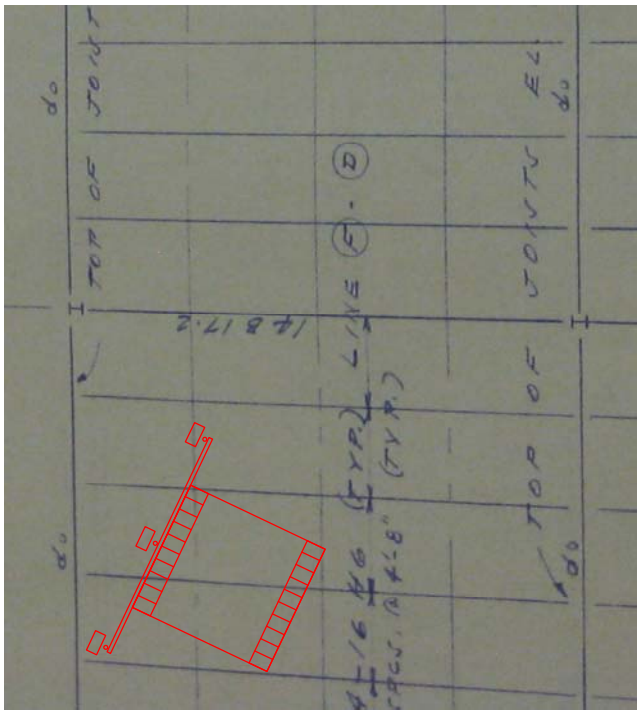
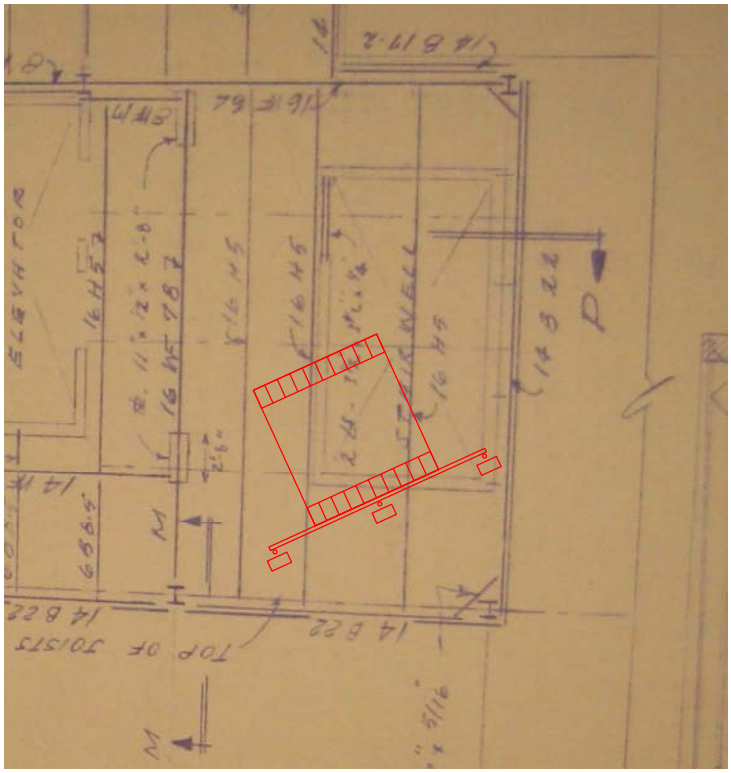


ALPHA SECTOR FRAME (1 / SDC)
SCALE: 1"=10'



GAMMA SECTOR FRAME (2 / SDC)
SCALE: 1"=10'



BETA SECTOR FRAME (3 / SDC)
SCALE: 1"=10'

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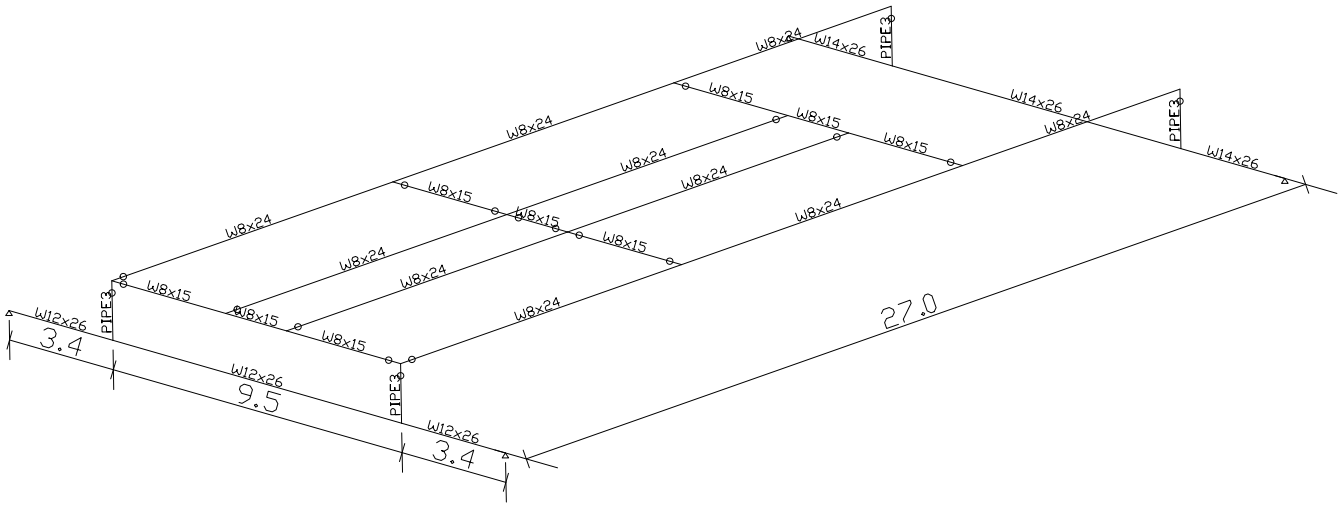
SCALE = 1:55

DATE:11/26/14



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

```
/ BEAM LOADS
/ BEAM LOADS
SELF   X3 -1. B 1 TO 29
/ BEAM LOADS
DIST GL FX3 -0.025 B 2 16 15
DIST GL FX3 -0.014 B 1 14 29
/ END

F O R C E   S U M M A T I O N
FX1=0.
FX2=0.
FX3=-4.2088
```

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

```
/ GLOBAL LOADS
DIST FX3 -0.01 PLANE 3.4167 0. 1.5    3.4167 19.5 1.5    12.917 19.5
1.5    PT 0. 9.5 BEAMS
/ END

F O R C E   S U M M A T I O N
FX1=0.
FX2=0.
FX3=-1.8525
```

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

```
/ GLOBAL LOADS
/ GLOBAL LOADS
DIST FX3 -0.095 PLANE 6.9167 9. 1.5    6.9167 12. 1.5    9.4167 12. 1.5 PT
0. 2.5 BEAMS
DIST FX3 -0.095 PLANE 6.9167 8. 1.5    9.4167 8. 1.5    9.4167 5. 1.5    PT
0. 3. BEAMS
DIST FX3 -0.094 PLANE 6.9167 12.5 1.5    6.9167 15.5 1.5    9.4167 15.5
1.5    PT 0. 2.5 BEAMS
DIST FX3 -0.33 PLANE 6.9167 16. 1.5    6.9167 19. 1.5    9.4167 19. 1.5 PT
0. 2.5 BEAMS
/ END

F O R C E   S U M M A T I O N
FX1=0.
FX2=0.
FX3=-4.605
```

Load no. 4: Snow Load (units - kips ft.)

```
/ BEAM LOADS
DIST GL FX3 -0.75 B 2 16 15
DIST GL FX3 -0.43 B 1 14 29
/ END STATIC

F O R C E   S U M M A T I O N
FX1=0.
FX2=0.
FX3=-19.273
```

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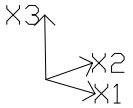
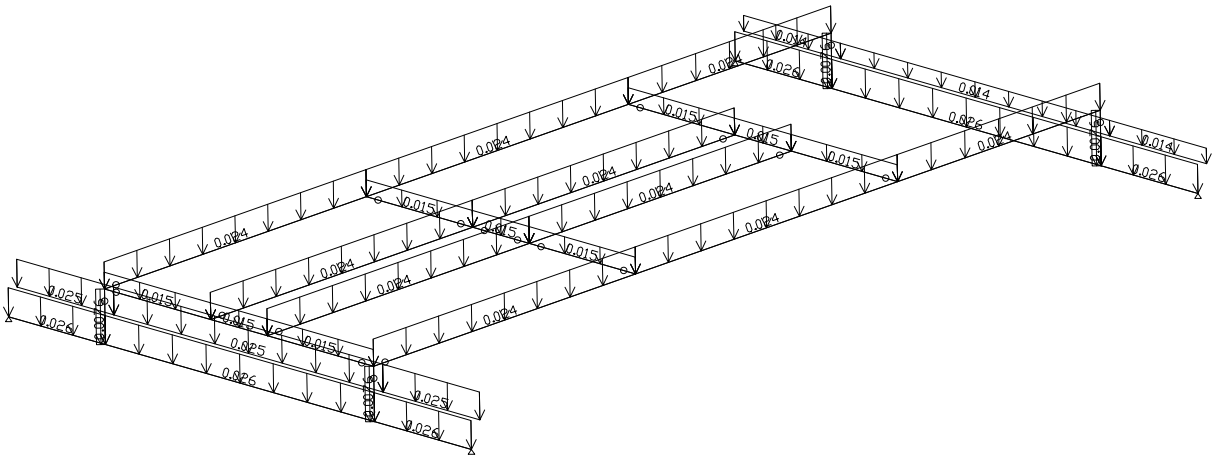
Load 1: Selfweight



