CUDDY
+FEDER
LLP

445 Hamilton Avenue, 14th Floor
White Plains, New York 10601
T 914 761 1300
F 914 761 5372
cuddyfeder.com

Daniel M. Laub
dlaub@cuddyfeder.com

March 8, 2017

VIA EMAIL AND OVERNIGHT DELIVERY
Hon. Robert Stein, Chairman
and Members of the Connecticut Siting Council
10 Franklin Square
New Britain, CT 06051

Re: New Cingular Wireless PCS, LLC ("AT&T")
Petition 1282 Seeking Declaratory Ruling
208 Harbor Drive, Stamford, Connecticut

Dear Chairman Stein and Members of the Council:

On behalf of New Cingular Wireless PCS, LLC (AT&T), we respectfully enclose an original and fifteen (15) copies of the first eight (8) pages of a revised structural report for the captioned Petition as well as four (4) full copies of the same report with calculations. Please note that project engineers Hudson Design Group LLC revised this structural in keeping with the 2016 Connecticut Building Code Amendments which incorporate the EIA/TIA-222-G Structural Standards for Steel Antenna Towers and Antenna Supporting Structures.

Should the Council or Staff have any questions about this matter please do not hesitate to contact me.

Very truly yours,

Daniel M. Laub

cc: AT&T
Hudson Design Group

C&F: 3239281.1
C&F: 3356536.1

(REVISED)
STRUCTURAL ANALYSIS REPORT

For

S2900 (NSB)
STAMFORD HARBOR
208 Harbor Drive
Stamford, CT 06902

Antennas within FRP Enclosures on Steel Frames; Equipment Shelter on Steel Platform on the Roof

Prepared for:

CENTERLINE COMMUNICATIONS

at&t

Dated: February 10, 2017 (Rev. 3)
September 1, 2016 (Rev. 2)
March 27, 2015 (Rev. 1)
December 2, 2014

STATE OF CONNECTICUT
DANIEL HAMM
24178
LICENSED PROFESSIONAL ENGINEER

Prepared by:

Hudson Design Group LLC
1600 Osgood Street Building 20 North, Suite 3090
North Andover, MA 01845
Phone: (978) 557-5553
www.hudsondesigngroupllc.com

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SCOPE OF WORK:

Hudson Design Group LLC (HDG) has been authorized by AT&T to conduct a structural evaluation of the structure that will support the existing AT&T equipment located in the areas depicted in the latest HDG's construction drawings.

This report represents this office's findings, conclusions and recommendations pertaining to the support of AT&T's proposed equipment.

An on-site visual survey and structural mapping of the existing building structure was performed by ProVertic LLC on October 9, 2014.

CONCLUSION SUMMARY:

Partial building plans prepared by Yankee Planning dated June 25, 1979 and partial building plans prepared by Pelliccione & Associates, LLC dated July 22, 2014 were available and were obtained for our use. A limited visual survey of the structure was completed in or near the areas of the proposed work.

Based on our evaluation, we have determined that the existing building structure **IS CAPABLE** of supporting the proposed equipment loading.

APPURTENACE/EQUIPMENT CONFIGURATION:

(12) HPA-65R-BUU-H8 Antennas (93"x15"x7" - Wt. = 68 lbs. /each) (Four per sector)

(9) RRH (RRUS-11) (19.7"x17"x7.2" – Wt. = 50 lbs. /each) (Three per sector)

(6) RRH (RRUS-12) (20.4"x18.5"x7.5" – Wt. = 58 lbs. /each) (Two per sector)

(3) RRH (RRUS-E2) (20.4"x18.5"x7.5" – Wt. = 58 lbs. /each) (One per sector)

(3) RRH (RRUS-32) (29.9"x13.3"x9.5" – Wt. = 77 lbs. /each) (One per sector)

(6) Surge Suppressor (Wt. = 43.5 lbs. / each)

(1) 11'-6" x 16'-0" Equipment Shelter (Wt. = 30000 lbs.) (Includes Equipment)

Referenced documents are attached.

ANALYSIS RESULTS SUMMARY → EXISTING CONDITIONS

- **Roof Beams**

Beam	Controlling Factor	Required (ft-lb)	Provided (ft-lb)	Ratio (%)	Pass/Fail
24-C-C'	Moment	12,954	110,279	11.8	PASS
24-D'-E	Moment	14,822	110,279	13.4	PASS
25-C-C'	Moment	15,514	238,024	6.5	PASS
25-D'-E	Moment	18,918	238,024	7.9	PASS
26-D'-E	Moment	75,181	238,024	31.6	PASS
26a-C-C'	Moment	71,844	165,918	43.3	PASS
26a-D'-E	Moment	88,931	165,918	53.6	PASS
C'-26-27	Moment	419,180	830,838	50.5	PASS
D'-26-27	Moment	417,640	830,838	50.3	PASS
27-C-D	Moment	507,452	573,852	88.4	PASS
27-D-E	Moment	421,305	573,852	73.4	PASS

- **4th Floor Building Columns**

Column	Axial Load Applied (lbs.)	Allowable Axial Load (lbs.)	Ratio (%)	Pass/Fail
C'24	73,642	324,000	22.7	PASS
D'24	75,632	324,000	23.3	PASS
C'25	104,923	291,000	36.1	PASS
D'25	111,993	291,000	38.5	PASS
C'26	135,016	207,000	65.2	PASS
D'26	130,881	207,000	63.2	PASS
C26	100,496	358,000	28.1	PASS
C27	113,102	443,000	25.5	PASS
D27	218,298	239,000	91.3	PASS

**** SEE PAGE 118 FOR BEAM AND COLUMN LAYOUT ****

DESIGN CRITERIA:

1. International Building Code 2012 with 2016 Connecticut State Building Code Amendments; ASCE 7-10 Minimum Design Loads for Buildings and Other Structures.

Wind Analysis:

Ultimate Wind Speed (V_{ult}):	120 mph	(CTSBC 2016 Appendix N)
Nominal Wind Speed (V_{asd}):	93 mph	(CTSBC 2016 Appendix N)
Category:	D	

Roof:

Live Loads:

Roof Snow Load:	30 psf	(CTSBC 2016 Appendix N)
Service Load:	60 psf	(Equipment Platform Only)
Penthouse Floor Load:	150 psf	(Assumed)

Dead Loads:

Roof (Type 0):	5 psf	(See pg. 170 for Breakdown)
Roof (Type 1):	46 psf	(See pg. 170 for Breakdown)
Roof (Type 2):	85 psf	(See pg. 170 for Breakdown)
Roof (Type 3):	95 psf	(See pg. 170 for Breakdown)
Penthouse Wall:	50 psf	
Penthouse Roof:	10 psf	
FRP Enclosure:	30 plf	

1. EIA/TIA-222-G Structural Standards for Steel Antenna Towers and Antenna Supporting Structures.

City/Town:	Stamford	
County:	Fairfield	
Wind Load:	110 mph	(fastest mile)
Nominal Ice Thickness:	3/4 inch	

2. Approximate height above grade to the top of the FRP enclosures:

82'-9"+/-

ANTENNA / RRH SUPPORT RECOMMENDATIONS:

The new antennas and RRH's are proposed to be mounted on new pipe masts within new FRP enclosures. The FRP enclosures are proposed to be mounted on new steel frames secured to the existing building structure.

EQUIPMENT SUPPORT RECOMMENDATIONS:

The new equipment shelter is proposed to be mounted on a new steel equipment platform secured to the existing building structure.

Limitations and assumptions:

1. Reference the latest HDG construction drawings for all the equipment locations and details.
2. Mount all equipment per manufacturer's specifications.
3. All structural members and their connections are assumed to be in good condition and are free from defects with no deterioration to its member capacities.
4. All antennas, coax cables and waveguide cables are assumed to be properly installed and supported as per the manufacturer requirements.
5. HDG is not responsible for any modifications completed prior to and hereafter which HDG was not directly involved.
6. If field conditions differ from what is assumed in this report, then the engineer of record is to be notified as soon as possible.

Hudson
Design Group LLC

FIELD PHOTOS:

Photo 1: Sample photo illustrating the location of the proposed Alpha sector frame and the new equipment platform (existing HVAC unit to be removed).

Photo 2: Sample photo illustrating the location of the proposed Beta/Gamma sector frame (existing ladder to be relocated).

FIELD PHOTOS (CONT.):

Photo 3: Sample photo illustrating the existing roof construction.

Photo 4: Sample photo illustrating the existing roof construction.

Alpha Sector
Antenna Frame Calculations

Date: 02-10-2016
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Wind Analysis → FRP Enclosure (ALPHA Sector)

Wind Blowing East - West

Reference Codes:

-Connecticut State Building Code 2016

-International Building Code 2012 (IBC 2012)

-Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)

Structure Classification	II		(ASCE 7-10 Table 1.5-1)
Nominal Wind Speed, V_{asd}	93	mph	(CTSBC Appendix N)
Importance Factor, I	1		(ASCE 7-10 Table 1.5-2)
Exposure Category	D		(ASCE 7-10 Section 26.7)
Height Above Ground Level, z	82.75	ft	(Top of Enclosure)
Exposure Coeficient, K_z	1.39		(ASCE 7-10 Table 29-3.1)
Wind Directionality Coef., K_d	0.90		(ASCE 7-10 Table 26.6-1)
Topographic Factor, K_{zt}	1.00		(ASCE 7-10 Section 26.8.2)

Velocity Pressure, q_z $= 0.00256K_zK_{zt}K_dV^2$ (ASCE 7-10 Equation 29.3-1)

$=$ **27.60 psf**

Gust Factor, G	0.85		(ASCE 7-10 Section 26.9)
Net Force Coeficient, C_f	1.30		(ASCE 7-10 Figure 29.5-1)
Projected Area Normal to Wind, A_f	205	ft^2	(20.5 ft. W x 10 ft. H)

Wind Force, F $= q_zGC_fA_f$ (ASCE 7-10 Equation 29.5-2)

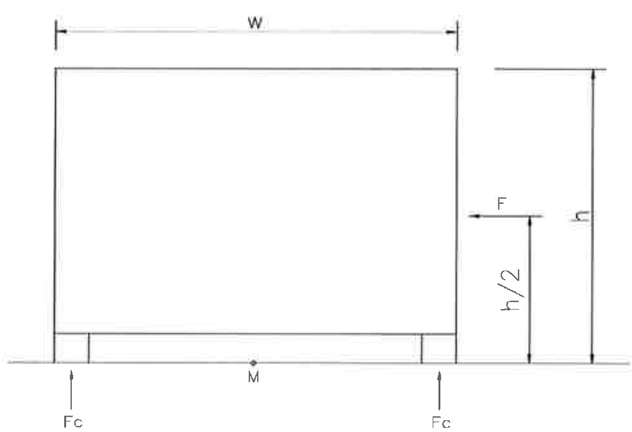
$=$ **6251.93 lbs**

Date: 02-10-2016
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Calculate Overturning Moment of Proposed FRP Enclosure (ALPHA)

Wind Blowing East - West

Dimensions (ft)	Wide, w	Depth, d	Height, h
	20.167	7	10



Moment, M = F x h/2
= **31259.66** **lb-ft**

Calculate Force Couple

Force Couple, F_c = M / d
= **4465.67** **lbs.**

Length of Frame: 20.1667 ft

Loading = **221 plf**

Date: 02-10-2016
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Wind Analysis → FRP Enclosure (ALPHA Sector)

Wind Blowing North - South

Reference Codes:

-Connecticut State Building Code 2016

-International Building Code 2012 (IBC 2012)

-Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)

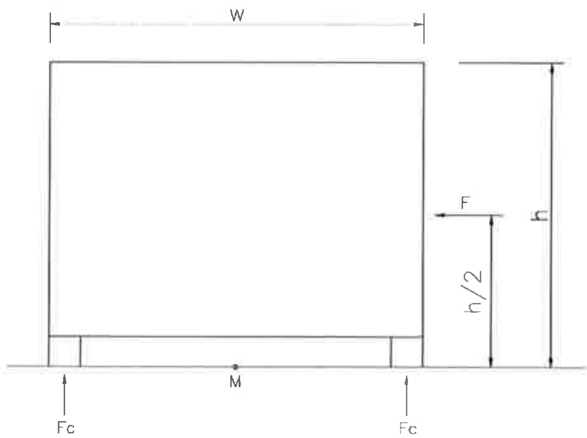
Structure Classification	II		(ASCE 7-10 Table 1.5-1)
Nominal Wind Speed, V_{asd}	93	mph	(CTSBC Appendix N)
Importance Factor, I	1		(ASCE 7-10 Table 1.5-2)
Exposure Category	D		(ASCE 7-10 Section 26.7)
Height Above Ground Level, z	82.75	ft	(Top of Enclosure)
Exposure Coeficient, K_z	1.39		(ASCE 7-10 Table 29-3.1)
Wind Directionality Coef., K_d	0.90		(ASCE 7-10 Table 26.6-1)
Topographic Factor, K_{zt}	1.00		(ASCE 7-10 Section 26.8.2)
Velocity Pressure, q_z	$= 0.00256K_zK_{zt}K_dV^2$		(ASCE 7-10 Equation 29.3-1)
	= **27.60 psf**		
Gust Factor, G	0.85		(ASCE 7-10 Section 26.9)
Net Force Coeficient, C_f	1.31		(ASCE 7-10 Figure 29.5-1)
Projected Area Normal to Wind, A_f	73	ft^2	(7.33 ft. W x 10 ft. H)
Wind Force, F	$= q_zGC_fA_f$		(ASCE 7-10 Equation 29.5-2)
	= **2252.64 lbs**		

Date: 02-10-2016
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Calculate Overturning Moment of Proposed FRP Enclosure (ALPHA)

Wind Blowing North - South

Dimensions (ft)	Wide, w	Depth, d	Height, h
	7	20.1667	10



Moment, M = F x h/2
= **11263.21** **lb-ft**

Calculate Force Couple

Force Couple, F_c = M / d
= **558.51** **lbs.**

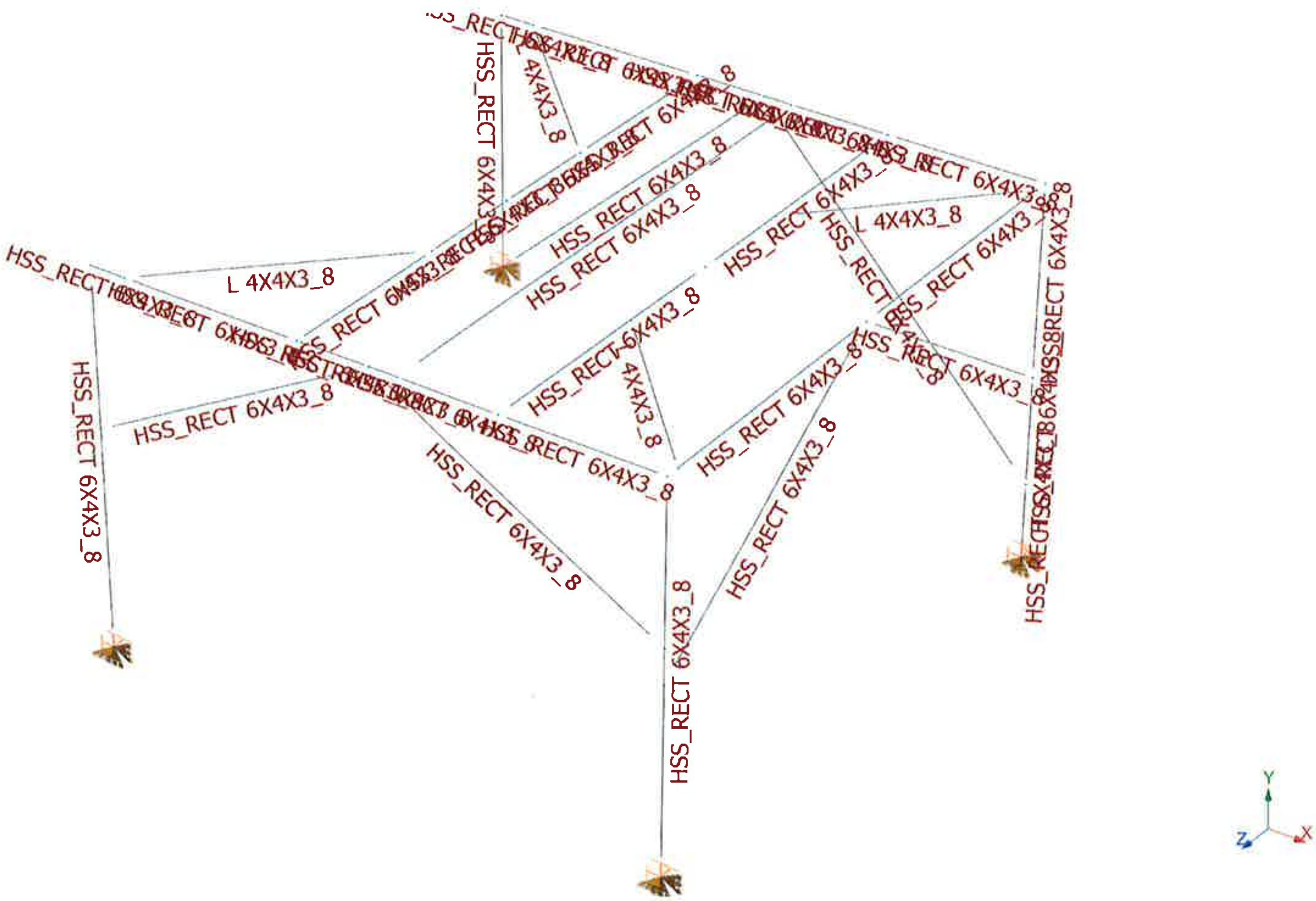
Length of Frame: 7 ft

Loading = **80 plf**

ALPHA FRAME
(EASTERLY WIND)

Bentley

Current Date: 2/8/2017 4:09 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind East) (Rev3).etz

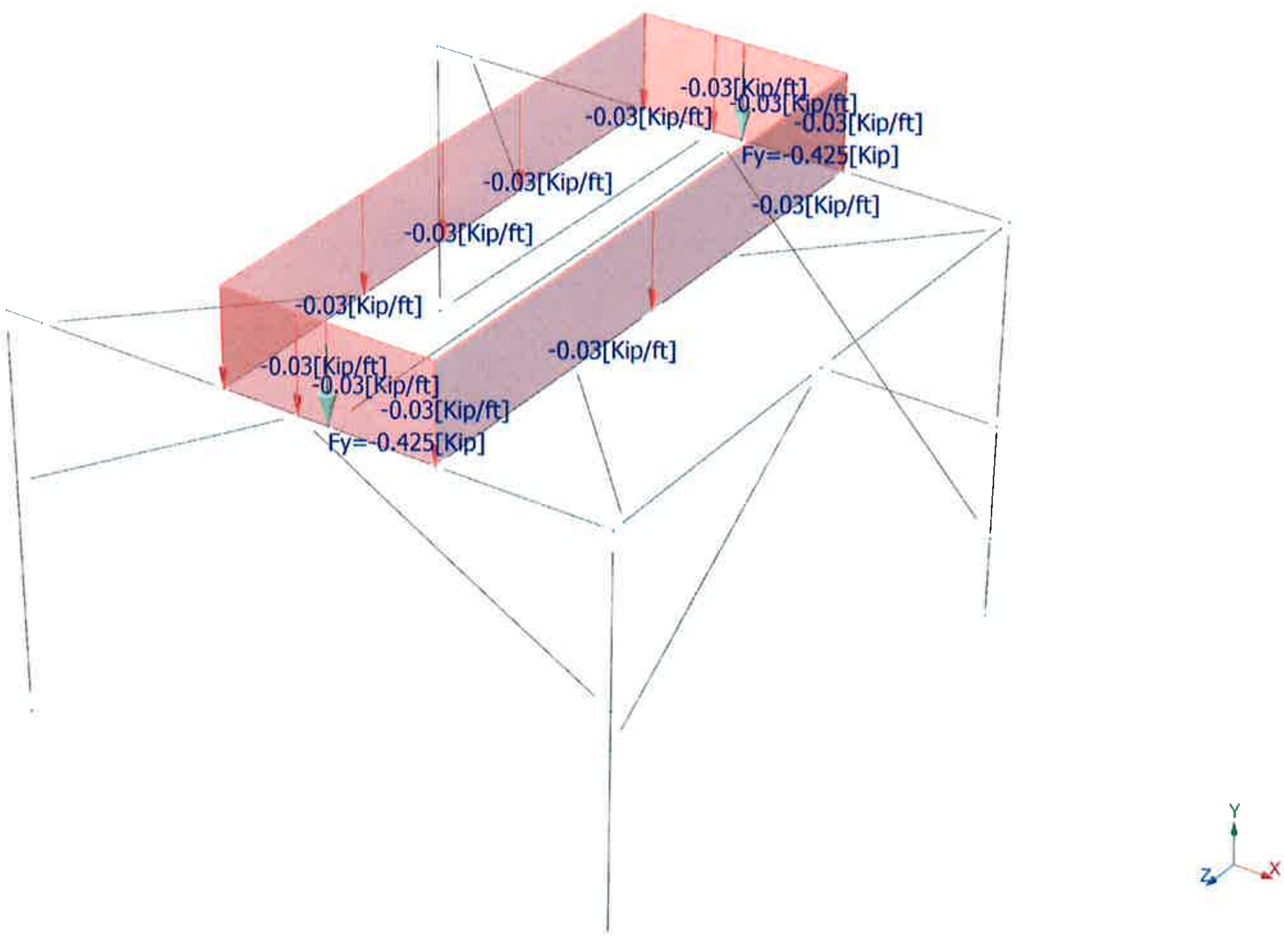


ALPHA FRAME
(EASTERLY WIND)

Current Date: 2/8/2017 4:10 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind East) (Rev3).etz
Load condition: DL=Dead Load

Loads

- Global distributed - Members
- Local distributed - Members
- Concentrated - Nodes



ALPHA FRAME
(EASTERLY WIND)

Current Date: 2/10/2017 9:21 AM
Units system: English
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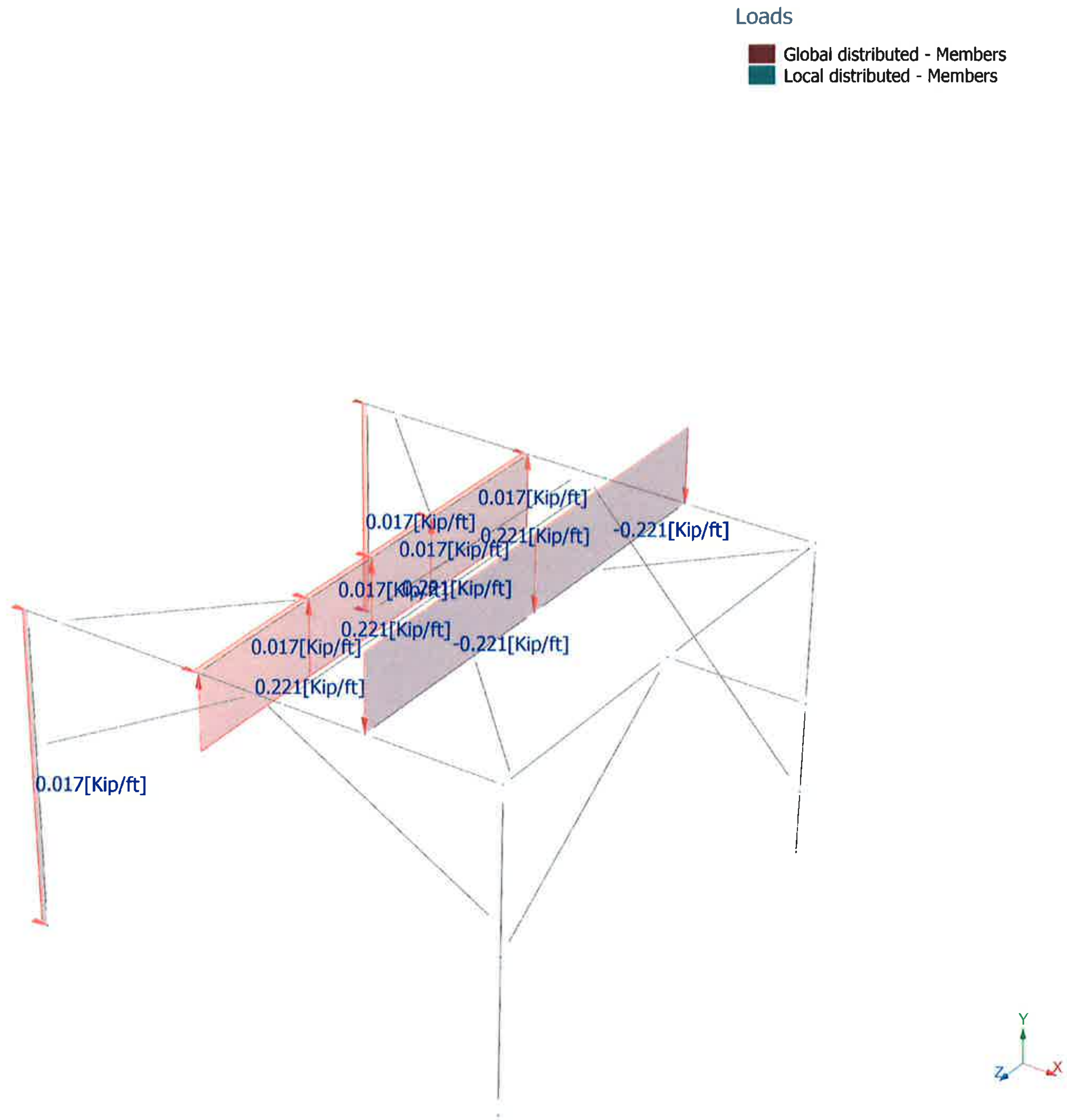
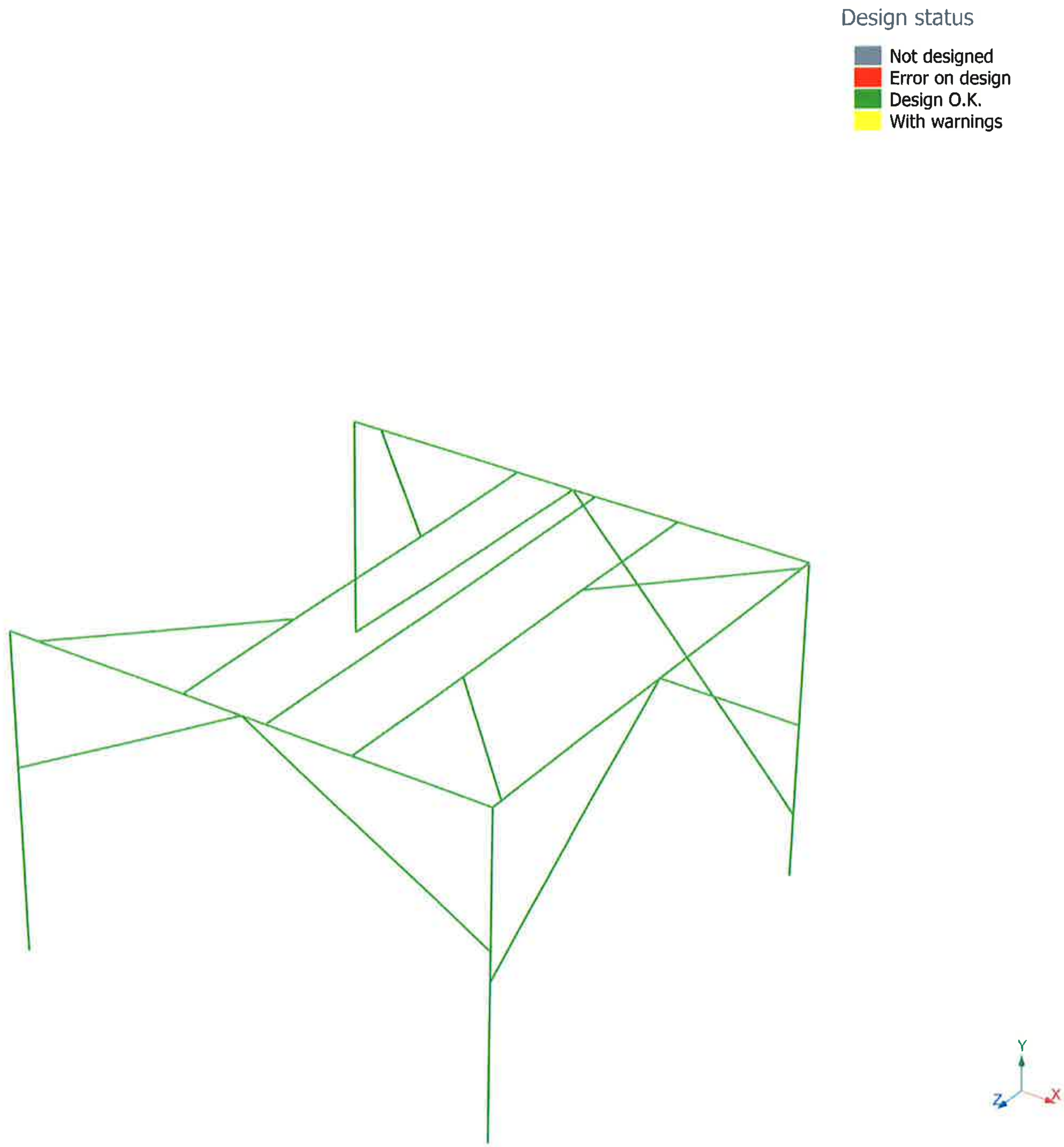
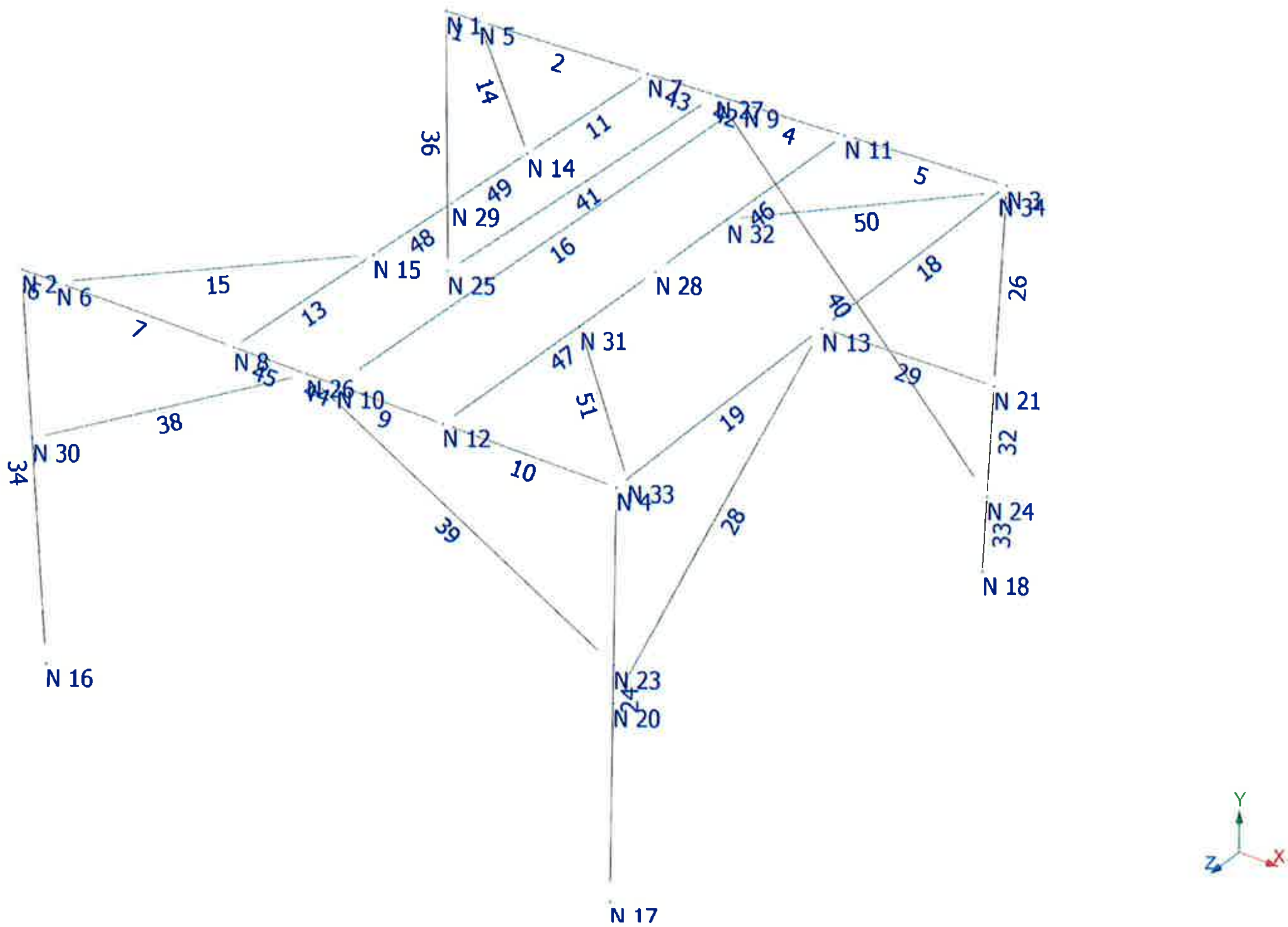
Load condition: W=Wind




Design status
Not designed
Error on design
Design O.K.
With warnings
Y
Z
X

Current Date: 2/8/2017 4:09 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind East) (Rev3).etz



ALPHA FRAME
(EASTERLY WIND)

Bentley

Current Date: 2/10/2017 9:21 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind East) (Rev3).etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :
D1=DL
D2=DL+W
D3=0.6DL+W

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_RECT 6X4X3_8	1	D1 at 0.00%	0.03	OK	
			D2 at 0.00%	0.03	OK	
			D3 at 0.00%	**0.04**	**OK**	
		2	D1 at 0.00%	0.02	OK	
			D2 at 100.00%	0.08	OK	Eq. H1-1b
			D3 at 100.00%	**0.08**	**OK**	Eq. H1-1b
		4	D1 at 100.00%	0.04	OK	Eq. H1-1b
			D2 at 100.00%	**0.20**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.19	OK	Eq. H1-1b
		5	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 100.00%	**0.17**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.16	OK	Eq. H1-1b
		6	D1 at 0.00%	0.02	OK	
			D2 at 0.00%	0.04	OK	
			D3 at 0.00%	**0.04**	**OK**	
		7	D1 at 100.00%	**0.05**	**OK**	Eq. H1-1b
			D2 at 100.00%	0.03	OK	Eq. H1-1b
			D3 at 100.00%	0.04	OK	Eq. H1-1b
		9	D1 at 100.00%	0.06	OK	Eq. H1-1b
			D2 at 100.00%	**0.22**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.19	OK	Eq. H1-1b
		10	D1 at 100.00%	0.06	OK	Eq. H1-1b
			D2 at 100.00%	**0.19**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.17	OK	Eq. H1-1b
		11	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 0.00%	0.12	OK	Eq. H1-1b
			D3 at 0.00%	**0.13**	**OK**	Eq. H1-1b
		13	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 0.00%	0.13	OK	Eq. H1-1b
			D3 at 0.00%	**0.14**	**OK**	Eq. H1-1b
		16	D1 at 56.25%	0.04	OK	Eq. H1-1b
			D2 at 50.00%	**0.07**	**OK**	Eq. H1-1b
			D3 at 50.00%	0.06	OK	Eq. H1-1b
		18	D1 at 34.38%	0.02	OK	Eq. H1-1b

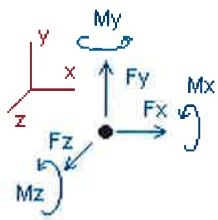
	D2 at 6.25%	**0.08**	**OK**	Eq. H1-1b
	D3 at 6.25%	0.08	OK	Eq. H1-1b
19	D1 at 18.75%	0.02	OK	Eq. H1-1b
	D2 at 6.25%	**0.08**	**OK**	Eq. H1-1b
	D3 at 6.25%	0.07	OK	Eq. H1-1b
24	D1 at 100.00%	0.15	OK	Eq. H1-1b
	D2 at 100.00%	**0.29**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.22	OK	Eq. H1-1b
26	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	**0.19**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.17	OK	Eq. H1-1b
28	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.09	OK	Eq. H1-1b
29	D1 at 100.00%	0.03	OK	Eq. H1-1b
	D2 at 100.00%	**0.09**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
32	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.14**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.12	OK	Eq. H1-1b
33	D1 at 0.00%	0.11	OK	Eq. H1-1b
	D2 at 0.00%	**0.22**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.17	OK	Eq. H1-1b
34	D1 at 43.75%	0.12	OK	Eq. H1-1b
	D2 at 43.75%	**0.13**	**OK**	Eq. H1-1b
	D3 at 43.75%	0.09	OK	Eq. H1-1b
36	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	0.05	OK	Eq. H1-1b
	D3 at 0.00%	**0.07**	**OK**	Eq. H1-1b
38	D1 at 100.00%	**0.08**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.07	OK	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
39	D1 at 0.00%	0.08	OK	Eq. H1-1b
	D2 at 0.00%	**0.16**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.13	OK	Eq. H1-1b
40	D1 at 100.00%	0.04	OK	Eq. H1-1b
	D2 at 100.00%	**0.09**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
41	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.10**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.08	OK	Eq. H1-1b
42	D1 at 100.00%	0.06	OK	Eq. H1-1b
	D2 at 0.00%	**0.18**	**OK**	
	D3 at 0.00%	0.17	OK	
43	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	0.14	OK	
	D3 at 0.00%	**0.15**	**OK**	

	44	D1 at 0.00%	0.03	OK	Eq. H1-1b
		D2 at 0.00%	**0.16**	**OK**	
		D3 at 0.00%	0.16	OK	
	45	D1 at 0.00%	0.06	OK	Eq. H1-1b
		D2 at 0.00%	0.15	OK	
		D3 at 0.00%	**0.16**	**OK**	
	46	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 100.00%	**0.32**	**OK**	Eq. H1-1b
		D3 at 100.00%	0.30	OK	Eq. H1-1b
	47	D1 at 90.63%	0.07	OK	Eq. H1-1b
		D2 at 100.00%	**0.32**	**OK**	Eq. H1-1b
		D3 at 100.00%	0.30	OK	Eq. H1-1b
	48	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 100.00%	0.13	OK	Eq. H1-1b
		D3 at 100.00%	**0.16**	**OK**	Eq. H1-1b
	49	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 100.00%	0.13	OK	Eq. H1-1b
		D3 at 93.75%	**0.16**	**OK**	Eq. H1-1b
L 4X4X3_8	**14**	D1 at 0.00%	0.12	OK	Eq. H1-1b
		D2 at 100.00%	0.19	OK	Eq. H1-1b
		D3 at 100.00%	**0.21**	**OK**	Eq. H1-1b
	15	D1 at 100.00%	0.10	OK	Eq. H1-1b
		D2 at 0.00%	0.17	OK	Eq. H1-1b
		D3 at 0.00%	**0.21**	**OK**	Eq. H1-1b
	50	D1 at 100.00%	0.17	OK	Eq. H1-1b
		D2 at 100.00%	**0.83**	**OK**	Eq. H1-1b
		D3 at 100.00%	0.76	OK	Eq. H1-1b
	51	D1 at 100.00%	0.18	OK	Eq. H1-1b
		D2 at 100.00%	**0.84**	**OK**	Eq. H1-1b
		D3 at 100.00%	0.77	OK	Eq. H1-1b

Current Date: 2/10/2017 9:21 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind East) (Rev3).etz\

Analysis result

Reactions



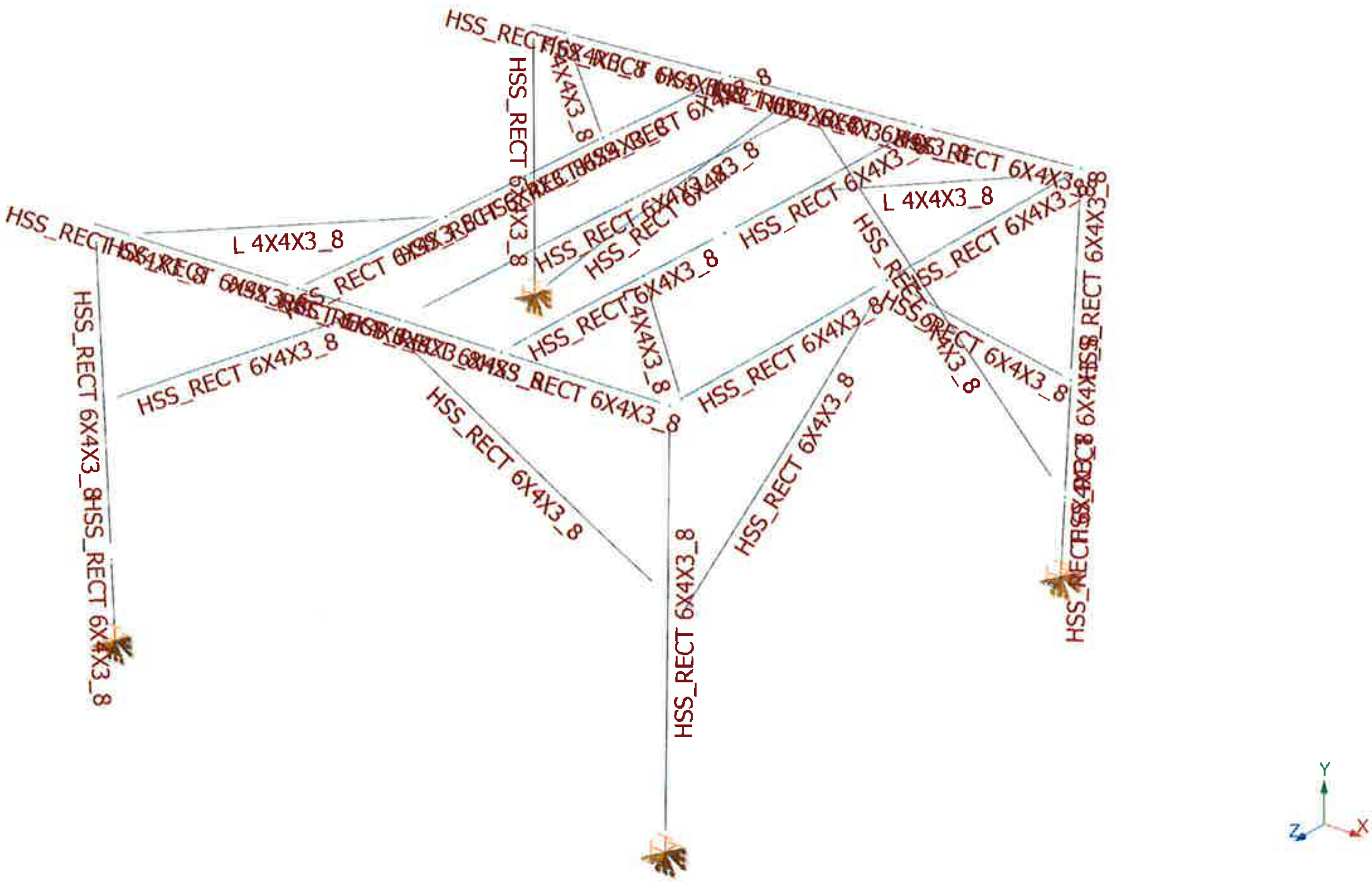
Direction of positive forces and moments

	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **D1=DL**						
16	0.74789	1.59434	-0.07895	-0.30265	0.08577	-2.68299
17	-0.78350	2.35302	-0.31619	-1.00498	-0.08825	2.77448
18	-1.10655	2.42763	0.26557	0.70694	0.10205	1.97656
25	1.14216	1.54669	0.12956	0.08125	-0.39380	0.21401
SUM	0.00000	7.92167	0.00000	-0.51944	-0.29424	2.28206
Condition **D2=DL+W**						
16	0.76524	1.80585	0.04829	0.19142	0.05887	-2.80707
17	-1.14451	4.81937	-0.70677	-2.24155	-0.07953	4.14376
18	-1.61574	5.09956	0.70917	1.99588	-0.02833	2.89952
25	1.27137	1.63856	-0.05068	-0.34013	-0.23686	0.39262
SUM	-0.72363	13.36335	0.00000	-0.39437	-0.28585	4.62883
Condition **D3=0.6DL+W**						
16	0.46608	1.16812	0.07987	0.31248	0.02456	-1.73388
17	-0.83111	3.87816	-0.58030	-1.83956	-0.04422	3.03397
18	-1.17312	4.12851	0.60294	1.71310	-0.06915	2.10890
25	0.81451	1.01989	-0.10251	-0.37263	-0.07934	0.30702
SUM	-0.72363	10.19468	0.00000	-0.18660	-0.16815	3.71601

Current Date: 2/8/2017 4:12 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind West) (Rev3).et:

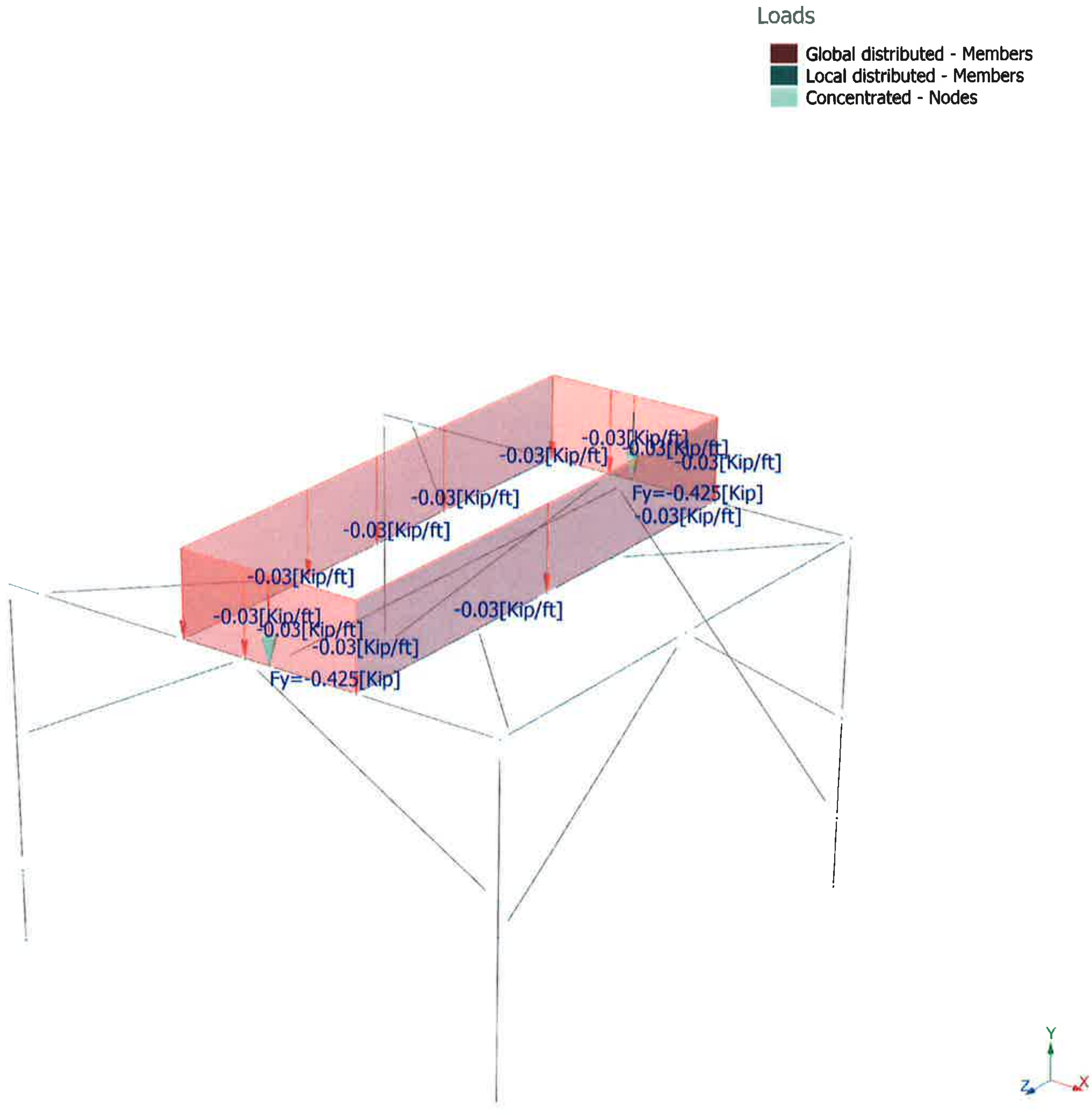
ALPHA FRAME
(WESTERLY WIND)