SCALE = 1:59	UNITS: kip ft	DATE:11/26/14



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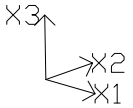
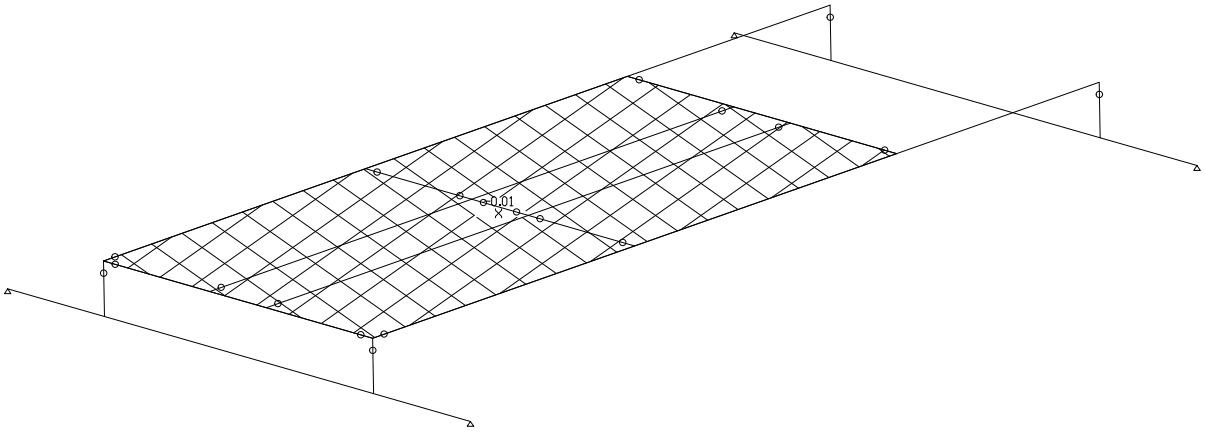
Load 2: Grating Load



SCALE = 1:59	UNITS: kip ft	DATE:11/26/14



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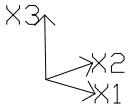
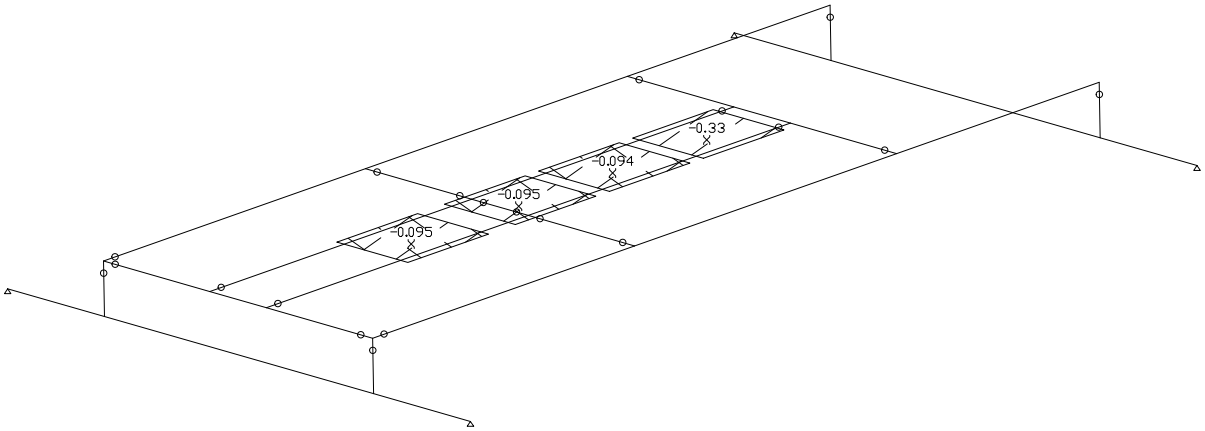
Load 3: Cabinet Loads



SCALE = 1:59	UNITS: kip ft	DATE:11/26/14



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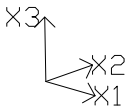
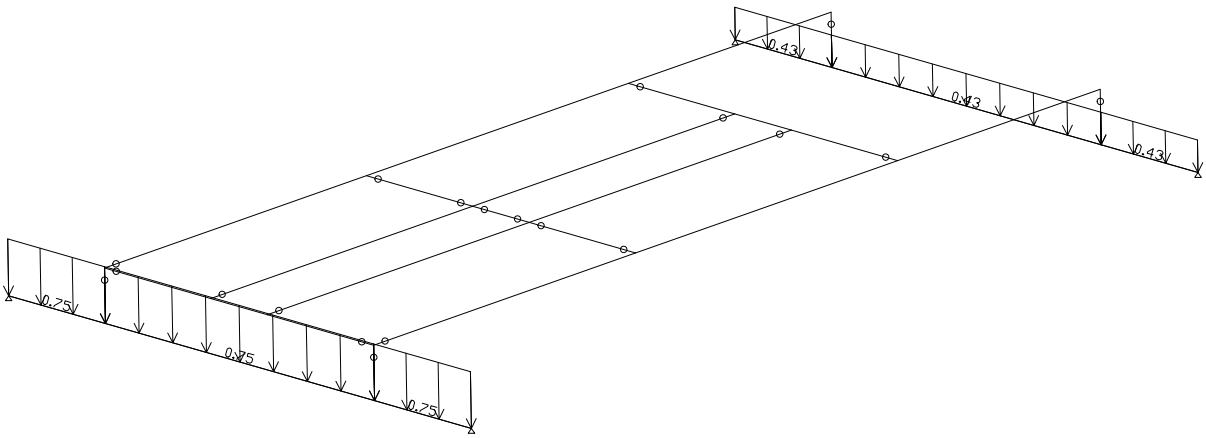
Load 4: Snow Load



SCALE = 1:59	UNITS: kip ft	DATE:11/26/14



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Date: 11/26/14

COMBINATIONS TABLE

Comb.	
Existing 1	4 * 1.00
Proposed 2	1 * 1.00 + 2 * 1.00 + 3 * 1.00 + 4 * 1.00

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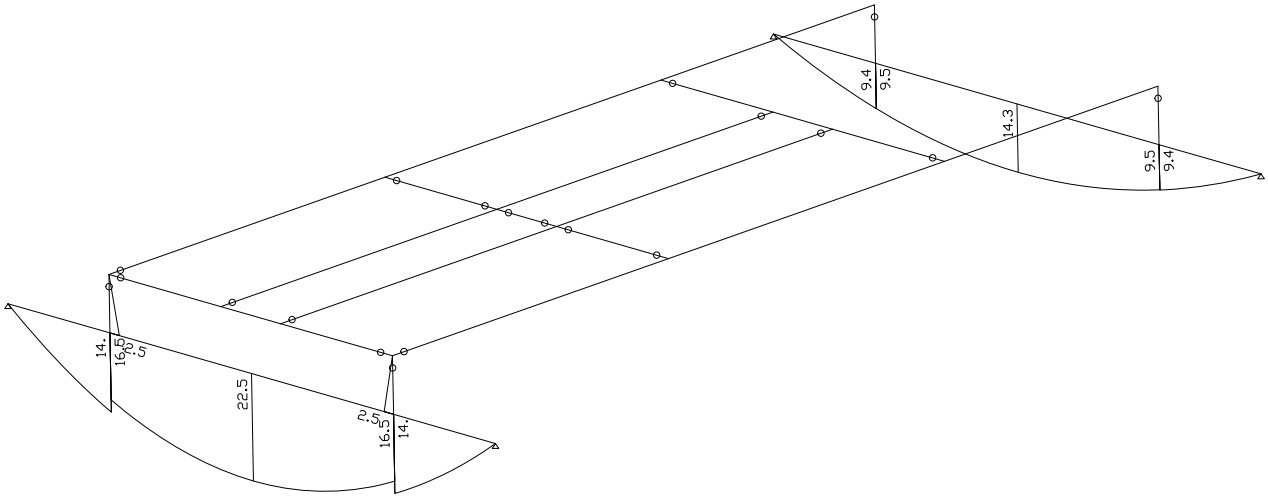
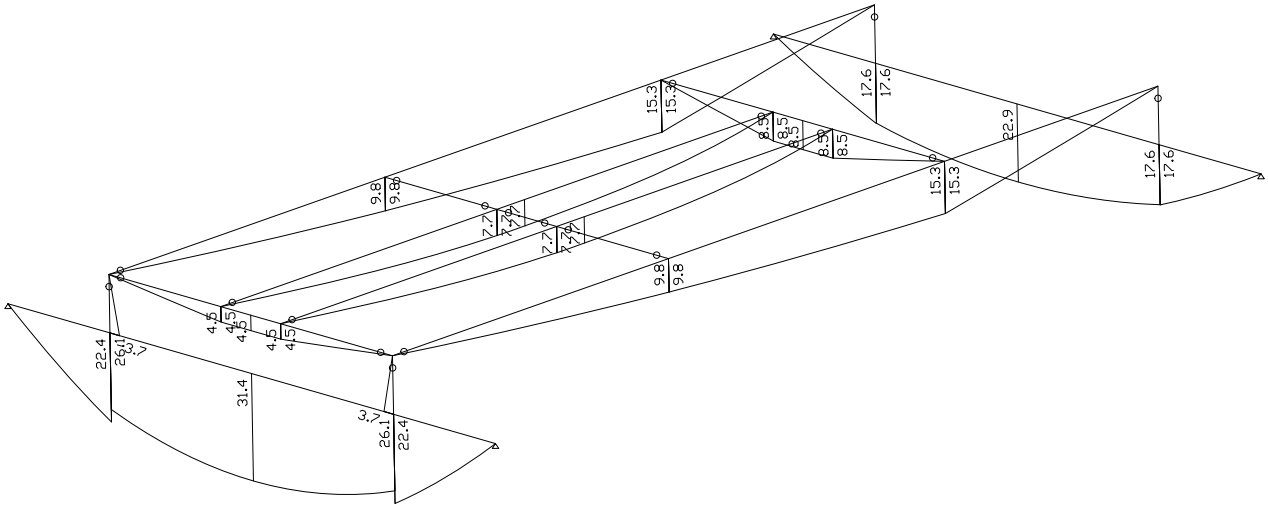
SCALE = 1:56 | UNITS: kip*ft | DATE:11/26/14



M2 MOMENT COMB. NO. 1 Existing Load

Harbor Terrace Portland - TMobile

SCALE = 1:56	UNITS: kip*ft	DATE:11/26/14



M2 MOMENT COMB. NO. 2 oposed Load

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	Page:
Prepared by:	**Date:** 11/26/14

Results Summary Table

Bea	Section	Co	Defl L/	Slen	CAPACITY Axial	Dir	Shea	Mom	LTB	Combined Axial+Mom
1	W 14x26	2		182	0.00	MJ	0.12	0.32	0.61	0.62
						MI	0.00	0.01	0.00	
2	W 12x26	2	713	130	-0.02	MJ	0.22	0.47	0.62	0.63
3	W 8x24	2	447	73	0.00	MJ	0.08	0.37	0.37	0.38
						MI	0.00	0.02	0.00	
4	W 8x24	2	447	73	0.00	MJ	0.08	0.37	0.37	0.38
						MI	0.00	0.02	0.00	
5	W 8x24	2	1043	73	0.00	MJ	0.08	0.18	0.18	0.18
6	W 8x24	2	1043	73	0.00	MJ	0.08	0.18	0.18	0.18
7	W 8x15	2	1150	51	0.00	MJ	0.08	0.35	0.35	0.35
8	PIPE 3	2		15	-0.05	MJ	0.00	0.02	0.02	0.04
9	PIPE 3	2	9999	15	-0.05	MJ	0.00	0.02	0.02	0.04
10	PIPE 3	2	1721	15	-0.05	MJ	0.17	0.90	0.90	0.93
11	PIPE 3	2	1721	15	-0.05	MJ	0.17	0.90	0.90	0.93
12	W 8x15	2	2144	51	-0.03	MJ	0.04	0.19	0.19	0.20
13	W 8x15	2	9999	51	0.00	MI	0.00	0.00	0.00	0.00
24	W 8x15	2	9999	27	0.00	MI	0.00	0.00	0.00	0.00
28	W 8x15	2	9999	51	0.00	MI	0.00	0.00	0.00	0.00

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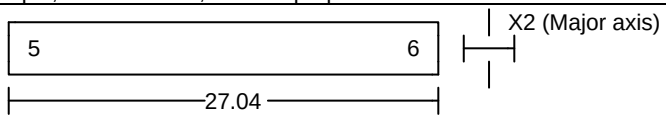
Harbor Terrace Portland - TMobile_	**Code:** AISC-ASD
	Page:
Prepared by:	**Date:** 11/26/14

Detailed Results Table

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

Beam:
3, 18, 17



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 =	9.75	19.50
Lat.-Tors.	+ -	+ -
Compress.	X Y	X Y

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

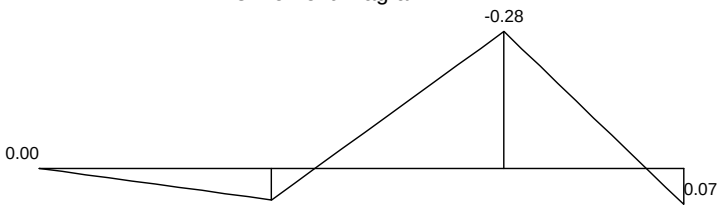
DESIGN COMBINATION = 2

M2 Moment Diagram

Moments at Intermediate Supports:
9.81 15.27

Max. AXIAL Force = 0.00 (tens.) Max. SHEAR Force = 2.12

M3 Moment Diagram



Moments at Intermediate Supports:
0.06 -0.28

Max. AXIAL Force = 0.00 (tens.) Max. SHEAR Force = 0.05

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Detailed Results Table

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

SECTION CLASSIFICATION: *** COMPACT ***

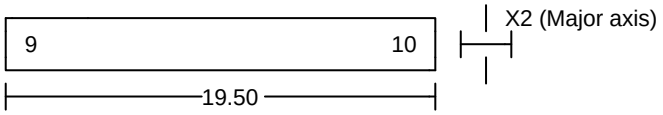
Limiting Ratios:	Compac		Non-Compact	
d/t= 25.90	<	106.7	161.7	(Fy= 36.0 R = 0.000)
b/t= 8.09	<	10.8	15.8	

DESIGN	EQUATION	FACTORS	VALUES	RESUL
M3 Moment (F6-1) without LTB	M / 0.6Mn < 1.00	Z = 8.57	M = 0.28 Mn = 25.73	0.02
V3 Shear (G2.1.a)	Vu/Vn/1.5<1.00 Vn=0.6*Fy*Aw	Av = 1.94	Vu = 2.12 Vn = 41.86	0.08
M2 Moment (F2-1) without LTB	M / 0.6Mn < 1.00	Z = 23.20	M = 15.27 Mn = 69.69	0.37
Deflection	defl. / L / 240 < 1.00		defl = 0.72558	0.54
Lateral Torsional Buckling (F2-2)	M / 0.6Mn < 1.00 Critical Segment from 9.75 to 19.50 on +z flange Segment End Moments: 9.81 and 15.27	Lb = 9.75 Lp = 6.70 Lr = 24.99 Cb = 1.14	M = 15.27 Mn = 69.55 Mr = 43.91 Mp = 69.69	0.37
Combined Forces (compress.) (H1-1b)	Pr / 2ϕPn + Mrx / ϕMnx + Mry / ϕMny < 1.00	Cmx = 1.00 Cmy = 1.00 Pex = 1761.76 Pey = 382.17	Mrx = 15.27 Mry = 0.28 B1x = 1.00 B1y = 1.00	0.38

Detailed Results Table

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

Beam:
5, 23



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

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Detailed Results Table

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

INTERMEDIATE SUPPORTS

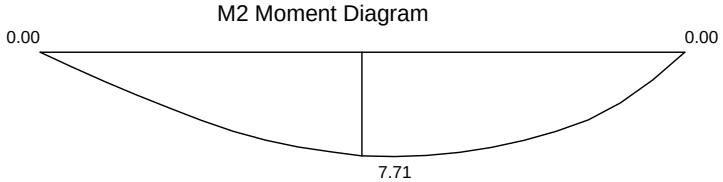
L =	9.75
Lat.-Tors.	+ -
Compress.	X Y

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 = 2

M2 Moment Diagram



Moments at Intermediate Supports:

7.66

Max. AXIAL Force = 0.00 (compr.) Max. SHEAR Force = 2.15

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)	Vu/Vn/1.5<1.00 Vn=0.6*Fy*Aw	Av = 1.94	Vu = 2.15 Vn = 41.86	0.08
M2 Moment (F2-1) without LTB	M / 0.6Mn < 1.00	Z = 23.20	M = 7.71 Mn = 69.69	0.18
Deflection	defl. / L / 240 < 1.00		defl = 0.22432	0.23
Lateral Torsional Buckling (F2-2)	M / 0.6Mn < 1.00 Critical Segment from 9.75 to 19.50 on +z flange Segment End Moments: 7.66 and 0.00	Lb = 9.75 Lp = 6.70 Lr = 24.99 Cb = 1.19	M = 7.71 Mn = 69.55 Mr = 43.91 Mp = 69.69	0.18

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Detailed Results Table

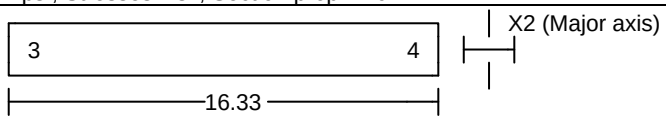
Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

DESIGN	EQUATION	FACTORS	VALUES	RESUL
Combined Forces (compress.) (H1-1b)	$\frac{Pr}{2\phi Pn} + \frac{Mrx}{\phi Mnx} + \frac{Mry}{\phi Mny} < 1.00$	Cmx = 1.00 Cmy = 1.00 Pex = 1761.76 Pey = 382.17	Mrx = 7.71 Mry = 0.00 B1x = 1.00 B1y = 1.00	0.18

Detailed Results Table

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

Beam:
2, 16, 15



CONSTRAINTS

- Sections : Check
- Steel Grade: A36

DESIGN DATA

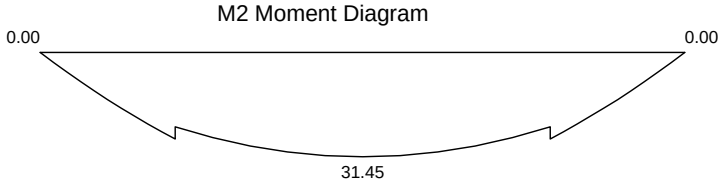
- Kx = 1.00 - Ky = 1.00
- Allow. Slend. : 200 (compr.) 300 (tens.)
- Allowable Deflection : 1/240
- Tension Area Reduction Factor : 1.00
- Building type : Unbraced

Section: W 12x26

Ix = 204.00 Iy = 17.30in4 Zx = 37.20 Zy = 8.17in3 Area = 7.66
hw = 12.22 bf = 6.49in tw = 0.23 tf = 0.38in
J = 0.30 Cw = 607.00in6

DESIGN COMBINATION = 2

M2 Moment Diagram



Max. AXIAL Force = 1.03 (tens.), -1.43 (compr.) Max. SHEAR Force =

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios:		Compac	Non-Compact	
d/t= 47.53	<	105.0	161.0	(Fy= 36.0 R = 0.005)
b/t= 8.49	<	10.8	15.8	