Current Date: 2/8/2017 4:12 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind West) (Rev3).et:



ALPHA FRAME
(WESTERLY WIND)

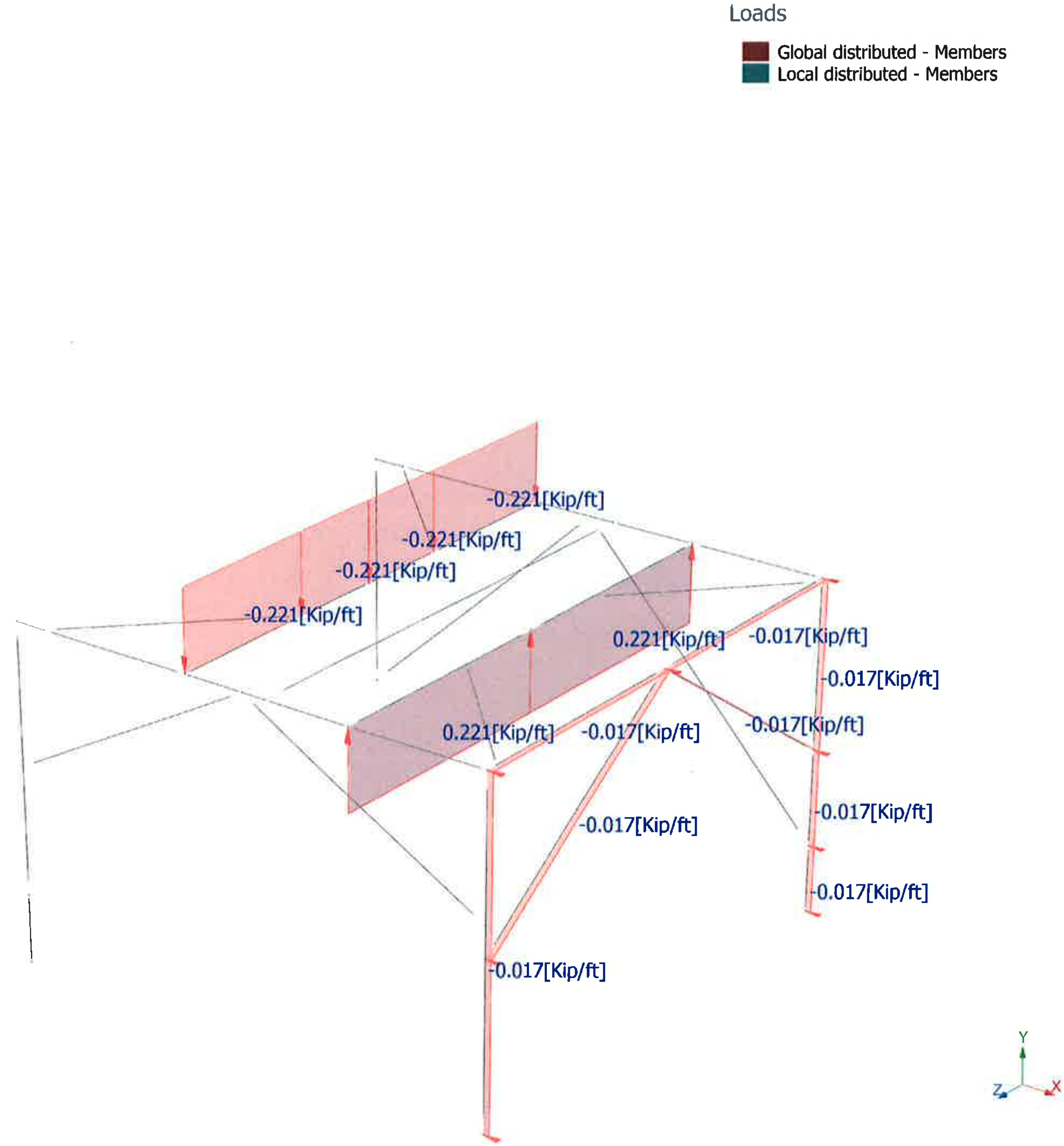
Current Date: 2/8/2017 4:12 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind West) (Rev3).etz
Load condition: DL=Dead Load

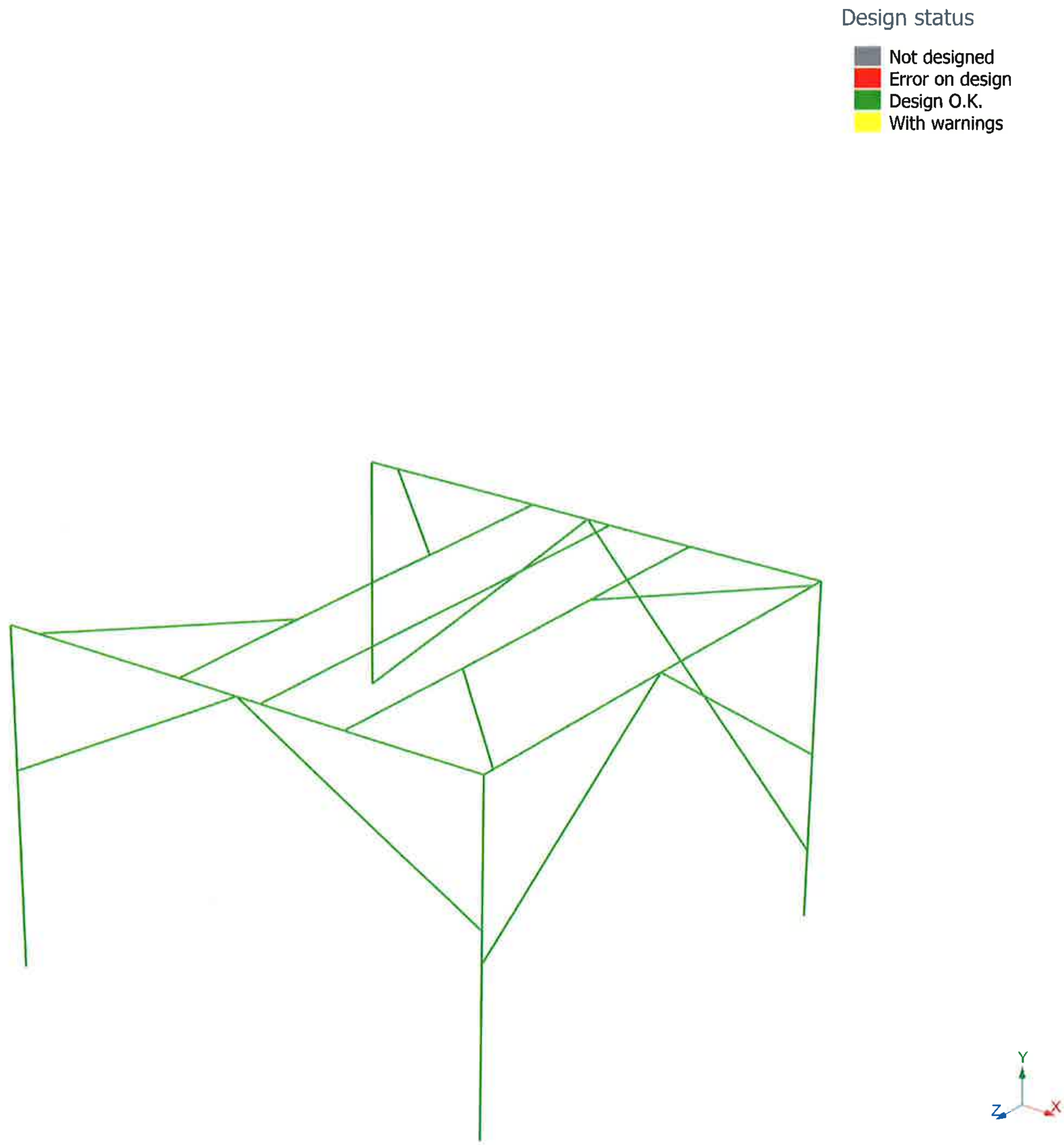


ALPHA FRAME
(WESTERLY WIND)

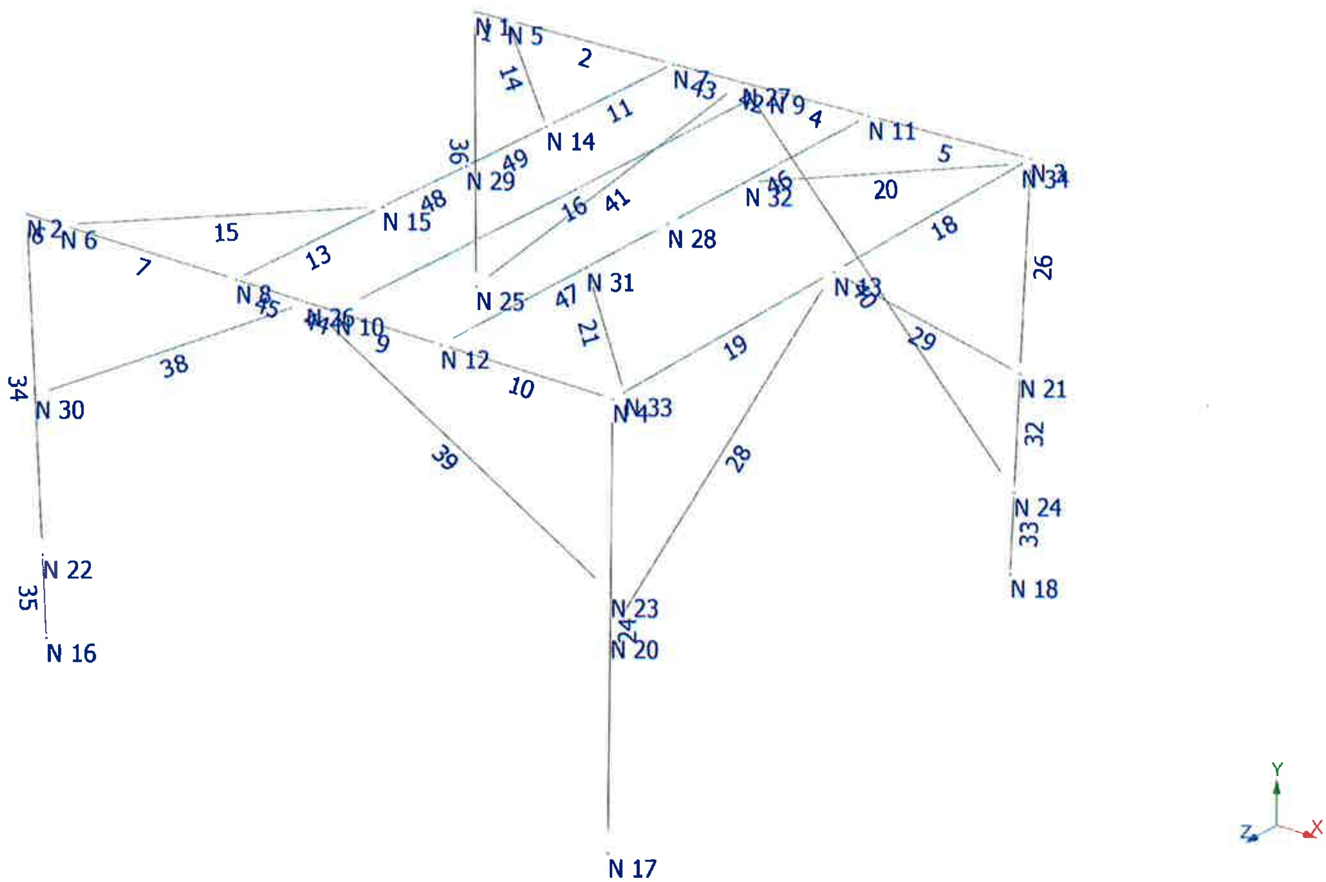
Bentley

Current Date: 2/10/2017 9:27 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind West) (Rev3).et:
Load condition: W=Wind




Design status
Not designed
Error on design
Design O.K.
With warnings
Y
Z
X

Current Date: 2/8/2017 4:12 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind West) (Rev3).et



ALPHA FRAME
(WESTERLY WIND)

Bentley

Current Date: 2/10/2017 9:28 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind West) (Rev3).etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

D1=DL
D2=DL+W
D3=0.6DL+W

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_RECT 6X4X3_8	**1**	D1 at 0.00%	0.03	OK	
			D2 at 0.00%	**0.10**	**OK**	
			D3 at 0.00%	0.09	OK	
		2	D1 at 0.00%	0.02	OK	
			D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.10	OK	Eq. H1-1b
		4	D1 at 100.00%	0.04	OK	Eq. H1-1b
			D2 at 0.00%	0.13	OK	
			D3 at 0.00%	**0.14**	**OK**	
		5	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 100.00%	0.09	OK	Eq. H1-1b
			D3 at 100.00%	**0.10**	**OK**	Eq. H1-1b
		6	D1 at 0.00%	0.02	OK	
			D2 at 0.00%	**0.09**	**OK**	
			D3 at 0.00%	0.08	OK	
		7	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 100.00%	**0.15**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.13	OK	Eq. H1-1b
		9	D1 at 100.00%	0.06	OK	Eq. H1-1b
			D2 at 0.00%	0.13	OK	
			D3 at 0.00%	**0.14**	**OK**	
		10	D1 at 100.00%	0.06	OK	Eq. H1-1b
			D2 at 100.00%	0.06	OK	Eq. H1-1b
			D3 at 100.00%	**0.09**	**OK**	Eq. H1-1b
		11	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 0.00%	**0.23**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.21	OK	Eq. H1-1b
		13	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 0.00%	**0.21**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.20	OK	Eq. H1-1b
		16	D1 at 56.25%	0.04	OK	Eq. H1-1b
			D2 at 62.50%	**0.04**	**OK**	Eq. H1-1b
			D3 at 68.75%	0.02	OK	Eq. H1-1b
		18	D1 at 34.38%	0.02	OK	Eq. H1-1b

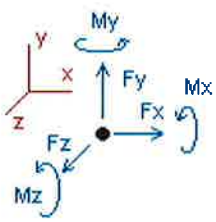
	D2 at 6.25%	0.07	OK	Eq. H1-1b
	D3 at 6.25%	**0.08**	**OK**	Eq. H1-1b
19	D1 at 18.75%	0.02	OK	Eq. H1-1b
	D2 at 6.25%	0.07	OK	Eq. H1-1b
	D3 at 6.25%	**0.08**	**OK**	Eq. H1-1b
24	D1 at 100.00%	**0.15**	**OK**	Eq. H1-1b
	D2 at 41.67%	0.12	OK	Eq. H1-1b
	D3 at 0.00%	0.10	OK	Eq. H1-1b
26	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	0.06	OK	Eq. H1-1b
	D3 at 0.00%	**0.08**	**OK**	Eq. H1-1b
28	D1 at 0.00%	**0.05**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.04	OK	Eq. H1-1b
	D3 at 100.00%	0.03	OK	Eq. H1-1b
29	D1 at 100.00%	**0.03**	**OK**	Eq. H1-1b
	D2 at 50.00%	0.02	OK	Eq. H1-1b
	D3 at 100.00%	0.02	OK	Eq. H1-1b
32	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	0.06	OK	Eq. H1-1b
	D3 at 100.00%	**0.06**	**OK**	Eq. H1-1b
33	D1 at 0.00%	**0.11**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.06	OK	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
34	D1 at 53.13%	0.12	OK	Eq. H1-1b
	D2 at 53.13%	**0.21**	**OK**	Eq. H1-1b
	D3 at 53.13%	0.16	OK	Eq. H1-1b
35	D1 at 0.00%	0.11	OK	Eq. H1-1b
	D2 at 0.00%	**0.19**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.14	OK	Eq. H1-1b
36	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	**0.15**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.14	OK	Eq. H1-1b
38	D1 at 100.00%	0.08	OK	Eq. H1-1b
	D2 at 100.00%	**0.17**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.14	OK	Eq. H1-1b
39	D1 at 0.00%	**0.08**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.05	OK	Eq. H1-1b
	D3 at 0.00%	0.05	OK	Eq. H1-1b
40	D1 at 100.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.10	OK	Eq. H1-1b
41	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.10	OK	Eq. H1-1b
42	D1 at 100.00%	0.06	OK	Eq. H1-1b
	D2 at 0.00%	0.11	OK	
	D3 at 0.00%	**0.12**	**OK**	

	43	D1 at 100.00%	0.05	OK	Eq. H1-1b
		D2 at 0.00%	**0.22**	**OK**	
		D3 at 0.00%	0.20	OK	
	44	D1 at 0.00%	0.03	OK	Eq. H1-1b
		D2 at 0.00%	0.14	OK	
		D3 at 0.00%	**0.15**	**OK**	
	45	D1 at 0.00%	0.06	OK	Eq. H1-1b
		D2 at 0.00%	**0.21**	**OK**	Eq. H3-6
		D3 at 0.00%	0.20	OK	
	46	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 93.75%	0.15	OK	Eq. H1-1b
		D3 at 96.88%	**0.18**	**OK**	Eq. H1-1b
	47	D1 at 90.63%	0.07	OK	Eq. H1-1b
		D2 at 100.00%	0.15	OK	Eq. H1-1b
		D3 at 100.00%	**0.18**	**OK**	Eq. H1-1b
	48	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 100.00%	**0.33**	**OK**	Eq. H1-1b
		D3 at 100.00%	0.30	OK	Eq. H1-1b
	49	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 100.00%	**0.33**	**OK**	Eq. H1-1b
		D3 at 100.00%	0.30	OK	Eq. H1-1b
L 4X4X3_8	**14**	D1 at 0.00%	0.12	OK	Eq. H1-1b
		D2 at 0.00%	**0.45**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.40	OK	Eq. H1-1b
	15	D1 at 100.00%	0.10	OK	Eq. H1-1b
		D2 at 0.00%	**0.39**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.35	OK	Eq. H1-1b
	20	D1 at 100.00%	0.17	OK	Eq. H1-1b
		D2 at 100.00%	0.48	OK	Eq. H1-1b
		D3 at 100.00%	**0.55**	**OK**	Eq. H1-1b
	21	D1 at 100.00%	0.18	OK	Eq. H1-1b
		D2 at 100.00%	0.47	OK	Eq. H1-1b
		D3 at 100.00%	**0.54**	**OK**	Eq. H1-1b

Current Date: 2/10/2017 9:28 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind West) (Rev3).etz\

Analysis result

Reactions



Direction of positive forces and moments

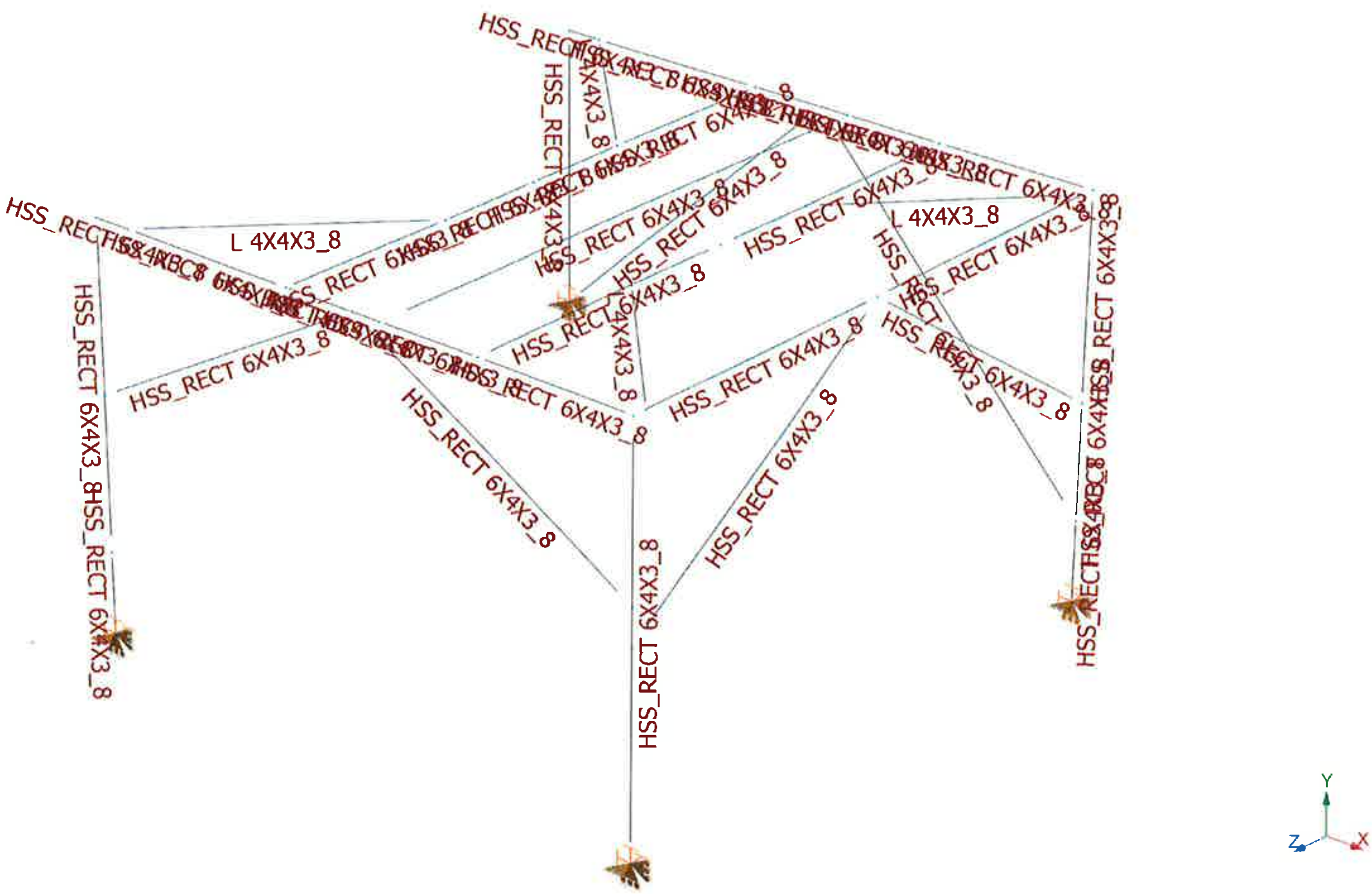
	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **D1=DL**						
16	0.74789	1.59434	-0.07895	-0.30265	0.08577	-2.68299
17	-0.78350	2.35302	-0.31619	-1.00498	-0.08825	2.77448
18	-1.10655	2.42763	0.26557	0.70694	0.10205	1.97656
25	1.14216	1.54669	0.12956	0.08125	-0.39380	0.21401
SUM	0.00000	7.92167	0.00000	-0.51944	-0.29424	2.28206
Condition **D2=DL+W**						
16	1.07566	2.49428	-0.23750	-0.83005	0.22750	-4.02275
17	-0.58512	1.48064	-0.18048	-0.56392	-0.27652	2.01688
18	-0.90267	1.38426	0.04444	-0.00087	0.43619	1.42225
25	1.61538	2.56250	0.37354	0.45530	-0.86777	0.33376
SUM	1.20325	7.92167	0.00000	-0.93954	-0.48060	-0.24985
Condition **D3=0.6DL+W**						
16	0.77651	1.85654	-0.20592	-0.70899	0.19320	-2.94955
17	-0.27172	0.53943	-0.05400	-0.16193	-0.24122	0.90709
18	-0.46005	0.41321	-0.06179	-0.28365	0.39537	0.63163
25	1.15852	1.94383	0.32171	0.42280	-0.71025	0.24816
SUM	1.20325	4.75300	0.00000	-0.73177	-0.36290	-1.16268

Bentley

Current Date: 2/8/2017 4:17 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind North) (Rev3).etz

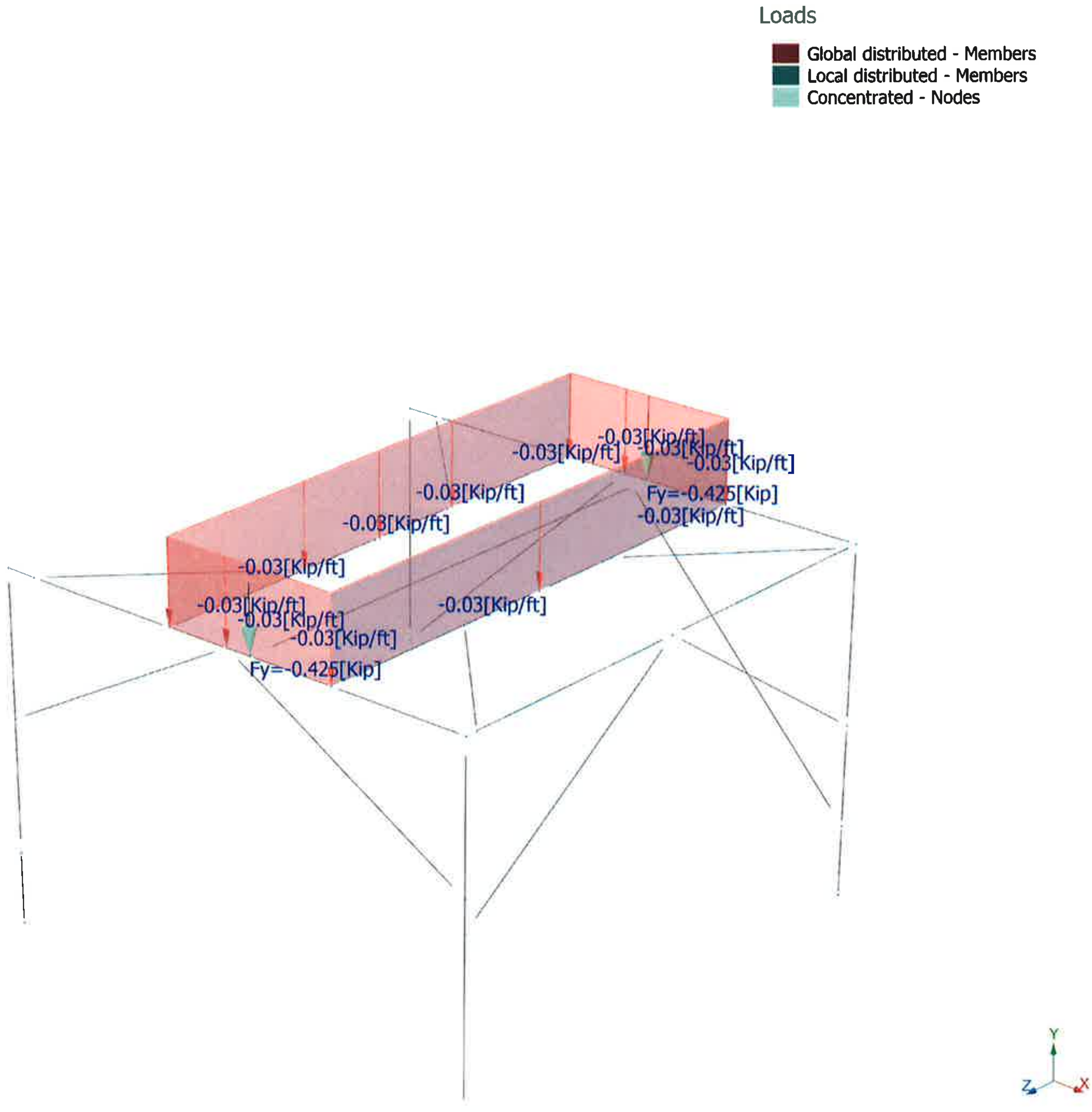
ALPHA FRAME
(NORTHERLY WIND)

Current Date: 2/8/2017 4:17 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind North) (Rev3).et



ALPHA FRAME
(NORTHERLY WIND)

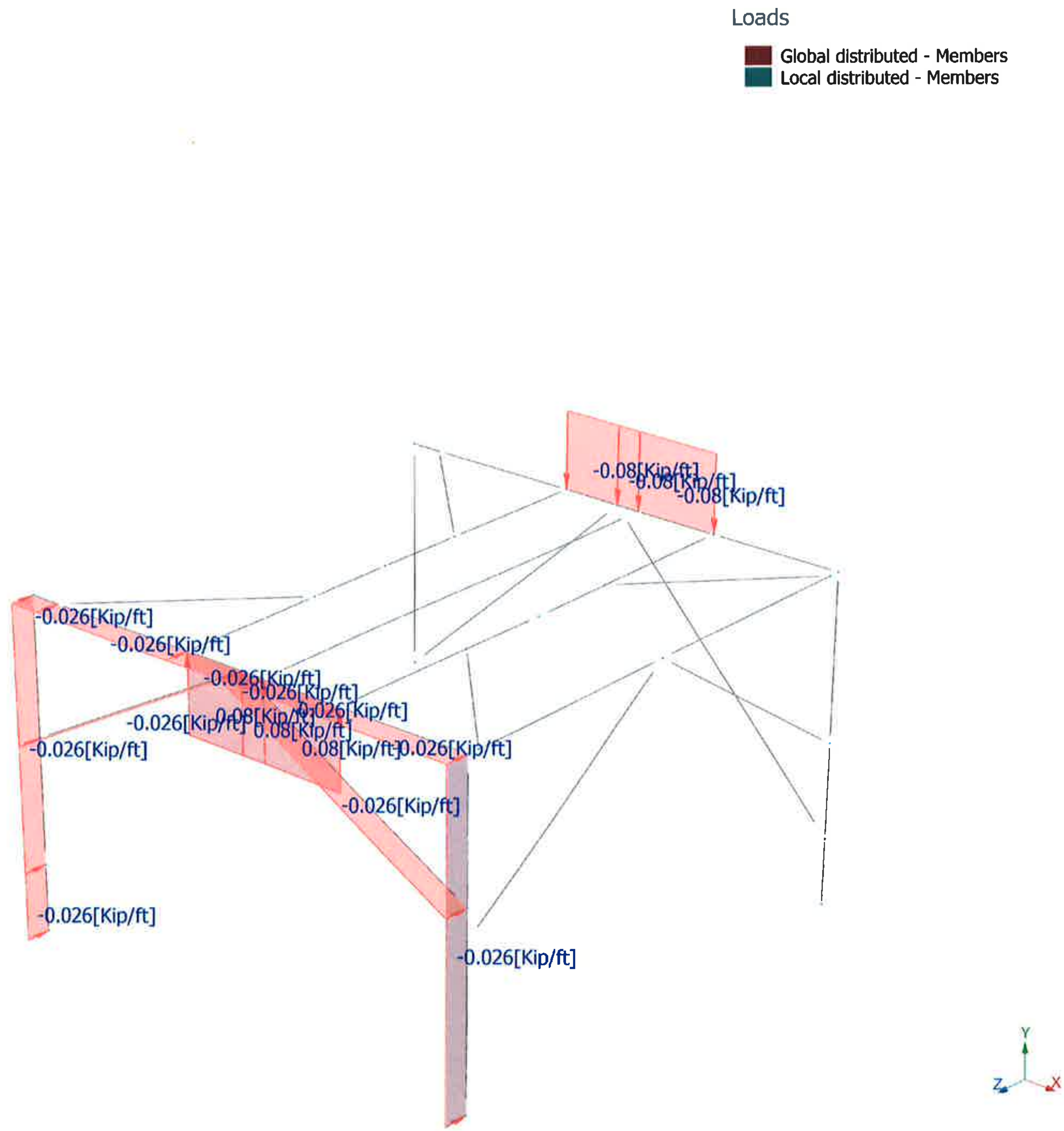
Current Date: 2/8/2017 4:18 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind North) (Rev3).et
Load condition: DL=Dead Load

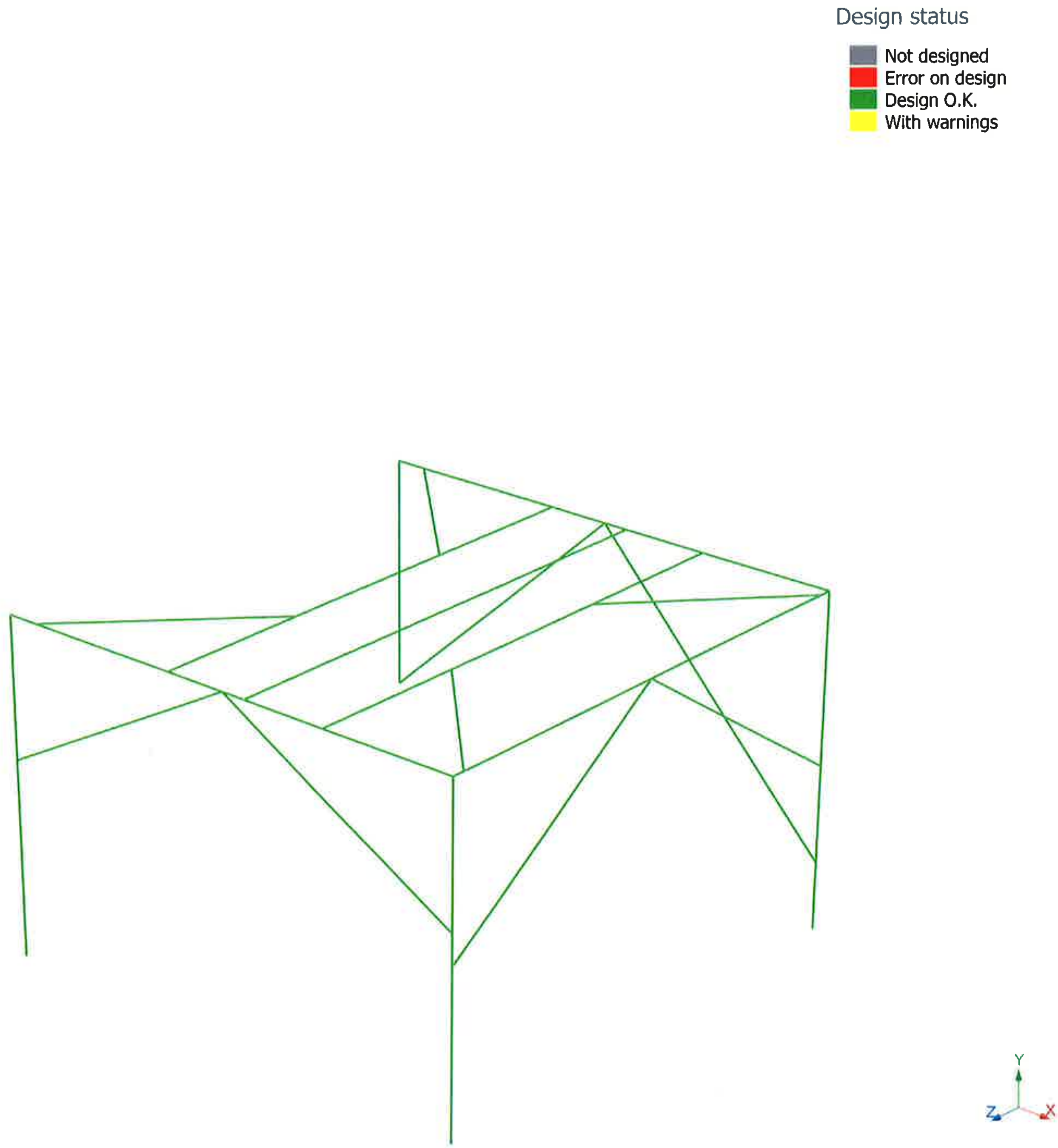


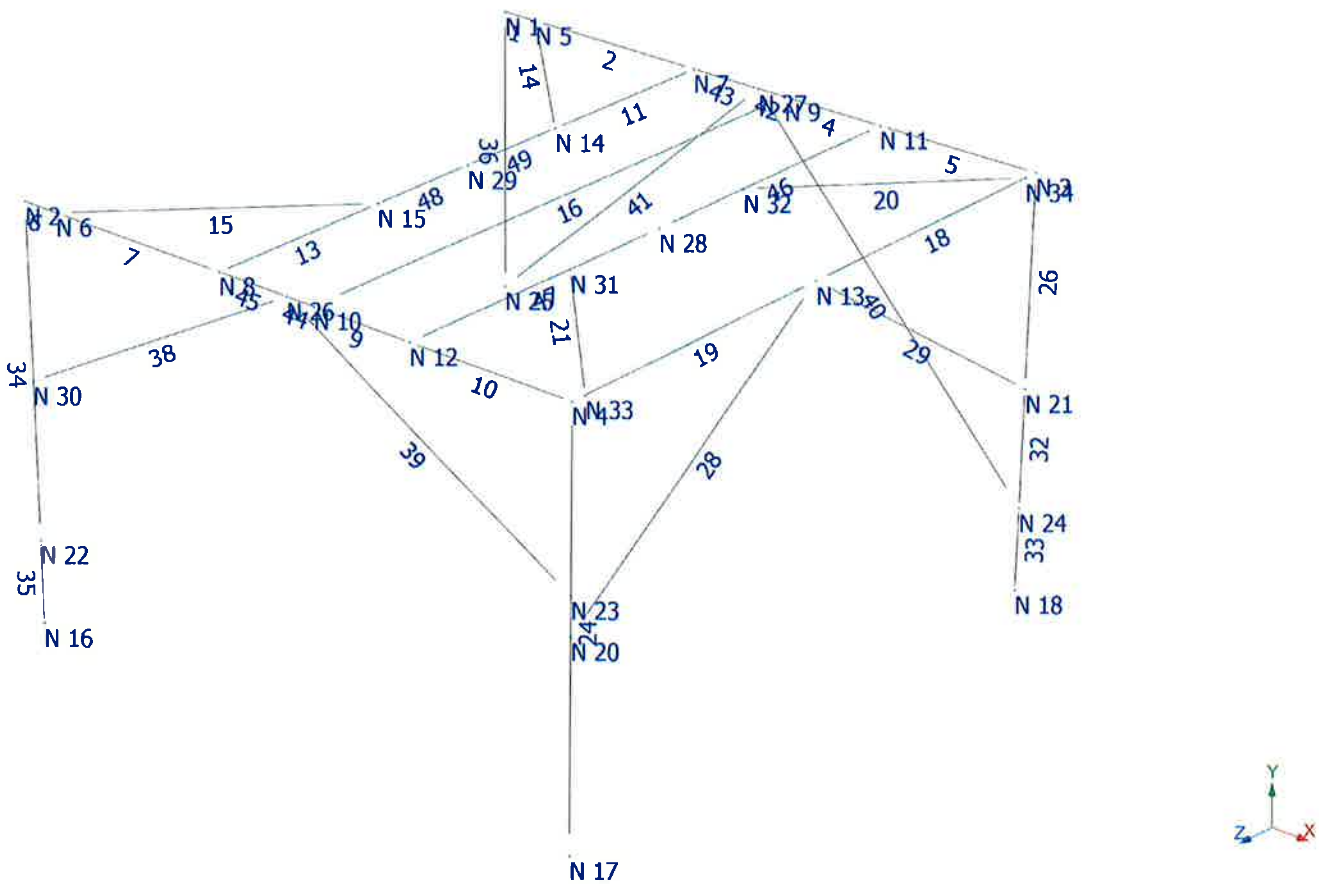
ALPHA FRAME
(NORTHERLY WIND)

Bentley

Current Date: 2/10/2017 9:36 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind North) (Rev3).etz
Load condition: W=Wind




Design status
Not designed
Error on design
Design O.K.
With warnings
Y
Z
X


N 1
N 5
2
14
N 14
11
N 27
N 9
4
N 11
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36
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N 29
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48
16
41
N 32
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N 34
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N 28
18
N 8
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N 26
N 10
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N 31
21
N 13
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N 12
19
34
N 30
38
10
N 33
29
N 21
39
28
32
N 24
N 22
35
N 16
N 23
N 20
33
N 18
N 17
Y
Z
X

Bentley

Current Date: 2/10/2017 9:37 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind North) (Rev3).etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

D1=DL
D2=DL+W
D3=0.6DL+W

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_RECT 6X4X3_8	1	D1 at 0.00%	0.03	OK	
			D2 at 0.00%	**0.04**	**OK**	
			D3 at 0.00%	0.03	OK	
		2	D1 at 0.00%	0.02	OK	
			D2 at 100.00%	**0.03**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.03	OK	Eq. H1-1b
		4	D1 at 100.00%	0.04	OK	Eq. H1-1b
			D2 at 100.00%	**0.06**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.05	OK	Eq. H1-1b
		5	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 100.00%	**0.05**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.04	OK	Eq. H1-1b
		6	D1 at 0.00%	0.02	OK	
			D2 at 0.00%	**0.02**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.01	OK	Eq. H1-1b
		7	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 100.00%	**0.05**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.03	OK	Eq. H1-1b
		9	D1 at 100.00%	**0.06**	**OK**	Eq. H1-1b
			D2 at 100.00%	0.06	OK	Eq. H1-1b
			D3 at 100.00%	0.04	OK	Eq. H1-1b
		10	D1 at 100.00%	**0.06**	**OK**	Eq. H1-1b
			D2 at 100.00%	0.06	OK	Eq. H1-1b
			D3 at 100.00%	0.03	OK	Eq. H1-1b
		11	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 0.00%	**0.08**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.06	OK	Eq. H1-1b
		13	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 100.00%	**0.07**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.05	OK	Eq. H1-1b
		16	D1 at 56.25%	**0.04**	**OK**	Eq. H1-1b
			D2 at 68.75%	0.04	OK	Eq. H1-1b
			D3 at 0.00%	0.03	OK	Eq. H1-1b
		18	D1 at 34.38%	0.02	OK	Eq. H1-1b

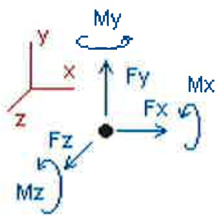
	D2 at 6.25%	**0.03**	**OK**	Eq. H1-1b
	D3 at 6.25%	0.02	OK	Eq. H1-1b
19	D1 at 18.75%	0.02	OK	Eq. H1-1b
	D2 at 0.00%	**0.02**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.02	OK	Eq. H1-1b
24	D1 at 100.00%	**0.15**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.12	OK	Eq. H1-1b
	D3 at 100.00%	0.10	OK	Eq. H1-1b
26	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 100.00%	**0.04**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.04	OK	Eq. H1-1b
28	D1 at 0.00%	**0.05**	**OK**	Eq. H1-1b
	D2 at 31.25%	0.02	OK	Eq. H1-1b
	D3 at 0.00%	0.02	OK	Eq. H1-1b
29	D1 at 100.00%	0.03	OK	Eq. H1-1b
	D2 at 100.00%	**0.06**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.04	OK	Eq. H1-1b
32	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.10	OK	Eq. H1-1b
33	D1 at 0.00%	0.11	OK	Eq. H1-1b
	D2 at 0.00%	**0.22**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.18	OK	Eq. H1-1b
34	D1 at 53.13%	**0.12**	**OK**	Eq. H1-1b
	D2 at 53.13%	0.11	OK	Eq. H1-1b
	D3 at 53.13%	0.07	OK	Eq. H1-1b
35	D1 at 0.00%	0.11	OK	Eq. H1-1b
	D2 at 0.00%	**0.17**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.14	OK	Eq. H1-1b
36	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 100.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.05	OK	Eq. H1-1b
38	D1 at 100.00%	**0.08**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.07	OK	Eq. H1-1b
	D3 at 100.00%	0.04	OK	Eq. H1-1b
39	D1 at 0.00%	**0.08**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.07	OK	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
40	D1 at 100.00%	0.04	OK	Eq. H1-1b
	D2 at 100.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.03	OK	Eq. H1-1b
41	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.09**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.07	OK	Eq. H1-1b
42	D1 at 100.00%	0.06	OK	Eq. H1-1b
	D2 at 100.00%	**0.10**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b

	43	D1 at 100.00%	0.05	OK	Eq. H1-1b
		D2 at 0.00%	**0.07**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.06	OK	Eq. H1-1b
	44	D1 at 0.00%	0.03	OK	Eq. H1-1b
		D2 at 0.00%	**0.04**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.02	OK	Eq. H1-1b
	45	D1 at 0.00%	0.06	OK	Eq. H1-1b
		D2 at 0.00%	**0.08**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.05	OK	Eq. H1-1b
	46	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 65.63%	**0.08**	**OK**	Eq. H1-1b
		D3 at 65.63%	0.06	OK	Eq. H1-1b
	47	D1 at 90.63%	0.07	OK	Eq. H1-1b
		D2 at 65.63%	**0.08**	**OK**	Eq. H1-1b
		D3 at 65.63%	0.06	OK	Eq. H1-1b
	48	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 62.50%	**0.09**	**OK**	Eq. H1-1b
		D3 at 37.50%	0.06	OK	Eq. H1-1b
	49	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 100.00%	**0.08**	**OK**	Eq. H1-1b
		D3 at 100.00%	0.05	OK	Eq. H1-1b
L 4X4X3_8	**14**	D1 at 0.00%	0.12	OK	Eq. H1-1b
		D2 at 0.00%	**0.16**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.12	OK	Eq. H1-1b
	15	D1 at 100.00%	**0.10**	**OK**	Eq. H1-1b
		D2 at 100.00%	0.09	OK	Eq. H1-1b
		D3 at 87.50%	0.06	OK	Eq. H1-1b
	20	D1 at 100.00%	0.17	OK	Eq. H1-1b
		D2 at 100.00%	**0.23**	**OK**	Eq. H1-1b
		D3 at 100.00%	0.16	OK	Eq. H1-1b
	21	D1 at 100.00%	**0.18**	**OK**	Eq. H1-1b
		D2 at 100.00%	0.14	OK	Eq. H1-1b
		D3 at 100.00%	0.07	OK	Eq. H1-1b

Current Date: 2/10/2017 9:37 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind North) (Rev3).etz\