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Harbor Terrace Portland - TMobile_	**Code:** AISC-ASD
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Detailed Results Table

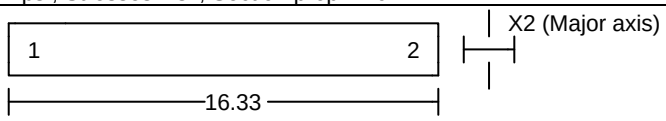
Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (G2.1.a)	Vu/Vn/1.5<1.00 Vn=0.6*Fy*Aw	Av = 2.79	Vu = 9.00 Vn = 60.35	0.22
M2 Moment (F2-1) without LTB	M / 0.6Mn < 1.00	Z = 37.20	M = 31.45 Mn = 111.73	0.47
Deflection	defl. / L / 240 < 1.00		defl = 0.27495	0.34
Axial Force (E3-1)	Pu / 0.6AgFcr < 1.00	(kL/r)x =38 (kL/r)y =130	Pu = 1.43 Ag = 7.66 Fcr = 14.83	0.02
Lateral Torsional Buckling (F2-2)	M / 0.6Mn < 1.00 Critical Segment from 0.00 to 16.33 on +z flange Segment End Moments: 0.00 and 0.00	Lb = 16.33 Lp = 6.26 Lr = 18.39 Cb = 1.09	M = 31.45 Mn = 84.37 Mr = 70.19 Mp = 111.73	0.62
Combined Forces (compress.) (H1-1b)	Pr / 2ϕPn + Mrx / ϕMnx + Mry / ϕMny < 1.00	Cmx = 1.00 Cmy = 1.00 Pex = 1524.57 Pey = 130.27	Mrx = 31.49 Mry = 0.00 B1x = 1.00 B1y = 1.02	0.63

Detailed Results Table

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

Beam:
1, 14, 29



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 14x26

Ix = 245.00 Iy = 8.91in4 Zx = 40.20 Zy = 5.54in3 Area = 7.69
hw = 13.91 bf = 5.02in tw = 0.26 tf = 0.42in
J = 0.36 Cw = 402.18in6

DESIGN COMBINATION = 2

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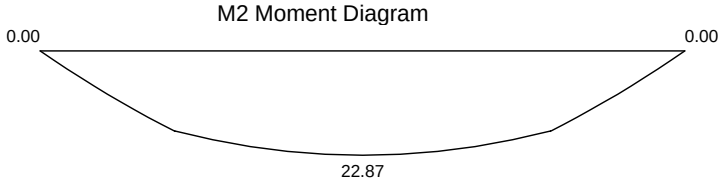
Harbor Terrace Portland - TMobile_	**Code:** AISC-ASD
	Page:
Prepared by:	**Date:** 11/26/14

Detailed Results Table

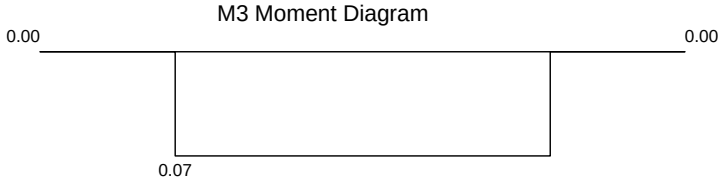
Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

M2 Moment Diagram



Max. AXIAL Force = 0.02 (tens.), -0.03 (compr.) Max. SHEAR Force =

M3 Moment Diagram



Max. AXIAL Force = 0.02 (tens.), -0.03 (compr.) Max. SHEAR Force =

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios:	Compac		Non-Compact	
d/t= 47.92	<	106.6	161.7	(Fy= 36.0 R = 0.000)
b/t= 5.96	<	10.8	15.8	

DESIGN	EQUATION	FACTORS	VALUES	RESUL
M3 Moment (F6-1) without LTB	M / 0.6Mn < 1.00	Z = 5.54	M = 0.07 Mn = 16.64	0.01
V3 Shear (G2.1.a)	Vu/Vn/1.5<1.00 Vn=0.6*Fy*Aw	Av = 3.56	Vu = 5.97 Vn = 76.98	0.12
M2 Moment (F2-1) without LTB	M / 0.6Mn < 1.00	Z = 40.20	M = 22.87 Mn = 120.75	0.32
Deflection	defl. / L / 240 < 1.00		defl = 0.16705	0.20
Axial Force (E3-1)	Pu / 0.6AgFcr < 1.00	(kL/r)x =35 (kL/r)y =182	Pu = 0.03 Ag = 7.69 Fcr = 7.60	0.00
Lateral Torsional Buckling (F2-3)	M / 0.6Mn < 1.00	Lb = 16.33 Lp = 4.49 Lr = 13.69 Cb = 1.09	M = 22.87 Mn = 62.14 Mr = 74.07 Fcr = 21.18	0.61
	Critical Segment from 0.00 to 16.33 on +z flange Segment End Moments: 0.00 and 0.00			

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Detailed Results Table

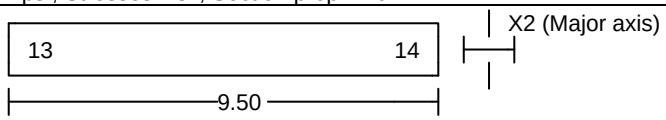
Moments: kips*foot , Forces: kips , Stresses: ksi , Section prop.: inch

DESIGN	EQUATION	FACTORS	VALUES	RESUL
Combined Forces (compress.) (H1-1b)	$\frac{Pr}{2\phi Pn} + \frac{Mrx}{\phi Mnx} + \frac{Mry}{\phi Mny} < 1.00$	Cmx = 1.00 Cmy = 1.00 Pex = 1804.41 Pey = 66.73	Mrx = 22.87 Mry = 0.07 B1x = 1.00 B1y = 1.00	0.62

Detailed Results Table

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

Beam:
7, 21, 25



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

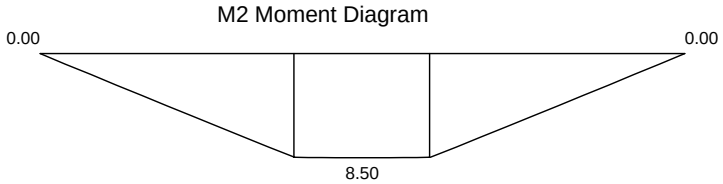
L =	3.75	5.75
Lat.-Tors.	+ -	+ -
Compress.	X Y	X Y

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 = 2

M2 Moment Diagram



Moments at Intermediate Supports:
8.46 8.46

Max. AXIAL Force = -0.08 (compr.) Max. SHEAR Force = 2.30

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Prepared by:	**Date:** 11/26/14

Detailed Results Table

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

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios:	Compac		Non-Compact	
d/t= 28.21	<	106.5	161.6	(Fy= 36.0 R = 0.001)
b/t= 6.38	<	10.8	15.8	

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (G2.1.a)	Vu/Vn/1.5<1.00 Vn=0.6*Fy*Aw	Av = 1.98	Vu = 2.30 Vn = 42.81	0.08
M2 Moment (F2-1) without LTB	$\frac{M}{0.6Mn} < 1.00$	Z = 13.60	M = 8.50 Mn = 40.85	0.35
Deflection	$\frac{defl.}{L / 240} < 1.00$		defl = 0.09916	0.21
Axial Force (E3-1)	$\frac{Pu}{0.6AgFcr} < 1.00$	(kL/r)x =14 (kL/r)y =51	Pu = 0.08 Ag = 4.43 Fcr = 31.41	0.00
Lateral Torsional Buckling	$\frac{M}{0.6Mn} < 1.00$ Critical Segment from 3.75 to 5.75 on +z flange Segment End Moments: 8.46 and 8.46	Lb = 2.00 Lp = 3.65	M = 8.50 Mn = 40.77	0.35
Combined Forces (compress.) (H1-1b)	$\frac{Pr}{2\phi Pn} + \frac{Mrx}{\phi Mnx} + \frac{Mry}{\phi Mny} < 1.00$	Cmx = 1.00 Cmy = 1.00 Pex = 6502.77 Pey = 490.02	Mrx = 8.50 Mry = 0.00 B1x = 1.00 B1y = 1.00	0.35

Site Name/Number:	**Harbor Terrace**
Site Address:	**284 Danforth Street, Portland, ME 04102**
CEA Job Number:	**1424.004**
Date:	**25-Nov-14**

CHAPPELL ENGINEERING ASSOCIATES, LLC
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Appurtenances Attached to Ballast Frame:

	CellMAX	Commscope Panel	Cell MAX	RRUS-11						
Depth, d =	5.2 in	7.1 in	5.2 in	7.0 in	in					
Width, w =	15.0 in	11.9 in	15.0 in	17.0 in	in					
Height, h =	81.0 in	96.4 in	81.0 in	20.0 in	in					
Height ARL =	8.6 ft	8 ft	8.6 ft	6 ft	ft					
Weight =	62 lbs	50.3 lbs	62 lbs	50.7 lbs	lbs					