Analysis result

Reactions



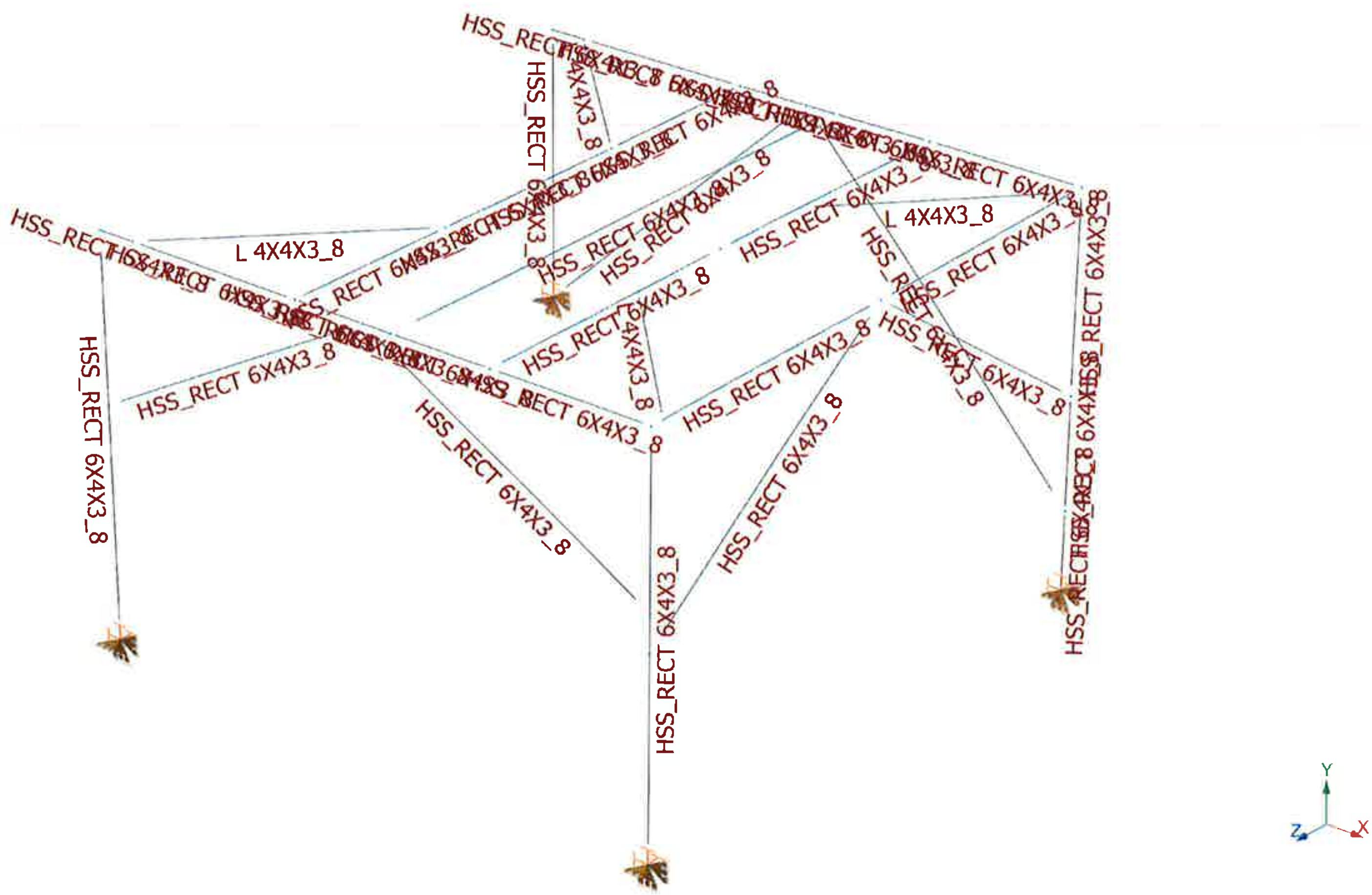
Direction of positive forces and moments

	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **D1=DL**						
16	0.74789	1.59434	-0.07895	-0.30265	0.08577	-2.68299
17	-0.78350	2.35302	-0.31619	-1.00498	-0.08825	2.77448
18	-1.10655	2.42763	0.26557	0.70694	0.10205	1.97656
25	1.14216	1.54669	0.12956	0.08125	-0.39380	0.21401
SUM	0.00000	7.92167	0.00000	-0.51944	-0.29424	2.28206
Condition **D2=DL+W**						
16	0.68403	1.43405	0.32077	1.49643	0.13096	-2.54361
17	-0.46442	1.49928	0.35491	1.20815	0.02149	1.54722
18	-1.47517	3.32342	0.72546	2.36070	0.13064	2.64941
25	1.25555	1.66492	0.39873	1.77842	-1.10319	0.24155
SUM	0.00000	7.92167	1.79986	6.84370	-0.82011	1.89457
Condition **D3=0.6DL+W**						
16	0.38487	0.79632	0.35235	1.61749	0.09665	-1.47041
17	-0.15102	0.55807	0.48138	1.61014	0.05679	0.43743
18	-1.03255	2.35237	0.61923	2.07792	0.08981	1.85879
25	0.79869	1.04625	0.34690	1.74592	-0.94567	0.15594
SUM	0.00000	4.75300	1.79986	7.05148	-0.70242	0.98174

ALPHA FRAME
(SOUTHERLY WIND)

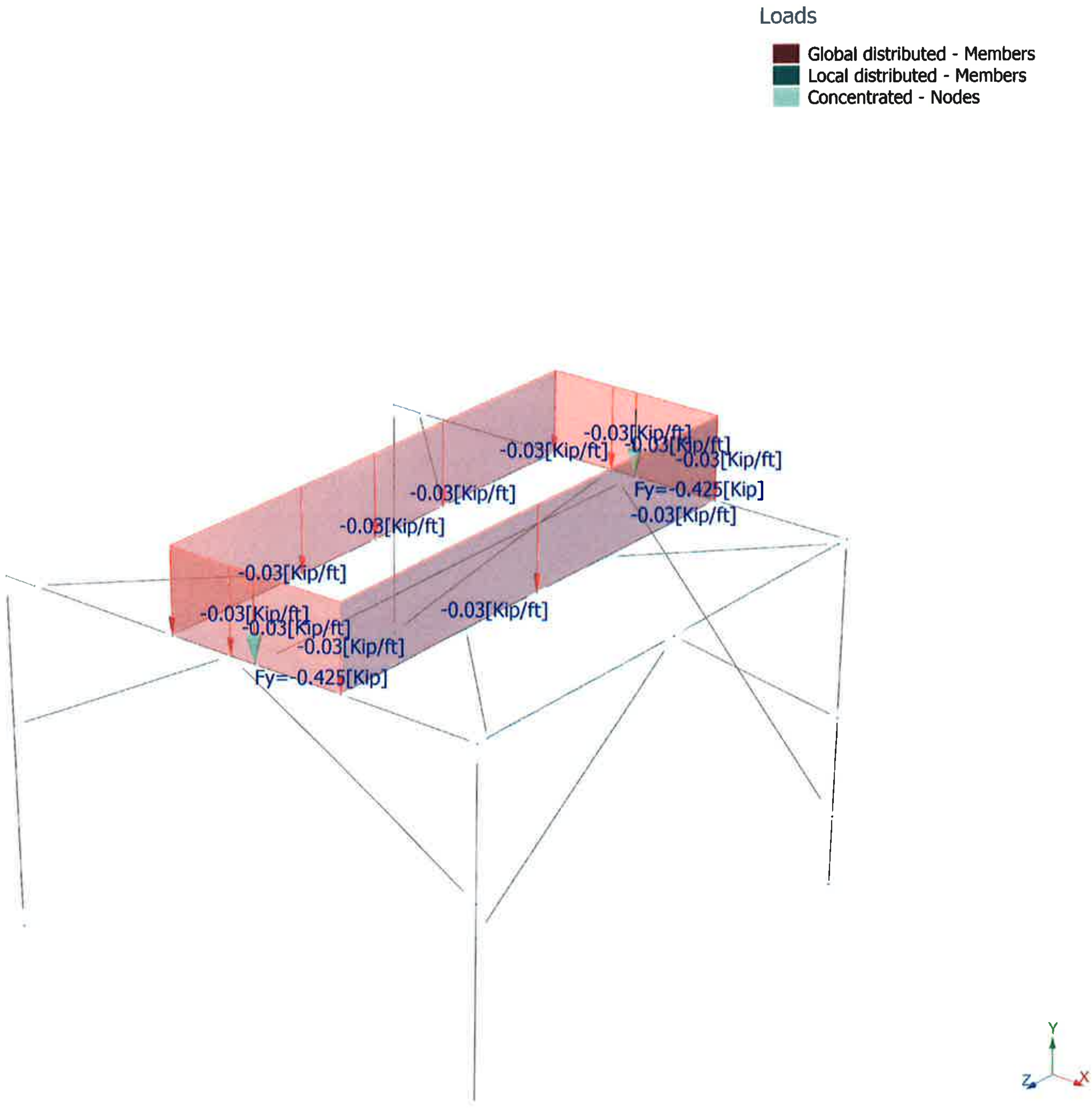
Bentley

Current Date: 2/8/2017 4:21 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind South) (Rev3).et



ALPHA FRAME
(SOUTHERLY WIND)

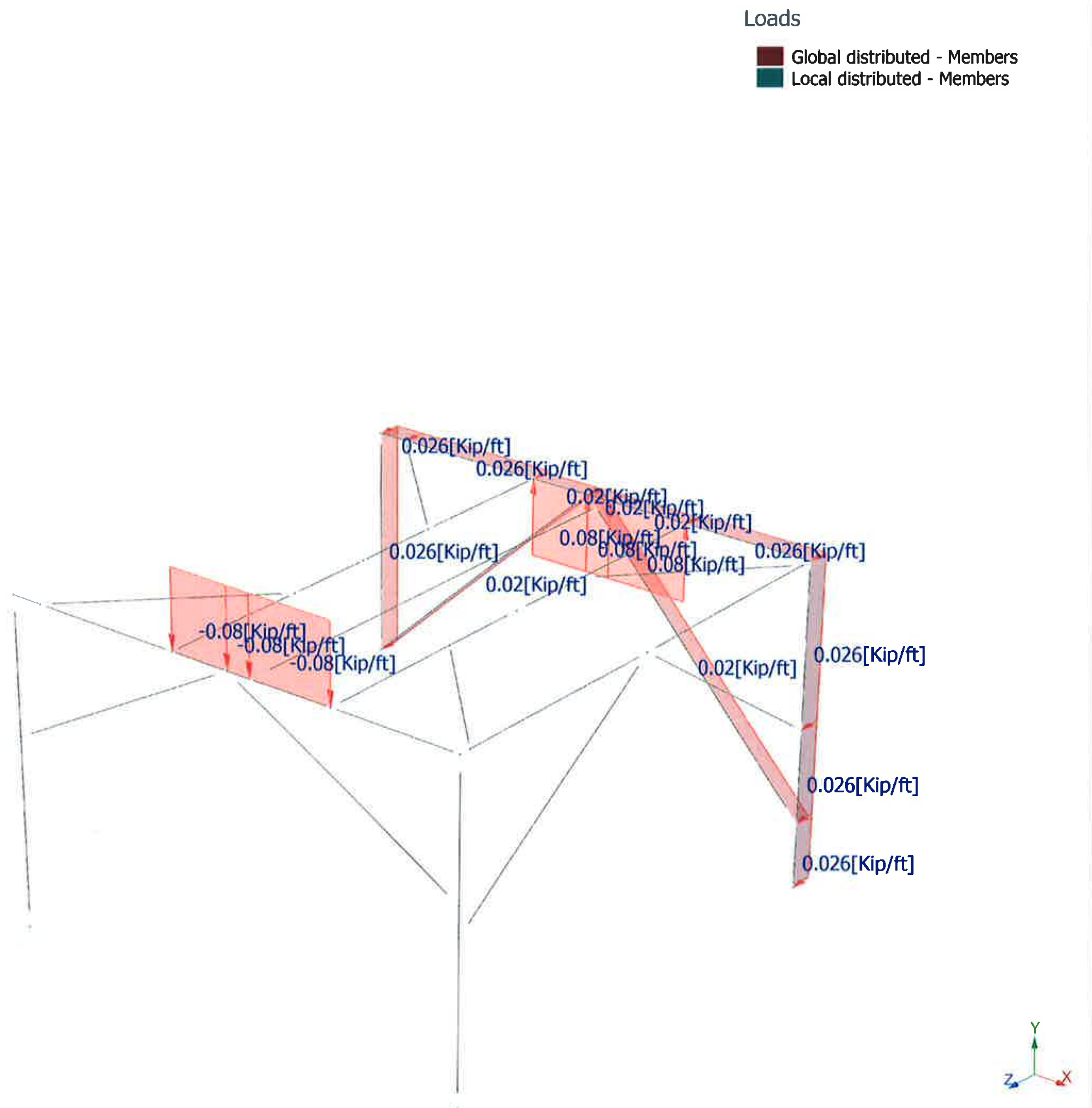
Current Date: 2/8/2017 4:22 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind South) (Rev3).et
Load condition: DL=Dead Load

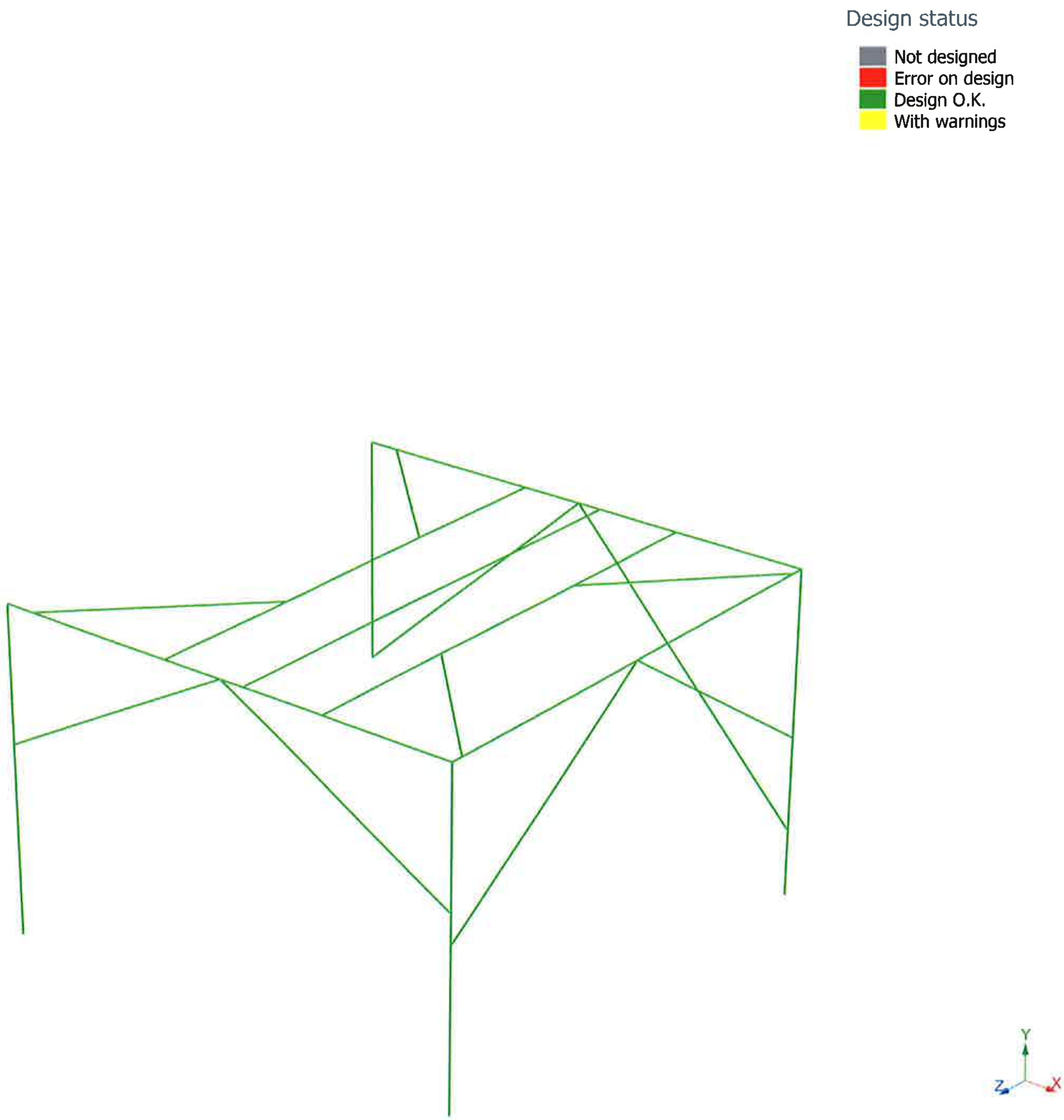


ALPHA FRAME
(SOUTHERLY WIND)

Bentley

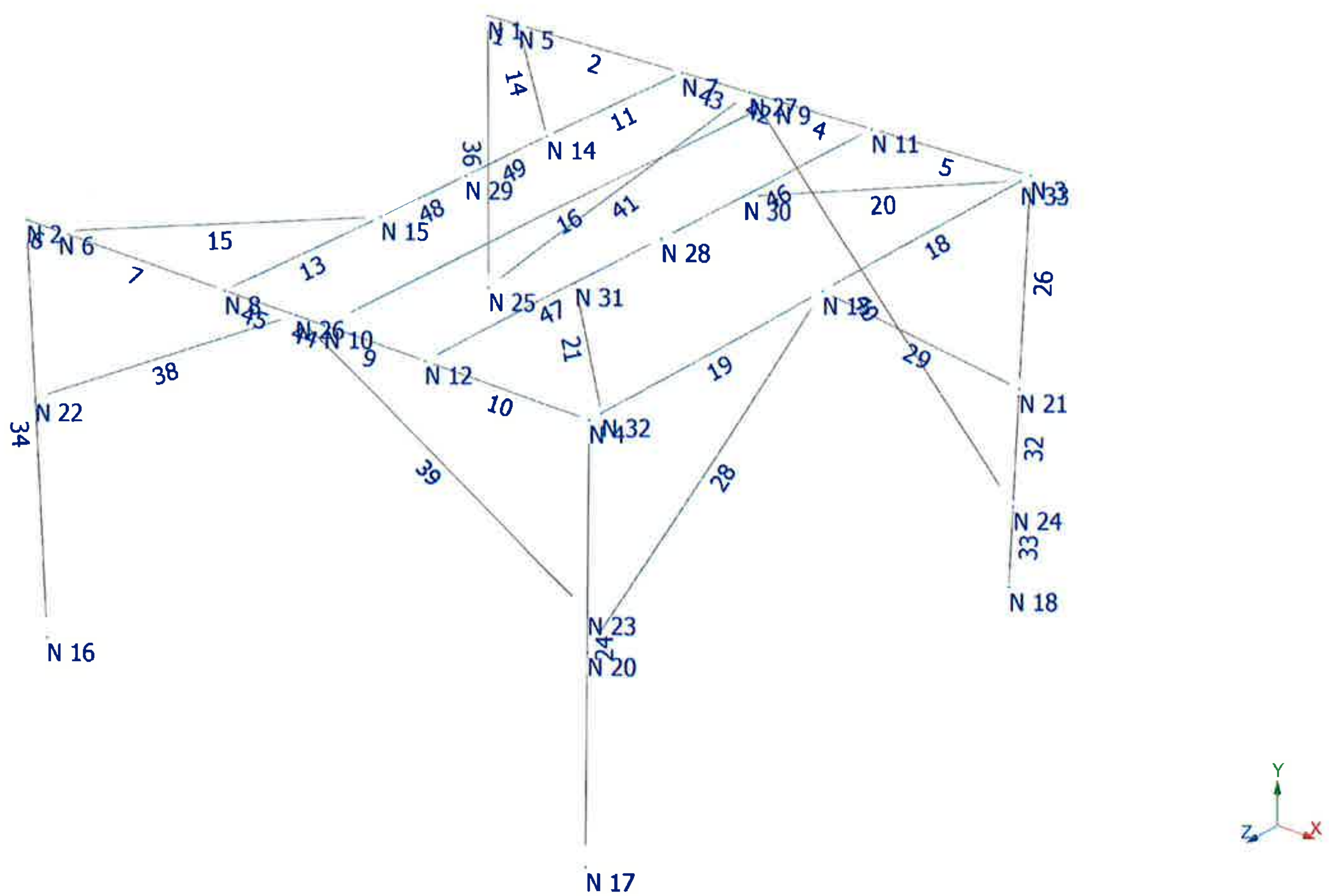
Current Date: 2/10/2017 9:39 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind South) (Rev3).et
Load condition: W=Wind




Design status
Not designed
Error on design
Design O.K.
With warnings
Y
Z
X

Bentley

Current Date: 2/8/2017 4:22 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind South) (Rev3).et



ALPHA FRAME
(SOUTHERLY WIND)

Current Date: 2/10/2017 9:39 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Alpha Frame (Wind South) (Rev3).etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :
D1=DL
D2=DL+W
D3=0.6DL+W

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_RECT 6X4X3_8	1	D1 at 0.00%	**0.03**	**OK**	
			D2 at 0.00%	0.01	OK	Eq. H1-1b
			D3 at 0.00%	0.01	OK	Eq. H1-1b
		2	D1 at 0.00%	**0.02**	**OK**	
			D2 at 100.00%	0.02	OK	Eq. H1-1b
			D3 at 100.00%	0.01	OK	Eq. H1-1b
		4	D1 at 100.00%	0.04	OK	Eq. H1-1b
			D2 at 100.00%	**0.05**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.03	OK	Eq. H1-1b
		5	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 100.00%	**0.04**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.02	OK	Eq. H1-1b
		6	D1 at 0.00%	0.02	OK	
			D2 at 0.00%	**0.03**	**OK**	
			D3 at 0.00%	0.02	OK	
		7	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 100.00%	**0.07**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.05	OK	Eq. H1-1b
		9	D1 at 100.00%	0.06	OK	Eq. H1-1b
			D2 at 100.00%	**0.08**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.06	OK	Eq. H1-1b
		10	D1 at 100.00%	0.06	OK	Eq. H1-1b
			D2 at 100.00%	**0.08**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.05	OK	Eq. H1-1b
		11	D1 at 100.00%	0.05	OK	Eq. H1-1b
			D2 at 100.00%	**0.06**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.04	OK	Eq. H1-1b
		13	D1 at 100.00%	**0.05**	**OK**	Eq. H1-1b
			D2 at 100.00%	0.05	OK	Eq. H1-1b
			D3 at 0.00%	0.03	OK	Eq. H1-1b
		16	D1 at 56.25%	0.04	OK	Eq. H1-1b
			D2 at 62.50%	**0.04**	**OK**	Eq. H1-1b
			D3 at 62.50%	0.02	OK	Eq. H1-1b
		18	D1 at 34.38%	**0.02**	**OK**	Eq. H1-1b

	D2 at 46.88%	0.02	OK	Eq. H1-1b
	D3 at 0.00%	0.02	OK	Eq. H1-1b
19	D1 at 18.75%	0.02	OK	Eq. H1-1b
	D2 at 6.25%	**0.03**	**OK**	Eq. H1-1b
	D3 at 6.25%	0.02	OK	Eq. H1-1b
24	D1 at 100.00%	0.15	OK	Eq. H1-1b
	D2 at 100.00%	**0.27**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.21	OK	Eq. H1-1b
26	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
28	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.07**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.05	OK	Eq. H1-1b
29	D1 at 100.00%	**0.03**	**OK**	Eq. H1-1b
	D2 at 37.50%	0.02	OK	Eq. H1-1b
	D3 at 62.50%	0.01	OK	Eq. H1-1b
32	D1 at 0.00%	**0.05**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.03	OK	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
33	D1 at 0.00%	0.11	OK	Eq. H1-1b
	D2 at 0.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.10	OK	Eq. H1-1b
34	D1 at 43.75%	0.12	OK	Eq. H1-1b
	D2 at 100.00%	**0.15**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.11	OK	Eq. H1-1b
36	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 100.00%	**0.07**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.07	OK	Eq. H1-1b
38	D1 at 100.00%	0.08	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
39	D1 at 0.00%	0.08	OK	Eq. H1-1b
	D2 at 0.00%	**0.10**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.07	OK	Eq. H1-1b
40	D1 at 100.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
41	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.05	OK	Eq. H1-1b
42	D1 at 100.00%	0.06	OK	Eq. H1-1b
	D2 at 100.00%	**0.07**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.05	OK	Eq. H1-1b
43	D1 at 100.00%	**0.05**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.04	OK	Eq. H1-1b
	D3 at 0.00%	0.02	OK	

	44	D1 at 0.00%	0.03	OK	Eq. H1-1b
		D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.04	OK	Eq. H1-1b
	45	D1 at 0.00%	0.06	OK	Eq. H1-1b
		D2 at 0.00%	**0.08**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.06	OK	Eq. H1-1b
	46	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 65.63%	**0.08**	**OK**	Eq. H1-1b
		D3 at 65.63%	0.06	OK	Eq. H1-1b
	47	D1 at 90.63%	0.07	OK	Eq. H1-1b
		D2 at 65.63%	**0.08**	**OK**	Eq. H1-1b
		D3 at 65.63%	0.06	OK	Eq. H1-1b
	48	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 100.00%	**0.08**	**OK**	Eq. H1-1b
		D3 at 100.00%	0.05	OK	Eq. H1-1b
	49	D1 at 100.00%	0.07	OK	Eq. H1-1b
		D2 at 87.50%	**0.08**	**OK**	Eq. H1-1b
		D3 at 75.00%	0.05	OK	Eq. H1-1b
L 4X4X3_8	**14**	D1 at 0.00%	**0.12**	**OK**	Eq. H1-1b
		D2 at 0.00%	0.06	OK	Eq. H1-1b
		D3 at 75.00%	0.03	OK	Eq. H1-1b
	15	D1 at 100.00%	0.10	OK	Eq. H1-1b
		D2 at 0.00%	**0.12**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.08	OK	Eq. H1-1b
	20	D1 at 100.00%	**0.17**	**OK**	Eq. H1-1b
		D2 at 100.00%	0.15	OK	Eq. H1-1b
		D3 at 100.00%	0.08	OK	Eq. H1-1b
	21	D1 at 100.00%	0.18	OK	Eq. H1-1b
		D2 at 100.00%	**0.23**	**OK**	Eq. H1-1b
		D3 at 0.00%	0.17	OK	Eq. H1-1b

Bentley

Current Date: 2/10/2017 9:39 AM
Units system: English
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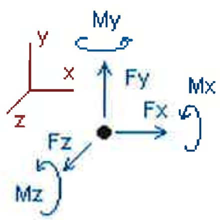
Analysis result

Reactions



Direction of positive forces and moments

	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **D1=DL**						
16	0.74789	1.59434	-0.07895	-0.30265	0.08577	-2.68299
17	-0.78350	2.35302	-0.31619	-1.00498	-0.08825	2.77448
18	-1.10655	2.42763	0.26557	0.70694	0.10205	1.97656
25	1.14216	1.54669	0.12956	0.08125	-0.39380	0.21401
SUM	0.00000	7.92167	0.00000	-0.51944	-0.29424	2.28206
Condition **D2=DL+W**						
16	0.85097	1.78670	-0.14869	-0.84811	0.13805	-2.99598
17	-1.05450	3.07581	-0.68162	-2.36975	-0.14235	3.79631
18	-0.78148	1.66840	-0.37520	-1.20982	-0.03117	1.38428
25	0.98502	1.39076	-0.41709	-2.01646	0.32101	0.19752
SUM	0.00000	7.92167	-1.62260	-6.44414	0.28555	2.38213
Condition **D3=0.6DL+W**						
16	0.55181	1.14897	-0.11711	-0.72705	0.10374	-1.92278
17	-0.74110	2.13460	-0.55514	-1.96776	-0.10704	2.68652
18	-0.33886	0.69735	-0.48143	-1.49260	-0.07199	0.59365
25	0.52815	0.77208	-0.46892	-2.04896	0.47853	0.11191
SUM	0.00000	4.75300	-1.62260	-6.23637	0.40324	1.46930

Hudson Design Group LLC

Beta/Gamma Sector
Antenna Frame Calculations

Date: 02-10-2016
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Wind Analysis → FRP Enclosure (BETA/GAMMA Sector)

Wind Blowing East - West

Reference Codes:

-Connecticut State Building Code 2016

-International Building Code 2012 (IBC 2012)

-Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)

Structure Classification	II		(ASCE 7-10 Table 1.5-1)
Nominal Wind Speed, V_{asd}	93	mph	(CTSBC Appendix N)
Importance Factor, I	1		(ASCE 7-10 Table 1.5-2)
Exposure Category	D		(ASCE 7-10 Section 26.7)
Height Above Ground Level, z	82.75	ft	(Top of Enclosure)
Exposure Coeficient, K_z	1.39		(ASCE 7-10 Table 29-3.1)
Wind Directionality Coef., K_d	0.90		(ASCE 7-10 Table 26.6-1)
Topographic Factor, K_{zt}	1.00		(ASCE 7-10 Section 26.8.2)

Velocity Pressure, q_z $= 0.00256K_zK_{zt}K_dV^2$ (ASCE 7-10 Equation 29.3-1)

$=$ **27.60 psf**

Gust Factor, G	0.85		(ASCE 7-10 Section 26.9)
Net Force Coeficient, C_f	1.30		(ASCE 7-10 Figure 29.5-1)
Projected Area Normal to Wind, A_f	205	ft^2	(20.5 ft. W x 10 ft. H)

Wind Force, F $= q_zGC_fA_f$ (ASCE 7-10 Equation 29.5-2)

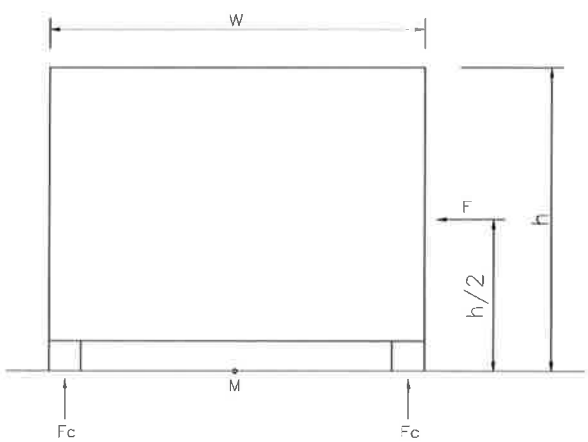
$=$ **6251.93 lbs**

Date: 02-10-2016
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Calculate Overturning Moment of Proposed FRP Enclosure (BETA/GAMMA)

Wind Blowing East - West

Dimensions (ft)	Wide, w	Depth, d	Height, h
	20.167	16	10



Moment, M = F x h/2
= **31259.66** lb-ft

Calculate Force Couple

Force Couple, F_c = M / d
= **1953.73** lbs.

Length of Frame: 20.1667 ft

Loading = 97 plf

Date: 02-10-2016
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Wind Analysis → FRP Enclosure (BETA/GAMMA Sector)

Wind Blowing North - South

Reference Codes:

-Connecticut State Building Code 2016

-International Building Code 2012 (IBC 2012)

-Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)

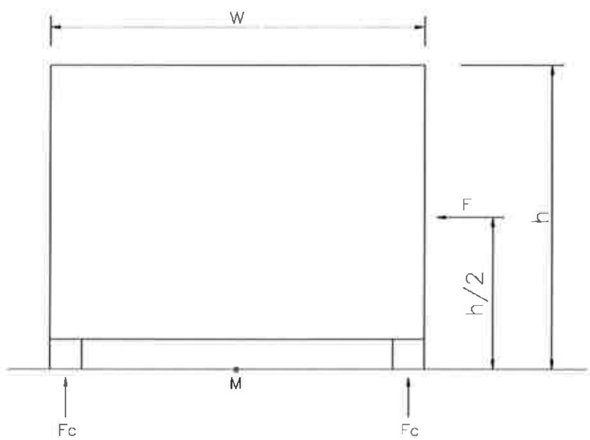
Structure Classification	II		(ASCE 7-10 Table 1.5-1)
Nominal Wind Speed, V_{asd}	93	mph	(CTSBC Appendix N)
Importance Factor, I	1		(ASCE 7-10 Table 1.5-2)
Exposure Category	D		(ASCE 7-10 Section 26.7)
Height Above Ground Level, z	82.75	ft	(Top of Enclosure)
Exposure Coeficient, K_z	1.39		(ASCE 7-10 Table 29-3.1)
Wind Directionality Coef., K_d	0.90		(ASCE 7-10 Table 26.6-1)
Topographic Factor, K_{zt}	1.00		(ASCE 7-10 Section 26.8.2)
Velocity Pressure, q_z	$= 0.00256K_zK_{zt}K_dV^2$		(ASCE 7-10 Equation 29.3-1)
	= **27.60 psf**		
Gust Factor, G	0.85		(ASCE 7-10 Section 26.9)
Net Force Coeficient, C_f	1.30		(ASCE 7-10 Figure 29.5-1)
Projected Area Normal to Wind, A_f	163	ft^2	(16.33 ft. W x 10 ft. H)
Wind Force, F	$= q_zGC_fA_f$		(ASCE 7-10 Equation 29.5-2)
	= **4980.20 lbs**		

Date: 02-10-2016
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Calculate Overturning Moment of Proposed FRP Enclosure (BETA/GAMMA)

Wind Blowing North - South

Dimensions (ft)	Wide, w	Depth, d	Height, h
	16	20.1667	10



Moment, M = F x h/2
= **24900.99** **lb-ft**

Calculate Force Couple

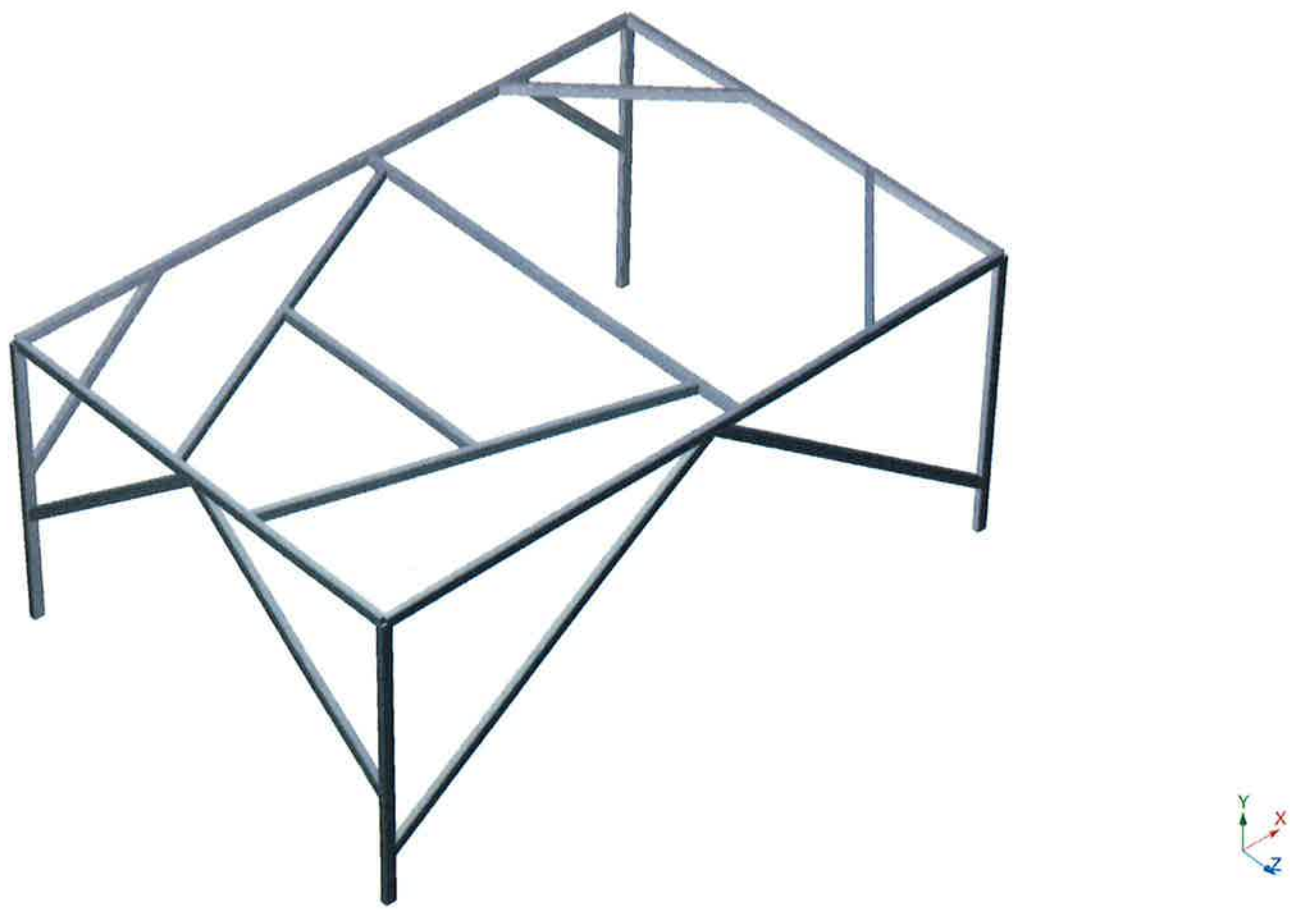
Force Couple, F_c = M / d
= **1234.76** **lbs.**

Length of Frame: 16 ft

Loading = **77 plf**

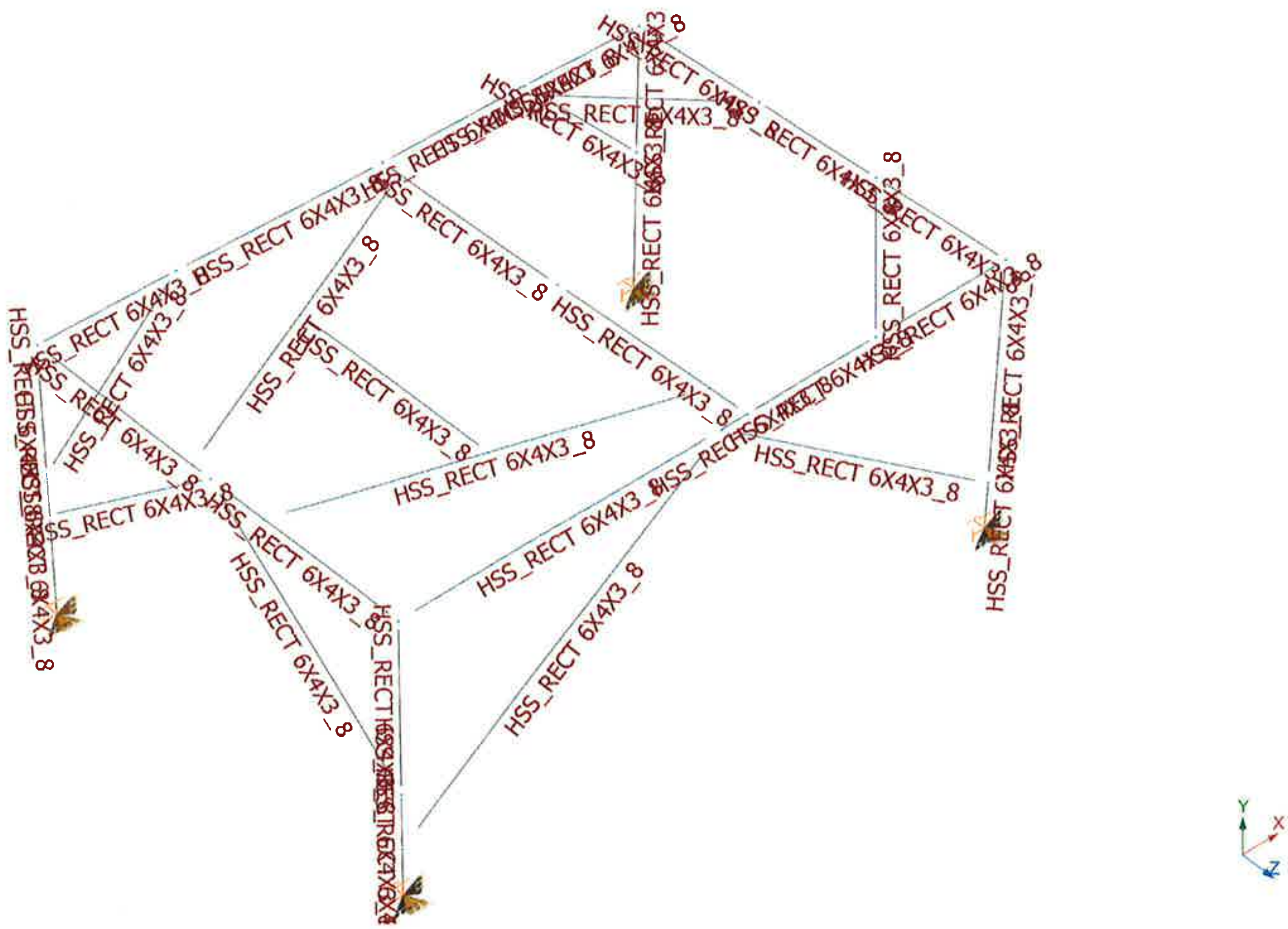
Bentley

Current Date: 2/8/2017 4:40 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind East) (R



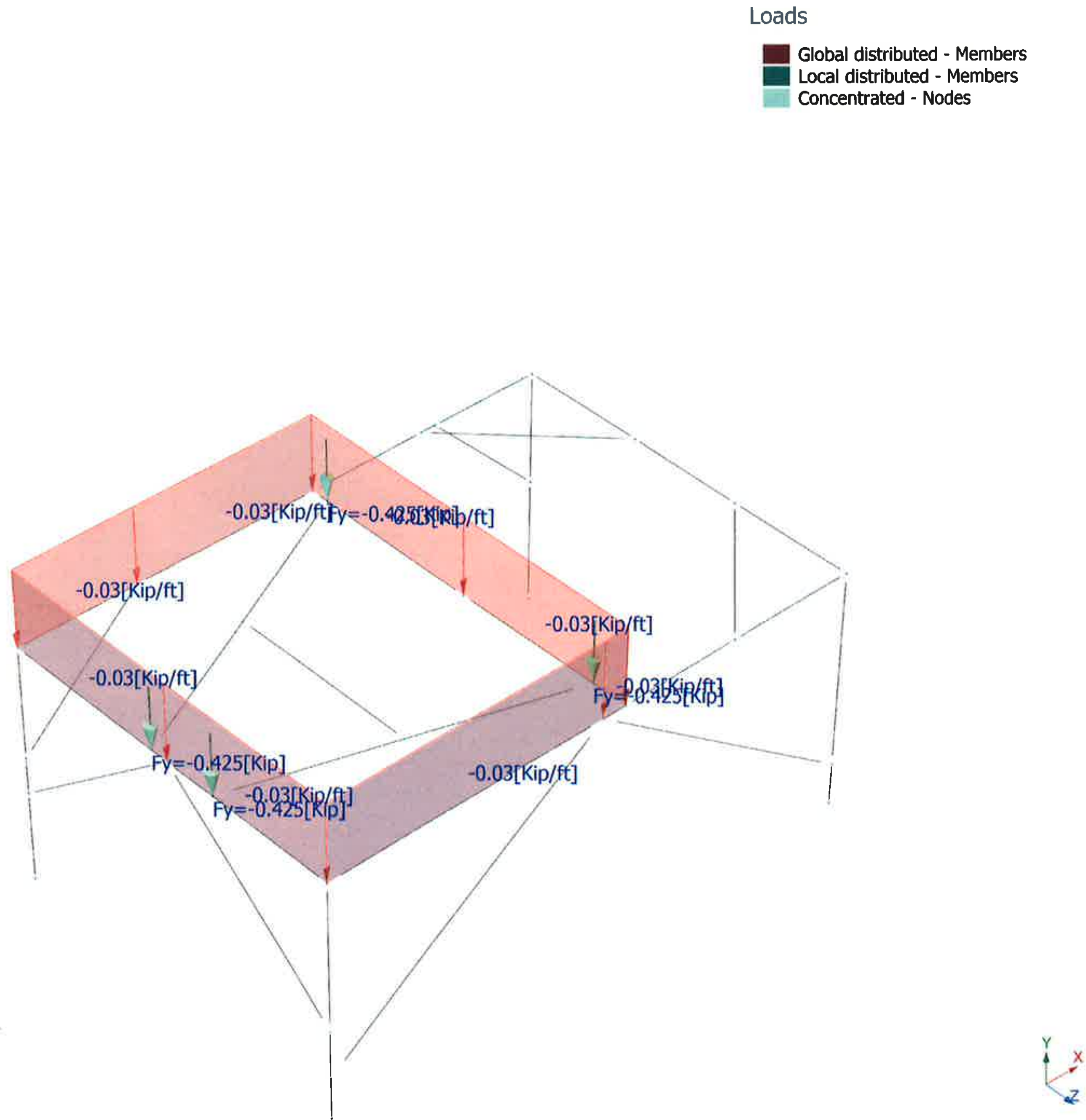
BETA/GAMMA FRAME
(EASTERLY WIND)

Current Date: 2/8/2017 4:40 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind East) (R



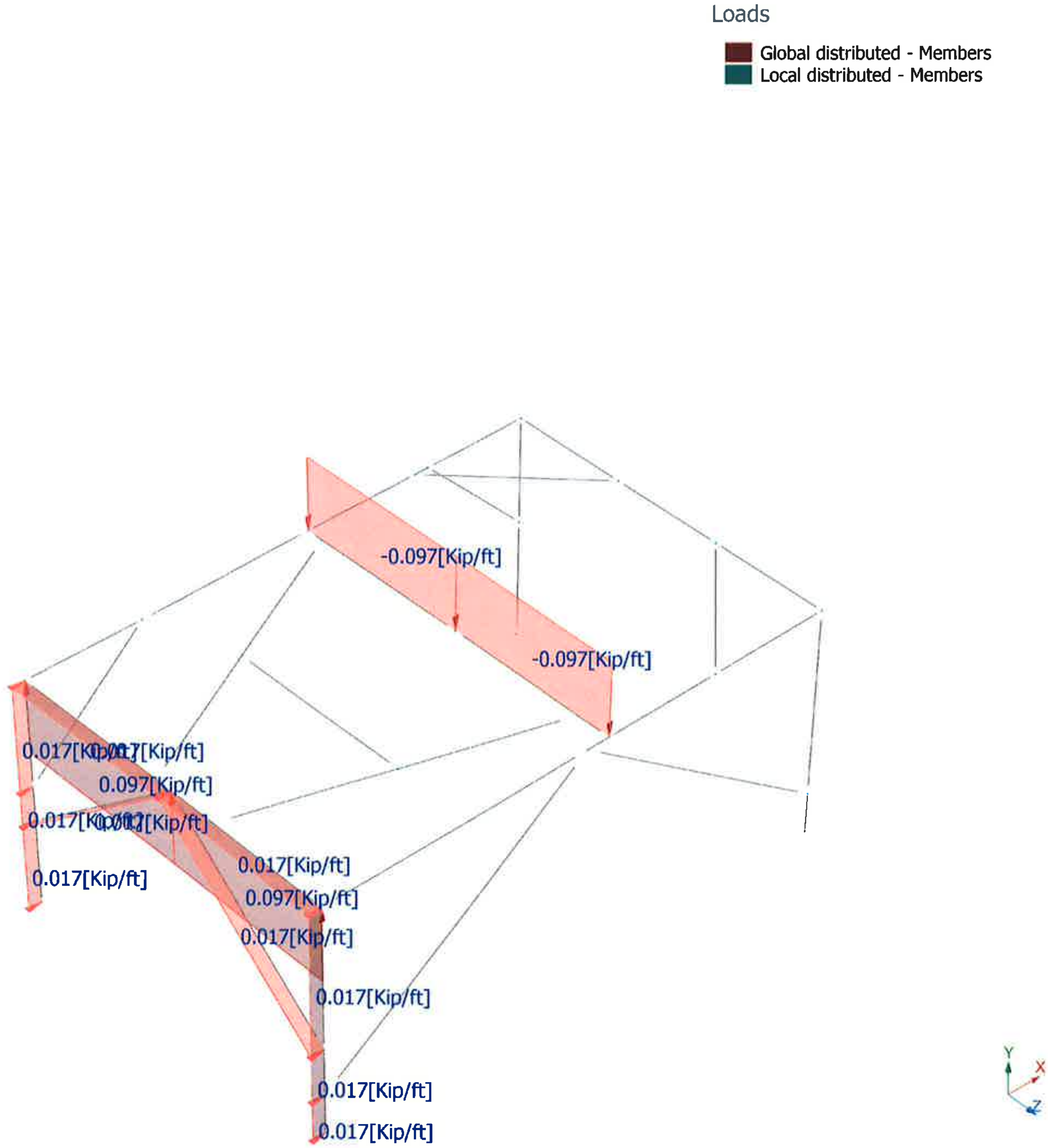
BETA/GAMMA FRAME
(EASTERLY WIND)

Current Date: 2/8/2017 4:40 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind East) (R
Load condition: DL=Dead Load



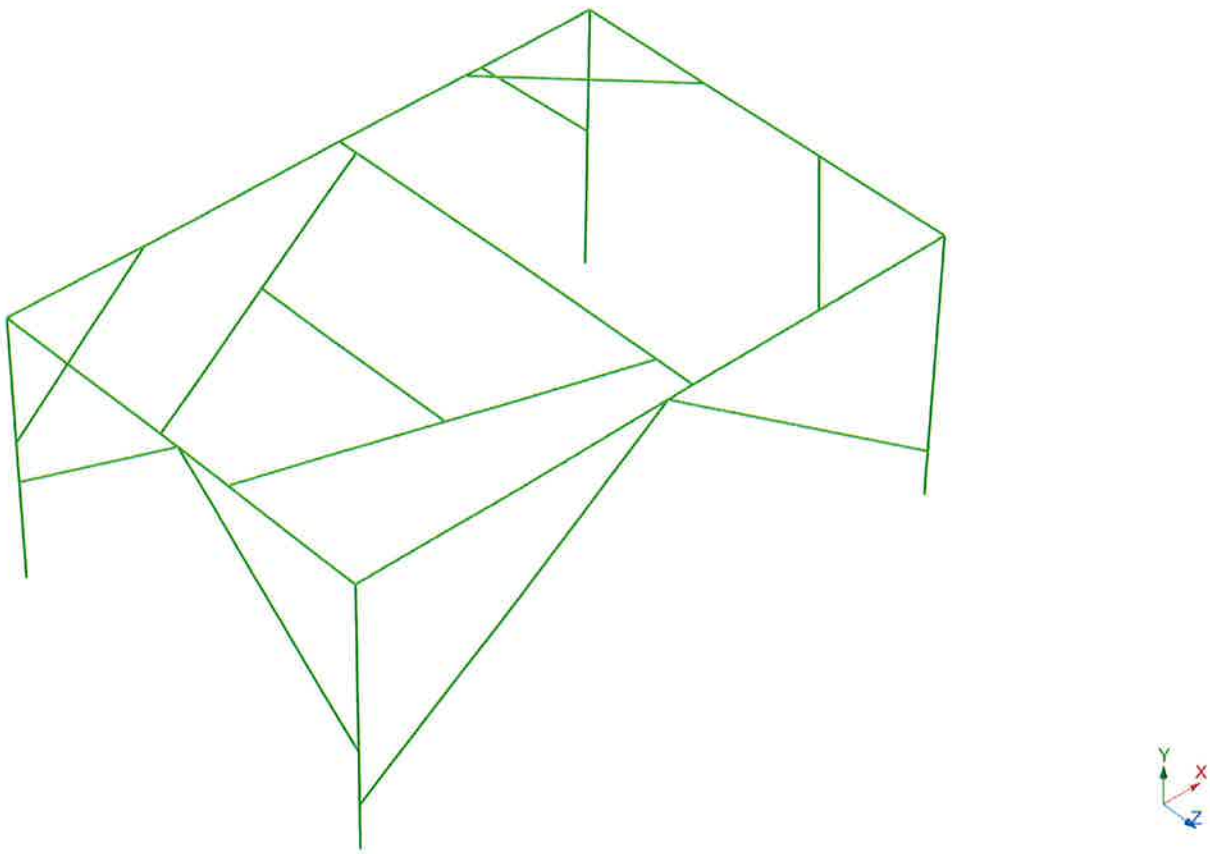
BETA/GAMMA FRAME
(EASTERLY WIND)

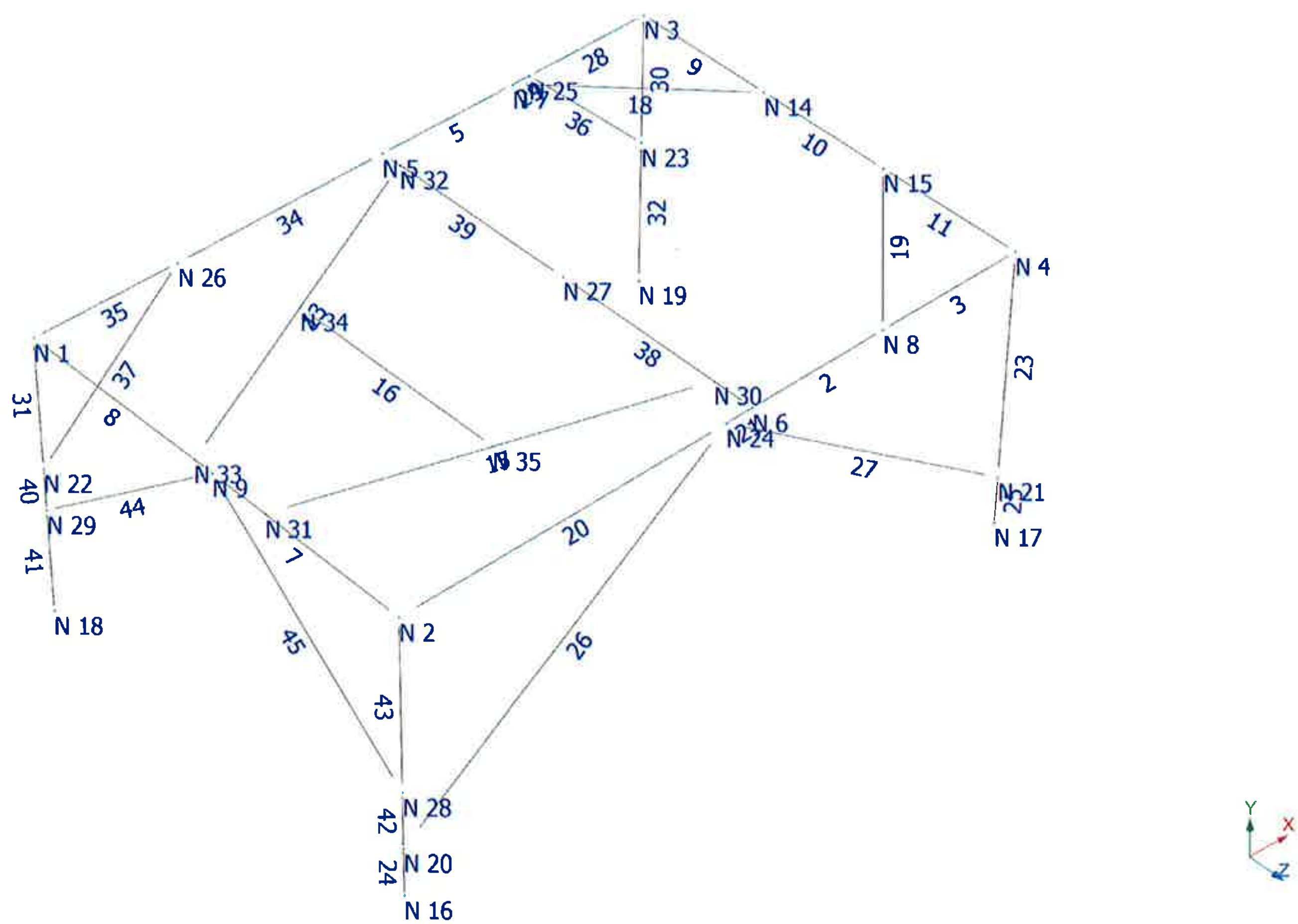
Current Date: 2/10/2017 9:43 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind East) (R
Load condition: W=Wind



Design status

- Not designed
- Error on design
- Design O.K.
- With warnings





BETA/GAMMA FRAME
(EASTERLY WIND)

Current Date: 2/10/2017 9:43 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind E: (Rev3).etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

D1=DL
D2=DL+W
D3=0.6DL+W

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_RECT 6X4X3_8	2	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 0.00%	**0.06**	**OK**	
			D3 at 0.00%	0.05	OK	
		3	D1 at 100.00%	0.02	OK	Eq. H1-1b
			D2 at 100.00%	**0.04**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.03	OK	Eq. H1-1b
		5	D1 at 0.00%	0.14	OK	Eq. H1-1b
			D2 at 0.00%	**0.29**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.24	OK	Eq. H1-1b
		7	D1 at 71.88%	0.04	OK	Eq. H1-1b
			D2 at 71.88%	**0.06**	**OK**	Eq. H1-1b
			D3 at 71.88%	0.04	OK	Eq. H1-1b
		8	D1 at 90.63%	0.04	OK	
			D2 at 90.63%	**0.09**	**OK**	
			D3 at 90.63%	0.08	OK	
		9	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 100.00%	**0.05**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.04	OK	Eq. H1-1b
		10	D1 at 93.75%	0.04	OK	Eq. H1-1b
			D2 at 75.00%	**0.09**	**OK**	Eq. H1-1b
			D3 at 68.75%	0.07	OK	Eq. H1-1b
		11	D1 at 100.00%	0.04	OK	Eq. H1-1b
			D2 at 100.00%	**0.08**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.06	OK	Eq. H1-1b
		13	D1 at 37.50%	0.05	OK	Eq. H1-1b
			D2 at 34.38%	**0.10**	**OK**	Eq. H1-1b
			D3 at 31.25%	0.09	OK	Eq. H1-1b
		15	D1 at 46.88%	0.04	OK	Eq. H1-1b
			D2 at 46.88%	**0.09**	**OK**	Eq. H1-1b
			D3 at 46.88%	0.07	OK	Eq. H1-1b
		16	D1 at 56.25%	0.02	OK	Eq. H1-1b
			D2 at 43.75%	**0.05**	**OK**	Eq. H1-1b
			D3 at 43.75%	0.04	OK	Eq. H1-1b
		18	D1 at 0.00%	0.03	OK	Eq. H1-1b

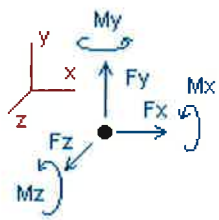
	D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
19	D1 at 37.50%	0.02	OK	Eq. H1-1b
	D2 at 25.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 25.00%	0.04	OK	Eq. H1-1b
20	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.09**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.07	OK	Eq. H1-1b
21	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 0.00%	**0.16**	**OK**	
	D3 at 0.00%	0.13	OK	
23	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.10	OK	Eq. H1-1b
24	D1 at 0.00%	0.21	OK	Eq. H1-1b
	D2 at 0.00%	**0.27**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.18	OK	Eq. H1-1b
25	D1 at 0.00%	0.10	OK	Eq. H1-1b
	D2 at 0.00%	**0.23**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.19	OK	Eq. H1-1b
26	D1 at 0.00%	0.05	OK	Eq. H1-2
	D2 at 100.00%	**0.13**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.11	OK	Eq. H1-1b
27	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.14**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.11	OK	Eq. H1-1b
28	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 100.00%	**0.15**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.12	OK	Eq. H1-1b
29	D1 at 100.00%	0.08	OK	Eq. H1-1b
	D2 at 100.00%	**0.17**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.14	OK	Eq. H1-1b
30	D1 at 100.00%	0.09	OK	Eq. H1-1b
	D2 at 100.00%	**0.19**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.15	OK	Eq. H1-1b
31	D1 at 100.00%	0.09	OK	Eq. H1-1b
	D2 at 100.00%	**0.15**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.11	OK	Eq. H1-1b
32	D1 at 100.00%	0.15	OK	Eq. H1-1b
	D2 at 100.00%	**0.31**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.25	OK	Eq. H1-1b
34	D1 at 0.00%	0.12	OK	Eq. H1-1b
	D2 at 0.00%	**0.23**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.19	OK	Eq. H1-1b
35	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 100.00%	**0.13**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.10	OK	Eq. H1-1b

36	D1 at 0.00%	0.06	OK	Eq. H1-1b
	D2 at 0.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.09	OK	Eq. H1-1b
37	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.08**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.06	OK	Eq. H1-1b
38	D1 at 100.00%	0.08	OK	Eq. H1-1b
	D2 at 100.00%	**0.21**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.18	OK	Eq. H1-1b
39	D1 at 96.88%	0.08	OK	Eq. H1-1b
	D2 at 96.88%	**0.21**	**OK**	Eq. H1-1b
	D3 at 96.88%	0.18	OK	Eq. H1-1b
40	D1 at 0.00%	0.14	OK	Eq. H1-1b
	D2 at 0.00%	**0.23**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.17	OK	Eq. H1-1b
41	D1 at 0.00%	0.25	OK	Eq. H1-1b
	D2 at 0.00%	**0.33**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.23	OK	Eq. H1-1b
42	D1 at 100.00%	0.12	OK	Eq. H1-1b
	D2 at 100.00%	**0.13**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
43	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
44	D1 at 0.00%	0.08	OK	Eq. H1-1b
	D2 at 0.00%	**0.10**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.07	OK	Eq. H1-1b
45	D1 at 100.00%	0.06	OK	Eq. H1-1b
	D2 at 0.00%	**0.09**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.07	OK	Eq. H1-1b

Current Date: 2/10/2017 9:43 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind Ea
(Rev3).etz\

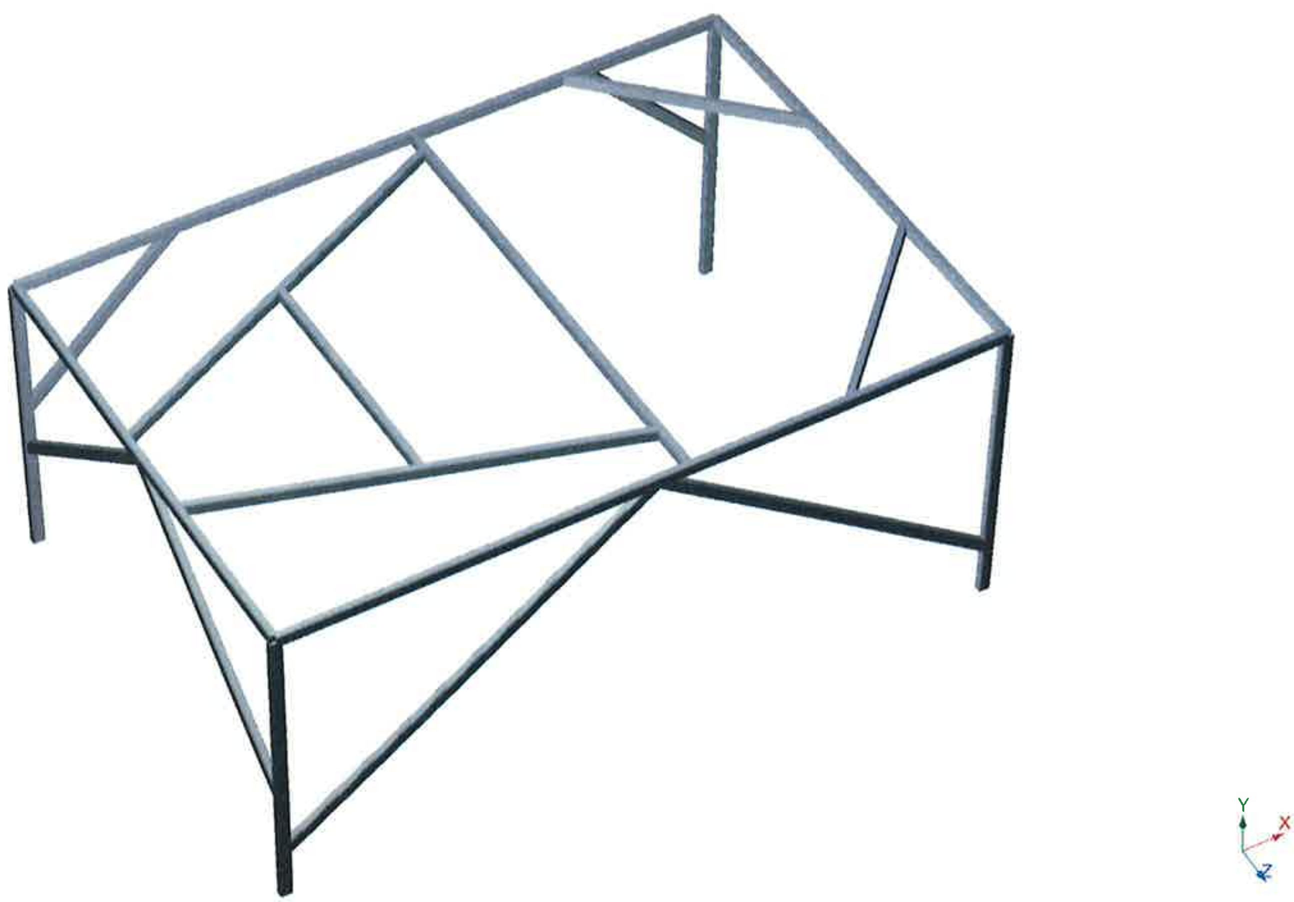
Analysis result

Reactions



Direction of positive forces and moments

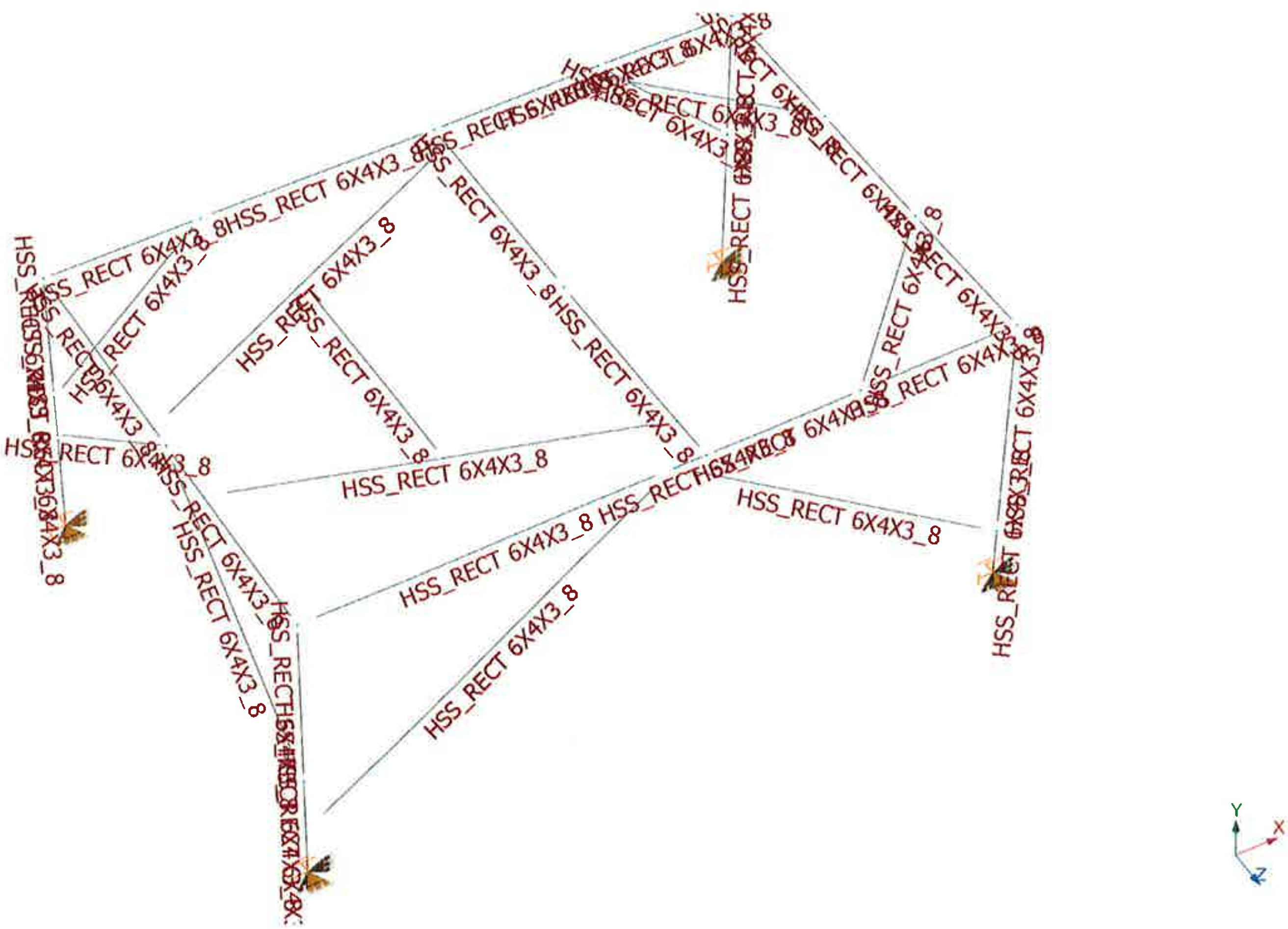
	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **D1=DL**						
16	1.55366	3.38751	-0.91907	-2.20076	0.01023	-2.46336
17	-1.54869	2.14805	-0.12952	-0.38983	-0.21539	2.33269
18	0.98850	3.08689	0.92333	2.39791	-0.05163	-3.30340
19	-0.99347	1.96410	0.12526	0.49449	0.02870	3.35452
SUM	0.00000	10.58654	0.00000	0.30181	-0.22809	-0.07953
Condition **D2=DL+W**						
16	2.60562	4.58034	-0.96670	-2.20453	0.25884	-3.95176
17	-3.28920	4.50760	-0.30598	-0.99249	-0.50153	4.97114
18	1.52856	4.18669	1.02140	2.63586	-0.15073	-5.09414
19	-2.07850	3.99361	0.25129	0.93431	0.10017	7.17909
SUM	-1.23352	17.26824	0.00000	0.37316	-0.29325	3.10433
Condition **D3=0.6DL+W**						
16	1.98415	3.22534	-0.59908	-1.32422	0.25475	-2.96642
17	-2.66973	3.64838	-0.25417	-0.83655	-0.41538	4.03806
18	1.13316	2.95193	0.65207	1.67670	-0.13008	-3.77278
19	-1.68111	3.20797	0.20118	0.73652	0.08869	5.83728
SUM	-1.23352	13.03362	0.00000	0.25244	-0.20202	3.13615



BETA/GAMMA FRAME
(WESTERLY WIND)

Bentley

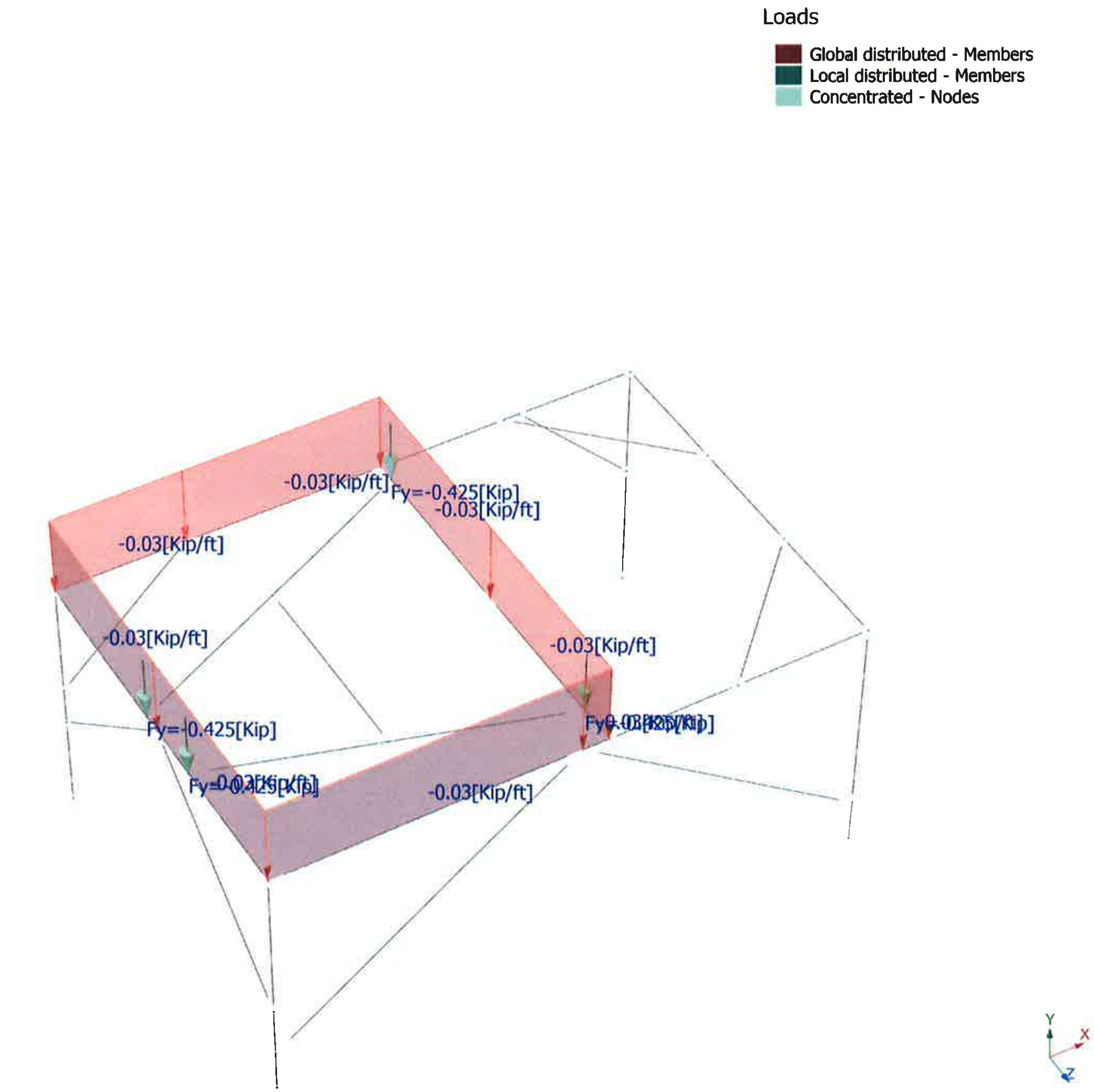
Current Date: 2/8/2017 4:43 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind West) (F



BETA/GAMMA FRAME
(WESTERLY WIND)

Bentley

Current Date: 2/8/2017 4:43 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind West) (F
Load condition: DL=Dead Load



BETA/GAMMA FRAME
(WESTERLY WIND)

Current Date: 2/10/2017 9:45 AM
Units system: English
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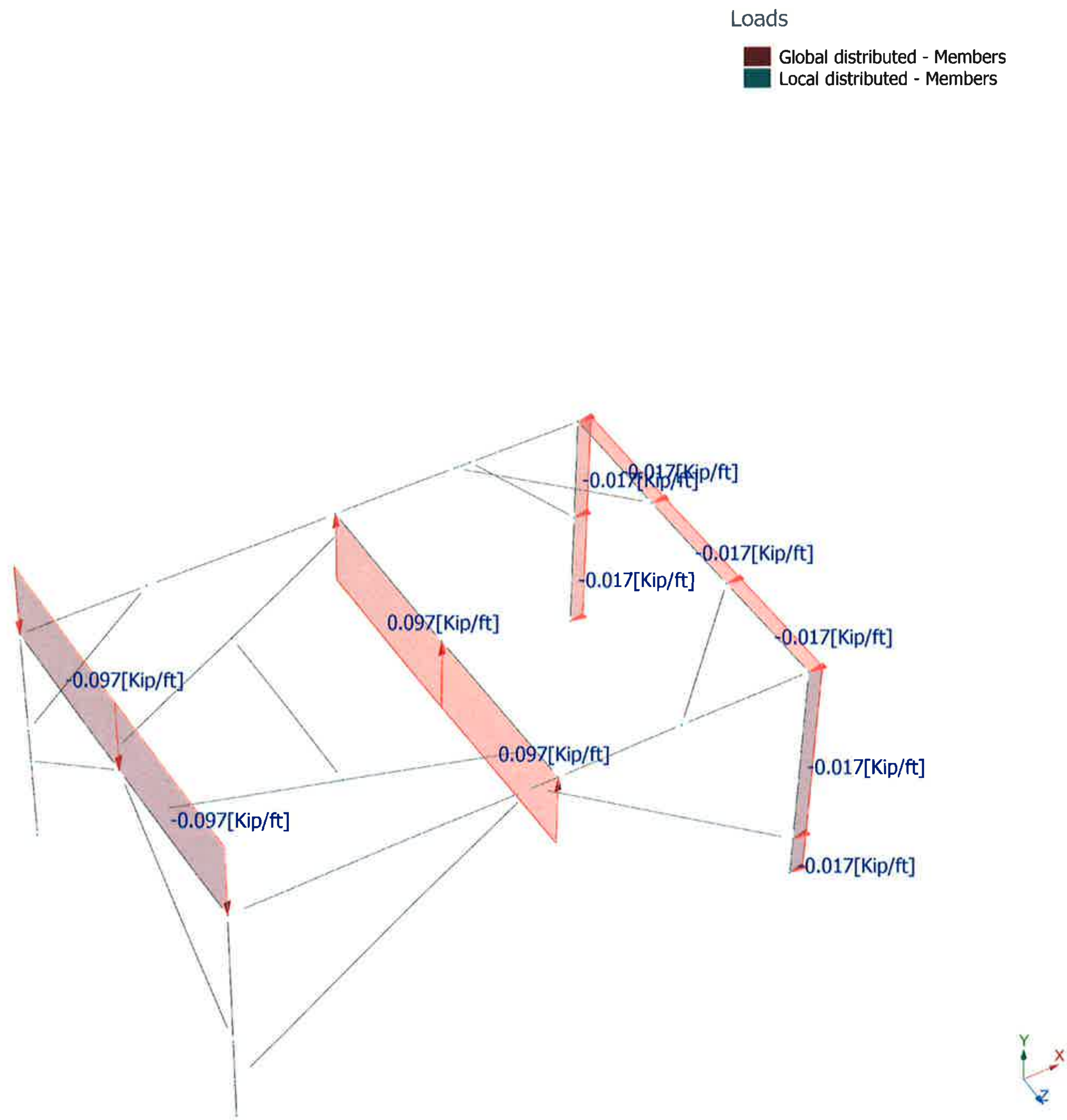
Load condition: W=Wind



Current Date: 2/10/2017 9:45 AM
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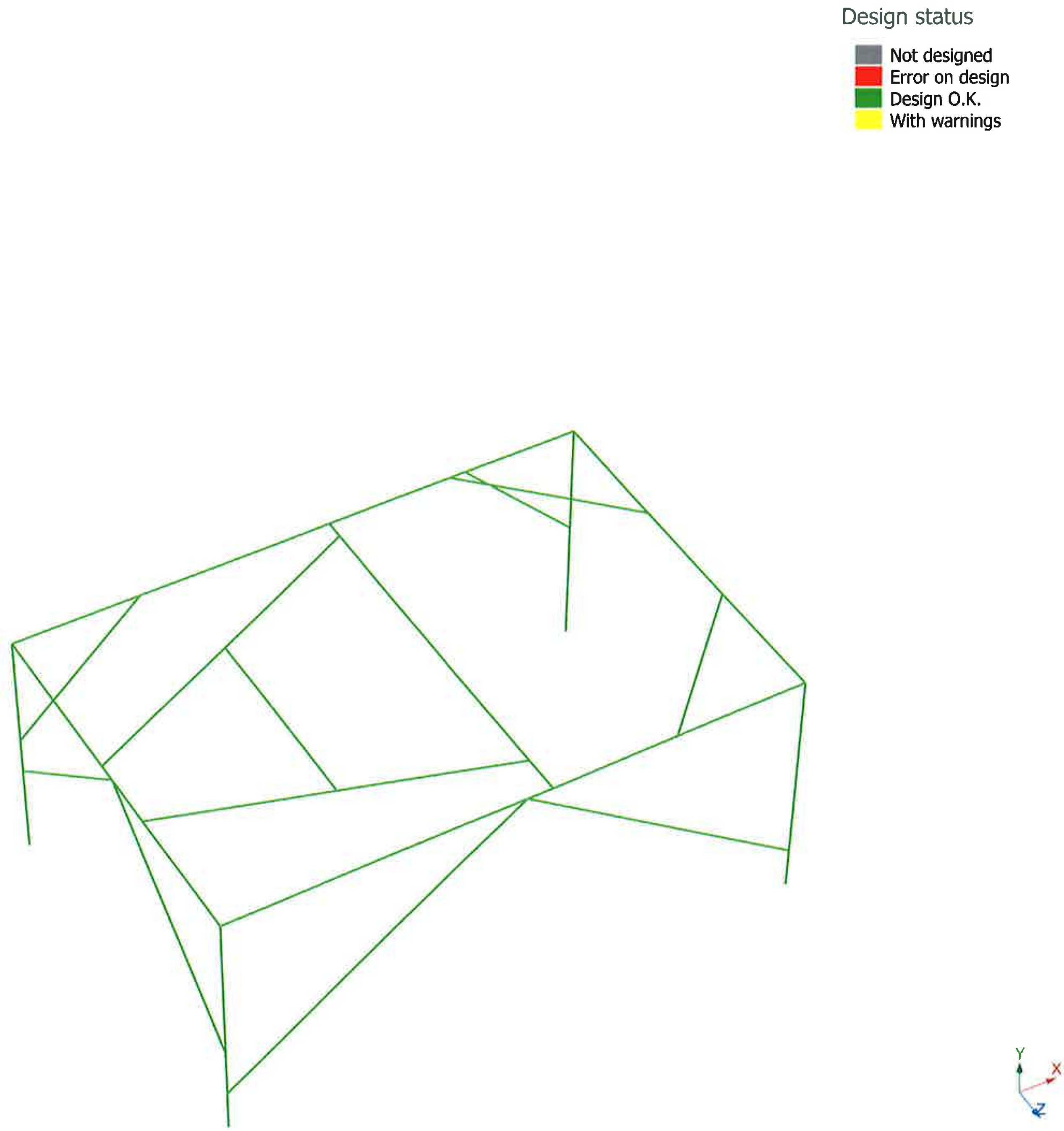

Current Date: 2/8/2017 4:43 PM
Units system: English
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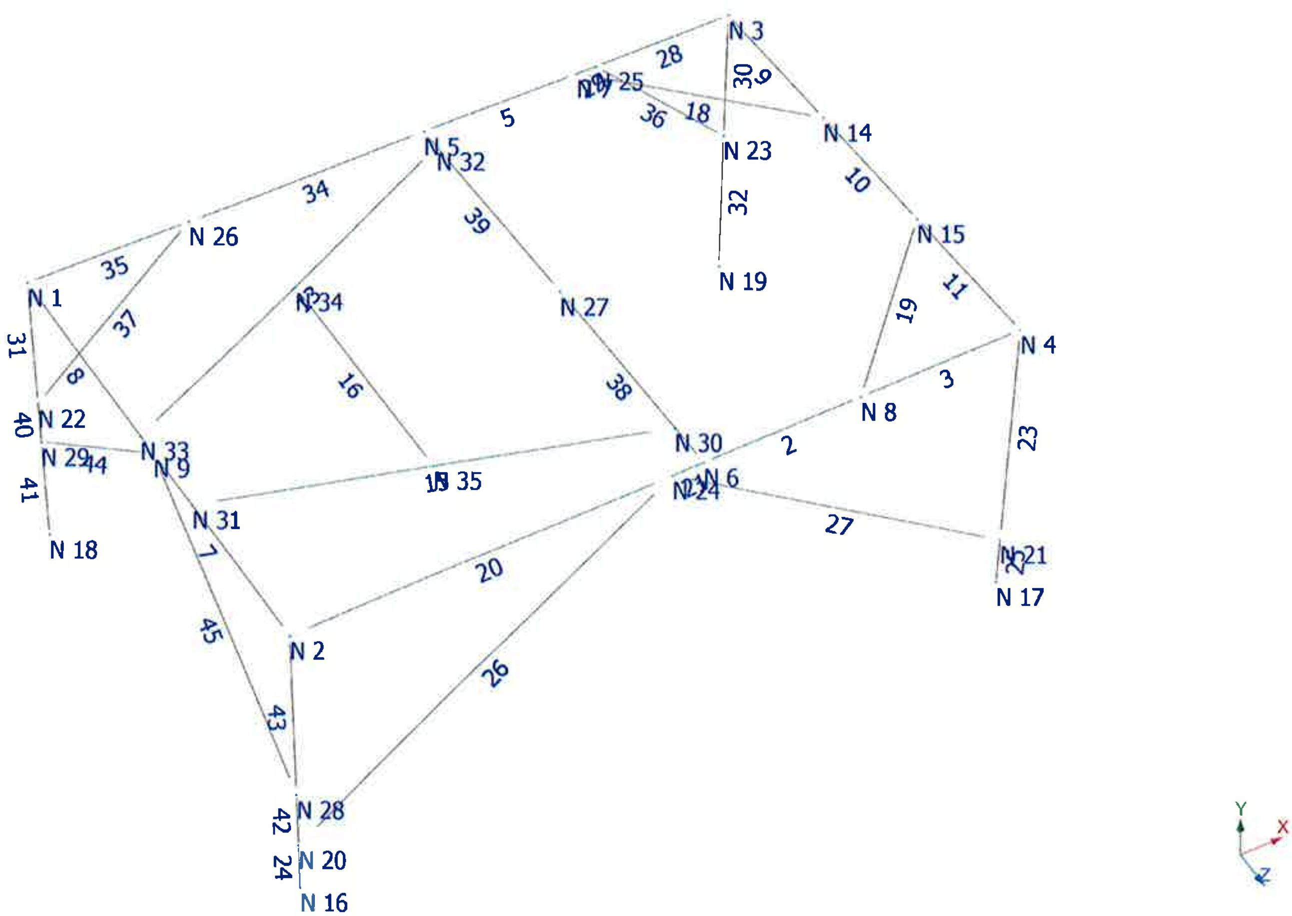


BETA/GAMMA FRAME
(WESTERLY WIND)

Bentley

Current Date: 2/10/2017 9:45 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind West) (Rev3).etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

D1=DL
D2=DL+W
D3=0.6DL+W

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_RECT 6X4X3_8	**2**	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.05	OK	Eq. H1-1b
		3	D1 at 100.00%	0.02	OK	Eq. H1-1b
			D2 at 100.00%	**0.04**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.04	OK	Eq. H1-1b
		5	D1 at 0.00%	**0.14**	**OK**	Eq. H1-1b
			D2 at 0.00%	0.09	OK	Eq. H1-1b
			D3 at 0.00%	0.04	OK	Eq. H1-1b
		7	D1 at 71.88%	0.04	OK	Eq. H1-1b
			D2 at 59.38%	**0.08**	**OK**	Eq. H1-1b
			D3 at 56.25%	0.06	OK	Eq. H1-1b
		8	D1 at 90.63%	0.04	OK	
			D2 at 100.00%	**0.07**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.05	OK	Eq. H1-1b
		9	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.04	OK	Eq. H1-1b
		10	D1 at 93.75%	0.04	OK	Eq. H1-1b
			D2 at 50.00%	**0.07**	**OK**	Eq. H1-1b
			D3 at 50.00%	0.06	OK	Eq. H1-1b
		11	D1 at 100.00%	0.04	OK	Eq. H1-1b
			D2 at 100.00%	**0.07**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.05	OK	Eq. H1-1b
		13	D1 at 37.50%	0.05	OK	Eq. H1-1b
			D2 at 50.00%	**0.07**	**OK**	Eq. H1-1b
			D3 at 53.13%	0.05	OK	Eq. H1-1b
		15	D1 at 46.88%	0.04	OK	Eq. H1-1b
			D2 at 50.00%	**0.07**	**OK**	Eq. H1-1b
			D3 at 50.00%	0.05	OK	Eq. H1-1b
		16	D1 at 56.25%	0.02	OK	Eq. H1-1b
			D2 at 56.25%	**0.02**	**OK**	Eq. H1-1b
			D3 at 56.25%	0.01	OK	Eq. H1-1b
		18	D1 at 0.00%	0.03	OK	Eq. H1-1b

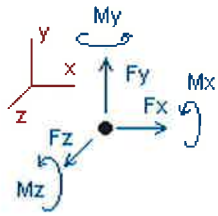
	D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
19	D1 at 37.50%	0.02	OK	Eq. H1-1b
	D2 at 50.00%	**0.02**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.02	OK	Eq. H1-1b
20	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.06**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.04	OK	Eq. H1-1b
21	D1 at 100.00%	**0.07**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.06	OK	Eq. H1-1b
	D3 at 100.00%	0.03	OK	Eq. H1-1b
23	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.09**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.07	OK	Eq. H1-1b
24	D1 at 0.00%	0.21	OK	Eq. H1-1b
	D2 at 0.00%	**0.33**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.25	OK	Eq. H1-1b
25	D1 at 0.00%	**0.10**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.10	OK	Eq. H1-1b
	D3 at 0.00%	0.06	OK	Eq. H1-1b
26	D1 at 0.00%	0.05	OK	Eq. H1-2
	D2 at 0.00%	**0.08**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.06	OK	Eq. H1-1b
27	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.06**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.04	OK	Eq. H1-1b
28	D1 at 100.00%	**0.07**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.05	OK	Eq. H1-1b
	D3 at 100.00%	0.02	OK	Eq. H1-1b
29	D1 at 100.00%	**0.08**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.06	OK	Eq. H1-1b
	D3 at 100.00%	0.03	OK	Eq. H1-1b
30	D1 at 100.00%	**0.09**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.09	OK	Eq. H1-1b
	D3 at 0.00%	0.06	OK	Eq. H1-1b
31	D1 at 100.00%	0.09	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
32	D1 at 100.00%	0.15	OK	Eq. H1-1b
	D2 at 100.00%	**0.16**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.10	OK	Eq. H1-1b
34	D1 at 0.00%	**0.12**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.12	OK	Eq. H1-1b
	D3 at 0.00%	0.07	OK	Eq. H1-1b
35	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 100.00%	**0.08**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.05	OK	Eq. H1-1b

36	D1 at 0.00%	0.06	OK	Eq. H1-1b
	D2 at 0.00%	**0.06**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
37	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.07**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.05	OK	Eq. H1-1b
38	D1 at 100.00%	**0.08**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.06	OK	
	D3 at 0.00%	0.05	OK	
39	D1 at 96.88%	**0.08**	**OK**	Eq. H1-1b
	D2 at 12.50%	0.05	OK	Eq. H1-1b
	D3 at 12.50%	0.05	OK	Eq. H1-1b
40	D1 at 0.00%	0.14	OK	Eq. H1-1b
	D2 at 0.00%	**0.19**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.13	OK	Eq. H1-1b
41	D1 at 0.00%	0.25	OK	Eq. H1-1b
	D2 at 0.00%	**0.38**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.28	OK	Eq. H1-1b
42	D1 at 100.00%	0.12	OK	Eq. H1-1b
	D2 at 100.00%	**0.22**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.17	OK	Eq. H1-1b
43	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	**0.07**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.05	OK	Eq. H1-1b
44	D1 at 0.00%	0.08	OK	Eq. H1-1b
	D2 at 0.00%	**0.13**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.10	OK	Eq. H1-1b
45	D1 at 100.00%	0.06	OK	Eq. H1-1b
	D2 at 100.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.10	OK	Eq. H1-1b

Current Date: 2/10/2017 9:45 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind West) (Rev3).etz\

Analysis result

Reactions

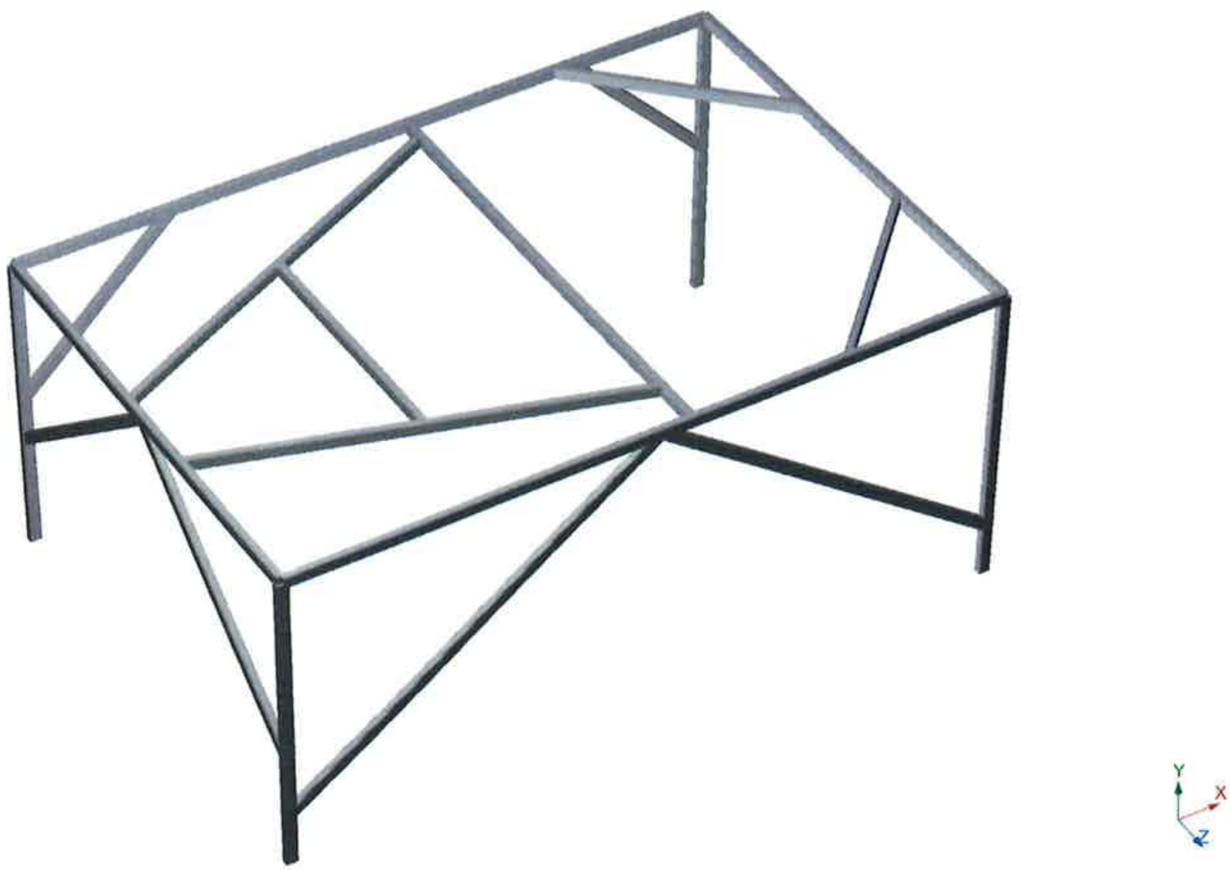


Direction of positive forces and moments

	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **D1=DL**						
16	1.55366	3.38751	-0.91907	-2.20076	0.01023	-2.46336
17	-1.54869	2.14805	-0.12952	-0.38983	-0.21539	2.33269
18	0.98850	3.08689	0.92333	2.39791	-0.05163	-3.30340
19	-0.99347	1.96410	0.12526	0.49449	0.02870	3.35452
SUM	0.00000	10.58654	0.00000	0.30181	-0.22809	-0.07953
Condition **D2=DL+W**						
16	1.98330	6.01963	-1.72635	-4.20419	-0.21329	-2.96889
17	-1.50848	3.07254	-0.15525	-0.50708	-0.12467	2.07700
18	1.17316	5.37473	1.68424	4.35778	-0.02928	-3.97530
19	-0.85635	2.80134	0.19737	0.81200	0.03744	2.92600
SUM	0.79163	17.26824	0.00000	0.45851	-0.32981	-1.94120
Condition **D3=0.6DL+W**						
16	1.36184	4.66462	-1.35873	-3.32388	-0.21738	-1.98355
17	-0.88900	2.21332	-0.10344	-0.35115	-0.03852	1.14392
18	0.77776	4.13998	1.31491	3.39862	-0.00863	-2.65394
19	-0.45896	2.01570	0.14726	0.61421	0.02595	1.58419
SUM	0.79163	13.03362	0.00000	0.33779	-0.23858	-1.90938