Design Code: ASCE 7

Z (Above Ground Level) =	71 ft	71 ft	71 ft	71 ft	71 ft	71 ft	71 ft	71 ft	71 ft	71 ft	
Height of Projection Area =	6.8 ft	8.0 ft	6.8 ft	1.7 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	
Width of Projection Area =	1.3 ft	1.0 ft	1.3 ft	1.4 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	
Af (Projected Area of Gross) =	8.4 s.f.	8.0 s.f.	8.4 s.f.	2.4 s.f.	0.0 s.f.	0.0 s.f.	0.0 s.f.	0.0 s.f.	0.0 s.f.	0.0 s.f.	
Reference Wind Velocity, V =	107 mph	107 mph	107 mph	107 mph	107 mph	107 mph	107 mph	107 mph	107 mph	107 mph	
Exposure =	B	B	B	B	B	B	B	B	B	B	Section 6.5.6.3
G (Gust effect factor) =	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	Section 6.5.8
Cf (Force Coeficient) =	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	Fig 6-20 to 6-23
K_z (Exposure Coefficients) =	1	1	1	1	1	1	1	1	1	1	6.5.6.6, Table 6-3
K_1 (Multiplier) =	0	0	0	0	0	0	0	0	0	0	Figure 6-2
K_2 (Multiplier) =	0	0	0	0	0	0	0	0	0	0	Figure 6-2
K_3 (Multiplier) =	0	0	0	0	0	0	0	0	0	0	Figure 6-2
K_{zt} (Topographic Factor) : $(1+K_1*K_2*K_3)^2$ =	1	1	1	1	1	1	1	1	1	1	Section 6.5.7.2
Kd =	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	Table 6-4
I (Importance Factor) =	1	1	1	1	1	1	1	1	1	1	Table 6-2
$q_z = .00256*K_z*K_{zt}*K_d*V^2*I$ (psf) =	24.9 psf	24.9 psf	24.9 psf	24.9 psf	24.9 psf	24.9 psf	24.9 psf	24.9 psf	24.9 psf	24.9 psf	psf, Section 6.5.10
Reference Wind Pressure, *p* =	29.6 psf	29.6 psf	29.6 psf	29.6 psf	29.6 psf	29.6 psf	29.6 psf	29.6 psf	29.6 psf	29.6 psf	
F, lbs =	**250**	**236**	**250**	**70**	**0**	**0**	**0**	**0**	**0**	**0**	

Required Minimum Ballast:

Ballast Frame Geometry

Frame width =	7.1 ft
Frame depth =	8.42 ft
Centroid of front ballast to toe, d_f =	0.67 ft
Centroid of rear ballast to toe, d_r =	7.75 ft
Frame Footprint Area =	59.76 ft^2
Weight of steel frame =	425 lbs

Safety Factor for Overturning = 1.5 Total Appurtenance Wgt = 225 lbs

Let W_t = total ballast required, lbs
Let W_f = 0.25 W_t
Let W_r = 0.75 W_t

For Stability;

$M_{causing} <= M_{resisting}$

$M_{causing} <= M_{frame\ wgt} + M_{rear\ ballast} + M_{front\ ballast}$

$M_{causing} <= M_{frame\ wgt} + 0.75\ (W_t)(d_r) + 0.25\ (W_t)(d_f)$

Solving for Wt;

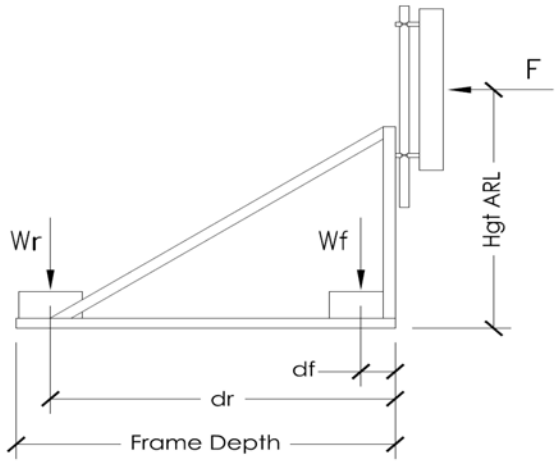
$$\frac{M_{causing} - M_{frame\ wgt}}{0.75\ d_r + 0.25\ d_f} <= W_t \quad \text{(min total ballast req'd)}$$

$M_{causing}$ = 9917.8 ft-lbs
$M_{frame\ wgt}$ = 283.3 ft-lbs
$0.75\ d_r$ = 5.81 ft
$0.25\ d_f$ = 0.17 ft

$$\frac{9634.43}{5.98} <= W_t$$

Min. Total Ballast Req'd (W_t) = 1611 lbs < = W_t

Min. Front Ballast Req'd (W_f) = 403 lbs = 5 Solid 8x8x16 blocks
Min. Rear Ballast Req'd (W_r) = 1209 lbs = 16 Solid 8x8x16 blocks
Total Loaded Frame Weight = 2261 lbs

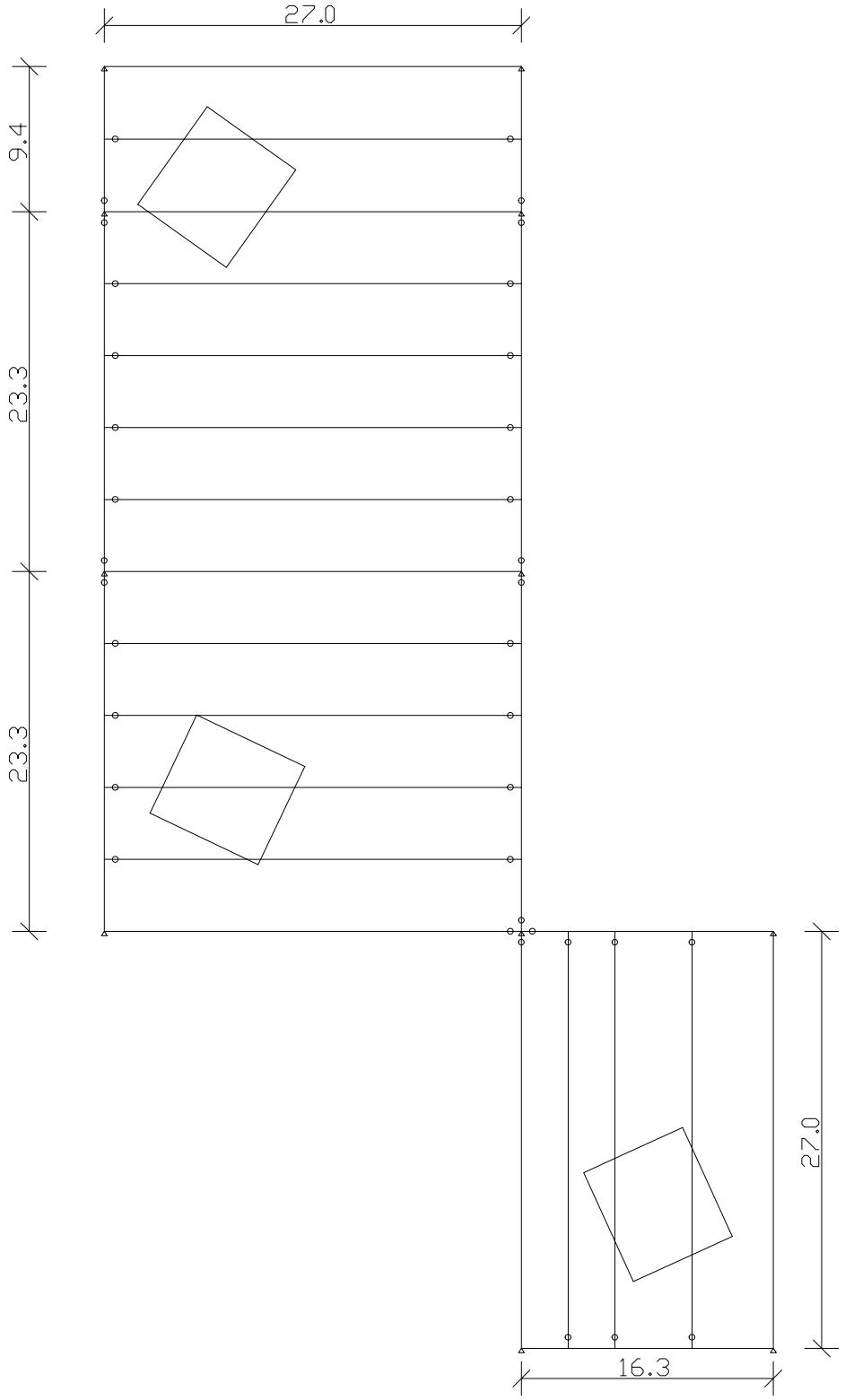


Frame Geometry

Harbor Terrace Ballast Mounts

SCALE = 1:134

DATE:11/26/14



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 12 19 TO 25 28 TO 46 50 TO 101
/ END
```

```
F O R C E   S U M M A T I O N
------------------------------
FX1=0.
FX2=0.
FX3=-10.356
```

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

```
/ GLOBAL LOADS
/ GLOBAL LOADS
DIST FX3 -0.01 PLANE 0. 0. 0.    0. -56.083 0.    27.042 -56.083 0.    PT
0. 27.042 BEAMS
DIST FX3 -0.01 PLANE 27.042 -56.083 0.    27.042 -83.125 0.    43.375
-83.125 0.    PT 0. 16.333 BEAMS
/ END
```

```
F O R C E   S U M M A T I O N
------------------------------
FX1=0.
FX2=0.
FX3=-19.616
```

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