Bentley

Current Date: 2/8/2017 4:46 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind North) (F



BETA/GAMMA FRAME
(NORTHERLY WIND)

Current Date: 2/8/2017 4:46 PM
Units system: English
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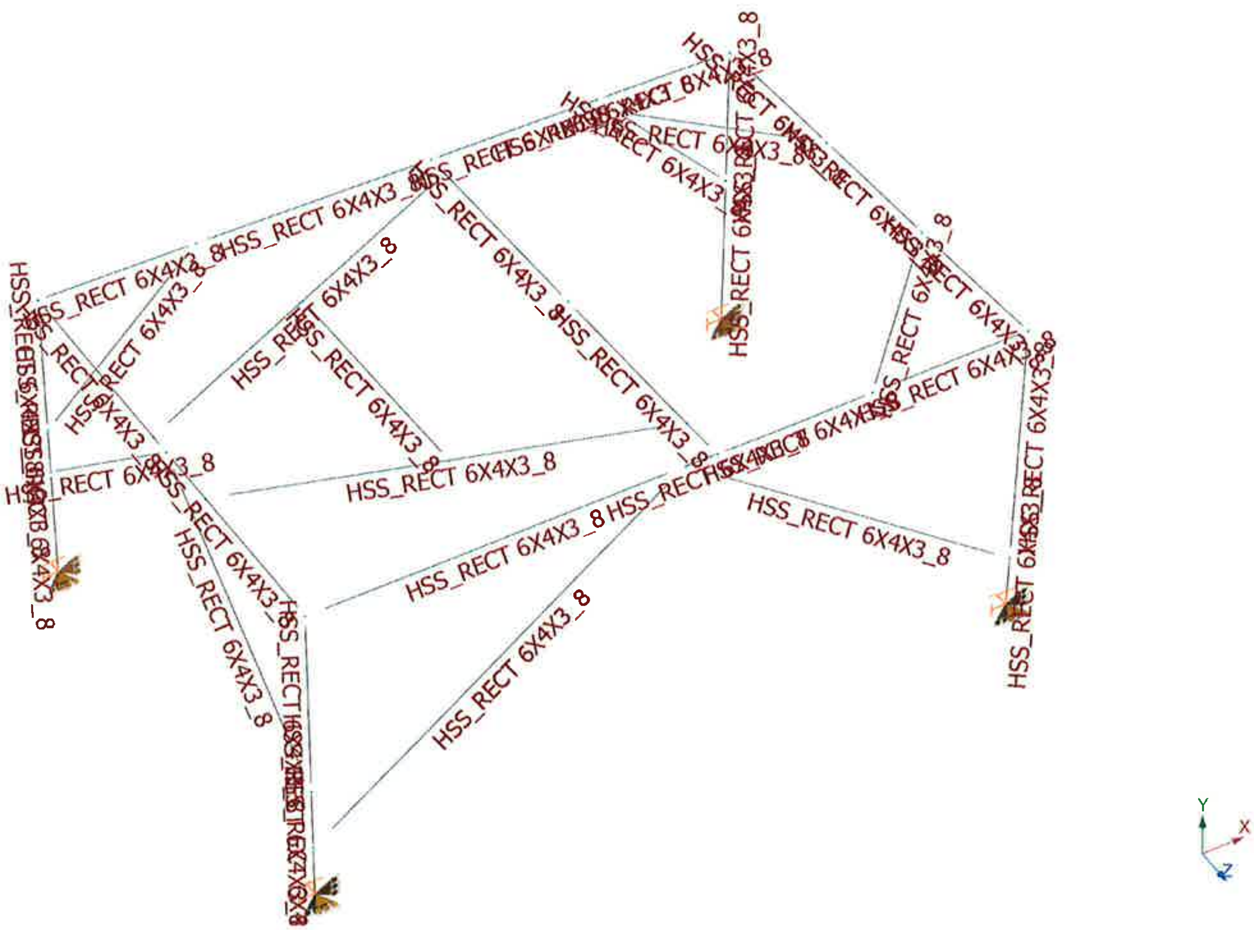
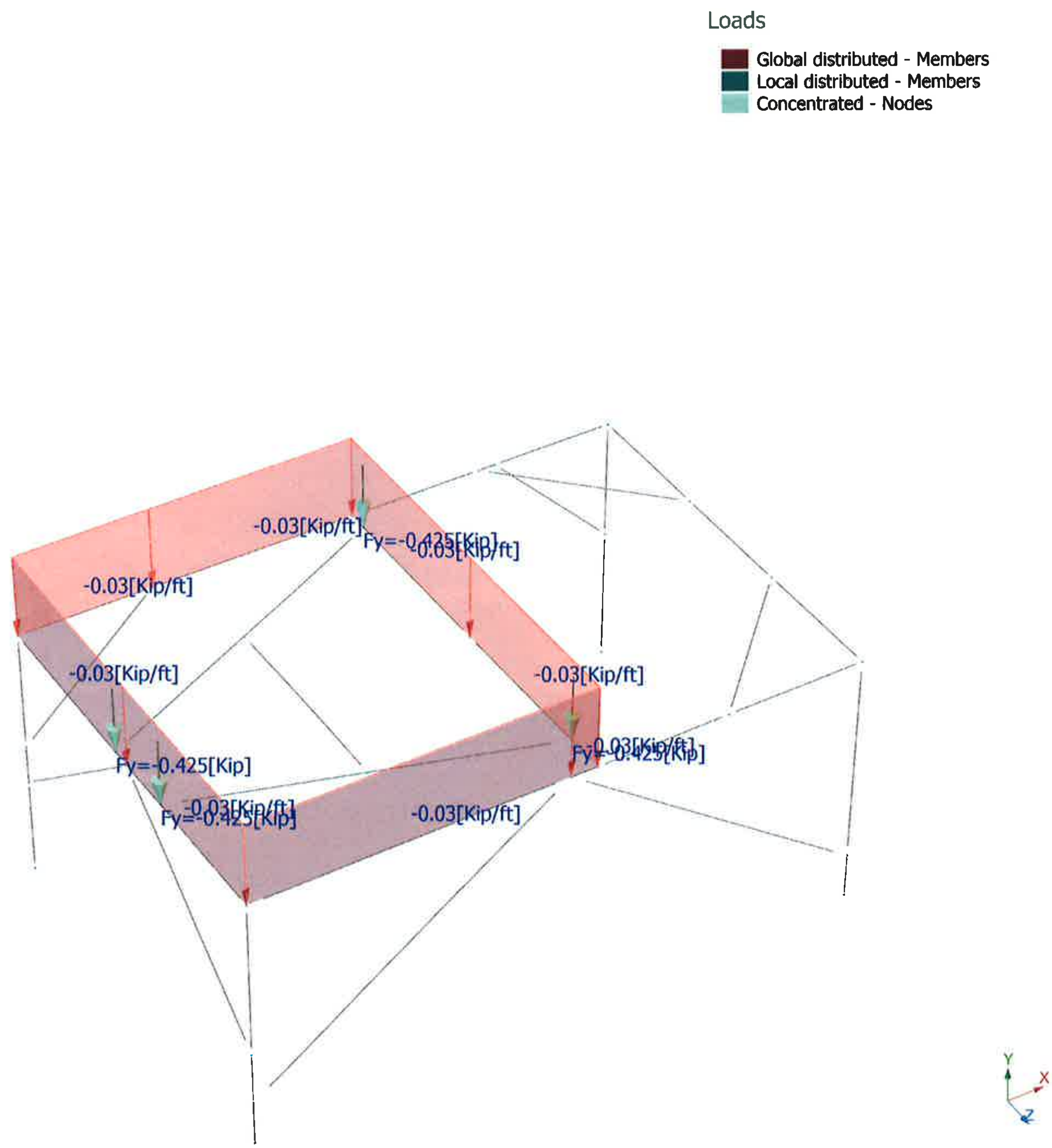


BETA/GAMMA FRAME
(NORTHERLY WIND)

Bentley

Current Date: 2/8/2017 4:46 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind North) (F
Load condition: DL=Dead Load



BETA/GAMMA FRAME
(NORTHERLY WIND)

Current Date: 2/10/2017 9:48 AM
Units system: English
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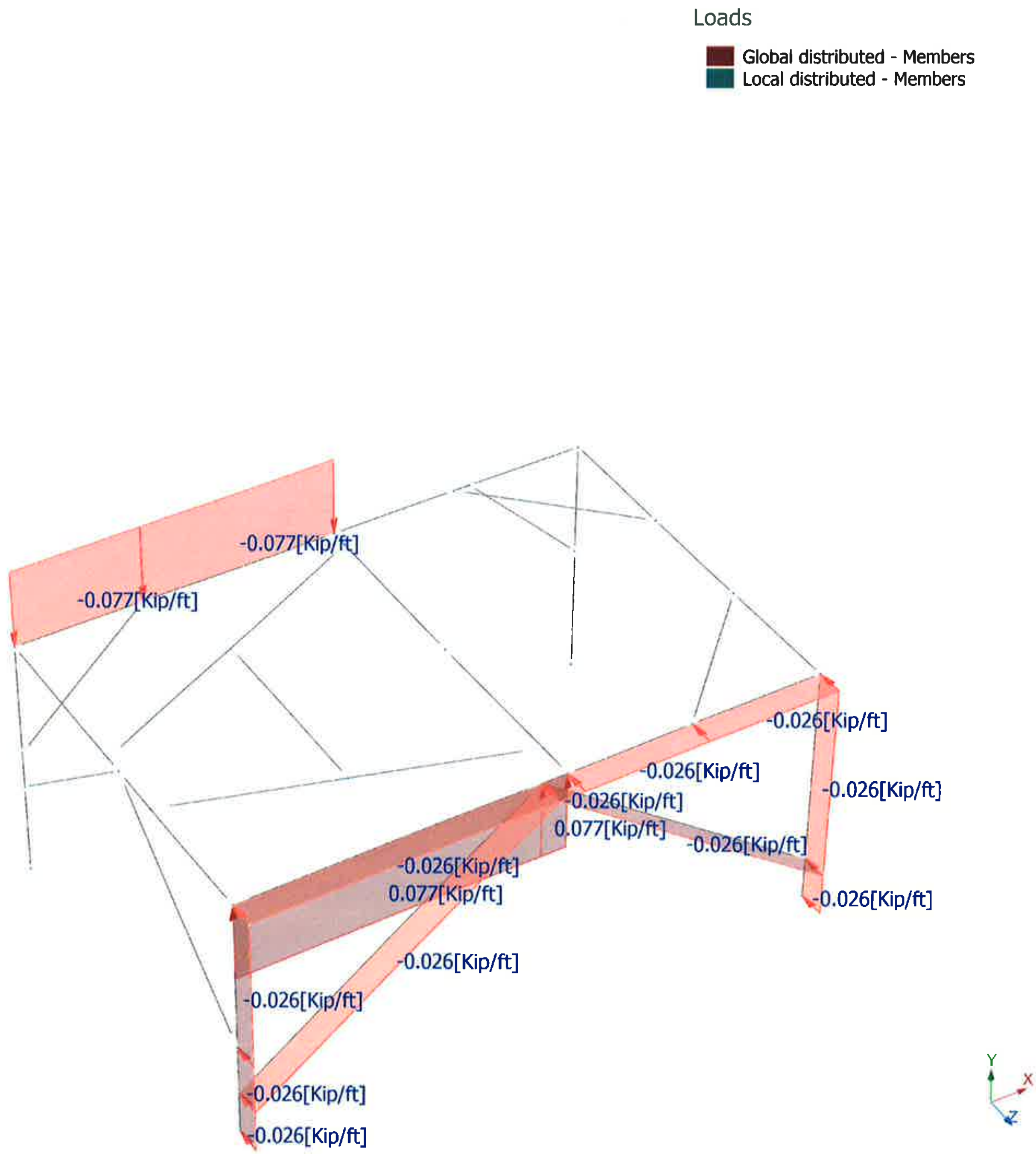
Load condition: W=Wind



Bentley
Current Date: 2/10/2017 9:48 AM
Units system: English
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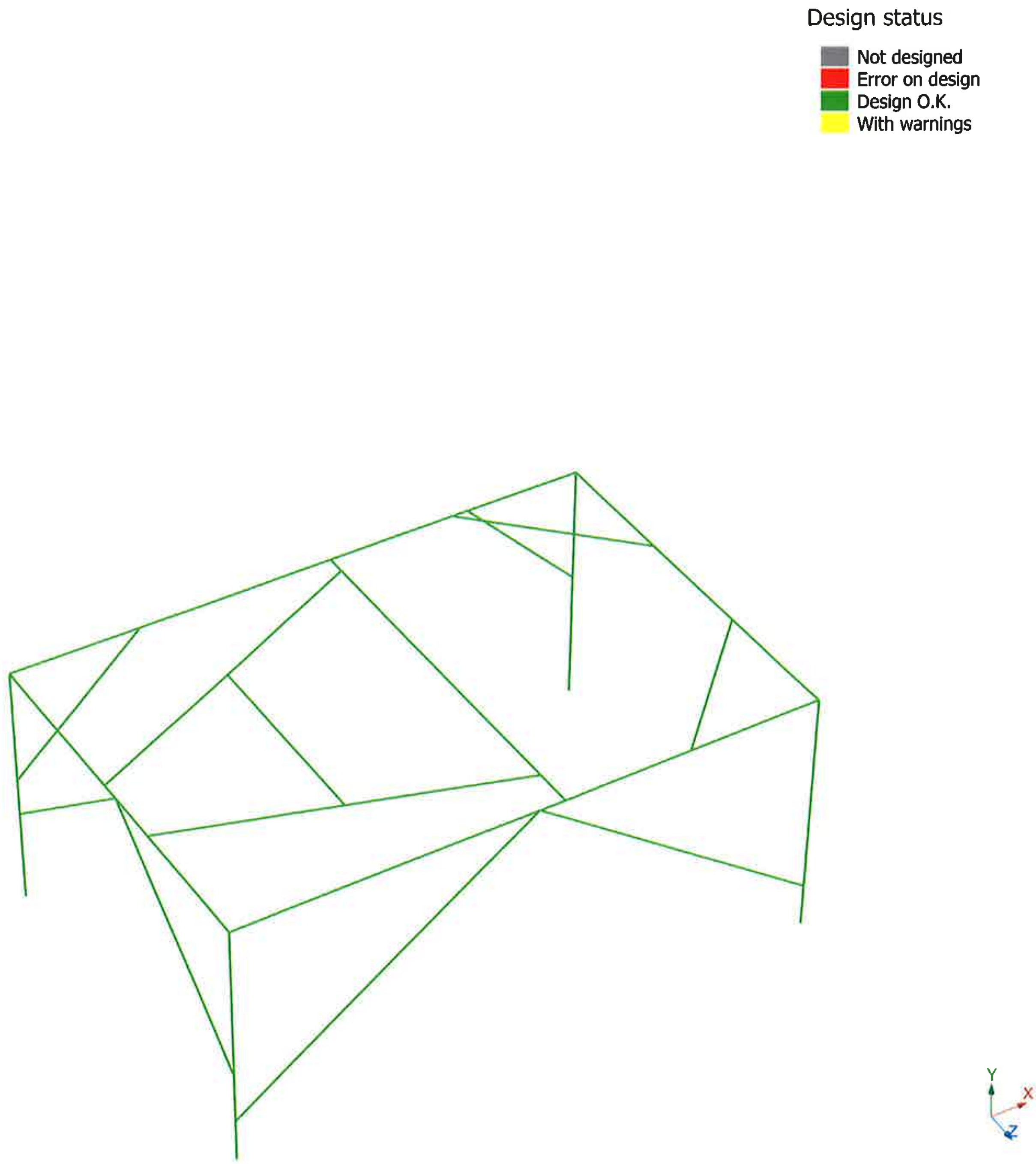
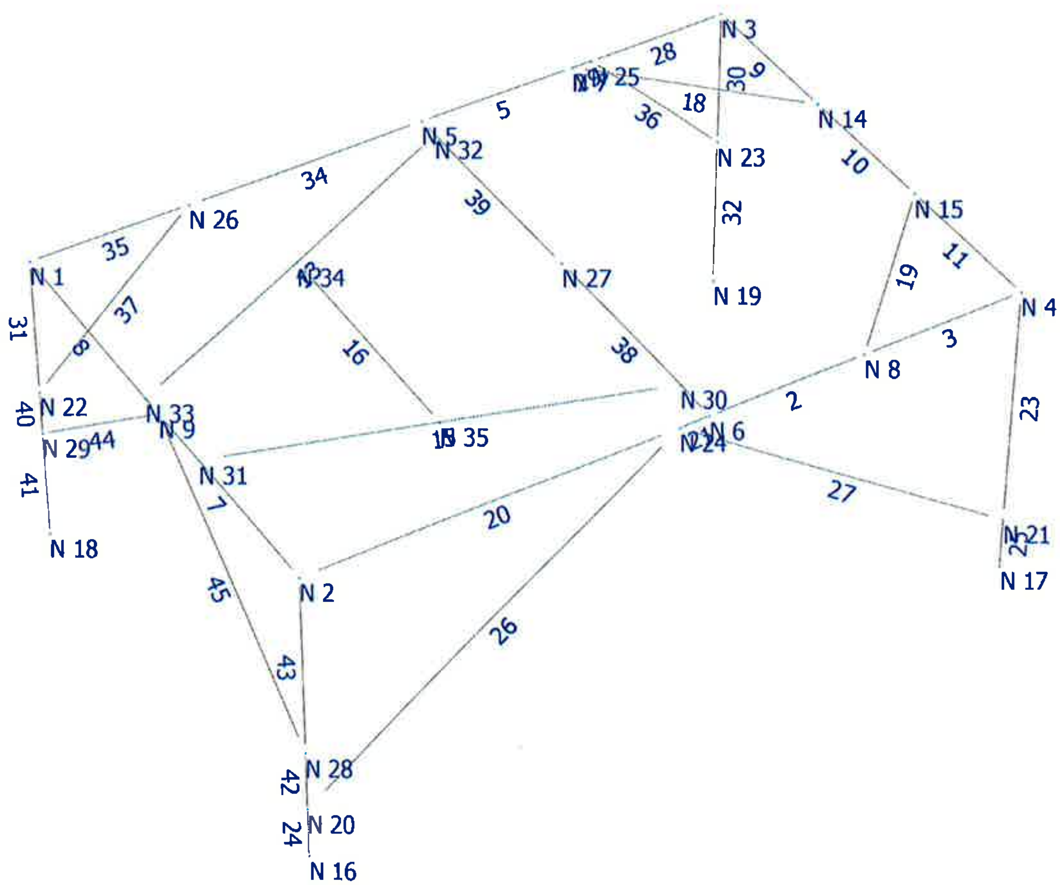
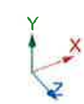






BETA/GAMMA FRAME
(NORTHERLY WIND)

Bentley

Current Date: 2/10/2017 9:48 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind North) (Rev3).etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

D1=DL
D2=DL+W
D3=0.6DL+W

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_RECT 6X4X3_8	2	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 100.00%	**0.05**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.04	OK	Eq. H1-1b
		3	D1 at 100.00%	0.02	OK	Eq. H1-1b
			D2 at 100.00%	**0.04**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.04	OK	Eq. H1-1b
		5	D1 at 0.00%	0.14	OK	Eq. H1-1b
			D2 at 0.00%	**0.23**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.17	OK	Eq. H1-1b
		7	D1 at 71.88%	0.04	OK	Eq. H1-1b
			D2 at 71.88%	**0.11**	**OK**	Eq. H1-1b
			D3 at 71.88%	0.09	OK	Eq. H1-1b
		8	D1 at 90.63%	0.04	OK	
			D2 at 90.63%	**0.07**	**OK**	
			D3 at 100.00%	0.06	OK	Eq. H1-1b
		9	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 0.00%	**0.08**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.07	OK	Eq. H1-1b
		10	D1 at 93.75%	0.04	OK	Eq. H1-1b
			D2 at 100.00%	**0.09**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.07	OK	Eq. H1-1b
		11	D1 at 100.00%	0.04	OK	Eq. H1-1b
			D2 at 100.00%	**0.07**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.06	OK	Eq. H1-1b
		13	D1 at 37.50%	0.05	OK	Eq. H1-1b
			D2 at 100.00%	**0.10**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.09	OK	Eq. H1-1b
		15	D1 at 46.88%	0.04	OK	Eq. H1-1b
			D2 at 50.00%	**0.09**	**OK**	Eq. H1-1b
			D3 at 50.00%	0.07	OK	Eq. H1-1b
		16	D1 at 56.25%	0.02	OK	Eq. H1-1b
			D2 at 81.25%	**0.04**	**OK**	Eq. H1-1b
			D3 at 93.75%	0.04	OK	Eq. H1-1b
		18	D1 at 0.00%	0.03	OK	Eq. H1-1b

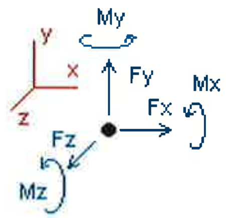
	D2 at 0.00%	**0.06**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.05	OK	Eq. H1-1b
19	D1 at 37.50%	0.02	OK	Eq. H1-1b
	D2 at 56.25%	**0.04**	**OK**	Eq. H1-1b
	D3 at 56.25%	0.03	OK	Eq. H1-1b
20	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	0.05	OK	Eq. H1-1b
	D3 at 0.00%	**0.06**	**OK**	Eq. H1-1b
21	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 0.00%	**0.11**	**OK**	
	D3 at 0.00%	0.09	OK	
23	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.06**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
24	D1 at 0.00%	**0.21**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.11	OK	Eq. H1-1b
	D3 at 0.00%	0.07	OK	Eq. H1-1b
25	D1 at 0.00%	0.10	OK	Eq. H1-1b
	D2 at 0.00%	**0.20**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.17	OK	Eq. H1-1b
26	D1 at 0.00%	0.05	OK	Eq. H1-2
	D2 at 100.00%	**0.10**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
27	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.10	OK	Eq. H1-1b
28	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
29	D1 at 100.00%	0.08	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
30	D1 at 100.00%	0.09	OK	Eq. H1-1b
	D2 at 0.00%	**0.16**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.13	OK	Eq. H1-1b
31	D1 at 100.00%	0.09	OK	Eq. H1-1b
	D2 at 100.00%	**0.20**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.16	OK	Eq. H1-1b
32	D1 at 100.00%	0.15	OK	Eq. H1-1b
	D2 at 0.00%	**0.30**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.24	OK	Eq. H1-1b
34	D1 at 0.00%	0.12	OK	Eq. H1-1b
	D2 at 0.00%	**0.21**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.16	OK	Eq. H1-1b
35	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 100.00%	**0.15**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.13	OK	Eq. H1-1b

36	D1 at 0.00%	0.06	OK	Eq. H1-1b
	D2 at 0.00%	**0.10**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.08	OK	Eq. H1-1b
37	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.09	OK	Eq. H1-1b
38	D1 at 100.00%	0.08	OK	Eq. H1-1b
	D2 at 0.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.09	OK	Eq. H1-1b
39	D1 at 96.88%	0.08	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
40	D1 at 0.00%	0.14	OK	Eq. H1-1b
	D2 at 0.00%	**0.32**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.26	OK	Eq. H1-1b
41	D1 at 0.00%	0.25	OK	Eq. H1-1b
	D2 at 0.00%	**0.57**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.47	OK	Eq. H1-1b
42	D1 at 100.00%	**0.12**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.09	OK	Eq. H1-1b
	D3 at 100.00%	0.04	OK	Eq. H1-1b
43	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	**0.04**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
44	D1 at 0.00%	0.08	OK	Eq. H1-1b
	D2 at 0.00%	**0.16**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.13	OK	Eq. H1-1b
45	D1 at 100.00%	0.06	OK	Eq. H1-1b
	D2 at 0.00%	**0.08**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.06	OK	Eq. H1-1b

Current Date: 2/10/2017 9:48 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind North) (Rev3).etz\

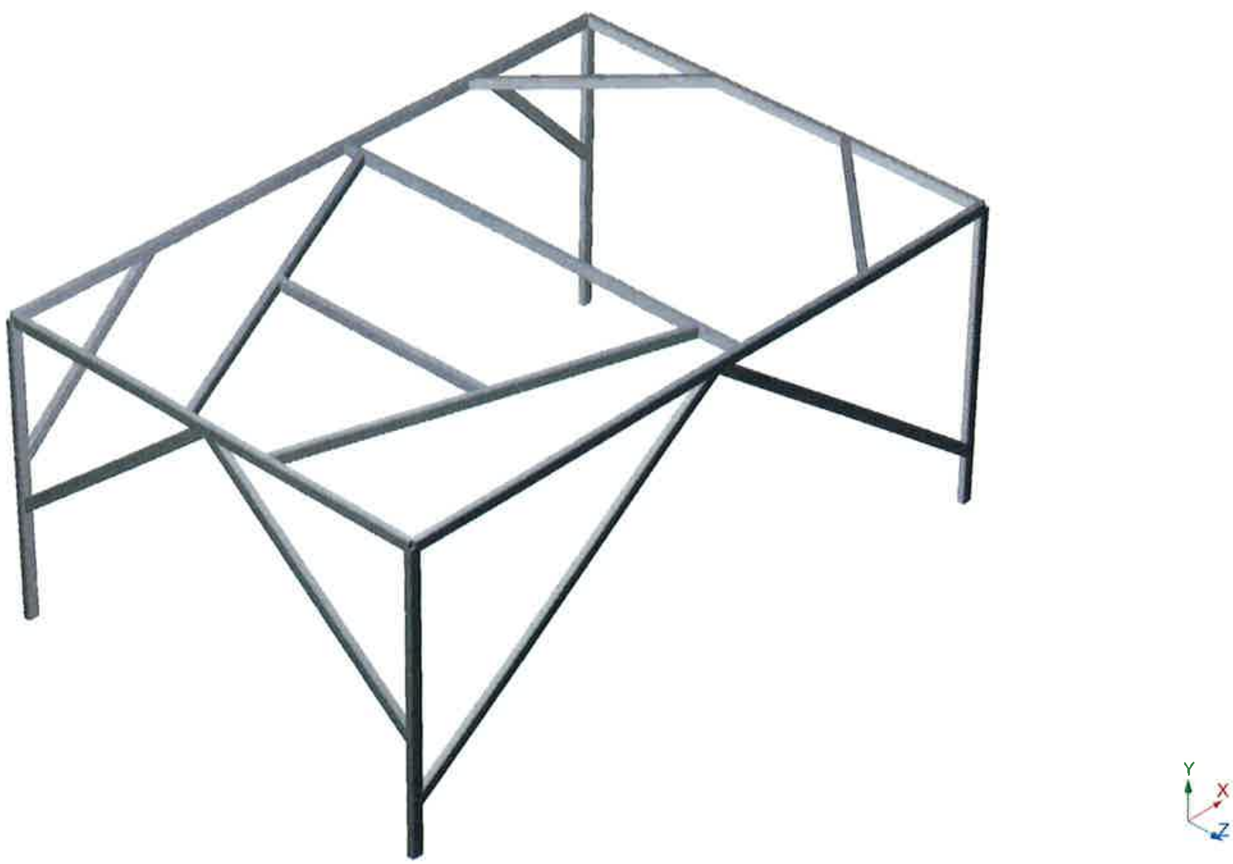
Analysis result

Reactions



Direction of positive forces and moments

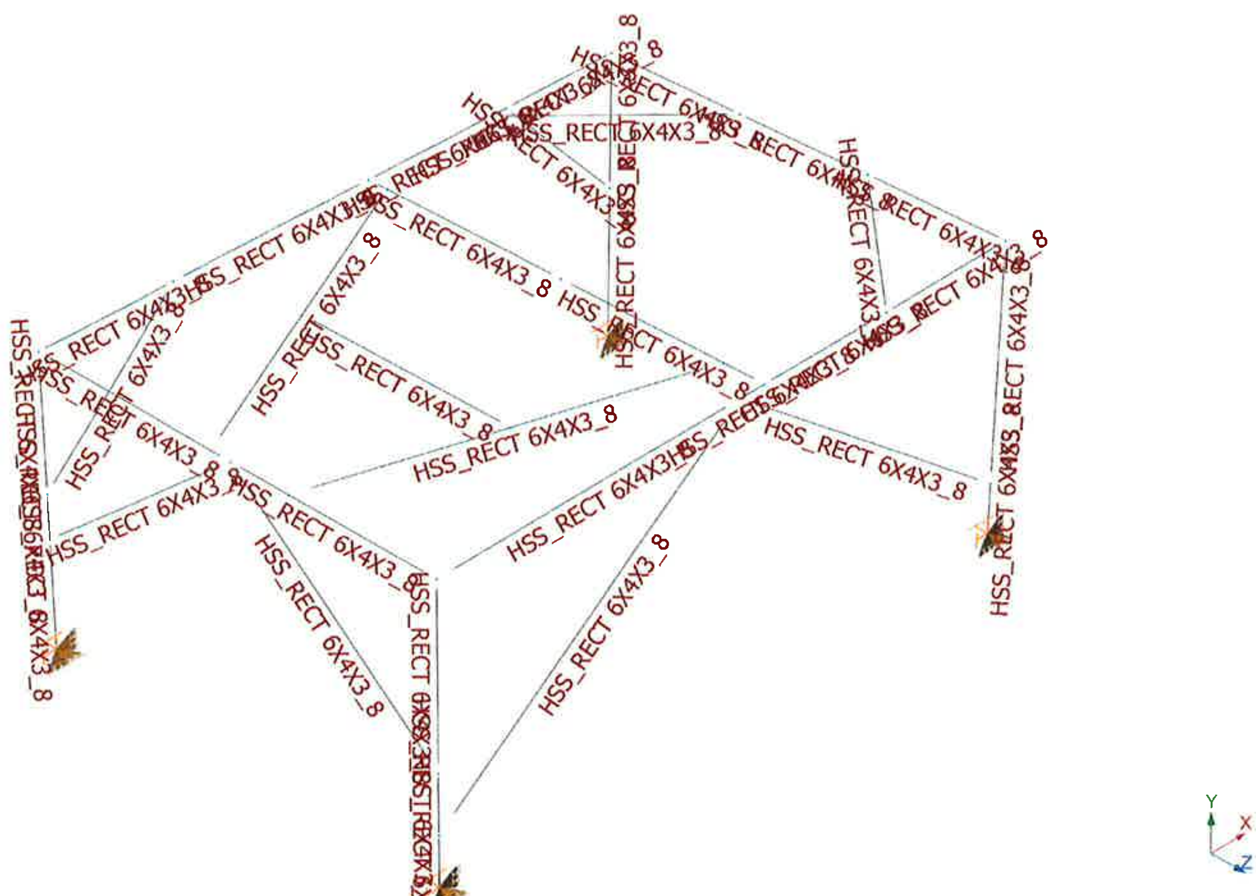
Node	Forces [Kip] FX	FY	FZ	Moments [Kip*ft] MX	MY	MZ
Condition **D1=DL**						
16	1.55366	3.38751	-0.91907	-2.20076	0.01023	-2.46336
17	-1.54869	2.14804	-0.12952	-0.38983	-0.21539	2.33269
18	0.98850	3.08689	0.92333	2.39791	-0.05163	-3.30340
19	-0.99347	1.96410	0.12526	0.49449	0.02870	3.35453
SUM	0.00000	10.58654	0.00000	0.30181	-0.22809	-0.07953
Condition **D2=DL+W**						
16	1.61883	3.53111	-0.31828	-0.42848	-0.61750	-2.33256
17	-2.27077	3.58051	0.34745	1.46585	0.14915	3.32560
18	2.21798	6.53946	1.99517	5.41595	-0.29380	-7.68562
19	-1.56604	3.61716	0.40629	2.11028	0.01407	4.95997
SUM	0.00000	17.26824	2.43063	8.56360	-0.74807	-1.73262
Condition **D3=0.6DL+W**						
16	0.99736	2.17611	0.04935	0.45182	-0.62159	-1.34722
17	-1.65130	2.72129	0.39926	1.62178	0.23531	2.39252
18	1.82258	5.30470	1.62584	4.45678	-0.27315	-6.36426
19	-1.16865	2.83151	0.35619	1.91249	0.00259	3.61816
SUM	0.00000	13.03362	2.43063	8.44287	-0.65684	-1.70080



BETA/GAMMA FRAME
(SOUTHERLY WIND)

Bentley

Current Date: 2/8/2017 4:48 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind South) (I



BETA/GAMMA FRAME
(SOUTHERLY WIND)

Current Date: 2/8/2017 4:49 PM
Units system: English
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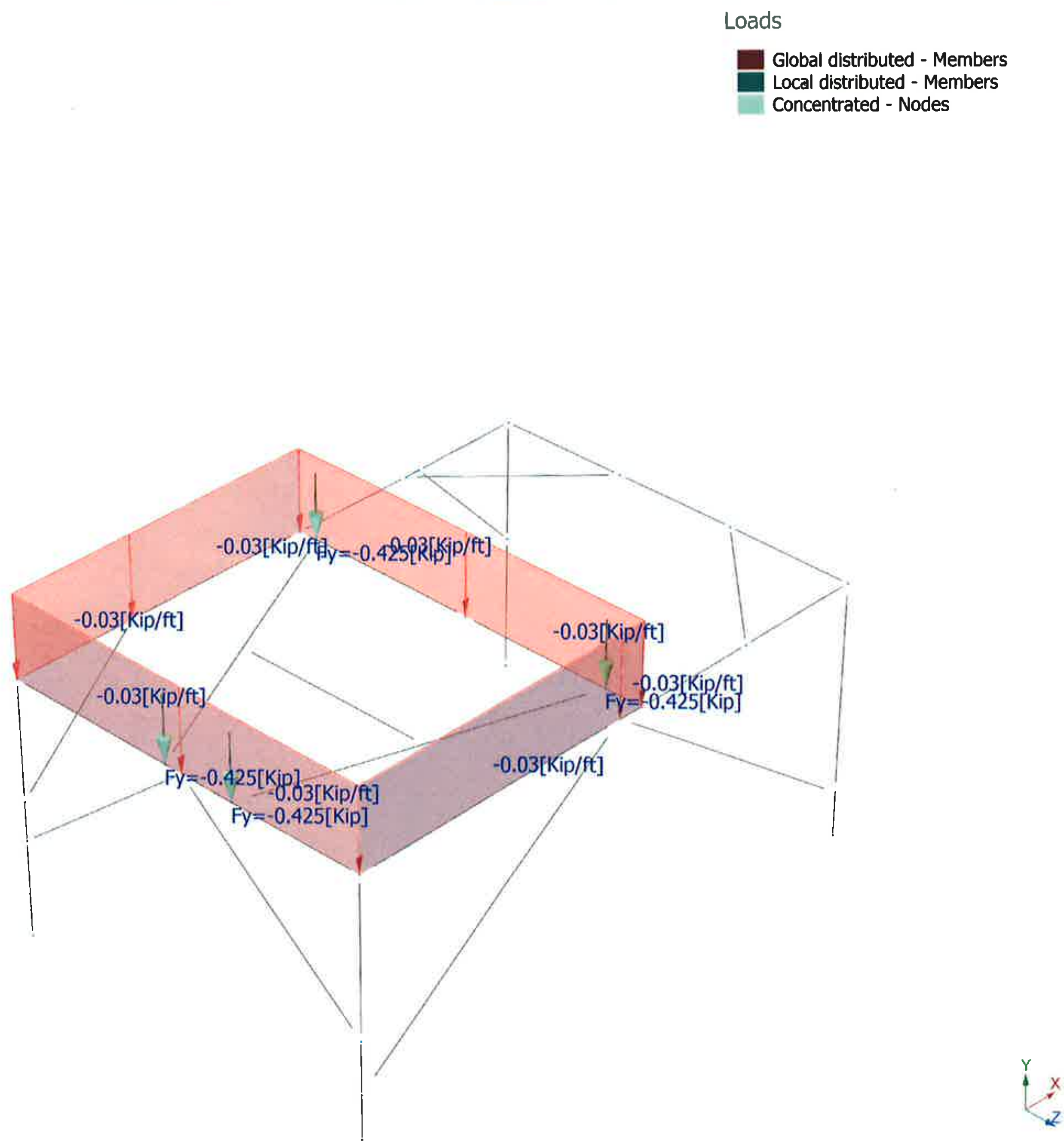
Load condition: DL=Dead Load



BETA/GAMMA FRAME
(SOUTHERLY WIND)

Current Date: 2/10/2017 9:50 AM
Units system: English
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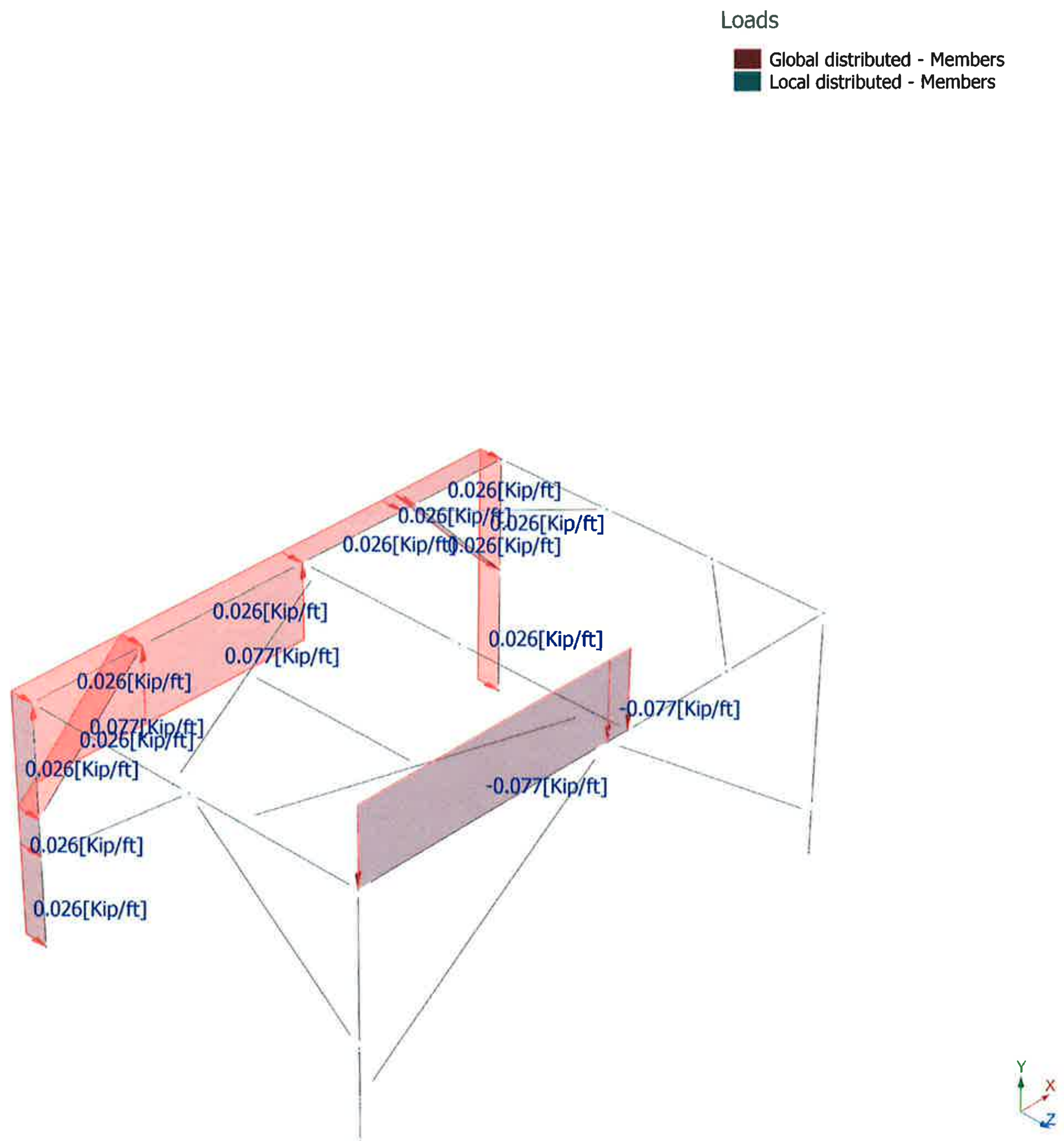
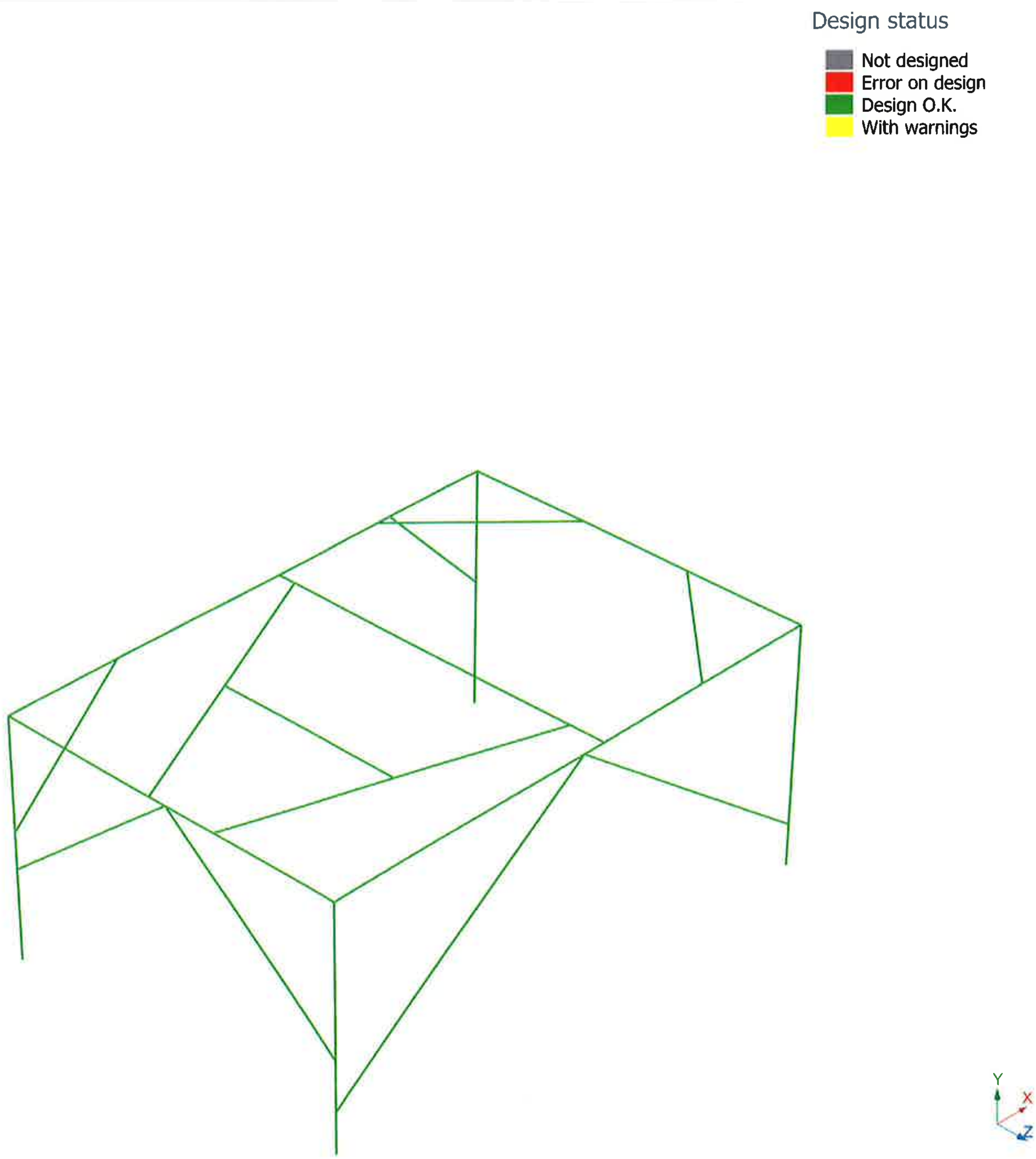
Load condition: W=Wind