```
/ GLOBAL LOADS
DIST FX3 -0.032 PLANE 0. 0. 0.    0. -56.083 0.    27.042 -56.083 0.    PT
0. 27.042 BEAMS
DIST FX3 -0.032 PLANE 27.042 -56.083 0.    27.042 -83.125 0.    43.375
-83.125 0.    PT 0. 16.333 BEAMS
/ END
```

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

F O R C E S U M M A T I O N

FX1=0.
FX2=0.
FX3=-62.772

Load no. 4: Ballast Frame Weights (units - kips ft.)

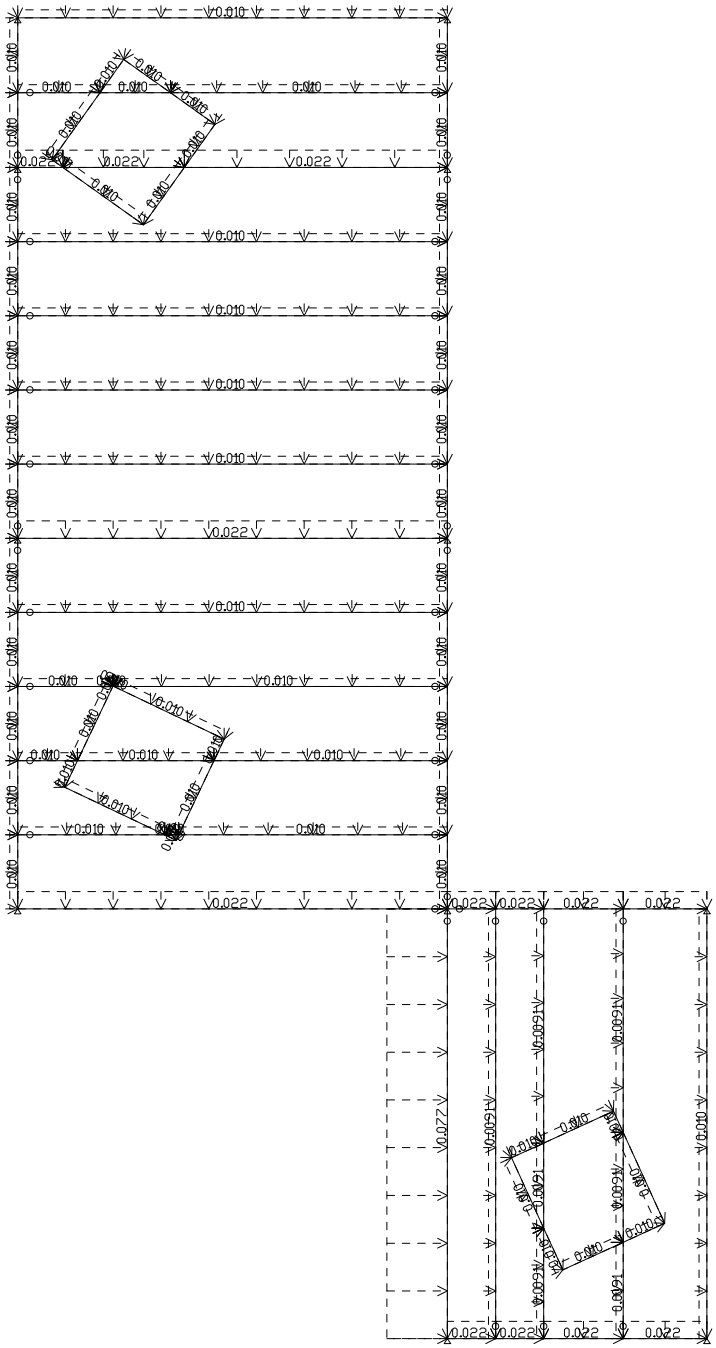
```
/ GLOBAL LOADS
/ GLOBAL LOADS
/ GLOBAL LOADS
/ GLOBAL LOADS
DIST FX3 -0.045 PLANE 2.9603 -48.422 0.    5.977 -42.083 0.    12.998
-45.39 0.    PT 0.0003 7.7605 BEAMS
DIST FX3 -0.045 PLANE 31.078 -71.738 0.    37.492 -68.808 0.    40.717
-75.867 0.    PT 0. 7.7603 BEAMS
DIST FX3 -0.045 PLANE 2.168 -8.9347 0.    6.6686 -2.6129 0.    12.413
-6.7028 0.    PT -0.0005 7.0514 BEAMS
/ END STATIC