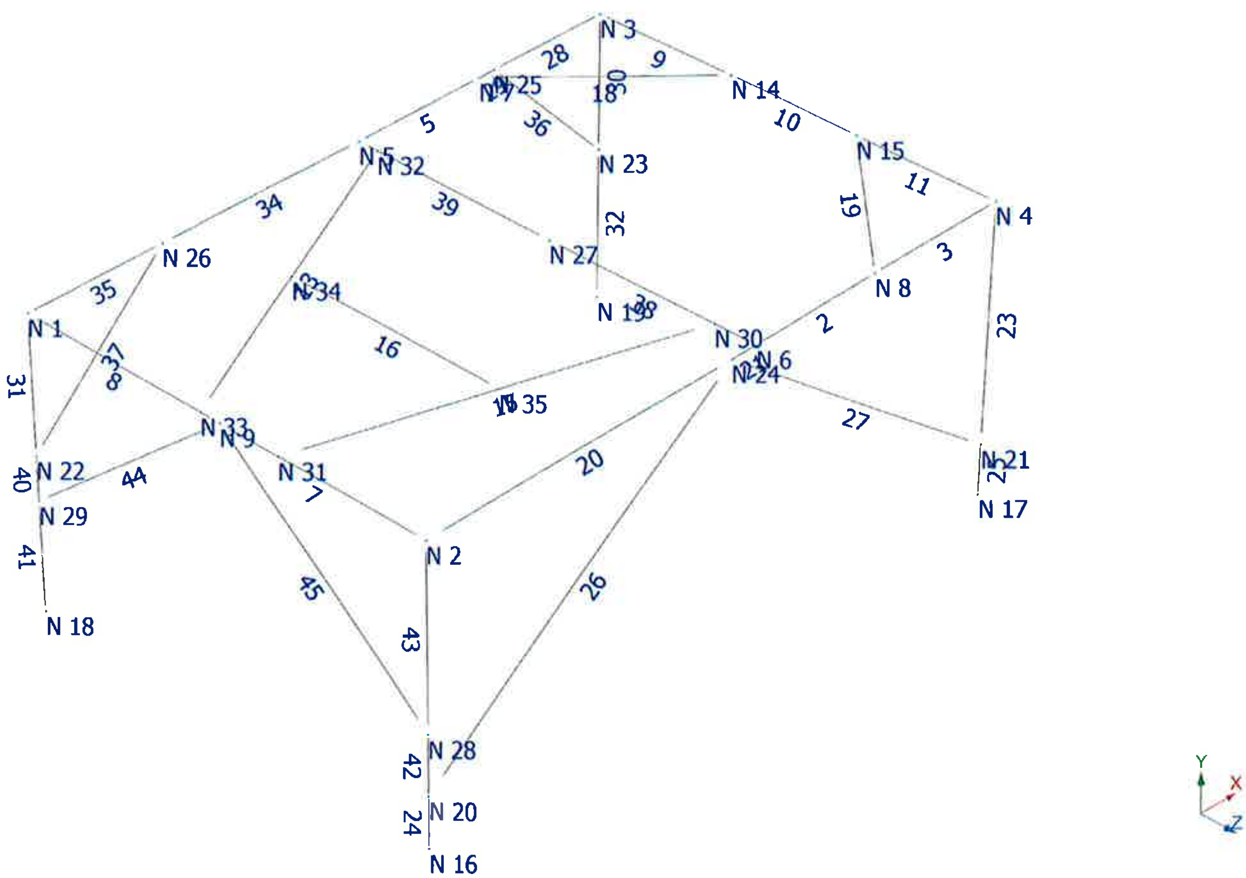


Current Date: 2/10/2017 9:50 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind South) (I



Bentley

Current Date: 2/8/2017 4:48 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind South) (I



BETA/GAMMA FRAME
(SOUTHERLY WIND)

Current Date: 2/10/2017 9:50 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind South) (Rev3).etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :
D1=DL
D2=DL+W
D3=0.6DL+W

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_RECT 6X4X3_8	2	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.05	OK	Eq. H1-1b
		3	D1 at 100.00%	0.02	OK	Eq. H1-1b
			D2 at 100.00%	**0.03**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.03	OK	Eq. H1-1b
		5	D1 at 0.00%	0.14	OK	Eq. H1-1b
			D2 at 0.00%	**0.21**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.15	OK	Eq. H1-1b
		7	D1 at 71.88%	0.04	OK	Eq. H1-1b
			D2 at 71.88%	**0.09**	**OK**	Eq. H1-1b
			D3 at 71.88%	0.08	OK	Eq. H1-1b
		8	D1 at 90.63%	0.04	OK	
			D2 at 65.63%	**0.05**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.04	OK	Eq. H1-1b
		9	D1 at 100.00%	0.03	OK	Eq. H1-1b
			D2 at 100.00%	**0.06**	**OK**	Eq. H1-1b
			D3 at 100.00%	0.05	OK	Eq. H1-1b
		10	D1 at 93.75%	0.04	OK	Eq. H1-1b
			D2 at 68.75%	**0.08**	**OK**	Eq. H1-1b
			D3 at 62.50%	0.06	OK	Eq. H1-1b
		11	D1 at 100.00%	0.04	OK	Eq. H1-1b
			D2 at 0.00%	**0.08**	**OK**	Eq. H1-1b
			D3 at 0.00%	0.07	OK	Eq. H1-1b
		13	D1 at 37.50%	0.05	OK	Eq. H1-1b
			D2 at 50.00%	**0.09**	**OK**	Eq. H1-1b
			D3 at 50.00%	0.07	OK	Eq. H1-1b
		15	D1 at 46.88%	0.04	OK	Eq. H1-1b
			D2 at 46.88%	**0.09**	**OK**	Eq. H1-1b
			D3 at 46.88%	0.07	OK	Eq. H1-1b
		16	D1 at 56.25%	0.02	OK	Eq. H1-1b
			D2 at 75.00%	**0.04**	**OK**	Eq. H1-1b
			D3 at 81.25%	0.03	OK	Eq. H1-1b
		18	D1 at 0.00%	0.03	OK	Eq. H1-1b

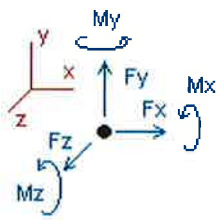
	D2 at 0.00%	**0.05**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
19	D1 at 37.50%	0.02	OK	Eq. H1-1b
	D2 at 43.75%	**0.03**	**OK**	Eq. H1-1b
	D3 at 43.75%	0.02	OK	Eq. H1-1b
20	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.13**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.11	OK	Eq. H1-1b
21	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
23	D1 at 0.00%	0.05	OK	Eq. H1-1b
	D2 at 0.00%	**0.13**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.11	OK	Eq. H1-1b
24	D1 at 0.00%	0.21	OK	Eq. H1-1b
	D2 at 0.00%	**0.45**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.36	OK	Eq. H1-1b
25	D1 at 0.00%	0.10	OK	Eq. H1-1b
	D2 at 0.00%	**0.24**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.20	OK	Eq. H1-1b
26	D1 at 0.00%	0.05	OK	Eq. H1-2
	D2 at 0.00%	**0.10**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.08	OK	Eq. H1-1b
27	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.06	OK	Eq. H1-2
28	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
29	D1 at 100.00%	0.08	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
30	D1 at 100.00%	0.09	OK	Eq. H1-1b
	D2 at 100.00%	**0.11**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
31	D1 at 100.00%	**0.09**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.07	OK	Eq. H1-1b
	D3 at 100.00%	0.05	OK	Eq. H1-1b
32	D1 at 100.00%	0.15	OK	Eq. H1-1b
	D2 at 0.00%	**0.22**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.19	OK	Eq. H1-1b
34	D1 at 0.00%	0.12	OK	Eq. H1-1b
	D2 at 0.00%	**0.19**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.14	OK	Eq. H1-1b
35	D1 at 100.00%	0.07	OK	Eq. H1-1b
	D2 at 100.00%	**0.09**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.06	OK	Eq. H1-1b

36	D1 at 0.00%	0.06	OK	Eq. H1-1b
	D2 at 0.00%	**0.08**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.06	OK	Eq. H1-1b
37	D1 at 100.00%	0.05	OK	Eq. H1-1b
	D2 at 100.00%	**0.06**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.04	OK	Eq. H1-1b
38	D1 at 100.00%	0.08	OK	Eq. H1-1b
	D2 at 0.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.10	OK	Eq. H1-1b
39	D1 at 96.88%	0.08	OK	Eq. H1-1b
	D2 at 96.88%	**0.11**	**OK**	Eq. H1-1b
	D3 at 96.88%	0.08	OK	Eq. H1-1b
40	D1 at 0.00%	**0.14**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.11	OK	Eq. H1-1b
	D3 at 0.00%	0.06	OK	Eq. H1-1b
41	D1 at 0.00%	**0.25**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.16	OK	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
42	D1 at 100.00%	0.12	OK	Eq. H1-1b
	D2 at 100.00%	**0.24**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.19	OK	Eq. H1-1b
43	D1 at 0.00%	0.04	OK	Eq. H1-1b
	D2 at 0.00%	**0.08**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.06	OK	Eq. H1-1b
44	D1 at 0.00%	**0.08**	**OK**	Eq. H1-1b
	D2 at 100.00%	0.07	OK	Eq. H1-1b
	D3 at 93.75%	0.05	OK	Eq. H1-1b
45	D1 at 100.00%	0.06	OK	Eq. H1-1b
	D2 at 100.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 100.00%	0.10	OK	Eq. H1-1b

Current Date: 2/10/2017 9:50 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 Beta-Gamma Frame (Wind South) (Rev3).etz\

Analysis result

Reactions



Direction of positive forces and moments

	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **D1=DL**						
16	1.55366	3.38751	-0.91907	-2.20076	0.01023	-2.46336
17	-1.54869	2.14805	-0.12952	-0.38983	-0.21539	2.33269
18	0.98850	3.08689	0.92333	2.39791	-0.05163	-3.30340
19	-0.99347	1.96410	0.12526	0.49449	0.02870	3.35452
SUM	0.00000	10.58654	0.00000	0.30181	-0.22809	-0.07953
Condition **D2=DL+W**						
16	3.23140	7.03166	-1.90586	-4.86006	0.10979	-5.14200
17	-2.64328	3.96836	-0.37852	-1.79742	-0.31745	3.96778
18	0.80180	3.13739	0.54462	1.25261	0.22212	-2.48924
19	-1.38992	3.13083	-0.18603	-0.87480	0.01480	4.88505
SUM	0.00000	17.26824	-1.92580	-6.27966	0.02926	1.22158
Condition **D3=0.6DL+W**						
16	2.60994	5.67665	-1.53824	-3.97976	0.10570	-4.15666
17	-2.02380	3.10915	-0.32671	-1.64149	-0.23130	3.03470
18	0.40640	1.90264	0.17528	0.29345	0.24277	-1.16788
19	-0.99253	2.34518	-0.23614	-1.07259	0.00332	3.54324
SUM	0.00000	13.03362	-1.92580	-6.40039	0.12049	1.25340

Equipment Platform Calculations

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Steel Frame Design Calculations

Live Loads:

Service Load	60 psf
Snow Load	30 psf
Wind Load	27.6 psf

Dead Loads:

Shelter + Equipment	30000 lbs.
Grating	15 psf
Handrail	10 plf

Load Breakdown

- **Beam A (0 ft - 15.5 ft)**

Live Load

→ Service 60 psf x 5.75 ft. (Half Shelter Width)
= **345.00 plf**

→ Snow 30 psf x 5.75 ft. (Half Shelter Width)
= **172.50 plf**

→ Wind 27.6 psf x 10.5 ft. (Shelter Height)
= **289.80 plf**

Dead Load

→ Shelter 30000 lbs / 2 beams / 15.5 ft (Shelter Length)
= **967.74 plf**

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Load Breakdown (Cont.)

- **Beam A (15.5 ft - 30 ft)**

Live Load

→ Service 60 psf x 1.5 ft. (Tributary Width)
= **90.00 plf**

→ Wind 27.6 psf x 1.33 ft. (Beam Height)
= **36.71 plf**

Dead Load

→ Grating 15 psf x 1.5 ft. (Tributary Width)
= **22.50 plf**

→ Handrail **10 plf**

- **Beam B**

Live Load

→ Service 60 psf x 5.75 ft. (Tributary Width)
= **345.00 plf**

→ Snow 30 psf x 5.75 ft. (Half Shelter Width)
= **172.50 plf**

Dead Load

→ Shelter 30000 lbs / 2 beams / 15.5 ft (Shelter Length)
= **967.74 plf**

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Load Breakdown (Cont.)

- **Beam C**

Live Load

→ Service 60 psf x 2.6 ft. (Tributary Width)

= **156.00 plf**

Dead Load

→ Grating 15 psf x 2.6 ft. (Tributary Width)

= **39.00 plf**

- **Beam D**

Live Load

→ Service 60 psf x 3.5 ft. (Tributary Width)

= **210.00 plf**

Dead Load

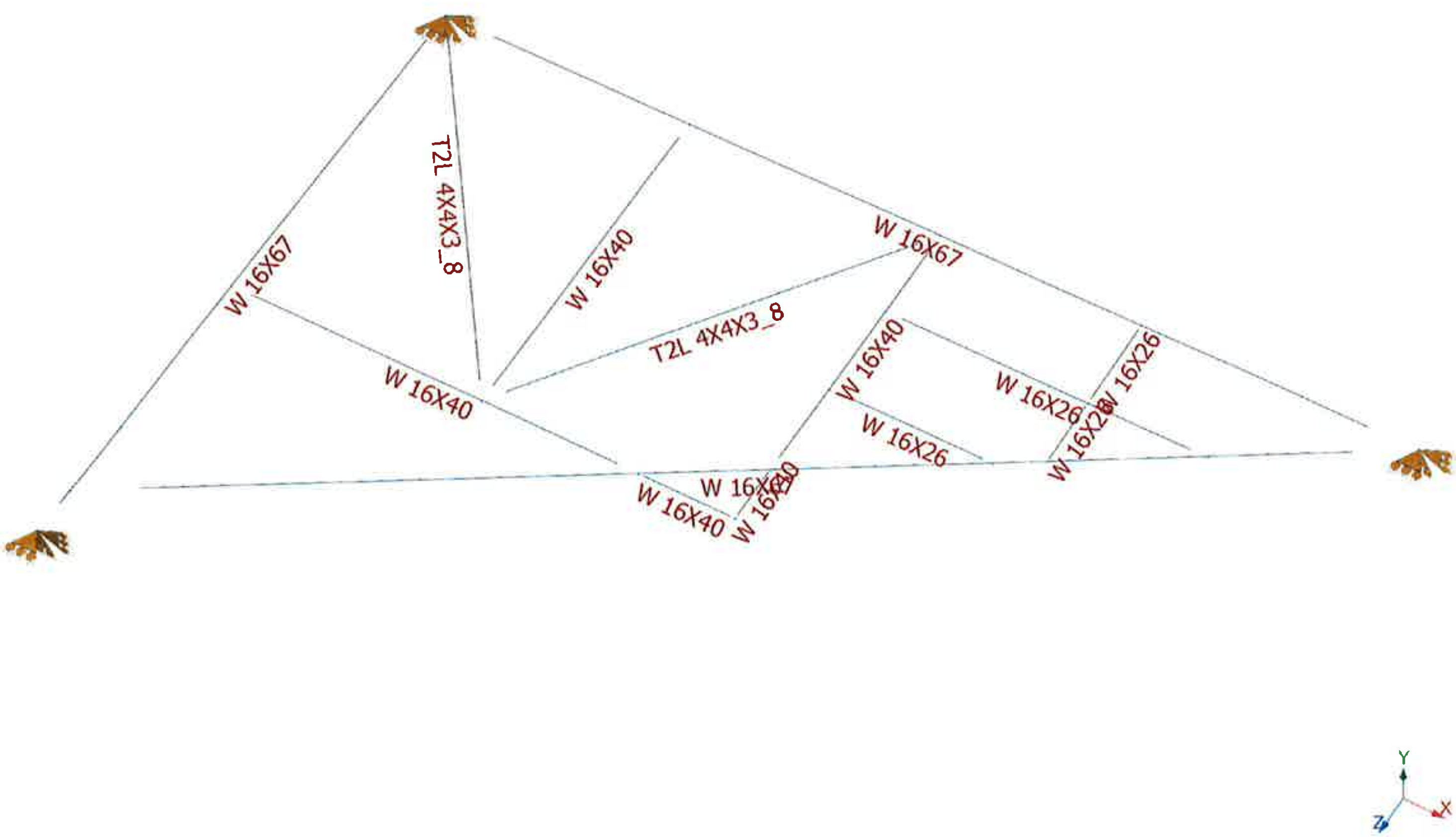
→ Grating 15 psf x 3.5 ft. (Tributary Width)

= **52.50 plf**


Y
X
Z

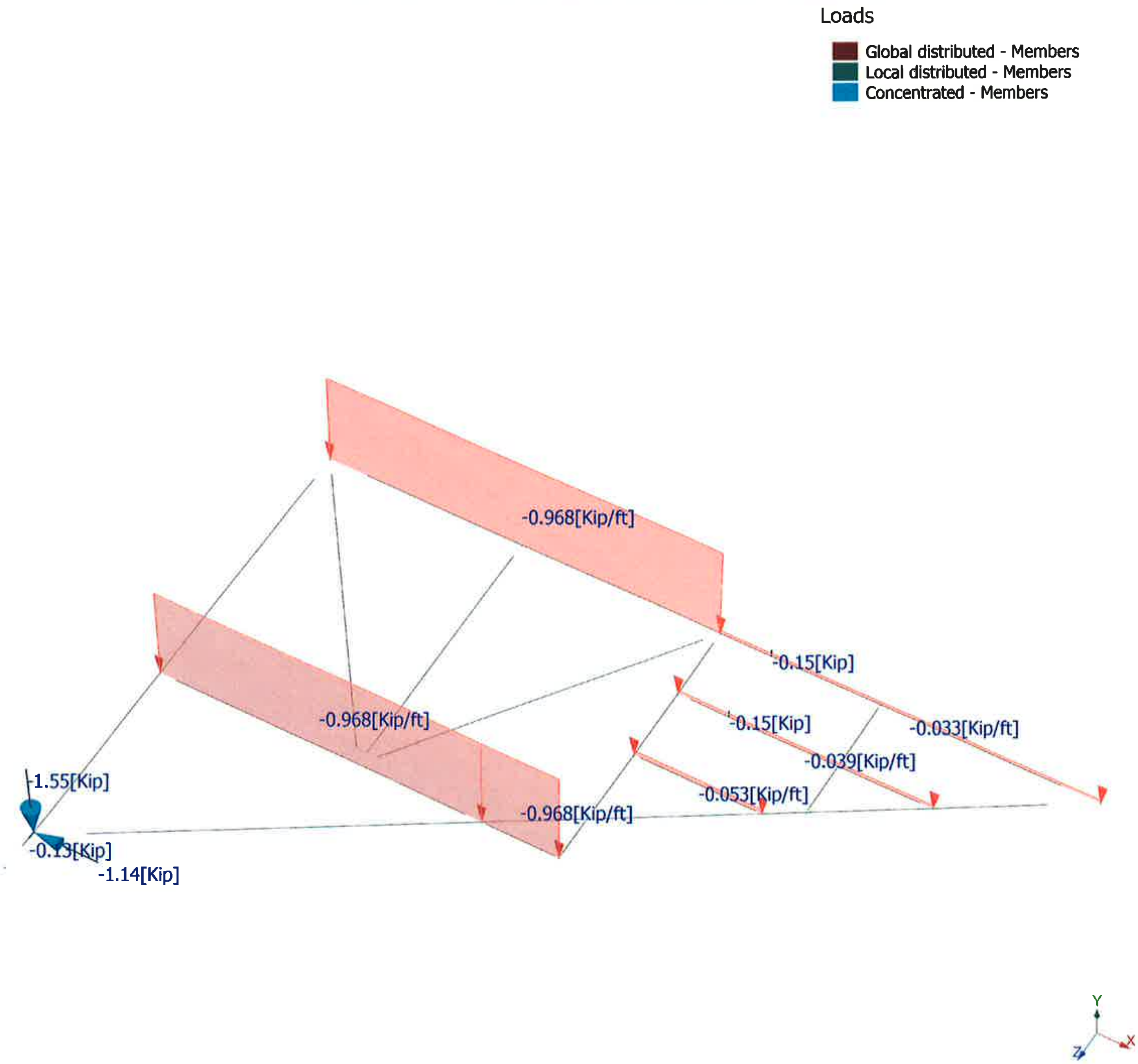
Bentley

Current Date: 2/8/2017 4:58 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 (Shelter Frame) (Rev3).etz\



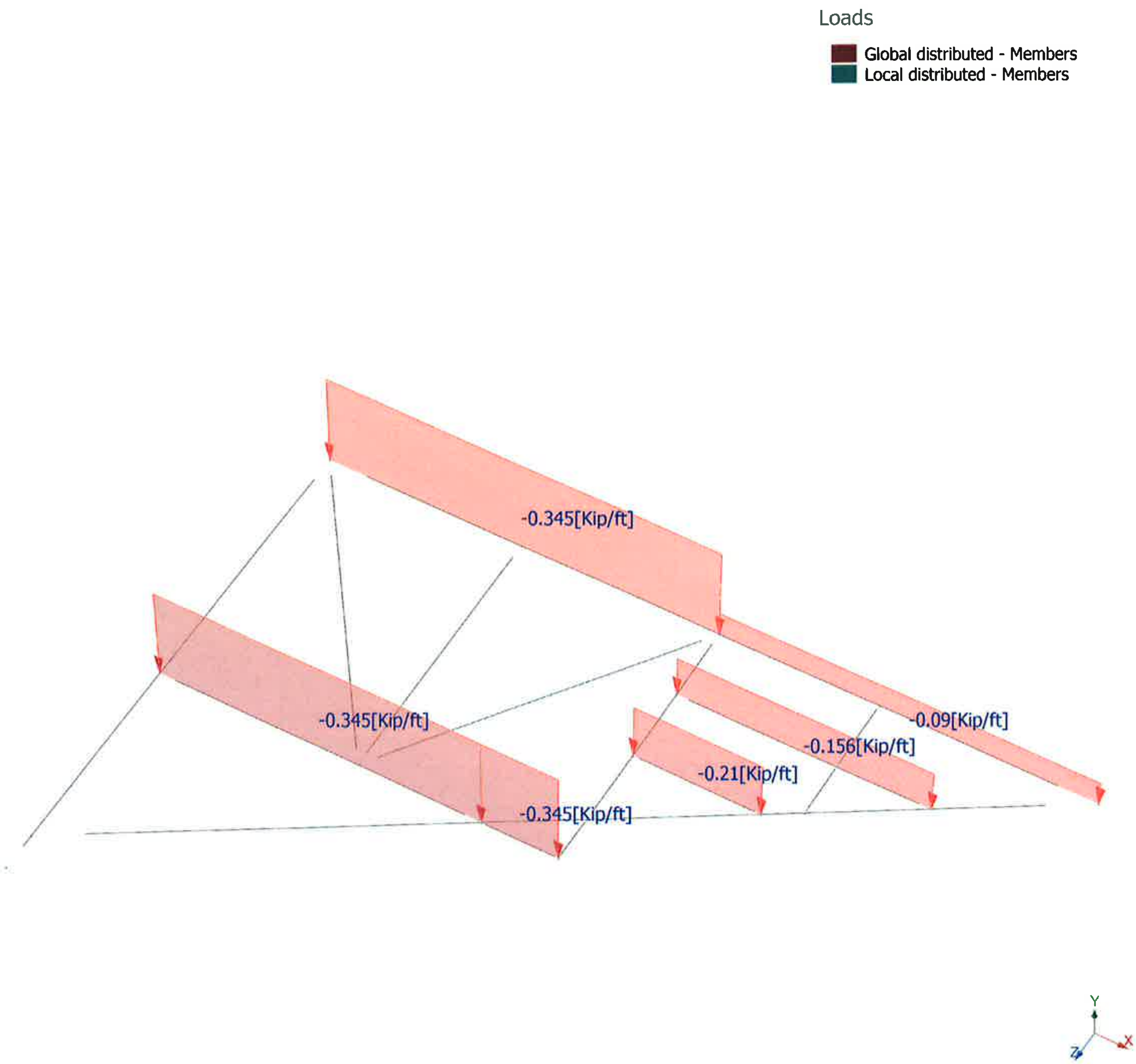
Bentley

Current Date: 2/8/2017 4:59 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 (Shelter Frame) (Rev3).etz\
Load condition: DL=Dead Load

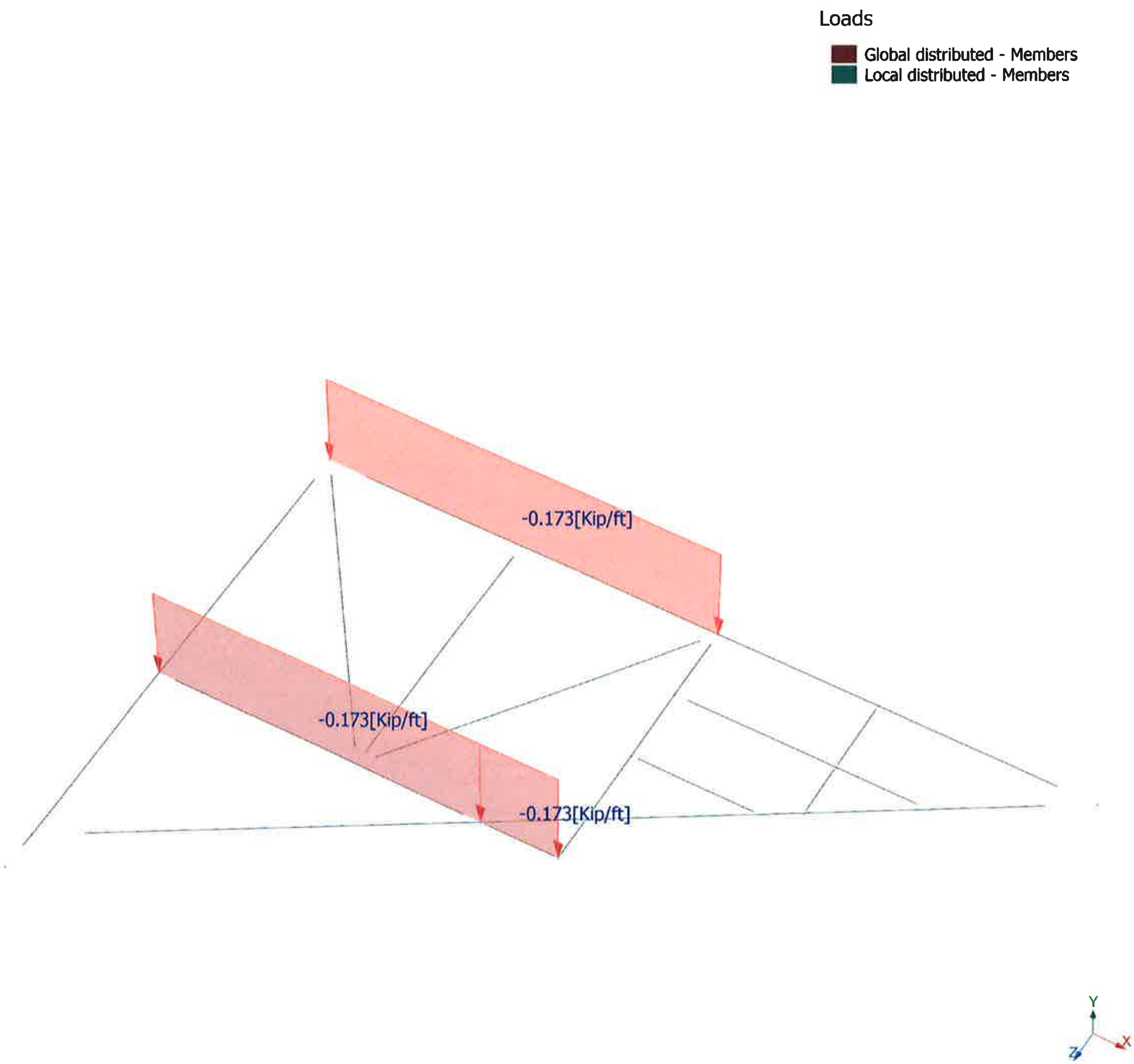


![Bentley]

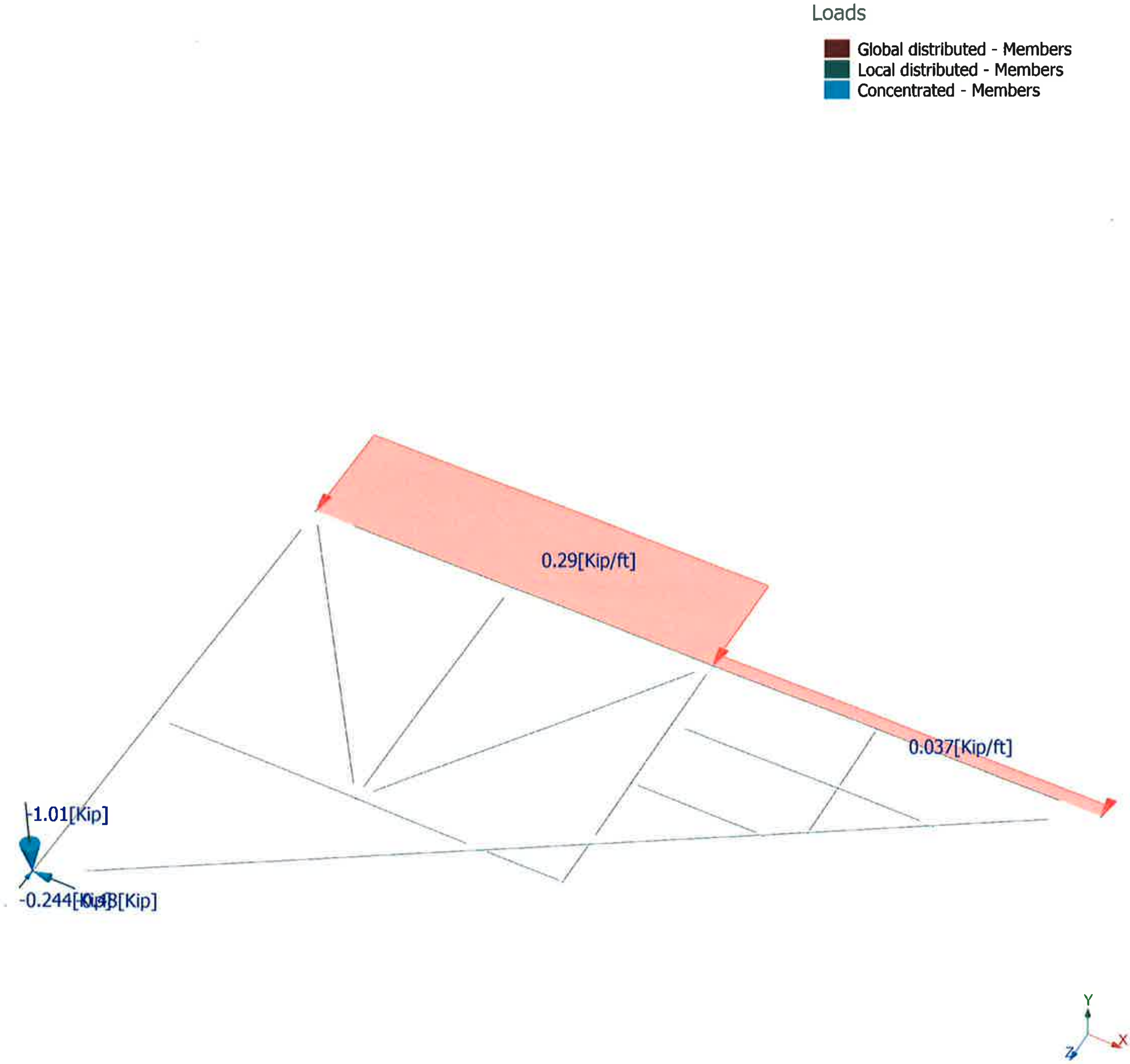
Current Date: 2/8/2017 4:59 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 (Shelter Frame) (Rev3).etz\
Load condition: LL=Service

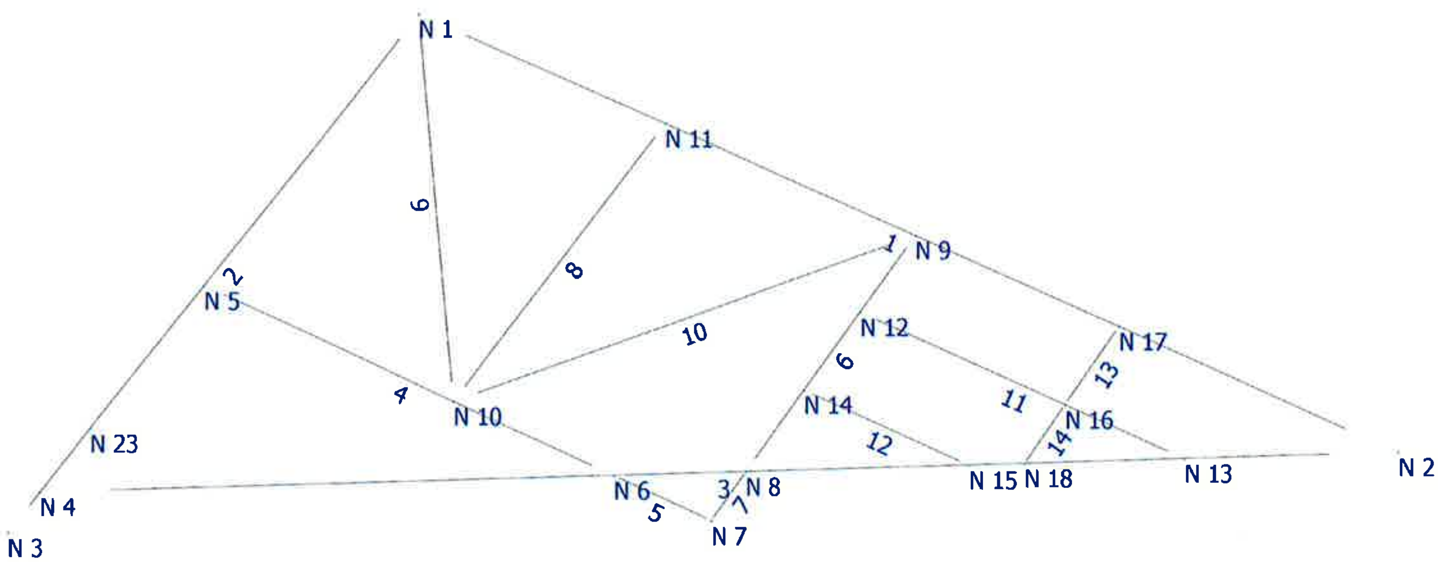


Current Date: 2/8/2017 4:59 PM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 (Shelter Frame) (Rev3).etz\
Load condition: S=Snow



Current Date: 2/10/2017 10:15 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 (Shelter Frame) (Rev3).etz\
Load condition: W=Wind




N 1
N 11
N 9
N 5
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N 12
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N 4
N 6
N 8
N 15
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N 2
N 3
N 7
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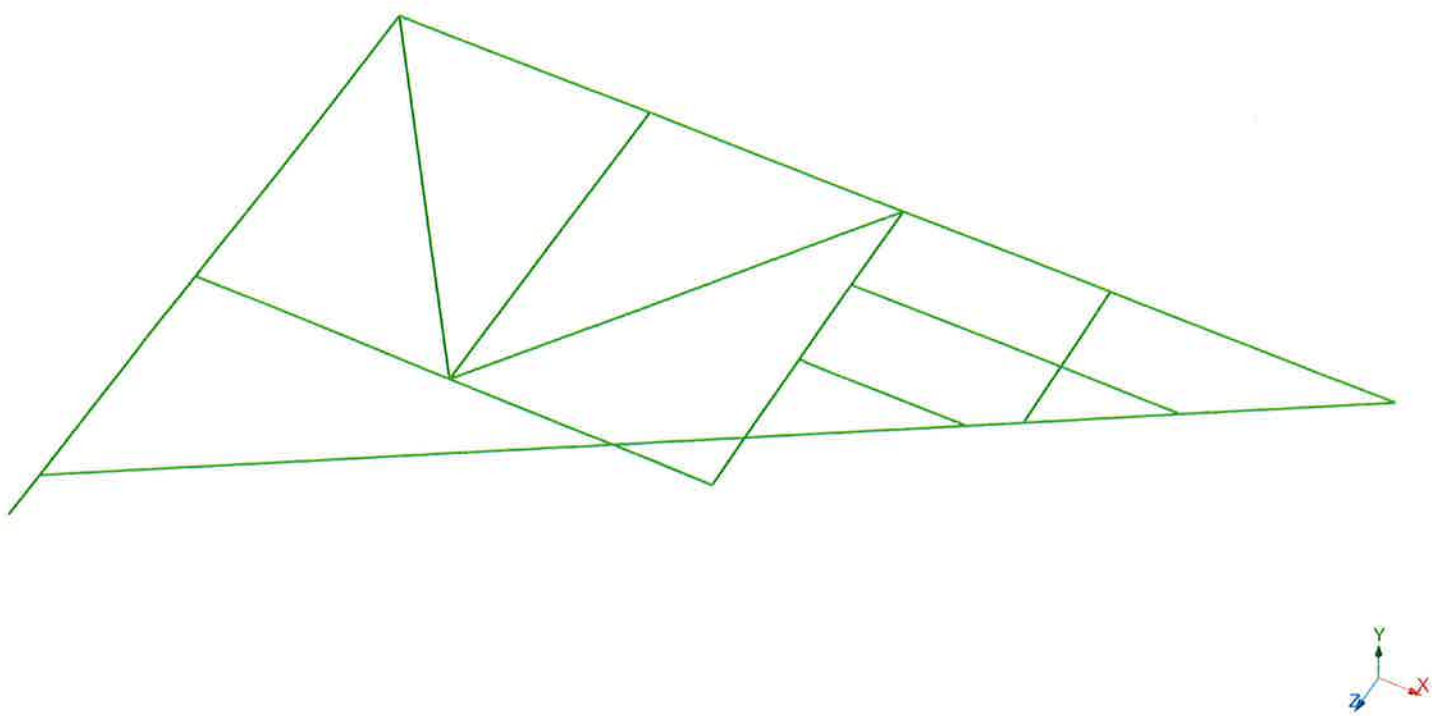
Y
X
Z

Design status

- Not designed
- Error on design
- Design O.K.
- With warnings



Bentley

Current Date: 2/10/2017 10:15 AM
Units system: English
File name: W:\STRUCTURAL DEPARTMENT\ANALYSIS SOFTWARE\RAM Elements\RAM Projects\S2900\Rev3\S2900 (Shelter Frame) (Rev3).etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

D1=DL
D2=DL+LL
D3=DL+S
D4=DL+0.75LL
D5=DL+0.75S
D6=DL+0.75LL+0.75S
D7=DL+W
D8=DL+0.75W+0.75LL
D9=DL+0.75W+0.75S
D10=DL+0.75W+0.75LL+0.75S
D11=0.6DL+W

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	T2L 4X4X3_8	**9**	D1 at 100.00%	0.08	OK	Eq. H1-1b
			D10 at 100.00%	**0.11**	**OK**	Eq. H1-1b
			D11 at 100.00%	0.07	OK	Eq. H1-1b
			D2 at 100.00%	0.10	OK	Eq. H1-1b
			D3 at 100.00%	0.09	OK	Eq. H1-1b
			D4 at 100.00%	0.09	OK	Eq. H1-1b
			D5 at 100.00%	0.09	OK	Eq. H1-1b
			D6 at 100.00%	0.10	OK	Eq. H1-1b
			D7 at 100.00%	0.10	OK	Eq. H1-1b
			D8 at 100.00%	0.10	OK	Eq. H1-1b
			D9 at 100.00%	0.10	OK	Eq. H1-1b
		10	D1 at 50.00%	0.08	OK	Eq. H1-1b
			D10 at 56.25%	**0.11**	**OK**	Eq. H1-1b
			D11 at 56.25%	0.05	OK	Eq. H1-1b
			D2 at 56.25%	0.11	OK	Eq. H1-1b
			D3 at 56.25%	0.09	OK	Eq. H1-1b
			D4 at 56.25%	0.10	OK	Eq. H1-1b
			D5 at 50.00%	0.09	OK	Eq. H1-1b
			D6 at 56.25%	0.11	OK	Eq. H1-1b
			D7 at 56.25%	0.09	OK	Eq. H1-1b
			D8 at 56.25%	0.10	OK	Eq. H1-1b
			D9 at 56.25%	0.09	OK	Eq. H1-1b
	W 16X26	**11**	D1 at 62.50%	0.13	OK	Eq. H1-1b
			D10 at 62.50%	**0.19**	**OK**	Eq. H1-1b
			D11 at 62.50%	0.08	OK	Eq. H1-1b
			D2 at 62.50%	0.18	OK	Eq. H1-1b
			D3 at 62.50%	0.14	OK	Eq. H1-1b
			D4 at 62.50%	0.17	OK	Eq. H1-1b
			D5 at 62.50%	0.14	OK	Eq. H1-1b
			D6 at 62.50%	0.18	OK	Eq. H1-1b
			D7 at 62.50%	0.13	OK	Eq. H1-1b
			D8 at 62.50%	0.18	OK	Eq. H1-1b
			D9 at 62.50%	0.14	OK	Eq. H1-1b
		12	D1 at 65.63%	0.00	OK	Eq. H1-1b
			D10 at 78.13%	**0.01**	**OK**	Eq. H1-1b

		D11 at 96.88%	0.01	OK	Eq. H1-1b
		D2 at 0.00%	0.01	OK	Sec. G2.1(a)
		D3 at 68.75%	0.00	OK	Eq. H1-1b
		D4 at 0.00%	0.01	OK	Sec. G2.1(a)
		D5 at 68.75%	0.00	OK	Eq. H1-1b
		D6 at 0.00%	0.01	OK	Sec. G2.1(a)
		D7 at 96.88%	0.01	OK	Eq. H1-1b
		D8 at 78.13%	0.01	OK	Eq. H1-1b
		D9 at 96.88%	0.01	OK	Eq. H1-1b
	13	D1 at 0.00%	0.06	OK	Eq. H1-1b
		D10 at 0.00%	**0.09**	**OK**	Eq. H1-1b
		D11 at 0.00%	0.05	OK	Eq. H1-1b
		D2 at 0.00%	0.08	OK	Eq. H1-1b
		D3 at 0.00%	0.07	OK	Eq. H1-1b
		D4 at 0.00%	0.08	OK	Eq. H1-1b
		D5 at 0.00%	0.07	OK	Eq. H1-1b
		D6 at 0.00%	0.09	OK	Eq. H1-1b
		D7 at 0.00%	0.07	OK	Eq. H1-1b
		D8 at 0.00%	0.08	OK	Eq. H1-1b
		D9 at 0.00%	0.08	OK	Eq. H1-1b
	14	D1 at 0.00%	0.06	OK	Eq. H1-1b
		D10 at 0.00%	**0.09**	**OK**	Eq. H1-1b
		D11 at 0.00%	0.04	OK	Eq. H1-1b
		D2 at 0.00%	0.08	OK	Eq. H1-1b
		D3 at 0.00%	0.07	OK	Eq. H1-1b
		D4 at 0.00%	0.08	OK	Eq. H1-1b
		D5 at 0.00%	0.07	OK	Eq. H1-1b
		D6 at 0.00%	0.09	OK	Eq. H1-1b
		D7 at 0.00%	0.06	OK	Eq. H1-1b
		D8 at 0.00%	0.08	OK	Eq. H1-1b
		D9 at 0.00%	0.07	OK	Eq. H1-1b
W 16X40	**4**	D1 at 100.00%	0.20	OK	Eq. H1-1b
		D10 at 100.00%	**0.28**	**OK**	Eq. H1-1b
		D11 at 62.50%	0.13	OK	Eq. H1-1b
		D2 at 100.00%	0.27	OK	Eq. H1-1b
		D3 at 100.00%	0.23	OK	Eq. H1-1b
		D4 at 100.00%	0.25	OK	Eq. H1-1b
		D5 at 100.00%	0.22	OK	Eq. H1-1b
		D6 at 100.00%	0.28	OK	Eq. H1-1b
		D7 at 100.00%	0.21	OK	Eq. H1-1b
		D8 at 100.00%	0.26	OK	Eq. H1-1b
		D9 at 100.00%	0.23	OK	Eq. H1-1b
	5	D1 at 0.00%	0.09	OK	Eq. H1-1b
		D10 at 0.00%	**0.13**	**OK**	Eq. H1-1b
		D11 at 0.00%	0.06	OK	Eq. H1-1b
		D2 at 0.00%	0.12	OK	Eq. H1-1b
		D3 at 0.00%	0.10	OK	Eq. H1-1b
		D4 at 0.00%	0.11	OK	Eq. H1-1b
		D5 at 0.00%	0.10	OK	Eq. H1-1b
		D6 at 0.00%	0.12	OK	Eq. H1-1b
		D7 at 0.00%	0.09	OK	Eq. H1-1b
		D8 at 0.00%	0.12	OK	Eq. H1-1b
		D9 at 0.00%	0.10	OK	Eq. H1-1b
	6	D1 at 0.00%	0.07	OK	Sec. G2.1(a)
		D10 at 0.00%	**0.11**	**OK**	Eq. H1-1b
		D11 at 1.56%	0.05	OK	Eq. H1-1b
		D2 at 0.00%	0.10	OK	Sec. G2.1(a)
		D3 at 0.00%	0.08	OK	Sec. G2.1(a)

		D4 at 0.00%	0.09	OK	Sec. G2.1(a)
		D5 at 0.00%	0.08	OK	Sec. G2.1(a)
		D6 at 0.00%	0.10	OK	Sec. G2.1(a)
		D7 at 0.00%	0.08	OK	Eq. H1-1b
		D8 at 0.00%	0.10	OK	Eq. H1-1b
		D9 at 0.00%	0.08	OK	Eq. H1-1b
	7	D1 at 100.00%	0.07	OK	Sec. G2.1(a)
		D10 at 100.00%	**0.11**	**OK**	Eq. H1-1b
		D11 at 100.00%	0.05	OK	Eq. H1-1b
		D2 at 100.00%	0.10	OK	Sec. G2.1(a)
		D3 at 100.00%	0.08	OK	Sec. G2.1(a)
		D4 at 100.00%	0.09	OK	Sec. G2.1(a)
		D5 at 100.00%	0.08	OK	Sec. G2.1(a)
		D6 at 100.00%	0.10	OK	Sec. G2.1(a)
		D7 at 100.00%	0.08	OK	Eq. H1-1b
		D8 at 100.00%	0.10	OK	Eq. H1-1b
		D9 at 100.00%	0.08	OK	Eq. H1-1b
	8	D1 at 43.75%	0.00	OK	Eq. H1-1b
		D10 at 50.00%	0.01	OK	Eq. H1-2
		D11 at 0.00%	0.01	OK	Sec. E1
		D2 at 43.75%	0.00	OK	Eq. H1-1b
		D3 at 43.75%	0.00	OK	Eq. H1-1b
		D4 at 43.75%	0.00	OK	Eq. H1-1b
		D5 at 43.75%	0.00	OK	Eq. H1-1b
		D6 at 43.75%	0.00	OK	Eq. H1-1b
		D7 at 43.75%	**0.01**	**OK**	Eq. H1-2
		D8 at 43.75%	0.01	OK	Eq. H1-2
		D9 at 43.75%	0.01	OK	Eq. H1-2
W 16X67	**1**	D1 at 39.06%	0.34	OK	Eq. H1-1b
		D10 at 39.06%	**0.48**	**OK**	Eq. H1-1b
		D11 at 39.06%	0.21	OK	Eq. H1-1b
		D2 at 39.06%	0.47	OK	Eq. H1-1b
		D3 at 37.50%	0.39	OK	Eq. H1-1b
		D4 at 39.06%	0.44	OK	Eq. H1-1b
		D5 at 37.50%	0.37	OK	Eq. H1-1b
		D6 at 39.06%	0.48	OK	Eq. H1-1b
		D7 at 39.06%	0.35	OK	Eq. H1-1b
		D8 at 39.06%	0.45	OK	Eq. H1-1b
		D9 at 39.06%	0.38	OK	Eq. H1-1b
	2	D1 at 53.13%	0.16	OK	Eq. H1-1b
		D10 at 53.13%	**0.22**	**OK**	Eq. H1-1b
		D11 at 53.13%	0.10	OK	Eq. H1-1b
		D2 at 53.13%	0.21	OK	Eq. H1-1b
		D3 at 53.13%	0.18	OK	Eq. H1-1b
		D4 at 53.13%	0.19	OK	Eq. H1-1b
		D5 at 53.13%	0.17	OK	Eq. H1-1b
		D6 at 53.13%	0.21	OK	Eq. H1-1b
		D7 at 53.13%	0.17	OK	Eq. H1-1b
		D8 at 53.13%	0.20	OK	Eq. H1-1b
		D9 at 53.13%	0.18	OK	Eq. H1-1b
	3	D1 at 51.79%	0.58	OK	Eq. H1-1b
		D10 at 51.79%	**0.80**	**OK**	Eq. H1-1b
		D11 at 51.79%	0.36	OK	Eq. H1-1b
		D2 at 51.79%	0.80	OK	Eq. H1-1b
		D3 at 51.79%	0.65	OK	Eq. H1-1b
		D4 at 51.79%	0.74	OK	Eq. H1-1b
		D5 at 51.79%	0.63	OK	Eq. H1-1b
		D6 at 51.79%	0.80	OK	Eq. H1-1b