F O R C E    S U M M A T I O N
FX1=0.
FX2=0.
FX3=-7.4025
```

Harbor Terrace Ballast Mounts

Load 1: Selfweight

X2
X1

SCALE = 1:145	UNITS: kip ft	DATE:11/26/14

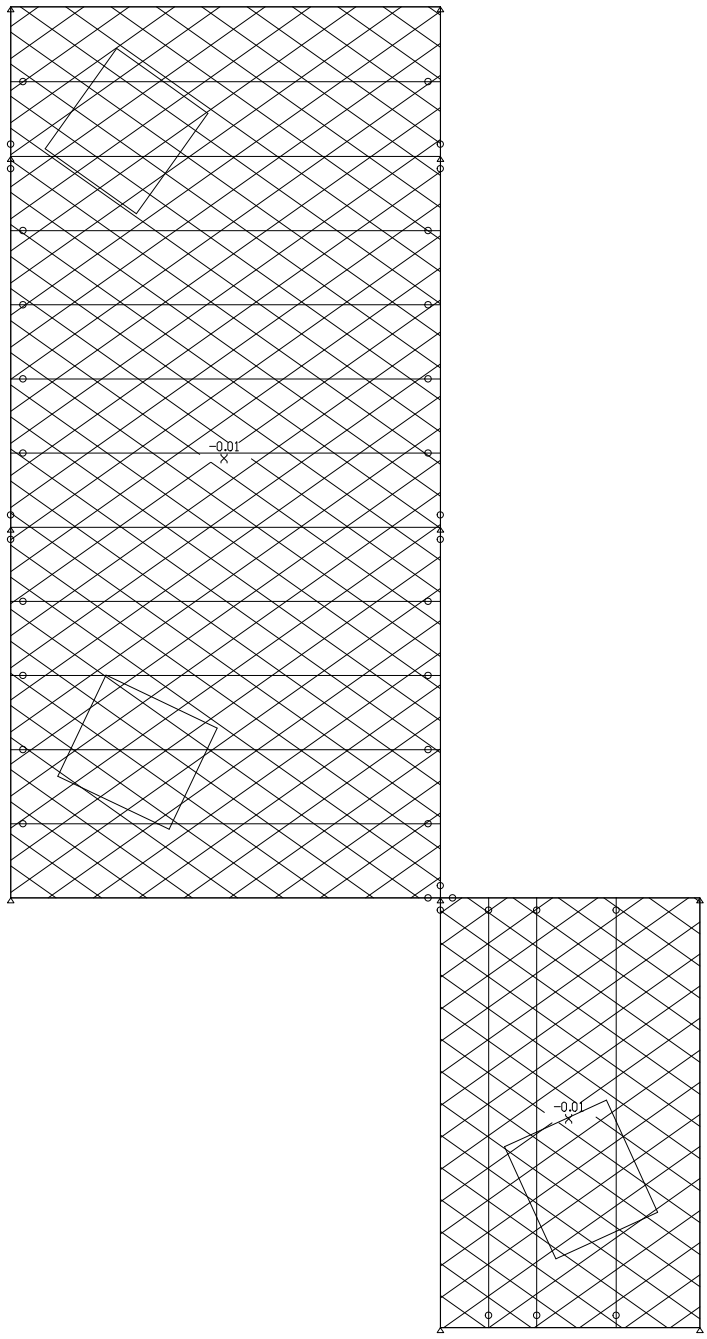


Harbor Terrace Ballast Mounts

Load 2: Steel Deck Weight

X2
X1

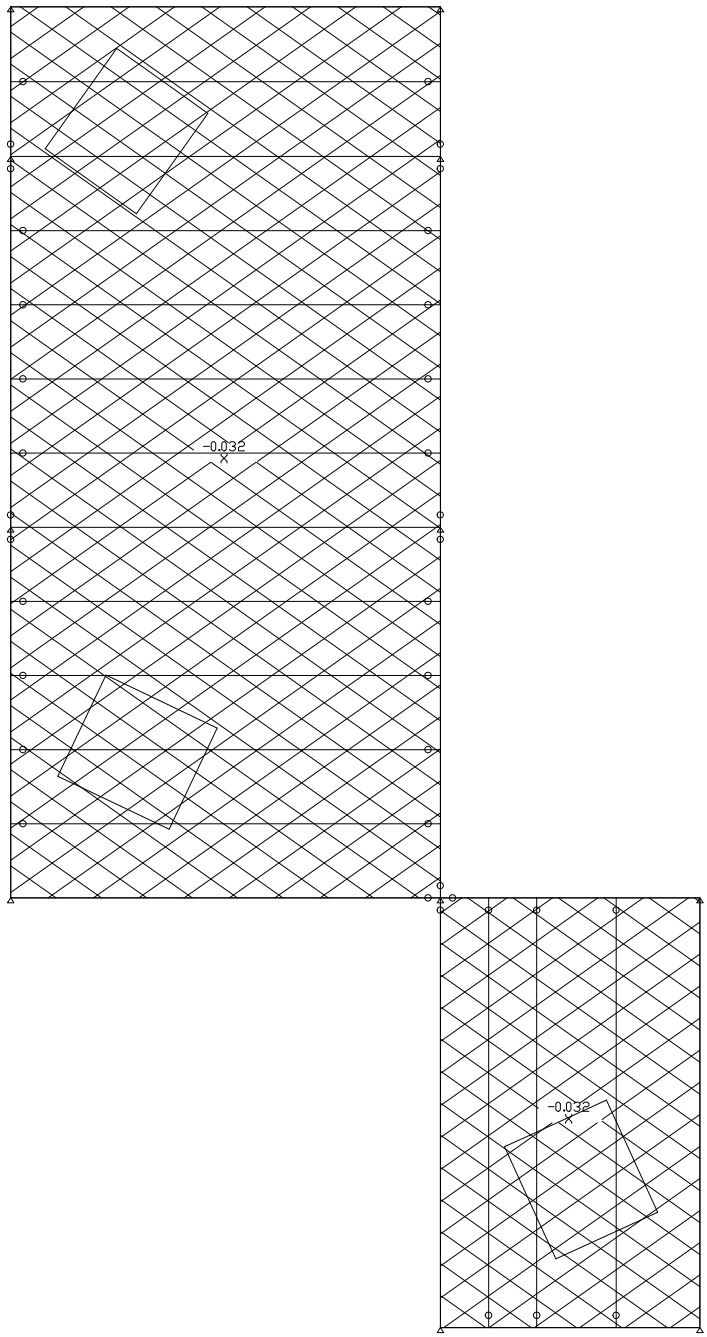
SCALE = 1:145	UNITS: kip ft	DATE:11/26/14



Harbor Terrace Ballast Mounts

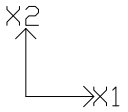
Load 3: Snow Load

SCALE = 1:145	UNITS: kip ft	DATE:11/26/14

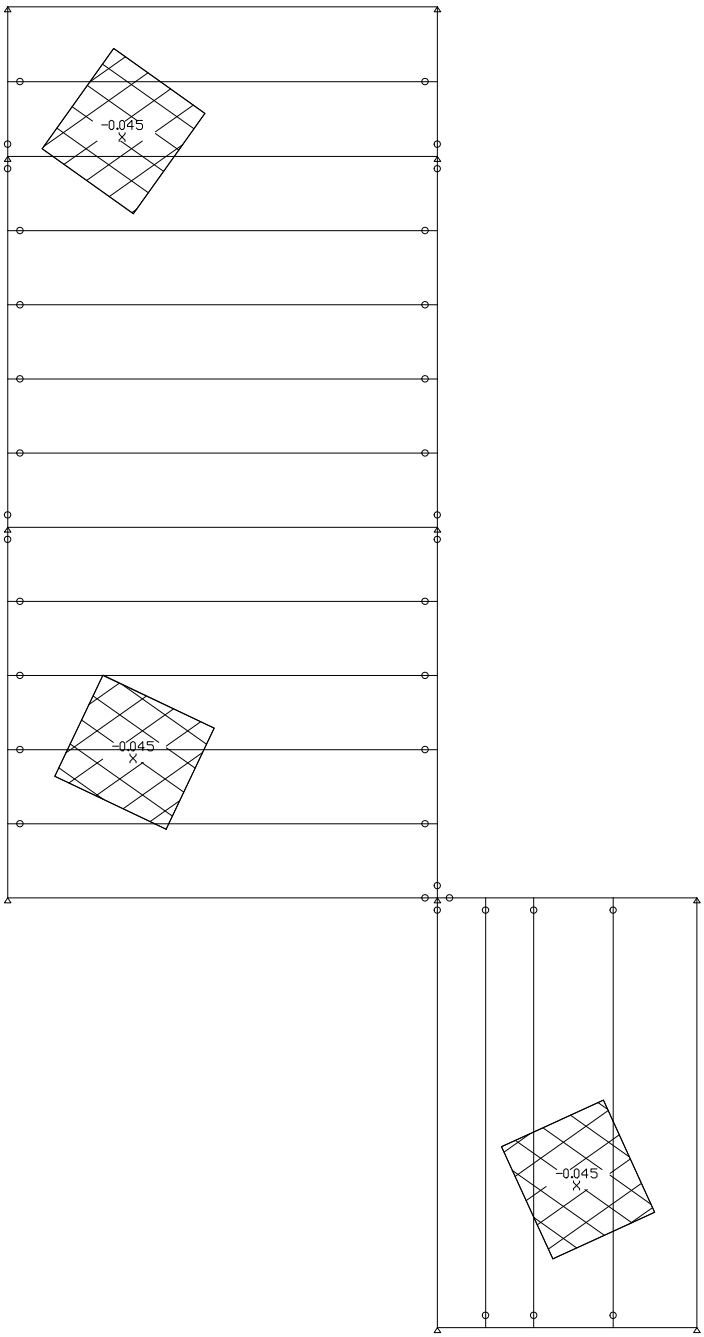


Harbor Terrace Ballast Mounts

Load 4: Ballast Frame Weights



SCALE = 1:145	UNITS: kip ft	DATE:11/26/14



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Civil · Structural · Land Surveying

Harbor Terrace Ballast Mounts

Prepared by:

Date: 11/26/14

COMBINATIONS TABLE

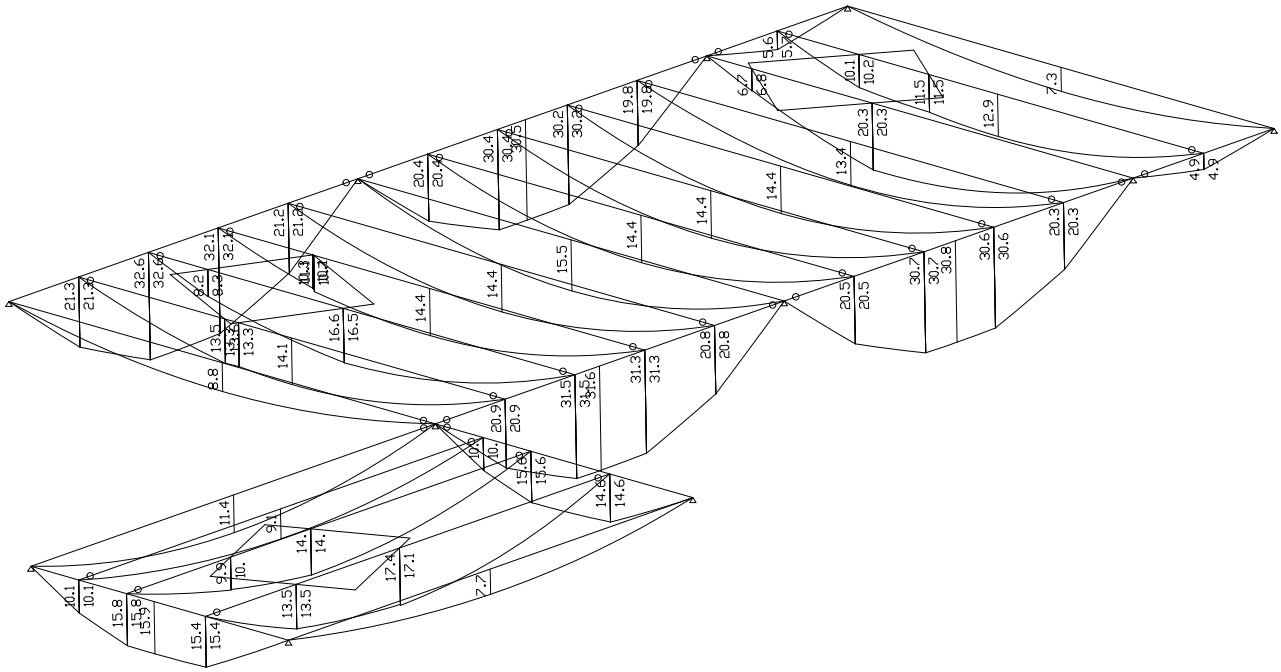
Comb.	
existing 1	1 * 1.00 + 3 * 1.00
Proposed 2	1 * 1.00 + 3 * 1.00 + 4 * 1.00

Harbor Terrace Ballast Mounts

SCALE = 1:106

UNITS: kip*ft

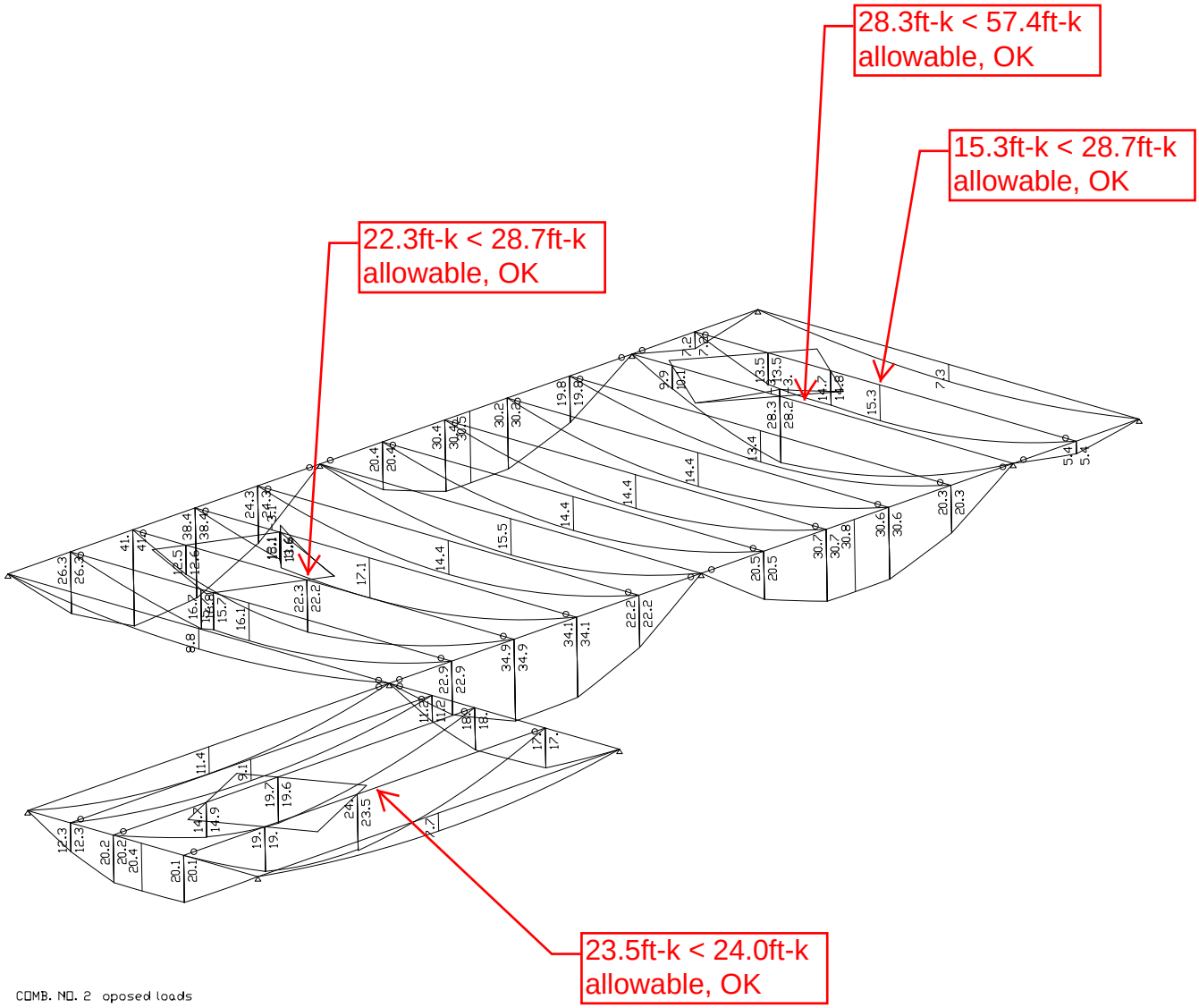
DATE:11/26/14



M2 MOMENT COMB. NO. 1 isting loads

Harbor Terrace Ballast Mounts

SCALE = 1:106 | UNITS: kip*ft | DATE:11/26/14



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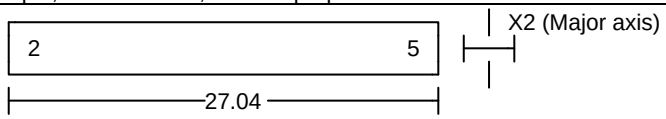
Harbor Terrace Ballast Mounts_	**Code:** AISC-ASD
	Page: **2**
Prepared by:	**Date:** 11/26/14

Detailed Results Table

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

Beam:
11, 60, 78



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

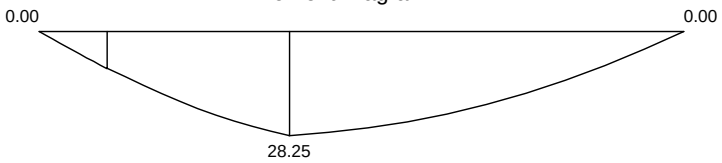
L =	2.88	10.50
Lat.-Tors.	+ -	+ -
Compress.	X Y	X Y

Section: W 14x22

Ix = 199.00 Iy = 7.00in4 Sx = 28.97 Sy = 2.80in3 Area = 6.49
hw = 13.74 bf = 5.00in tw = 0.23 tf = 0.33in
J = 0.21 Cw = 312.81in6

DESIGN COMBINATION = 2

M2 Moment Diagram



Moments at Intermediate Supports:
9.99 28.24

Max. AXIAL Force = 0.00 (tens.) Max. SHEAR Force = 3.64

SECTION CLASSIFICATION: *** COMPACT ***

Limiting Ratios:		Compac	Non-Compact	
d/t= 60.17	<	106.7	163.5	(Fy= 36.0 R = 0.000)
b/t= 7.47	<	10.8	15.8	

DESIGN	EQUATION	FACTORS	VALUES	RESUL
V3 Shear (F4-1)	V/(Av*Fv) <1.00 Fv=0.4*Fy	Av = 3.14	V = 3.64 Fv = 14.40	0.08
M2 Moment (F1-1)	$\frac{M}{S*Fb} < 1.00$	S = 28.97 Fb =0.660 *Fy	M = 28.25 S*Fb = 57.42	0.49

Detailed Results Table

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

DESIGN	EQUATION	FACTORS	VALUES	RESUL
Deflection	defl. / L / 240 < 1.00		defl = 0.60370	0.45
Combined Stresses (Local) (H1-2) (H2-1)	fa / 0.6Fy + fbx / Fbx + fby / Fby (Ft) < 1.00	fbx = 11.69 Fbx= 23.76 fby = 0.00 Fby= 0.00	P = 0.00 A = 6.49 Fu = 58.00 fb = M/S	0.49
Moment - noncompact (F1-8)	M / S*Fb < 1.00 Critical Segment from 10.50 to 27.04 on +z flange Segment End Moments: 28.24 and 0.00	S = 28.97 RT = 1.26 Lb = 16.54	M = 28.25 S*Fb= 31.13 Cb = 1.75	0.91
Combined Stresses (tension) (H2-1)	fa / Ft + fbx / Fbx + fby / Fby < 1.00	Fbx = 12.88 Fby = 27.00	fbx = 11.69 fby = 0.00	0.91

Allowable Total Safe Loads in Pounds Per Linear Foot of H-SERIES Steel Joists — *For Joist Depths 16" to 24" inclusive.

Joist Designation	16H4	16H5	16H6	16H7	16H8	18H5	18H6	18H7	18H8	20H5	20H6	20H7	20H8	22H6	22H7	22H8	24H6	24H7	24H8
*Depth in Inches	16	16	16	16	16	18	18	18	18	20	20	20	20	22	22	22	24	24	24
Resisting Moment In Inch-Pounds	221,000	289,000	344,000	413,000	478,000	325,000	383,000	466,000	540,000	365,000	406,000	499,000	602,000	422,000	526,000	653,000	462,000	576,000	716,000
Maximum End Reaction In Pounds	3800	4300	4600	4900	5200	4500	4800	5200	5400	4800	5100	5400	5600	5400	5600	5800	5600	5800	6000
†Approximate Weight in Pounds per Foot	6.6	7.8	8.6	10.3	11.4	8.0	9.2	10.4	11.6	8.4	9.6	10.7	12.2	9.7	10.7	12.0	10.3	11.5	12.7
Span in Feet																			
16	475	538	575	613	650														
17	447	506	541	576	612														
18	422	478	511	544	578	500	533	578	600										