D7 at 51.79%	0.58	OK	Eq. H1-1b
D8 at 51.79%	0.75	OK	Eq. H1-1b
D9 at 51.79%	0.64	OK	Eq. H1-1b

Current Date: 2/10/2017 10:15 AM
Units system: English
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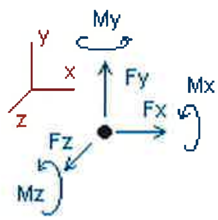
Analysis result

Reactions



Direction of positive forces and moments

	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **D1=DL**						
1	0.02821	17.16020	0.08582	0.00000	0.00000	0.00000
2	0.81848	11.88500	-0.51054	0.00000	0.00000	0.00000
3	0.29331	12.21483	0.55472	0.00000	0.00000	0.00000
SUM	1.14000	41.26003	0.13000	0.00000	0.00000	0.00000
Condition **D2=DL+LL**						
1	0.02821	22.90982	0.08582	0.00000	0.00000	0.00000
2	0.81848	17.29619	-0.51054	0.00000	0.00000	0.00000
3	0.29331	15.62013	0.55472	0.00000	0.00000	0.00000
SUM	1.14000	55.82615	0.13000	0.00000	0.00000	0.00000
Condition **D3=DL+S**						
1	0.02821	19.69940	0.08582	0.00000	0.00000	0.00000
2	0.81848	13.27441	-0.51054	0.00000	0.00000	0.00000
3	0.29331	13.65690	0.55472	0.00000	0.00000	0.00000
SUM	1.14000	46.63071	0.13000	0.00000	0.00000	0.00000
Condition **D4=DL+0.75LL**						
1	0.02821	21.47242	0.08582	0.00000	0.00000	0.00000
2	0.81848	15.94339	-0.51054	0.00000	0.00000	0.00000
3	0.29331	14.76881	0.55472	0.00000	0.00000	0.00000
SUM	1.14000	52.18462	0.13000	0.00000	0.00000	0.00000
Condition **D5=DL+0.75S**						
1	0.02821	19.06460	0.08582	0.00000	0.00000	0.00000
2	0.81848	12.92706	-0.51054	0.00000	0.00000	0.00000
3	0.29331	13.29638	0.55472	0.00000	0.00000	0.00000
SUM	1.14000	45.28804	0.13000	0.00000	0.00000	0.00000

Condition **D6=DL+0.75LL+0.75S**

1	0.02821	23.37681	0.08582	0.00000	0.00000	0.00000
2	0.81848	16.98545	-0.51054	0.00000	0.00000	0.00000
3	0.29331	15.85036	0.55472	0.00000	0.00000	0.00000
SUM	1.14000	56.21263	0.13000	0.00000	0.00000	0.00000

Condition **D7=DL+W**

1	-1.92224	17.24299	-3.31476	0.00000	0.00000	0.00000
2	2.88414	11.88500	-2.13292	0.00000	0.00000	0.00000
3	0.65810	13.14204	0.78456	0.00000	0.00000	0.00000
SUM	1.62000	42.27003	-4.66312	0.00000	0.00000	0.00000

Condition **D8=DL+0.75W+0.75LL**

1	-1.43463	21.53450	-2.46462	0.00000	0.00000	0.00000
2	2.36773	15.94339	-1.72732	0.00000	0.00000	0.00000
3	0.56690	15.46422	0.72710	0.00000	0.00000	0.00000
SUM	1.50000	52.94212	-3.46484	0.00000	0.00000	0.00000

Condition **D9=DL+0.75W+0.75S**

1	-1.43463	19.12669	-2.46462	0.00000	0.00000	0.00000
2	2.36773	12.92706	-1.72732	0.00000	0.00000	0.00000
3	0.56690	13.99179	0.72710	0.00000	0.00000	0.00000
SUM	1.50000	46.04554	-3.46484	0.00000	0.00000	0.00000

Condition **D10=DL+0.75W+0.75LL+0.75S**

1	-1.43463	23.43890	-2.46462	0.00000	0.00000	0.00000
2	2.36773	16.98545	-1.72732	0.00000	0.00000	0.00000
3	0.56690	16.54577	0.72710	0.00000	0.00000	0.00000
SUM	1.50000	56.97013	-3.46484	0.00000	0.00000	0.00000

Condition **D11=0.6DL+W**

1	-1.93352	10.37891	-3.34909	0.00000	0.00000	0.00000
2	2.55675	7.13100	-1.92870	0.00000	0.00000	0.00000
3	0.54078	8.25611	0.56267	0.00000	0.00000	0.00000
SUM	1.16400	25.76602	-4.71512	0.00000	0.00000	0.00000

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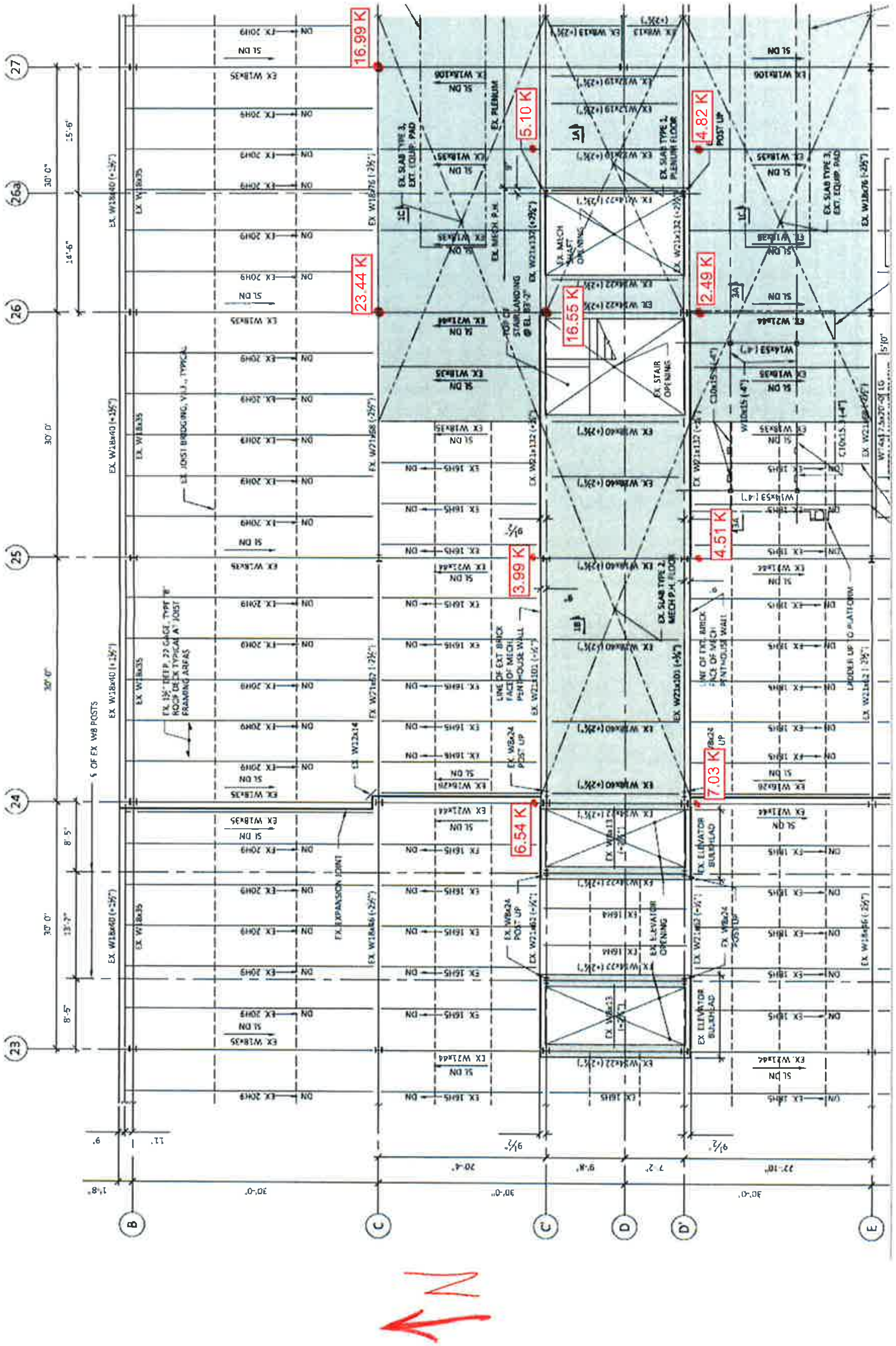
Roof Beam Calculations



PARTIAL ROOF FRAMING PLAN 1 SK

Date: 02-10-2017

Project Name: Stamford Harbor

Project Number: S2900

Designed By: GH Checked By: MSC

Hudson Design Group LLC

Roof Loading Calculations

Live Loads:

Snow Load	30 psf	
Penthouse Floor	150 psf	
Drift Load	100 plf	(Max Drift Intensity = 30 psf)

Dead Loads:

Roof (Type 0)	5 psf
Roof (Type 1)	46 psf
Roof (Type 2)	85 psf
Roof (Type 3)	95 psf
Penthouse Wall	50 psf
Penthouse Roof	10 psf

Load Breakdown:

- **Beam 24-C-C' & 24-D'-E**

  Live Load

  → Snow 30 psf x 2.5 ft. (Tributary Width)
  = **75.00 plf**

  → Drift 30 psf x 2.5 ft. (Tributary Width)
  = **75.00 plf**

  Dead Load

  → Roof (0) 5 psf x 2.5 ft. (Tributary Width)
  = **12.50 plf**

- **Beam 25-C-C' & 25-D'-E**

  Live Load

  → Snow 30 psf x 5 ft. (Tributary Width)
  = **150.00 plf**

  → Drift 30 psf x 5 ft. (Tributary Width)
  = **150.00 plf**

  Dead Load

  → Roof (0) 5 psf x 5 ft. (Tributary Width)
  = **25.00 plf**

Date: 02-10-2017

Project Name: Stamford Harbor

Project Number: S2900

Designed By: GH Checked By: MSC

Load Breakdown (Cont.):

- **Beam 26-D'-E**

Live Load

→ Snow 30 psf x 8.5 ft. (Tributary Width)

= **255.00 plf**

→ Drift 30 psf x 8.5 ft. (Tributary Width)

= **255.00 plf**

Dead Load

→ Roof (3) 95 psf x 8.5 ft. (Tributary Width)

= **807.50 plf**

- **Beam 26a-C-C' & 26a-D'-E**

Live Load

→ Snow 30 psf x 10 ft. (Tributary Width)

= **300.00 plf**

→ Drift 30 psf x 10 ft. (Tributary Width)

= **300.00 plf**

Dead Load

→ Roof (3) 95 psf x 10 ft. (Tributary Width)

= **950.00 plf**

- **Beam C'-26-27 & D'-26-27**

Live Load

→ Snow 30 psf x 18.6 ft. (Tributary Width)

= **558.00 plf**

→ Drift **100 plf**

→ Penthouse 150 psf x 8.42 ft. (Half Penthouse Floor Width)

= **1263.00 plf**

Dead Load

→ Roof (1) 46 psf x 8.42 ft. (Half Penthouse Floor Width)

= **387.32 plf**

→ Roof (3) 95 psf x 10.67 ft. (Width of slab acting on beam)

= **1013.65 plf**

→ Pent. Wall 50 psf x 11.5 ft. (Penthouse Height)

= **575.00 plf**

→ Pent. Roof 10 psf x 8.42 ft. (Half Penthouse Roof Width)

= 84.20 plf + W10x26 = **100 plf**

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Load Breakdown (Cont.):

- **Beam 27-C-D (0 ft - 10.17 ft)**

Live Load

→ Snow 30 psf x 10 ft. (Tributary Width)
= **300.00 plf**

Dead Load

→ Roof (3) 95 psf x 10 ft. (Tributary Width)
= **950.00 plf**

- **Beam 27-C-D (10.17 ft - 20.33 ft)**

Live Load

→ Snow 30 psf x 5 ft. (Tributary Width)
= **150.00 plf**

→ Drift 30 psf x 5 ft. (Tributary Width)
= **150.00 plf**

Dead Load

→ Roof (3) 95 psf x 5 ft. (Tributary Width)
= **475.00 plf**

- **Beam 27-C-D (20.33 ft - 30.0 ft)**

Live Load

→ Snow 30 psf x 5 ft. (Tributary Width)
= **150.00 plf**

→ Penthouse 150 psf x 5 ft. (Tributary Width)
= **750.00 plf**

Dead Load

→ Roof (1) 46 psf x 5 ft. (Tributary Width)
= **230.00 plf**

→ Pent. Roof 10 psf x 5 ft. (Tributary Width)
= **50.00 plf**

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Load Breakdown (Cont.):

- **Beam 27-D-E (0 ft - 7.17 ft)**

Live Load

→ Snow	30 psf	x	5	ft.	(Tributary Width)
=	**150.00 plf**				
→ Penthouse	150 psf	x	5	ft.	(Tributary Width)
=	**750.00 plf**				

Dead Load

→ Roof (1)	46 psf	x	5	ft.	(Tributary Width)
=	**230.00 plf**				
→ Pent. Roof	10 psf	x	5	ft.	(Tributary Width)
=	**50.00 plf**				

- **Beam 27-D-E (7.17 ft - 18.58 ft)**

Live Load

→ Snow	30 psf	x	5	ft.	(Tributary Width)
=	**150.00 plf**				
→ Drift	30 psf	x	5	ft.	(Tributary Width)
=	**150.00 plf**				

Dead Load

→ Roof (3)	95 psf	x	5	ft.	(Tributary Width)
=	**475.00 plf**				

- **Beam 27-D-E (18.58 ft - 30.0 ft)**

Live Load

→ Snow	30 psf	x	10	ft.	(Tributary Width)
=	**300.00 plf**				

Dead Load

→ Roof (3)	95 psf	x	10	ft.	(Tributary Width)
=	**950.00 plf**				

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam 24-C-C'
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)]
A992-50 W16x26 x 20.33 FT
Section Adequate By: 751.3%
Controlling Factor: Moment

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

StruCalc Version 8.0.113.0 — 2/10/2017 10:41:02 AM

DEFLECTIONS	Center	
Live Load	0.04	IN L/6825
Dead Load	0.07	in
Total Load	0.11	IN L/2249

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	790	lb	985	lb
Dead Load	925	lb	6397	lb
Total Load	1715	lb	7383	lb
Bearing Length	0.75	in	0.75	in

BEAM DATA	Center	
Span Length	20.33	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	20.33	ft

STEEL PROPERTIES

W16x26 - A992-50

Properties:

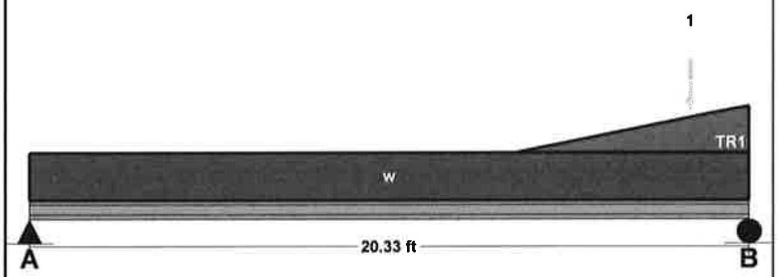
Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	15.7	in
Web Thickness:	tw =	0.25	in
Flange Width:	bf =	5.5	in
Flange Thickness:	tf =	0.35	in
Distance to Web Toe of Fillet:	k =	0.75	in
Moment of Inertia About X-X Axis:	Ix =	301	in4
Section Modulus About X-X Axis:	Sx =	38.4	in3
Plastic Section Modulus About X-X Axis:	Zx =	44.2	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	7.97	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	56.82	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	3.96	ft
Nominal Flexural Strength w/ safety factor:	Mn =	110279	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	56.82	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-3		
Nominal Shear Strength w/ safety factor:	Vn =	70509	lb

Controlling Moment: 12954 ft-lb
15.04 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Controlling Shear: -7383 lb
20.0 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	32.11 in4	301 in4
Moment:	12954 ft-lb	110279 ft-lb
Shear:	-7383 lb	70509 lb

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	75	plf
Uniform Dead Load	13	plf
Beam Self Weight	26	plf
Total Uniform Load	114	plf

POINT LOADS - CENTER SPAN	
Load Number	One
Live Load	0 lb
Dead Load	6540 lb
Location	18.67 ft

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	0 plf
Left Dead Load	0 plf
Right Live Load	75 plf
Right Dead Load	0 plf
Load Start	13.65 ft
Load End	20.33 ft
Load Length	6.68 ft

NOTES

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam 24-D'-E
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W16x26 x 22.83 FT
Section Adequate By: 617.8%
Controlling Factor: Deflection

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

StruCalc Version 8.0.113.0 2/10/2017 10:42:05 AM

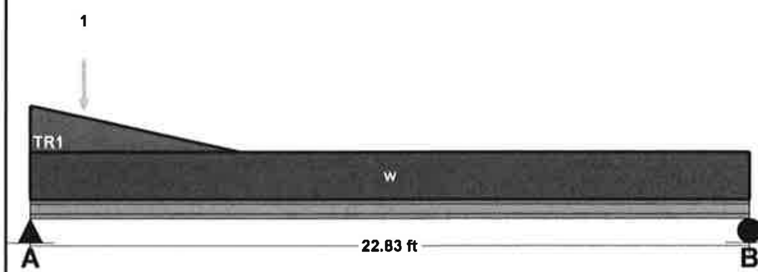
DEFLECTIONS	Center	
Live Load	0.06	IN L/4894
Dead Load	0.10	in
Total Load	0.16	IN L/1723

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	1082	lb	881	lb
Dead Load	6955	lb	954	lb
Total Load	8037	lb	1834	lb
Bearing Length	0.75	in	0.75	in

BEAM DATA	Center	
Span Length	22.83	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	22.83	ft

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	75	plf
Uniform Dead Load	13	plf
Beam Self Weight	26	plf
Total Uniform Load	114	plf

POINT LOADS - CENTER SPAN	
Load Number	One
Live Load	0 lb
Dead Load	7030 lb
Location	1.67 ft

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	75 plf
Left Dead Load	0 plf
Right Live Load	0 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	6.68 ft
Load Length	6.68 ft

STEEL PROPERTIES

W16x26 - A572-50

Properties:

Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	15.7	in
Web Thickness:	tw =	0.25	in
Flange Width:	bf =	5.5	in
Flange Thickness:	tf =	0.35	in
Distance to Web Toe of Fillet:	k =	0.75	in
Moment of Inertia About X-X Axis:	Ix =	301	in4
Section Modulus About X-X Axis:	Sx =	38.4	in3
Plastic Section Modulus About X-X Axis:	Zx =	44.2	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	7.97	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	56.82	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	3.96	ft
Nominal Flexural Strength w/ safety factor:	Mn =	110279	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	56.82	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-3		
Nominal Shear Strength w/ safety factor:	Vn =	70509	lb

Controlling Moment: 14822 ft-lb
6.62 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Controlling Shear: 8037 lb
At left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	41.93 in4	301 in4
Moment:	14822 ft-lb	110279 ft-lb
Shear:	8037 lb	70509 lb

NOTES

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam 25-C-C'
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W21x44 x 20.33 FT
Section Adequate By: 1434.3%
Controlling Factor: Moment

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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DEFLECTIONS

	Center	
Live Load	0.03	IN L/9557
Dead Load	0.02	in
Total Load	0.05	IN L/5035

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

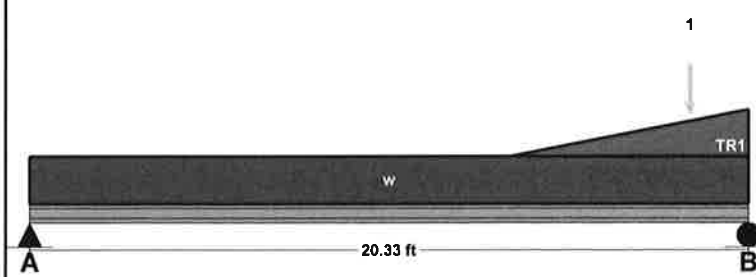
REACTIONS

	A		B	
Live Load	1580	lb	1971	lb
Dead Load	1027	lb	4366	lb
Total Load	2607	lb	6336	lb
Bearing Length	0.95	in	0.95	in

BEAM DATA

	Center	
Span Length	20.33	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	20.33	ft

LOADING DIAGRAM



UNIFORM LOADS

	Center	
Uniform Live Load	150	plf
Uniform Dead Load	25	plf
Beam Self Weight	44	plf
Total Uniform Load	219	plf

POINT LOADS - CENTER SPAN

Load Number	One
Live Load	0 lb
Dead Load	3990 lb
Location	18.67 ft

TRAPEZOIDAL LOADS - CENTER SPAN

Load Number	One
Left Live Load	0 plf
Left Dead Load	0 plf
Right Live Load	150 plf
Right Dead Load	0 plf
Load Start	13.65 ft
Load End	20.33 ft
Load Length	6.68 ft

STEEL PROPERTIES

W21x44 - A572-50

Properties:

Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	20.7	in
Web Thickness:	tw =	0.35	in
Flange Width:	bf =	6.5	in
Flange Thickness:	tf =	0.45	in
Distance to Web Toe of Fillet:	k =	0.95	in
Moment of Inertia About X-X Axis:	Ix =	843	in4
Section Modulus About X-X Axis:	Sx =	81.6	in3
Plastic Section Modulus About X-X Axis:	Zx =	95.4	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	7.22	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	53.71	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	4.45	ft
Nominal Flexural Strength w/ safety factor:	Mn =	238024	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	53.71	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	144900	lb

Controlling Moment: 15514 ft-lb
11.99 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Controlling Shear: -6336 lb
20.0 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	40.18 in4	843 in4
Moment:	15514 ft-lb	238024 ft-lb
Shear:	-6336 lb	144900 lb

NOTES

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam 25-D'-E
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W21x44 x 22.83 FT
Section Adequate By: 1158.2%
Controlling Factor: Moment

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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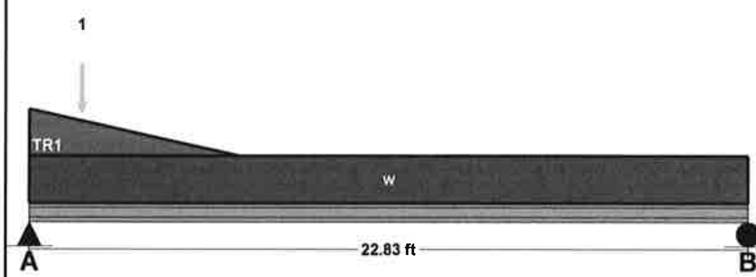
DEFLECTIONS	Center	
Live Load	0.04	IN L/6853
Dead Load	0.03	in
Total Load	0.07	IN L/3674

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	2164	lb	1761	lb
Dead Load	4968	lb	1118	lb
Total Load	7132	lb	2879	lb
Bearing Length	0.95	in	0.95	in

BEAM DATA	Center	
Span Length	22.83	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	22.83	ft

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	150	plf
Uniform Dead Load	25	plf
Beam Self Weight	44	plf
Total Uniform Load	219	plf

POINT LOADS - CENTER SPAN	
Load Number	One
Live Load	0 lb
Dead Load	4510 lb
Location	1.67 ft

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	150 plf
Left Dead Load	0 plf
Right Live Load	0 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	6.68 ft
Load Length	6.68 ft

STEEL PROPERTIES

W21x44 - A572-50

Properties:

Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	20.7	in
Web Thickness:	tw =	0.35	in
Flange Width:	bf =	6.5	in
Flange Thickness:	tf =	0.45	in
Distance to Web Toe of Fillet:	k =	0.95	in
Moment of Inertia About X-X Axis:	Ix =	843	in4
Section Modulus About X-X Axis:	Sx =	81.6	in3
Plastic Section Modulus About X-X Axis:	Zx =	95.4	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	7.22	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	53.71	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	4.45	ft
Nominal Flexural Strength w/ safety factor:	Mn =	238024	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	53.71	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	144900	lb

Controlling Moment: 18918 ft-lb
9.59 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Controlling Shear: 7132 lb
At left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	55.06 in4	843 in4
Moment:	18918 ft-lb	238024 ft-lb
Shear:	7132 lb	144900 lb

NOTES

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam 26-D'-E
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W21x44 x 22.83 FT
Section Adequate By: 216.6%
Controlling Factor: Moment

DEFLECTIONS	Center	
Live Load	0.07	IN L/4031
Dead Load	0.22	in
Total Load	0.29	IN L/943

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	3679	lb	2994	lb
Dead Load	12033	lb	9908	lb
Total Load	15713	lb	12902	lb
Bearing Length	0.95	in	0.95	in

BEAM DATA	Center	
Span Length	22.83	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	22.83	ft

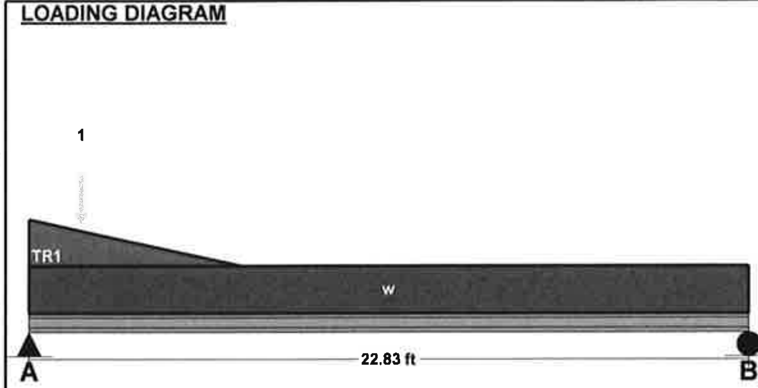
STEEL PROPERTIES
W21x44 - A572-50

Properties:			
Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	20.7	in
Web Thickness:	tw =	0.35	in
Flange Width:	bf =	6.5	in
Flange Thickness:	tf =	0.45	in
Distance to Web Toe of Fillet:	k =	0.95	in
Moment of Inertia About X-X Axis:	Ix =	843	in4
Section Modulus About X-X Axis:	Sx =	81.6	in3
Plastic Section Modulus About X-X Axis:	Zx =	95.4	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	7.22	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	53.71	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	4.45	ft
Nominal Flexural Strength w/ safety factor:	Mn =	238024	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	53.71	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	144900	lb

Controlling Moment: 75181 ft-lb
11.19 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: 15713 lb
At left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	214.5 in4	843 in4
Moment:	75181 ft-lb	238024 ft-lb
Shear:	15713 lb	144900 lb

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	255	plf
Uniform Dead Load	808	plf
Beam Self Weight	44	plf
Total Uniform Load	1107	plf

POINT LOADS - CENTER SPAN	
Load Number	One
Live Load	0 lb
Dead Load	2490 lb
Location	1.67 ft

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	255 plf
Left Dead Load	0 plf
Right Live Load	0 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	6.68 ft
Load Length	6.68 ft

NOTES

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam 26a-C-C'
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W18x35 x 20.33 FT
Section Adequate By: 130.9%
Controlling Factor: Moment

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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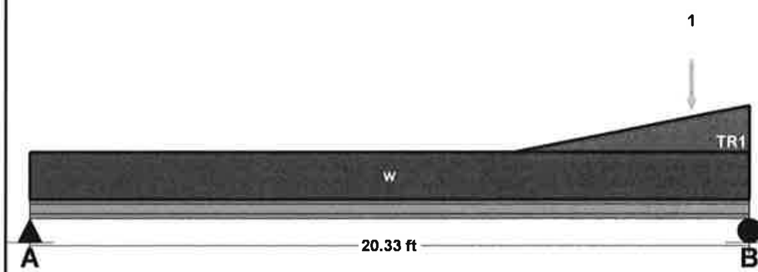
DEFLECTIONS	Center	
Live Load	0.08	IN L/2891
Dead Load	0.28	in
Total Load	0.37	IN L/667

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	3159	lb	3942	lb
Dead Load	10429	lb	14696	lb
Total Load	13588	lb	18638	lb
Bearing Length	0.83	in	0.83	in

BEAM DATA	Center	
Span Length	20.33	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	20.33	ft

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	300	plf
Uniform Dead Load	950	plf
Beam Self Weight	35	plf
Total Uniform Load	1285	plf

POINT LOADS - CENTER SPAN	
Load Number	One
Live Load	0 lb
Dead Load	5100 lb
Location	18.67 ft

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	0 plf
Left Dead Load	0 plf
Right Live Load	300 plf
Right Dead Load	0 plf
Load Start	13.65 ft
Load End	20.33 ft
Load Length	6.68 ft

STEEL PROPERTIES

W18x35 - A572-50

Properties:			
Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	17.7	in
Web Thickness:	tw =	0.3	in
Flange Width:	bf =	6	in
Flange Thickness:	tf =	0.43	in
Distance to Web Toe of Fillet:	k =	0.83	in
Moment of Inertia About X-X Axis:	Ix =	510	in4
Section Modulus About X-X Axis:	Sx =	57.6	in3
Plastic Section Modulus About X-X Axis:	Zx =	66.5	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	7.06	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	53.49	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	4.31	ft
Nominal Flexural Strength w/ safety factor:	Mn =	165918	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	53.49	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	106200	lb

Controlling Moment: 71844 ft-lb
10.57 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: -18638 lb
20.0 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	183.47 in4	510 in4
Moment:	71844 ft-lb	165918 ft-lb
Shear:	-18638 lb	106200 lb

NOTES

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam 26a-D'-E
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W18x35 x 22.83 FT
Section Adequate By: 86.6%
Controlling Factor: Moment

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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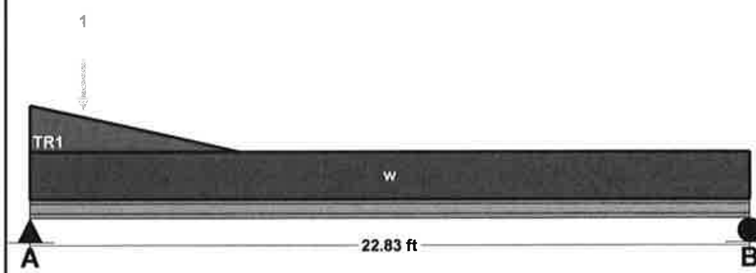
DEFLECTIONS	Center	
Live Load	0.13	IN L/2073
Dead Load	0.44	in
Total Load	0.57	IN L/481

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	4329	lb	3522	lb
Dead Load	15711	lb	11596	lb
Total Load	20040	lb	15119	lb
Bearing Length	0.83	in	0.83	in

BEAM DATA	Center	
Span Length	22.83	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	22.83	ft

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	300	plf
Uniform Dead Load	950	plf
Beam Self Weight	35	plf
Total Uniform Load	1285	plf

POINT LOADS - CENTER SPAN	
Load Number	One
Live Load	0 lb
Dead Load	4820 lb
Location	1.67 ft

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	300 plf
Left Dead Load	0 plf
Right Live Load	0 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	6.68 ft
Load Length	6.68 ft

STEEL PROPERTIES

W18x35 - A572-50

Properties:			
Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	17.7	in
Web Thickness:	tw =	0.3	in
Flange Width:	bf =	6	in
Flange Thickness:	tf =	0.43	in
Distance to Web Toe of Fillet:	k =	0.83	in
Moment of Inertia About X-X Axis:	Ix =	510	in4
Section Modulus About X-X Axis:	Sx =	57.6	in3
Plastic Section Modulus About X-X Axis:	Zx =	66.5	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	7.06	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	53.49	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	4.31	ft
Nominal Flexural Strength w/ safety factor:	Mn =	165918	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	53.49	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	106200	lb

Controlling Moment: 88931 ft-lb
10.96 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: 20040 lb
At left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	254.48 in4	510 in4
Moment:	88931 ft-lb	165918 ft-lb
Shear:	20040 lb	106200 lb

NOTES

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam C'-26-27
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W21x132 x 30.0 FT
Section Adequate By: 98.2%
Controlling Factor: Moment

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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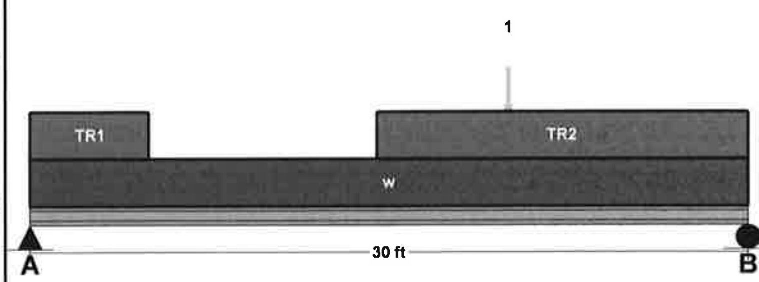
DEFLECTIONS	Center	
Live Load	0.27	IN L/1311
Dead Load	0.45	in
Total Load	0.72	IN L/500

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	20716	lb	24915	lb
Dead Load	32347	lb	35337	lb
Total Load	53063	lb	60253	lb
Bearing Length	1.54	in	1.54	in

BEAM DATA	Center	
Span Length	30	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	30	ft

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	658	plf
Uniform Dead Load	1689	plf
Beam Self Weight	132	plf
Total Uniform Load	2479	plf

POINT LOADS - CENTER SPAN	
Load Number	One
Live Load	0 lb
Dead Load	5100 lb
Location	20 ft

TRAPEZOIDAL LOADS - CENTER SPAN		
Load Number	One	Two
Left Live Load	1263 plf	1263 plf
Left Dead Load	388 plf	388 plf
Right Live Load	1263 plf	1263 plf
Right Dead Load	388 plf	388 plf
Load Start	0 ft	14.5 ft
Load End	5 ft	30 ft
Load Length	5 ft	15.5 ft

STEEL PROPERTIES
W21x132 - A572-50

Properties:			
Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	21.8	in
Web Thickness:	tw =	0.65	in
Flange Width:	bf =	12.4	in
Flange Thickness:	tf =	1.04	in
Distance to Web Toe of Fillet:	k =	1.54	in
Moment of Inertia About X-X Axis:	Ix =	3220	in4
Section Modulus About X-X Axis:	Sx =	295	in3
Plastic Section Modulus About X-X Axis:	Zx =	333	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	5.96	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	28.8	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	10.35	ft
Nominal Flexural Strength w/ safety factor:	Mn =	830838	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	28.8	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	283400	lb

Controlling Moment: 419180 ft-lb
16.5 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: -60253 lb
At right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	1545.62 in4	3220 in4
Moment:	419180 ft-lb	830838 ft-lb
Shear:	-60253 lb	283400 lb

NOTES

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam D'-26-27
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W21x132 x 30.0 FT
Section Adequate By: 98.9%
Controlling Factor: Moment

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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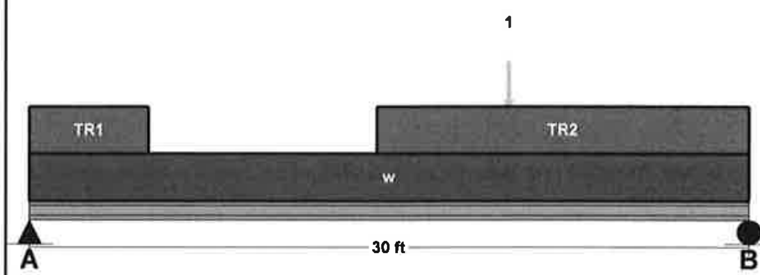
DEFLECTIONS	Center	
Live Load	0.27	IN L/1311
Dead Load	0.44	in
Total Load	0.72	IN L/502

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	20716	lb	24915	lb
Dead Load	32254	lb	35150	lb
Total Load	52970	lb	60066	lb
Bearing Length	1.54	in	1.54	in

BEAM DATA	Center	
Span Length	30	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	30	ft

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	658	plf
Uniform Dead Load	1689	plf
Beam Self Weight	132	plf
Total Uniform Load	2479	plf

POINT LOADS - CENTER SPAN

Load Number	One
Live Load	0 lb
Dead Load	4820 lb
Location	20 ft

TRAPEZOIDAL LOADS - CENTER SPAN

Load Number	One	Two
Left Live Load	1263 plf	1263 plf
Left Dead Load	388 plf	388 plf
Right Live Load	1263 plf	1263 plf
Right Dead Load	388 plf	388 plf
Load Start	0 ft	14.5 ft
Load End	5 ft	30 ft
Load Length	5 ft	15.5 ft

STEEL PROPERTIES
W21x132 - A572-50

Properties:

Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	21.8	in
Web Thickness:	tw =	0.65	in
Flange Width:	bf =	12.4	in
Flange Thickness:	tf =	1.04	in
Distance to Web Toe of Fillet:	k =	1.54	in
Moment of Inertia About X-X Axis:	Ix =	3220	in4
Section Modulus About X-X Axis:	Sx =	295	in3
Plastic Section Modulus About X-X Axis:	Zx =	333	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	5.96	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	28.8	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	10.35	ft
Nominal Flexural Strength w/ safety factor:	Mn =	830838	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	28.8	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	283400	lb

Controlling Moment: 417640 ft-lb
16.5 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: -60066 lb
At right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	1540.27 in4	3220 in4
Moment:	417640 ft-lb	830838 ft-lb
Shear:	-60066 lb	283400 lb

NOTES

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam 27-C-D
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W18x106 x 30.0 FT
Section Adequate By: 13.1%
Controlling Factor: Moment

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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DEFLECTIONS	Center	
Live Load	0.50	IN L/727
Dead Load	0.77	in
Total Load	1.26	IN L/286

Live Load Deflection Criteria: L/360   Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	12915	lb	25779	lb
Dead Load	24458	lb	33254	lb
Total Load	37373	lb	59033	lb
Bearing Length	1.34	in	1.34	in

BEAM DATA	Center	
Span Length	30	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	30	ft

STEEL PROPERTIES

W18x106 - A572-50

Properties:

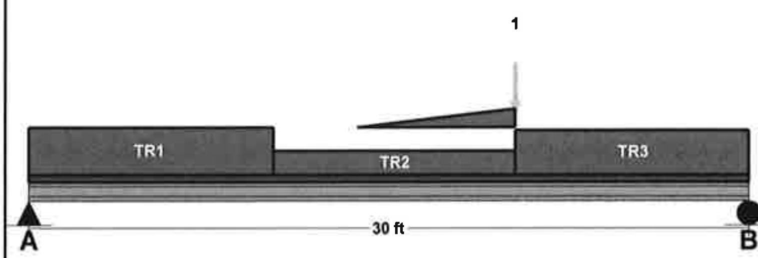
Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	18.7	in
Web Thickness:	tw =	0.59	in
Flange Width:	bf =	11.2	in
Flange Thickness:	tf =	0.94	in
Distance to Web Toe of Fillet:	k =	1.34	in
Moment of Inertia About X-X Axis:	Ix =	1910	in4
Section Modulus About X-X Axis:	Sx =	204	in3
Plastic Section Modulus About X-X Axis:	Zx =	230	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	5.96	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	27.15	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	9.4	ft
Nominal Flexural Strength w/ safety factor:	Mn =	573852	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	27.15	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	220660	lb

Controlling Moment: 507452 ft-lb
20.4 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Controlling Shear: -59033 lb
At right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	1604.63 in4	1910 in4
Moment:	507452 ft-lb	573852 ft-lb
Shear:	-59033 lb	220660 lb

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	0	plf
Uniform Dead Load	0	plf
Beam Self Weight	106	plf
Total Uniform Load	106	plf

POINT LOADS - CENTER SPAN		
Load Number	One	Two
Live Load	24915 lb	0 lb
Dead Load	35337 lb	2000 lb
Location	20.33 ft	20.33 ft

TRAPEZOIDAL LOADS - CENTER SPAN				
Load Number	One	Two	Three	Four
Left Live Load	300 plf	150 plf	900 plf	0 plf
Left Dead Load	950 plf	475 plf	280 plf	0 plf
Right Live Load	300 plf	150 plf	900 plf	150 plf
Right Dead Load	950 plf	475 plf	280 plf	0 plf
Load Start	0 ft	10.17 ft	20.33 ft	13.65 ft
Load End	10.17 ft	20.33 ft	30 ft	20.33 ft
Load Length	10.17 ft	10.16 ft	9.67 ft	6.68 ft

NOTES

Point Load 1: Reaction from Beam C'-26-27

Point Load 2: Penthouse Wall (8 ft tall section) (8 ft * 50 psf * 5 ft = 2000 lbs)

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Beam 27-D-E
Multi-Loaded Multi-Span Beam
[2003 International Building Code(AISC 13th Ed ASD)
A572-50 W18x106 x 30.0 FT
Section Adequate By: 36.2%
Controlling Factor: Moment

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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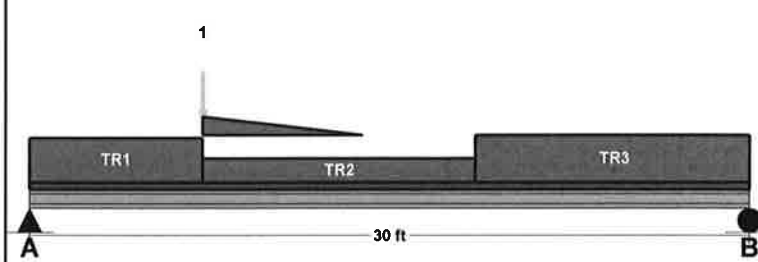
DEFLECTIONS	Center	
Live Load	0.40	IN L/904
Dead Load	0.67	in
Total Load	1.07	IN L/338

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	26615	lb	10391	lb
Dead Load	36788	lb	21819	lb
Total Load	63403	lb	32210	lb
Bearing Length	1.34	in	1.34	in

BEAM DATA	Center	
Span Length	30	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	30	ft

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	0	plf
Uniform Dead Load	0	plf
Beam Self Weight	106	plf
Total Uniform Load	106	plf

POINT LOADS - CENTER SPAN		
Load Number	One	Two
Live Load	24915 lb	0 lb
Dead Load	35150 lb	2000 lb
Location	7.17 ft	7.17 ft

TRAPEZOIDAL LOADS - CENTER SPAN				
Load Number	One	Two	Three	Four
Left Live Load	900 plf	150 plf	300 plf	150 plf
Left Dead Load	280 plf	475 plf	950 plf	0 plf
Right Live Load	900 plf	150 plf	300 plf	0 plf
Right Dead Load	280 plf	475 plf	950 plf	0 plf
Load Start	0 ft	7.17 ft	18.58 ft	7.17 ft
Load End	7.17 ft	18.58 ft	30 ft	13.85 ft
Load Length	7.17 ft	11.41 ft	11.42 ft	6.68 ft

STEEL PROPERTIES
W18x106 - A572-50

Properties:

Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	18.7	in
Web Thickness:	tw =	0.59	in
Flange Width:	bf =	11.2	in
Flange Thickness:	tf =	0.94	in
Distance to Web Toe of Fillet:	k =	1.34	in
Moment of Inertia About X-X Axis:	Ix =	1910	in4
Section Modulus About X-X Axis:	Sx =	204	in3
Plastic Section Modulus About X-X Axis:	Zx =	230	in3
Design Properties per AISC 13th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	5.96	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	27.15	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	9.4	ft
Nominal Flexural Strength w/ safety factor:	Mn =	573852	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	27.15	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	220660	lb

Controlling Moment: 421305 ft-lb
7.2 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: 63403 lb
At left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	1356.58 in4	1910 in4
Moment:	421305 ft-lb	573852 ft-lb
Shear:	63403 lb	220660 lb

NOTES

Point Load 1: Reaction from Beam D'-26-27

Point Load 2: Penthouse Wall (8 ft tall section)(8 ft * 50 psf * 5 ft = 2000 lbs)

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Column Calculations

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

COLUMN ANALYSIS

Column C'24 (W12x72):

Tributary Area = (30'/2 + 30'/2) * (20.33'/2 + 16.83'/2) = 557.4 ft^2

Live Loads:

Snow =	30 psf	x	557.4 ft^2	=	16722 lbs
Penthouse Floor =	150 psf	x	127 ft^2	=	19050 lbs
				Total =	**35772 lbs**

Dead Loads:

Roof (Type 0) =	5 psf	x	430.4 ft^2	=	2152 lbs
Roof (Type 2) =	85 psf	x	127 ft^2	=	10795 lbs
Penthouse Roof =	10 psf	x	127 ft^2	=	1270 lbs
Penthouse Walls =	575 plf	x	23.4 ft	=	13455 lbs
Misc. =	5 psf	x	557.4 ft^2	=	2787 lbs
Reaction From Proposed Antenna Frame =					6540 lbs
				Total =	**36999 lbs**

Column D'24 (W12x72):

Tributary Area = (30'/2 + 30'/2) * (22.83'/2 + 16.83'/2) = 594.9 ft^2

Live Loads:

Snow =	30 psf	x	594.9 ft^2	=	17847 lbs
Penthouse Floor =	150 psf	x	127 ft^2	=	19050 lbs
				Total =	**36897 lbs**

Dead Loads:

Roof (Type 0) =	5 psf	x	467.9 ft^2	=	2339.5 lbs
Roof (Type 2) =	85 psf	x	127 ft^2	=	10795 lbs
Penthouse Roof =	10 psf	x	127 ft^2	=	1270 lbs
Penthouse Walls =	575 plf	x	23.4 ft	=	13455 lbs
Misc. =	5 psf	x	594.9 ft^2	=	2974.5 lbs
Reaction From Proposed Antenna Frame =					7030 lbs
				Total =	**37864 lbs**

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

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COLUMN ANALYSIS (Cont.)