19	400 399	453	484	516	547	474	505	547	568										
20	368 342	430 413	460	490	520	450	480	520	540	480	510	540	560						
21	334 295	410 357	438 429	467	495	429	457	495	514	457	486	514	533						
22	304 257	391 310	418 373	445 434	473	409 398	436	473	491	436	464	491	509	491	509	527			
23	279 225	364 272	400 326	426 379	452 434	391 348	417	452	470	417	443	470	487	470	487	504			
24	256 198	334 239	383 287	408 334	433 382	375 306	400 367	433 427	450	400 380	425	450	467	450	467	483	467	483	500
25	236 175	308 211	367 254	392 295	416 338	347 271	384 325	416 378	432	384 336	408 405	432	448	432	448	464	448	464	480
26	218 155	285 188	339 226	377 262	400 300	321 241	369 289	400 336	415 385	360 299	392 360	415	431	415	431	446	431	446	462
27	202 139	264 168	315 201	363 234	385 268	297 215	350 258	385 300	400 344	334 267	371 321	400 375	415	386	415	430	415	430	444
28	188 124	246 150	293 181	350 210	371 240	276 193	326 231	371 269	386 308	310 239	345 288	386 336	400 385	359 351	400	414	393	414	429
29	175 112	229 135	273 162	327 189	359 216	258 173	304 208	359 242	372 278	289 215	322 259	372 302	386 346	335 316	386 368	400	366	400	414
30	164 101	214 122	255 147	306 171	347 195	241 157	284 188	345 219	360 251	270 194	301 234	360 273	373 313	313 286	373 332	387 382	342 342	387	400
31	153 91	200 111	239 133	287 155	332 177	225 142	266 170	323 198	348 227	253 176	282 212	346 247	361 284	293 259	361 301	374 346	320 310	374 361	387
32	144 83	188 101	224 121	269 141	311 161	212 129	249 155	303 180	338 206	238 160	264 193	325 225	350 258	275 235	342 274	363 315	301 282	363 329	375
33						199 117	234 141	285 164	327 188	223 146	249 176	305 205	339 235	258 214	322 250	352 287	283 257	352 300	364 344
34						187 107	221 129	269 150	311 172	210 133	234 161	288 187	329 215	243 196	303 228	341 263	266 235	332 274	353 315
35						177 98	208 118	254 137	294 158	199 122	221 147	272 172	320 197	230 180	286 209	331 241	251 215	313 251	343 288
36						167 90	197 108	240 126	278 145	188 112	209 135	257 158	310 181	217 165	271 192	322 221	238 198	296 231	333 265
37										178 103	198 125	243 145	293 167	206 152	256 177	314 204	225 182	280 212	324 244
38										169 95	187 115	230 134	278 154	195 140	243 163	301 188	213 168	266 196	316 225
39										160 88	178 106	219 124	264 142	185 130	231 151	286 174	202 155	252 181	308 208
40										152 82	169 98	208 115	251 132	176 120	219 140	272 161	193 144	240 168	298 193
41														167 112	209 130	259 149	183 134	228 156	284 179
42														159 104	199 121	247 139	175 124	218 145	271 167
43														152 97	190 113	235 130	167 116	208 135	258 155
44														145 90	181 105	225 121	159 108	198 126	247 145
45																	152 101	190 118	236 135
46																	146 94	181 110	226 127
47																	139 89	174 103	216 119
48																	134 83	167 97	207 111

*Indicates Nominal Depth of Steel Joists only.
†Approximate Weights per Linear Foot of Steel Joists only. Accessories and nailer strip not included.
*†See Manufacturers' Catalog for detailed information on specific joist types.

LOADS ABOVE COLORED LINES ARE GOVERNED BY SHEAR.

Tests on steel joists designed in accordance with the Steel Joist Institute Standard Specifications have demonstrated that the Steel Joist Institute Load Tables are applicable for concentrated top chord loadings (such as are developed in bulb-tee roof construction) when the sum of the equal concentrated top chord loadings does not exceed the allowable uniform loading for the joist type and span and the loads are placed at spacings not exceeding 33" along the top chord.

Adopted by the Steel Joist Institute May 31, 1961.

This Table in accordance with Simplified Practice Recommendation filed with the Commodity Standards Division, Office of Technical Services, U. S. Department of Commerce.