Column C'25 (W12x65):

Tributary Area = (30'/2 + 30'/2) * (20.33'/2 + 16.83'/2) = 557.4 ft^2

Live Loads:

Snow =	30 psf	x	557.4 ft^2	=	16722 lbs
Penthouse Floor =	150 psf	x	252.5 ft^2	=	37875 lbs
				Total =	**54597 lbs**

Dead Loads:

Roof (Type 0) =	5 psf	x	304.9 ft^2	=	1524.5 lbs
Roof (Type 2) =	85 psf	x	252.5 ft^2	=	21462.5 lbs
Penthouse Roof =	10 psf	x	252.5 ft^2	=	2525 lbs
Penthouse Walls =	575 plf	x	30 ft	=	17250 lbs
Misc. =	5 psf	x	557.4 ft^2	=	2787 lbs
Reaction From Proposed Antenna Frame =					3990 lbs
				Total =	**49539 lbs**

Column D'25 (W12x65):

Tributary Area = (30'/2 + 30'/2) * (22.83'/2 + 16.83'/2) = 594.9 ft^2

Live Loads:

Snow =	30 psf	x	594.9 ft^2	=	17847 lbs
Penthouse Floor =	150 psf	x	252.5 ft^2	=	37875 lbs
				Total =	**55722 lbs**

Dead Loads:

Roof (Type 0) =	5 psf	x	342.4 ft^2	=	1712 lbs
Roof (Type 2) =	85 psf	x	252.5 ft^2	=	21462.5 lbs
Penthouse Roof =	10 psf	x	252.5 ft^2	=	2525 lbs
Penthouse Walls =	575 plf	x	30 ft	=	17250 lbs
Misc. =	5 psf	x	594.9 ft^2	=	2974.5 lbs
Reaction From Cooling Tower =					5050 lbs
Reaction From Proposed Antenna Frame =					4510 lbs
				Total =	**55484 lbs**

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

COLUMN ANALYSIS (Cont.)

Column C'26 (W10x60):

Tributary Area = (30'/2 + 30'/2) * (20.33'/2 + 16.83'/2) = 557.4 ft^2

Live Loads:

Snow =	30 psf	x	557.4 ft^2	=	16722 lbs
Penthouse Floor =	150 psf	x	252.5 ft^2	=	37875 lbs
				Total =	**54597 lbs**

Dead Loads:

Roof (Type 3) =	95 psf	x	304.9 ft^2	=	28965.5 lbs
Roof (Type 1) =	46 psf	x	252.5 ft^2	=	11615 lbs
Penthouse Roof =	10 psf	x	252.5 ft^2	=	2525 lbs
Penthouse Walls =	575 plf	x	30 ft	=	17250 lbs
Misc. =	5 psf	x	557.4 ft^2	=	2787 lbs
Reaction From Proposed Equipment Frame =					16550 lbs
				Total =	**79693 lbs**

Column D'26 (W10x60):

Tributary Area = (30'/2 + 30'/2) * (22.83'/2 + 16.83'/2) = 594.9 ft^2

Live Loads:

Snow =	30 psf	x	594.9 ft^2	=	17847 lbs
Penthouse Floor =	150 psf	x	252.5 ft^2	=	37875 lbs
				Total =	**55722 lbs**

Dead Loads:

Roof (Type 3) =	95 psf	x	342.4 ft^2	=	32528 lbs
Roof (Type 1) =	46 psf	x	252.5 ft^2	=	11615 lbs
Penthouse Roof =	10 psf	x	252.5 ft^2	=	2525 lbs
Penthouse Walls =	575 plf	x	30 ft	=	17250 lbs
Misc. =	5 psf	x	594.9 ft^2	=	2974.5 lbs
Reaction From Cooling Tower =					5050 lbs
Reaction From Proposed Antenna Frame =					2490 lbs
				Total =	**74433 lbs**

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

COLUMN ANALYSIS (Cont.)

Column C26 (W12x79):

Tributary Area = (30'/2 + 30'/2) * (20.33'/2 + 30'/2) = 755 ft^2

Live Loads:

Snow =	30 psf	x	755 ft^2	=	22650 lbs
				Total =	**22650 lbs**

Dead Loads:

Roof (Type 3) =	95 psf	x	305 ft^2	=	28975 lbs
Roof (Type 0) =	46 psf	x	450 ft^2	=	20700 lbs
Misc. =	5 psf	x	755 ft^2	=	3775 lbs
Reaction From Proposed Equipment Frame =					23440 lbs
				Total =	**76890 lbs**

Column C27 (W12x96):

Tributary Area = (30'/2 + 30'/2) * (20.33'/2 + 30'/2) = 900 ft^2

Live Loads:

Snow =	30 psf	x	900 ft^2	=	27000 lbs
				Total =	**27000 lbs**

Dead Loads:

Roof (Type 3) =	95 psf	x	450 ft^2	=	42750 lbs
Roof (Type 0) =	46 psf	x	450 ft^2	=	20700 lbs
Misc. =	5 psf	x	900 ft^2	=	4500 lbs
Reaction From Proposed Equipment Frame =					16990 lbs
				Total =	**84940 lbs**

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

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COLUMN ANALYSIS (Cont.)

Column D27 (W10x68):

Tributary Area = (30'/2 + 30'/2) * (20.33'/2 + 16.83'/2) = 900 ft^2

Live Loads:

Snow =	30 psf	x	900 ft^2	=	27000 lbs
Penthouse Floor =	150 psf	x	505 ft^2	=	75750 lbs
				Total =	**102750 lbs**

Dead Loads:

Roof (Type 1) =	46 psf	x	505 ft^2	=	23230 lbs
Roof (Type 3) =	95 psf	x	395 ft^2	=	37525 lbs
Penthouse Roof =	10 psf	x	505 ft^2	=	5050 lbs
Penthouse Walls =	575 plf	x	60 ft	=	34500 lbs
Misc. =	5 psf	x	900 ft^2	=	4500 lbs
Reaction From Proposed Antenna Frame = 5100 lb + 4820 lb =					9920 lbs
				Total =	**114725 lbs**

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Column C'24
Column
[2003 International Building Code(AISC 13th Ed ASD)]
A572-50 W12x72 x 12.1 FT
Section Adequate By: 77.3%

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	35772	lb
Dead Load:	Vert-DL-Rxn =	37870	lb
Total Load:	Vert-TL-Rxn =	73642	lb

COLUMN DATA

Total Column Length:	12.1	ft
Unbraced Length (X-Axis) Lx:	12.1	ft
Unbraced Length (Y-Axis) Ly:	12.1	ft
Column End Condtion-K (e):	2	
Column Bending Coefficient	1	

COLUMN PROPERTIES

W12x72 - W Shapes

Yield Stress:	Fy =	50 ksi		
Modulus of Elasticity:	E =	29 ksi		
Depth:	d =	12.3 in		
Web Thickness:	tw =	0.43 in		
Flange Width:	bf =	12 in		
Flange Thickness:	tf =	0.67 in		
Distance to Web Toe of Fillet:	k =	1.27 in		
Moment of Inertia (deflection):	Ix =	597 in4	Iy =	195 in4
Section Modulus:	Sx =	97.4 in3	Sy =	32.4 in3
Plastic Section Modulus:	Zx =	108 in3	Zy =	49.2 in3
Rad. of Gyration:	rx =	5.31 in	ry =	3.04 in

Column Compression Calculations:

KL/r Ratio: KLx/rx = 54.69 KLy/ry = 95.53

Controlling Direction for Compr. Calcs: (Y-Y Axis)

Flexural Buckling Stress: Fcr = 25.66 ksi

Controlling Equation F2-2

Nominal Compressive Strength: Pc = 324 kip

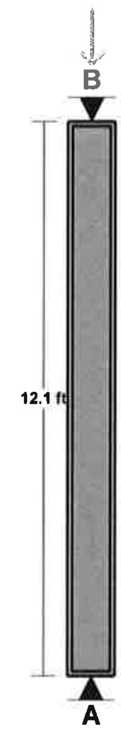
Combined Stress Calculations:

H1-1a Controls : 0.23

Controlling Combined Stress Factor: 0.23

NOTES

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	35772 lb
Dead Load:	PD =	36999 lb
Column Self Weight:	CSW =	871 lb
Total Load:	PT =	73642 lb

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Column D'24
Column
[2003 International Building Code(AISC 13th Ed ASD)]
A572-50 W12x72 x 12.1 FT
Section Adequate By: 76.7%

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	36897	lb
Dead Load:	Vert-DL-Rxn =	38735	lb
Total Load:	Vert-TL-Rxn =	75632	lb

COLUMN DATA

Total Column Length:	12.1	ft
Unbraced Length (X-Axis) Lx:	12.1	ft
Unbraced Length (Y-Axis) Ly:	12.1	ft
Column End Condtion-K (e):	2	
Column Bending Coefficient	1	

COLUMN PROPERTIES

W12x72 - W Shapes

Yield Stress:	Fy =	50	ksi			
Modulus of Elasticity:	E =	29	ksi			
Depth:	d =	12.3	in			
Web Thickness:	tw =	0.43	in			
Flange Width:	bf =	12	in			
Flange Thickness:	tf =	0.67	in			
Distance to Web Toe of Fillet:	k =	1.27	in			
Moment of Inertia (deflection):	Ix =	597	in4	Iy =	195	in4
Section Modulus:	Sx =	97.4	in3	Sy =	32.4	in3
Plastic Section Modulus:	Zx =	108	in3	Zy =	49.2	in3
Rad. of Gyration:	rx =	5.31	in	ry =	3.04	in

Column Compression Calculations:

KL/r Ratio: KLx/rx = 54.69 KLy/ry = 95.53

Controlling Direction for Compr. Calcs: (Y-Y Axis)

Flexural Buckling Stress: Fcr = 25.66 ksi

Controlling Equation F2-2

Nominal Compressive Strength: Pc = 324 kip

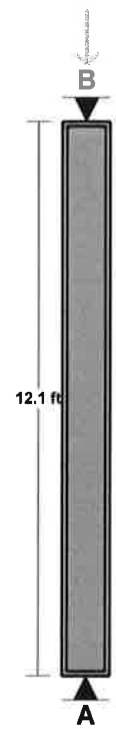
Combined Stress Calculations:

H1-1a Controls : 0.23

Controlling Combined Stress Factor: 0.23

NOTES

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	36897	lb
Dead Load:	PD =	37864	lb
Column Self Weight:	CSW =	871	lb
Total Load:	PT =	75632	lb

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Column C'25
Column
[2003 International Building Code(AISC 13th Ed ASD)]
A572-50 W12x65 x 12.1 FT
Section Adequate By: 63.9%

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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StruCalc Version 8.0.113.0 2/10/2017 11:08:53 AM

VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	54597	lb
Dead Load:	Vert-DL-Rxn =	50326	lb
Total Load:	Vert-TL-Rxn =	104923	lb

COLUMN DATA

Total Column Length:	12.1	ft
Unbraced Length (X-Axis) Lx:	12.1	ft
Unbraced Length (Y-Axis) Ly:	12.1	ft
Column End Condtion-K (e):	2	
Column Bending Coefficient	1	

COLUMN PROPERTIES

W12x65 - W Shapes

Yield Stress:	Fy =	50	ksi			
Modulus of Elasticity:	E =	29	ksi			
Depth:	d =	12.1	in			
Web Thickness:	tw =	0.39	in			
Flange Width:	bf =	12	in			
Flange Thickness:	tf =	0.61	in			
Distance to Web Toe of Fillet:	k =	1.2	in			
Moment of Inertia (deflection):	Ix =	533	in4	Iy =	174	in4
Section Modulus:	Sx =	87.9	in3	Sy =	29.1	in3
Plastic Section Modulus:	Zx =	96.8	in3	Zy =	44.1	in3
Rad. of Gyration:	rx =	5.28	in	ry =	3.02	in

Column Compression Calculations:

KL/r Ratio: KLx/rx = 55 KLy/ry = 96.16

Controlling Direction for Compr. Calcs: (Y-Y Axis)

Flexural Buckling Stress: Fcr = 25.43 ksi

Controlling Equation F2-2

Nominal Compressive Strength: Pc = 291 kip

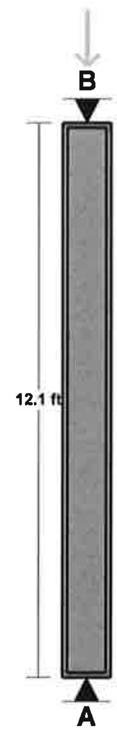
Combined Stress Calculations:

H1-1a Controls : 0.36

Controlling Combined Stress Factor: 0.36

NOTES

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	54597	lb
Dead Load:	PD =	49539	lb
Column Self Weight:	CSW =	787	lb
Total Load:	PT =	104923	lb

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Column D'25
Column
[2003 International Building Code(AISC 13th Ed ASD)]
A572-50 W12x65 x 12.1 FT
Section Adequate By: 61.5%

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	55722	lb
Dead Load:	Vert-DL-Rxn =	56271	lb
Total Load:	Vert-TL-Rxn =	111993	lb

COLUMN DATA

Total Column Length:	12.1	ft
Unbraced Length (X-Axis) Lx:	12.1	ft
Unbraced Length (Y-Axis) Ly:	12.1	ft
Column End Condtion-K (e):	2	
Column Bending Coefficient	1	

COLUMN PROPERTIES

W12x65 - W Shapes

Yield Stress:	Fy =	50 ksi		
Modulus of Elasticity:	E =	29 ksi		
Depth:	d =	12.1 in		
Web Thickness:	tw =	0.39 in		
Flange Width:	bf =	12 in		
Flange Thickness:	tf =	0.61 in		
Distance to Web Toe of Fillet:	k =	1.2 in		
Moment of Inertia (deflection):	Ix =	533 in4	Iy =	174 in4
Section Modulus:	Sx =	87.9 in3	Sy =	29.1 in3
Plastic Section Modulus:	Zx =	96.8 in3	Zy =	44.1 in3
Rad. of Gyration:	rx =	5.28 in	ry =	3.02 in

Column Compression Calculations:

KL/r Ratio: KLx/rx = 55 KLy/ry = 96.16

Controlling Direction for Compr. Calcs: (Y-Y Axis)

Flexural Buckling Stress: Fcr = 25.43 ksi

Controlling Equation F2-2

Nominal Compressive Strength: Pc = 291 kip

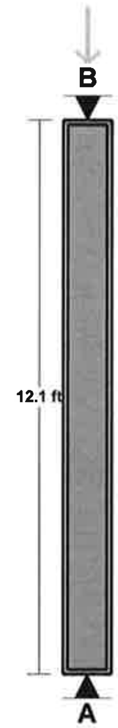
Combined Stress Calculations:

H1-1a Controls : 0.39

Controlling Combined Stress Factor: 0.39

NOTES

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	55722	lb
Dead Load:	PD =	55484	lb
Column Self Weight:	CSW =	787	lb
Total Load:	PT =	111993	lb

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Column C'26
Column
[2003 International Building Code(AISC 13th Ed ASD)]
A572-50 W10x60 x 12.1 FT
Section Adequate By: 34.8%

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

StruCalc Version 8.0.113.0 2/10/2017 11:09:03 AM

VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	54597	lb
Dead Load:	Vert-DL-Rxn =	80419	lb
Total Load:	Vert-TL-Rxn =	135016	lb

COLUMN DATA

Total Column Length:	12.1	ft
Unbraced Length (X-Axis) Lx:	12.1	ft
Unbraced Length (Y-Axis) Ly:	12.1	ft
Column End Condtion-K (e):	2	
Column Bending Coefficient	1	

COLUMN PROPERTIES

W10x60 - W Shapes

Yield Stress:	Fy =	50 ksi		
Modulus of Elasticity:	E =	29 ksi		
Depth:	d =	10.2 in		
Web Thickness:	tw =	0.42 in		
Flange Width:	bf =	10.1 in		
Flange Thickness:	tf =	0.68 in		
Distance to Web Toe of Fillet:	k =	1.18 in		
Moment of Inertia (deflection):	Ix =	341 in4	Iy =	116 in4
Section Modulus:	Sx =	66.7 in3	Sy =	23 in3
Plastic Section Modulus:	Zx =	74.6 in3	Zy =	35 in3
Rad. of Gyration:	rx =	4.39 in	ry =	2.57 in

Column Compression Calculations:

KL/r Ratio: KLx/rx = 66.15 KLy/ry = 113

Controlling Direction for Compr. Calcs: (Y-Y Axis)

Flexural Buckling Stress: Fcr = 19.66 ksi

Controlling Equation F2-2

Nominal Compressive Strength: Pc = 207 kip

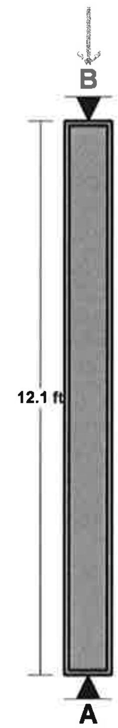
Combined Stress Calculations:

H1-1a Controls : 0.65

Controlling Combined Stress Factor: 0.65

NOTES

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	54597	lb
Dead Load:	PD =	79693	lb
Column Self Weight:	CSW =	726	lb
Total Load:	PT =	135016	lb

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Column D'26
Column
[2003 International Building Code(AISC 13th Ed ASD)]
A572-50 W10x60 x 12.1 FT
Section Adequate By: 36.8%

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

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VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	55722	lb
Dead Load:	Vert-DL-Rxn =	75159	lb
Total Load:	Vert-TL-Rxn =	130881	lb

COLUMN DATA

Total Column Length:	12.1	ft
Unbraced Length (X-Axis) Lx:	12.1	ft
Unbraced Length (Y-Axis) Ly:	12.1	ft
Column End Condtion-K (e):	2	
Column Bending Coefficient	1	

COLUMN PROPERTIES

W10x60 - W Shapes

Yield Stress:	Fy =	50 ksi		
Modulus of Elasticity:	E =	29 ksi		
Depth:	d =	10.2 in		
Web Thickness:	tw =	0.42 in		
Flange Width:	bf =	10.1 in		
Flange Thickness:	tf =	0.68 in		
Distance to Web Toe of Fillet:	k =	1.18 in		
Moment of Inertia (deflection):	Ix =	341 in4	Iy =	116 in4
Section Modulus:	Sx =	66.7 in3	Sy =	23 in3
Plastic Section Modulus:	Zx =	74.6 in3	Zy =	35 in3
Rad. of Gyration:	rx =	4.39 in	ry =	2.57 in

Column Compression Calculations:

KL/r Ratio:	KLx/rx = 66.15	KLy/ry =	113

Controlling Direction for Compr. Calcs: (Y-Y Axis)

Flexural Buckling Stress:	Fcr =	19.66 ksi
Controlling Equation	F2-2	
Nominal Compressive Strength:	Pc =	207 kip

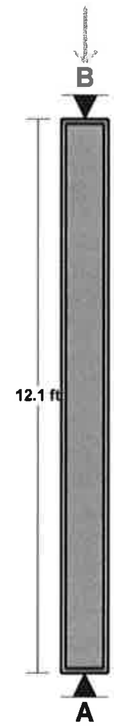
Combined Stress Calculations:

H1-1a Controls : 0.63

Controlling Combined Stress Factor: 0.63

NOTES

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	55722	lb
Dead Load:	PD =	74433	lb
Column Self Weight:	CSW =	726	lb
Total Load:	PT =	130881	lb

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Column C26
Column
[2003 International Building Code(AISC 13th Ed ASD)]
A572-50 W12x79 x 12.1 FT
Section Adequate By: 71.9%

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

StruCalc Version 8.0.113.0 2/10/2017 11:09:13 AM

VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	22650	lb
Dead Load:	Vert-DL-Rxn =	77846	lb
Total Load:	Vert-TL-Rxn =	100496	lb

COLUMN DATA

Total Column Length:	12.1	ft
Unbraced Length (X-Axis) Lx:	12.1	ft
Unbraced Length (Y-Axis) Ly:	12.1	ft
Column End Condtion-K (e):	2	
Column Bending Coefficient	1	

COLUMN PROPERTIES

W12x79 - W Shapes

Yield Stress:	Fy =	50 ksi		
Modulus of Elasticity:	E =	29 ksi		
Depth:	d =	12.4 in		
Web Thickness:	tw =	0.47 in		
Flange Width:	bf =	12.1 in		
Flange Thickness:	tf =	0.74 in		
Distance to Web Toe of Fillet:	k =	1.33 in		
Moment of Inertia (deflection):	Ix =	662 in4	Iy =	216 in4
Section Modulus:	Sx =	107 in3	Sy =	35.8 in3
Plastic Section Modulus:	Zx =	119 in3	Zy =	54.3 in3
Rad. of Gyration:	rx =	5.34 in	ry =	3.05 in

Column Compression Calculations:

KL/r Ratio: KLx/rx = 54.38 KLy/ry = 95.21
Controlling Direction for Compr. Calcs: (Y-Y Axis)
Flexural Buckling Stress: Fcr = 25.77 ksi
Controlling Equation F2-2
Nominal Compressive Strength: Pc = 358 kip

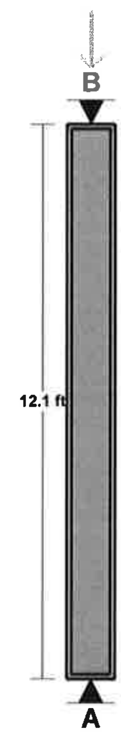
Combined Stress Calculations:

H1-1a Controls : 0.28

Controlling Combined Stress Factor: 0.28

NOTES

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	22650 lb
Dead Load:	PD =	76890 lb
Column Self Weight:	CSW =	956 lb
Total Load:	PT =	100496 lb

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Column C27
Column
[2003 International Building Code(AISC 13th Ed ASD)]
A572-50 W12x96 x 12.1 FT
Section Adequate By: 74.4%

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

StruCalc Version 8.0.113.0 2/10/2017 11:09:18 AM

VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	27000	lb
Dead Load:	Vert-DL-Rxn =	86102	lb
Total Load:	Vert-TL-Rxn =	113102	lb

COLUMN DATA

Total Column Length:	12.1	ft
Unbraced Length (X-Axis) Lx:	12.1	ft
Unbraced Length (Y-Axis) Ly:	12.1	ft
Column End Condtion-K (e):	2	
Column Bending Coefficient	1	

COLUMN PROPERTIES

W12x96 - W Shapes

Yield Stress:	Fy =	50 ksi		
Modulus of Elasticity:	E =	29 ksi		
Depth:	d =	12.7 in		
Web Thickness:	tw =	0.55 in		
Flange Width:	bf =	12.2 in		
Flange Thickness:	tf =	0.9 in		
Distance to Web Toe of Fillet:	k =	1.5 in		
Moment of Inertia (deflection):	Ix =	833 in4	Iy =	270 in4
Section Modulus:	Sx =	131 in3	Sy =	44.4 in3
Plastic Section Modulus:	Zx =	147 in3	Zy =	67.5 in3
Rad. of Gyration:	rx =	5.44 in	ry =	3.09 in

Column Compression Calculations:

KL/r Ratio: KLx/rx = 53.38 KLy/ry = 93.98

Controlling Direction for Compr. Calcs: (Y-Y Axis)

Flexural Buckling Stress: Fcr = 26.21 ksi

Controlling Equation F2-2

Nominal Compressive Strength: Pc = 443 kip

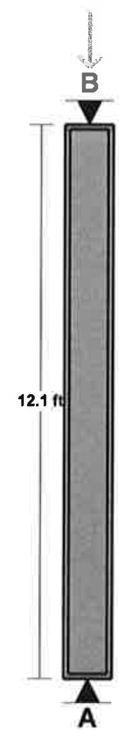
Combined Stress Calculations:

H1-1a Controls : 0.26

Controlling Combined Stress Factor: 0.26

NOTES

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	27000	lb
Dead Load:	PD =	84940	lb
Column Self Weight:	CSW =	1162	lb
Total Load:	PT =	113102	lb

Project: S2900 - Stamford Harbor (Rev3 with updated code)

Location: Column D27
Column
[2003 International Building Code(AISC 13th Ed ASD)]
A572-50 W10x68 x 12.1 FT
Section Adequate By: 8.6%

Gregory Haggstrom, EIT
Hudson Design Group LLC
1600 Osgood Street, Bldg 20N, Suite 3090
North Andover, MA 01845

StruCalc Version 8.0.113.0 2/10/2017 11:09:23 AM

VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	102750	lb
Dead Load:	Vert-DL-Rxn =	115548	lb
Total Load:	Vert-TL-Rxn =	218298	lb

COLUMN DATA

Total Column Length:	12.1	ft
Unbraced Length (X-Axis) Lx:	12.1	ft
Unbraced Length (Y-Axis) Ly:	12.1	ft
Column End Condtion-K (e):	2	
Column Bending Coefficient	1	

COLUMN PROPERTIES

W10x68 - W Shapes

Yield Stress:	Fy =	50 ksi		
Modulus of Elasticity:	E =	29 ksi		
Depth:	d =	10.4 in		
Web Thickness:	tw =	0.47 in		
Flange Width:	bf =	10.1 in		
Flange Thickness:	tf =	0.77 in		
Distance to Web Toe of Fillet:	k =	1.27 in		
Moment of Inertia (deflection):	Ix =	394 in4	Iy =	134 in4
Section Modulus:	Sx =	75.7 in3	Sy =	26.4 in3
Plastic Section Modulus:	Zx =	85.3 in3	Zy =	40.1 in3
Rad. of Gyration:	rx =	4.44 in	ry =	2.59 in

Column Compression Calculations:

KL/r Ratio: KLx/rx = 65.41 KLy/ry = 112.12

Controlling Direction for Compr. Calcs: (Y-Y Axis)

Flexural Buckling Stress: Fcr = 19.94 ksi

Controlling Equation F2-2

Nominal Compressive Strength: Pc = 239 kip

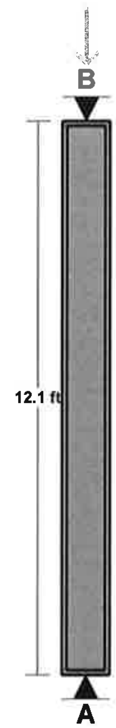
Combined Stress Calculations:

H1-1a Controls : 0.91

Controlling Combined Stress Factor: 0.91

NOTES

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	102750	lb
Dead Load:	PD =	114725	lb
Column Self Weight:	CSW =	823	lb
Total Load:	PT =	218298	lb

Hudson Design Group LLC

Miscellaneous Calculations

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Determine the Weight of the Proposed Equipment (Per Sector)

Antennas:	4	x	68 lbs	=	272 lbs
RRUS-11:	3	x	50 lbs	=	150 lbs
RRUS-12:	2	x	58 lbs	=	116 lbs
RRUS-E2:	1	x	58 lbs	=	58 lbs
RRUS-32:	1	x	77 lbs	=	77 lbs
Pipes:	4	x	30 lbs	=	120 lbs
Misc.:	1	x	44 lbs	=	44 lbs
TOTAL =					**837 lbs**

Determine the Weight of the Cooling Tower & Supporting Frame:

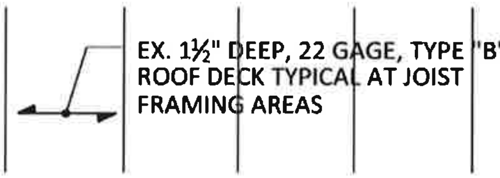
W10x12:	57	ft	x	12 plf	=	684 lbs
W14x34:	16.2	ft	x	34 plf	=	550.8 lbs
W14x61:	28	ft	x	61 plf	=	1708 lbs
W12x26:	16.2	ft	x	26 plf	=	421.2 lbs
C10x20:	81	ft	x	20 plf	=	1620 lbs
Grating	6.1	psf	x	302 sqft	=	1842 lbs
Cooling Tower Weight =						12940 lbs
Misc. =						400 lbs
TOTAL =						**20166 lbs**

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

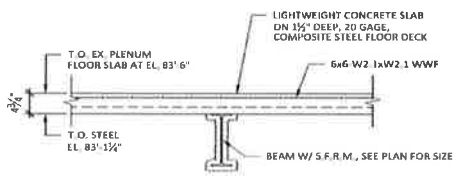
Determine the Weight of the Existing Roof

TYPE 0:



Deck Weight =	2 psf
Misc. =	3 psf
Total =	5 psf

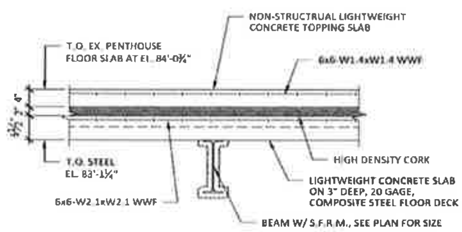
TYPE 1:



EXISTING PLENUM FLOOR SLAB TYPE 1 SECTION 1A
¼"=1' 0"

Composite Deck Weight =	43 psf
Misc. =	3 psf
Total =	46 psf

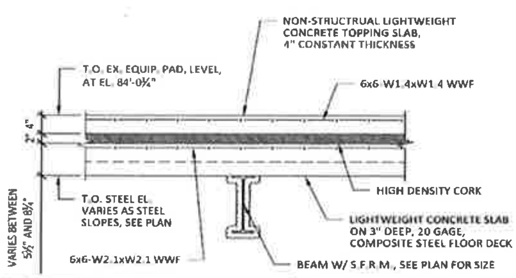
TYPE 2:



EXISTING MECHANICAL PENTHOUSE FLOOR SLAB TYPE 2 SECTION 1B
¾"=1'-0"

Composite Deck Weight =	43 psf
Upper Slab Weight =	37 psf
Misc. =	5 psf
Total =	85 psf

TYPE 3:



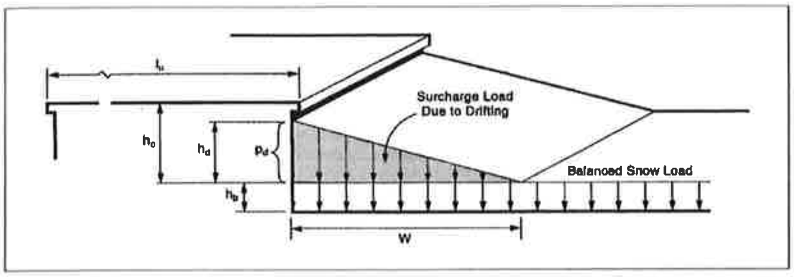
EXISTING EXTERIOR EQUIPMENT PAD/SLAB TYPE 3 SECTION 1C
¾"=1'-0"

Avg. Composite Deck Weight =	53 psf
Upper Slab Weight =	37 psf
Misc. =	5 psf
Total =	95 psf

Date: 02-10-2017
Project Name: Stamford Harbor
Project Number: S2900
Designed By: GH Checked By: MSC

Hudson Design Group LLC

Determine the Surcharge Load Due to Drift



l_u =	18.42 ft
h_c =	11.5 ft
P_g =	30 psf

FIGURE 7-8 CONFIGURATION OF SNOW DRIFTS ON LOWER ROOFS

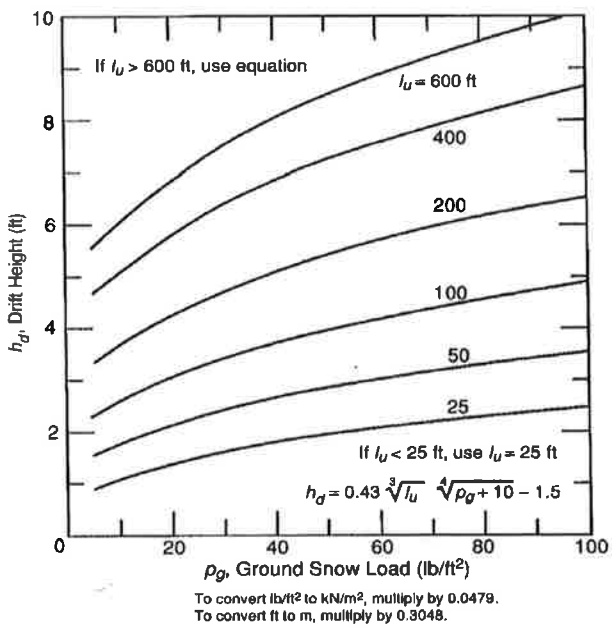


FIGURE 7-9 GRAPH AND EQUATION FOR DETERMINING DRIFT HEIGHT, h_d

From Figure 7-9: h_d = 1.67 ft

W = 6.68 ft (4 * h_d)

Snow Density (γ) = 0.13 * P_g + 14 = 17.9 pcf

Maximum Intensity of Surcharge Load (**P_d**) = h_d * γ = 29.9 psf

Distributed Load Caused by Drift = (hd * W/2) * γ = **99.8 plf**

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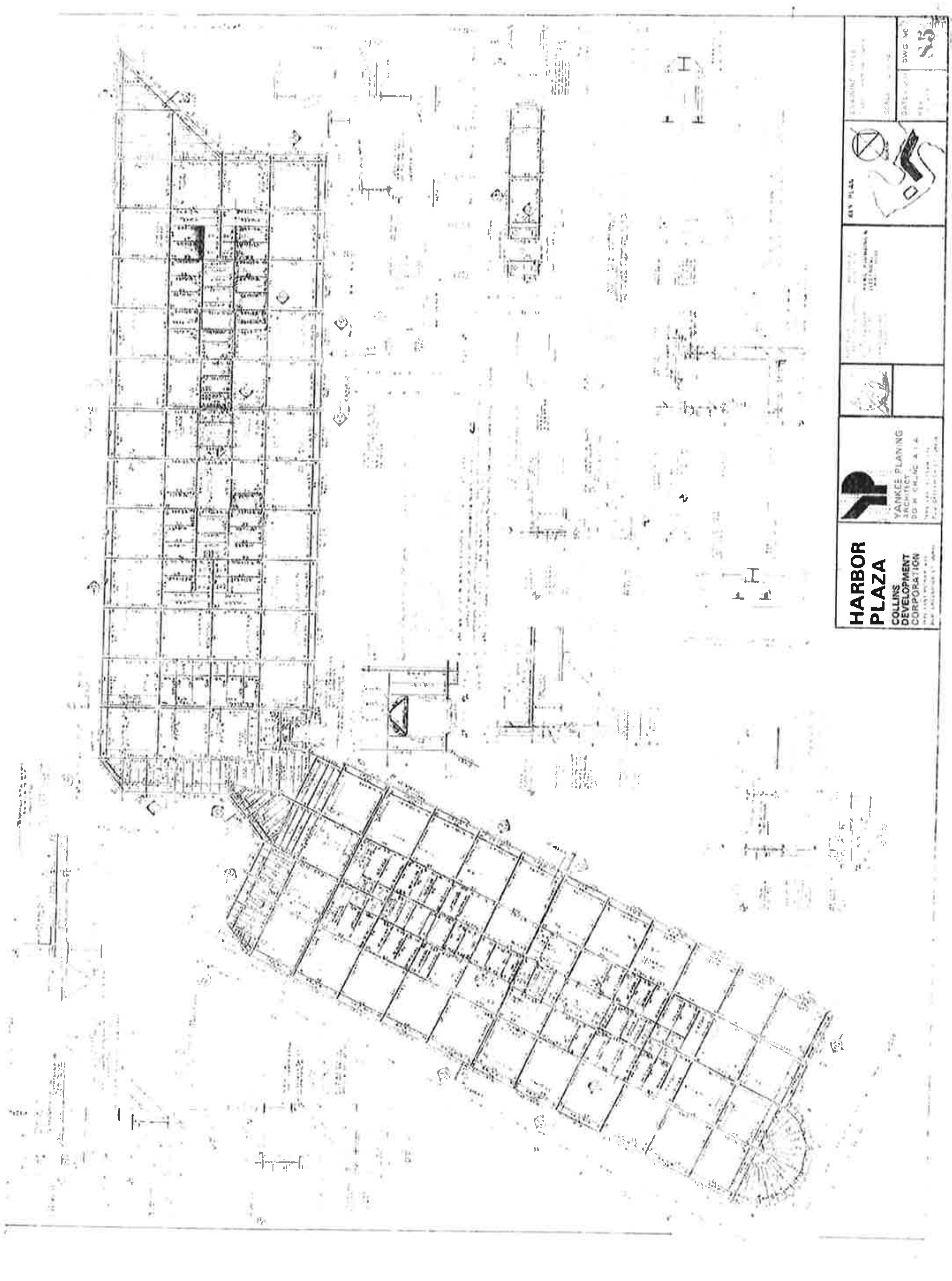
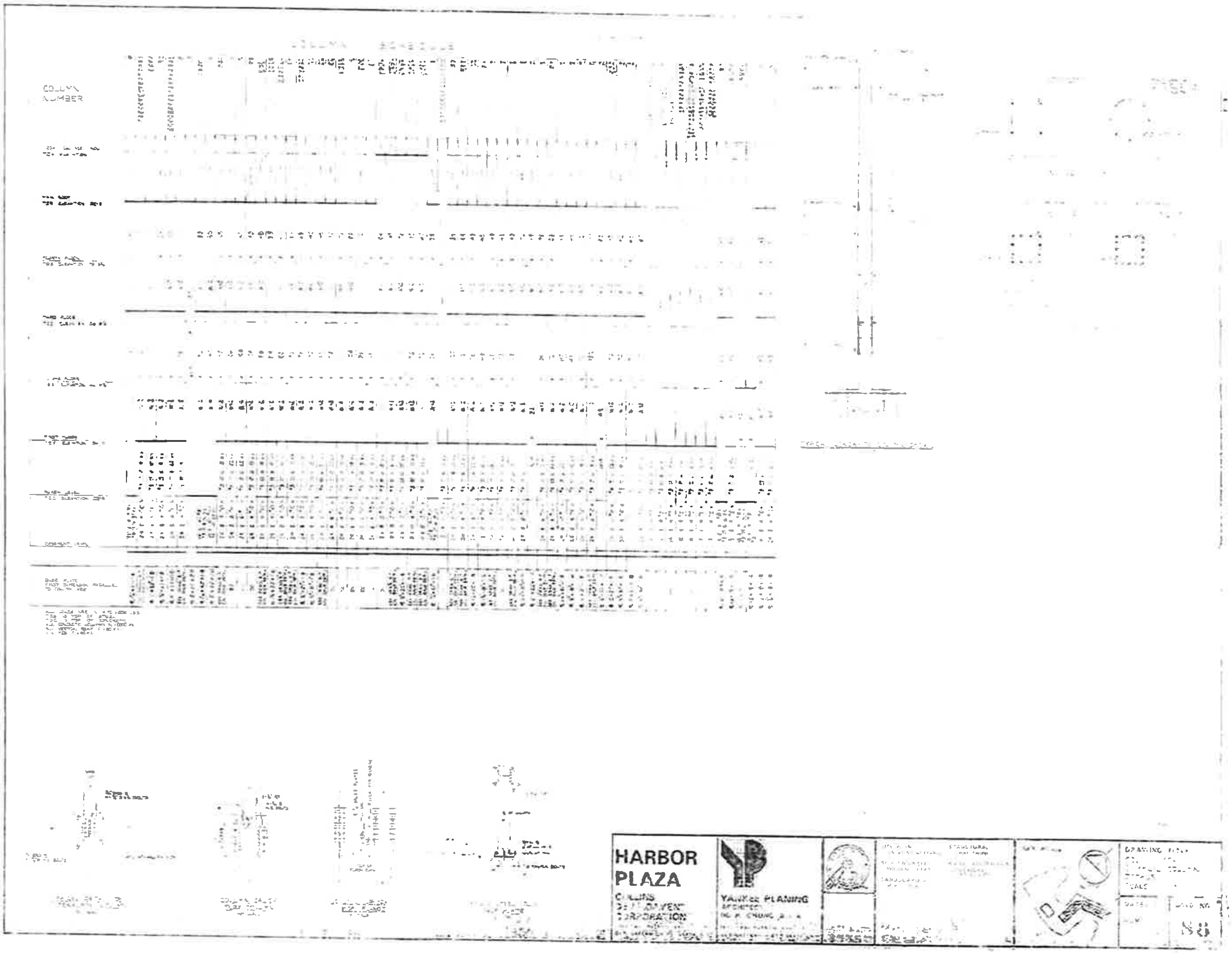
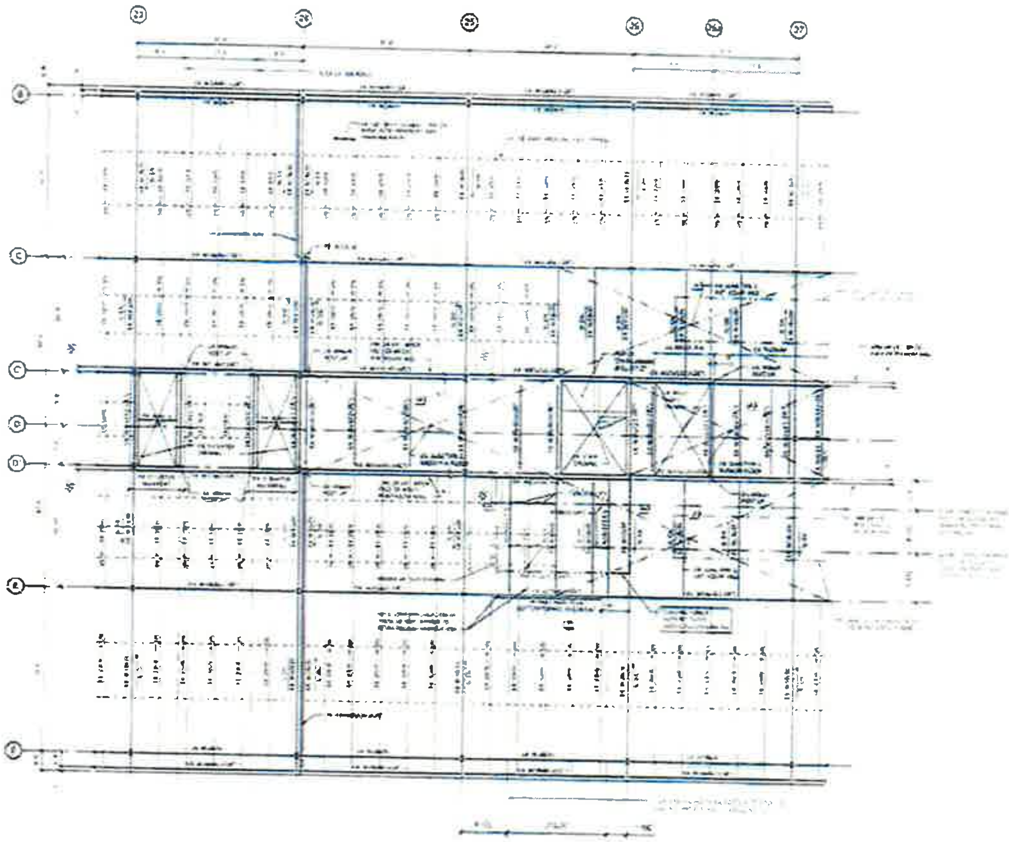
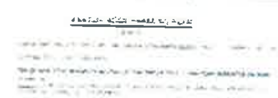
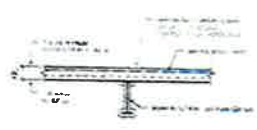
Referenced Documents


HARBOR PLAZA
COLLINS DEVELOPMENT CORPORATION
YANKEE PLANING
ARCHITECT
KEY PLAN

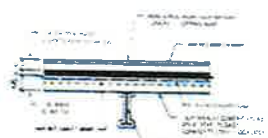

COLUMN NUMBER
HARBOR PLAZA
COLLINS DEVELOPMENT CORPORATION
YANKEE PLANING
ARCHITECT
S8



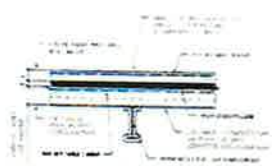




EXISTING PLENUM FLOOR SLAB TYPE 1 SECTION 1A



EXISTING MECHANICAL PENTHOUSE FLOOR SLAB TYPE 2 SECTION 1B



EXISTING EXTERIOR EQUIPMENT PAD SLAB TYPE 3 SECTION 1C

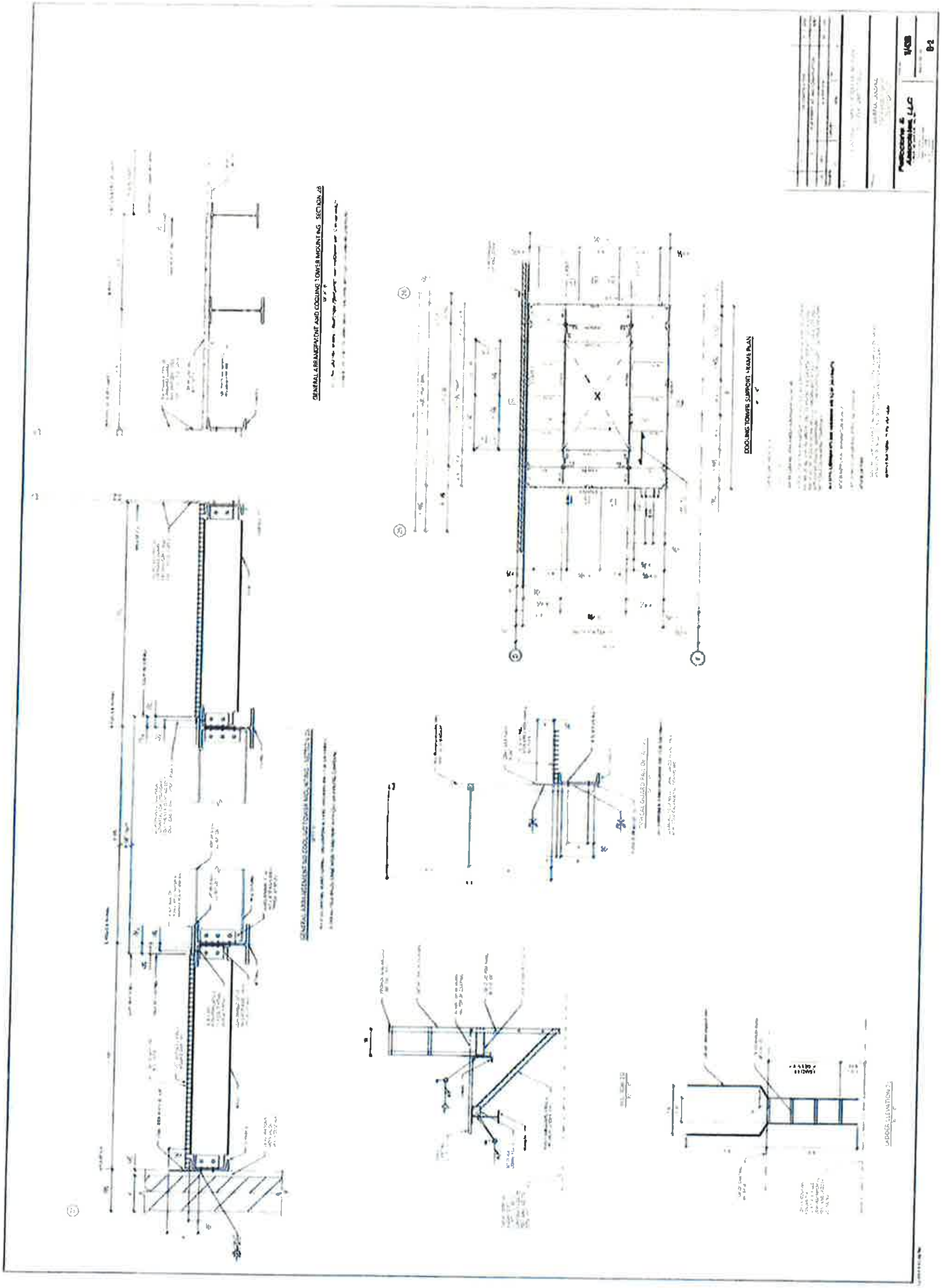
GENERAL NOTES:

CONCRETE NOTES:

STEEL NOTES:

Pellicciotta & Associates, LLC

S-1


COOLING TOWER SUPPORT FRAME PLAN
Pellicone & Associates, LLC
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