Project Design Document

Location

Bahama Sound Of Exuma

Example

Job Number: 242201 Date:
24/04/2024

Index

1 Project Details

The project is a Single Story structure, located at Bahama Sound Of Exuma. The superstructure is CFS enclosed structure & substructure in concrete by other. All the walls, floor and roof are of light gauge and the CFS metal studs will be manufactured and fabricated by certified manufacturers in the UK.

The calculations produced have been based on the drawings provided by the client and where information and details are based on RFI (Request for Information) forms will be issued. To prevent the delay of deliverables it can be agreed between the parties that certain details are to be confirmed. It should be noted that at the time of issuance of this document, there were several RFIs outstanding, and assumptions have been made in lieu of a response to the RFIs.

2 Scope of Work

The scope of the work is design of steel structure using cold form steel walls, roof trusses/joist. Hot rolled steel to be used for gravity and lateral load resistance of the building. Design of foundation is not a part of scope. The design will be based on US building requirements.

Services Include:

- Structural Design calculations for light gauge steel and hot rolled steel under Gravity and Lateral Loading
- Structural GA Drawings and Details
- 3-D structural model
- Fabrication Drawings and machine file
- ACS
- BOQs for CFS & HRS.

3 Excluded Items

Items excluded from this scope include, but limited to:

- Site visits and field inspections.
- RC Foundation / Substructure design.
- Remedial engineering services.
- Redesigns, changes and/or revisions made to the drawings initiated by the Owner, the Owner's representative, the Architect/Designer, the Builder/Contractor, or any Sub- Contractor.
- Mechanical, Electrical & Plumbing Designs and Drafting (By Others).
- Energy Calculations and Cost Estimates (By Others).
- Design of Balustrade and supporting arrangement.
- Design of Sun shader.
- Flood load design.
- Staircase design.
- Any item not identified as part of scope.

4 Assumptions

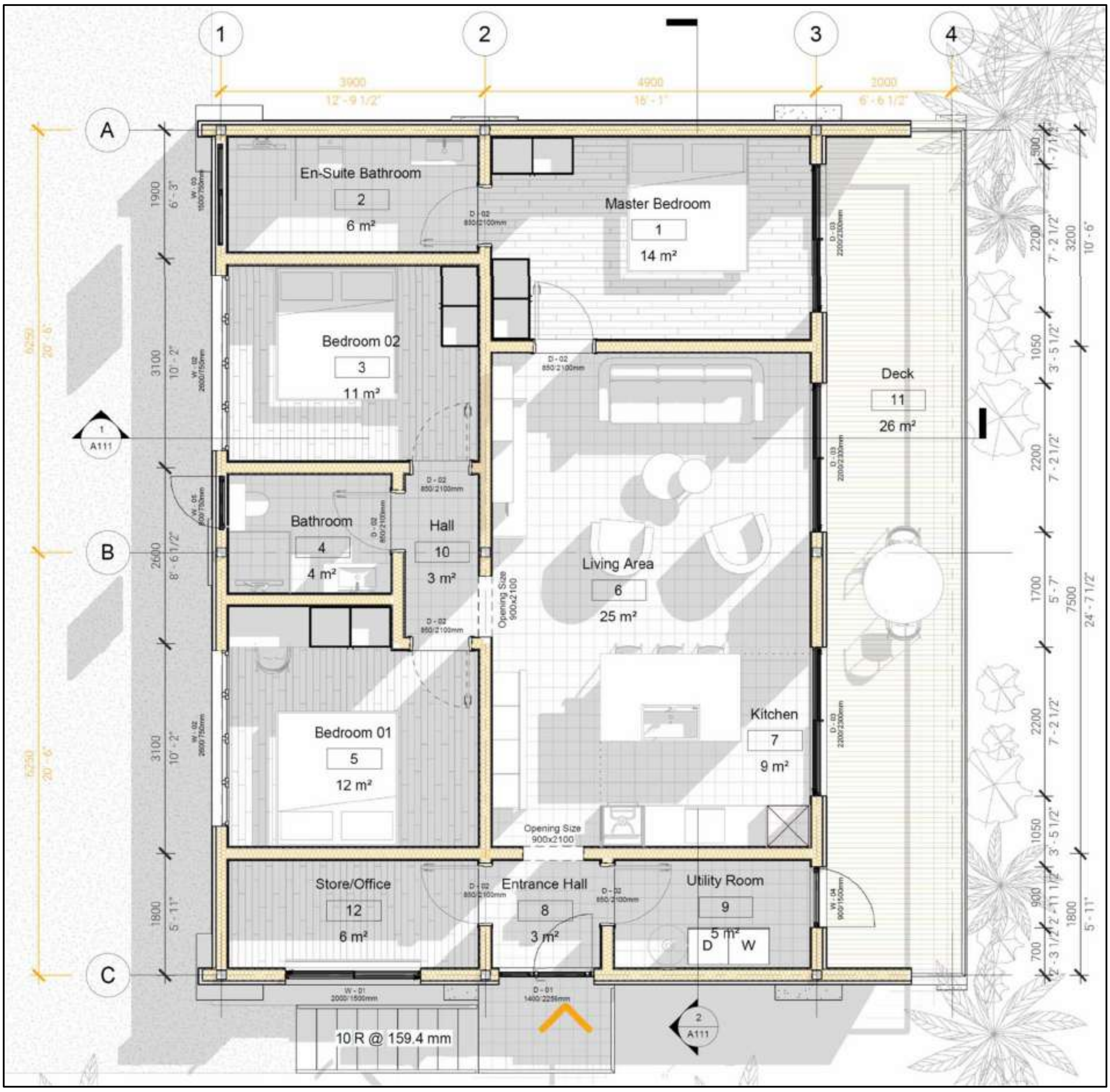
The intent of this document is to detail the structural design requirement and design specifications of the building with the following assumptions:

1. Maximum wind speed assumed as 200 mph.
2. The risk category was assumed as II.
3. Exposure category was assumed as D.
4. Site Class was assumed as Region D.
5. Wall set-out plans and elevations are considered as per the architectural plan.
6. Building sectional details are considered as per the architectural drawings provided, unless details are absent, and it is agreed to progress with assumptions.
7. A non-accessible type of roof is used.
8. The minimum ultimate compressive strength of concrete used will be 5000 to 6500 psi.
9. Sizes of structural openings are assumed to be the same as the openings in the architectural drawings.
10. All dimensions used within the calculations are approximate for design purposes only. No structural members are to be fabricated and no setting out is to be carried out based upon the dimensions contained herein. Accurate on-site measurements are to be taken where necessary, and reference is to be made to the architectural drawings for all setting out information where applicable.
11. Level of light gauge steel as per the architectural drawings provided, unless details are absent, and it is agreed to progress with assumptions.
12. Foundation design and detail by others.
13. Readily available technical data on fixings have been used.
14. The deflection allowance for the deflection head has been assumed to be 1” for the external and internal wall.

5 Geometry of Building

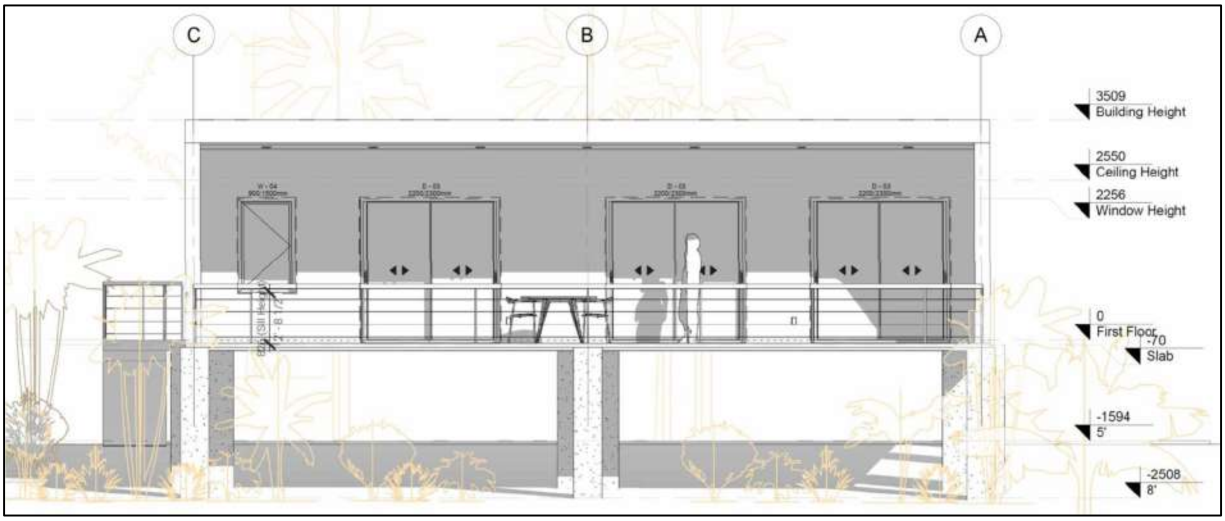
5.1 Plan

5.1.1 Ground Floor Plan

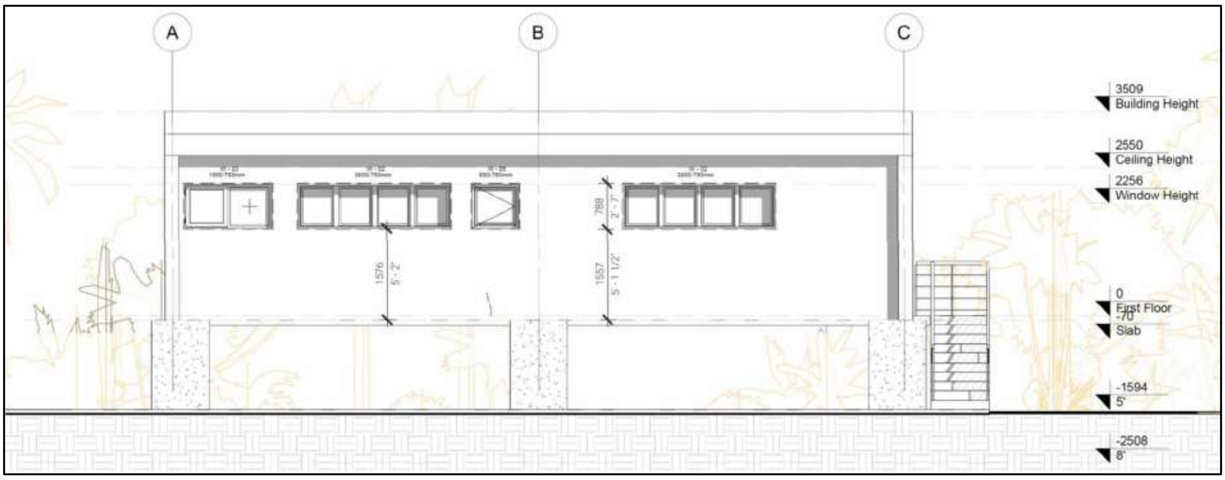


5.2 Elevation

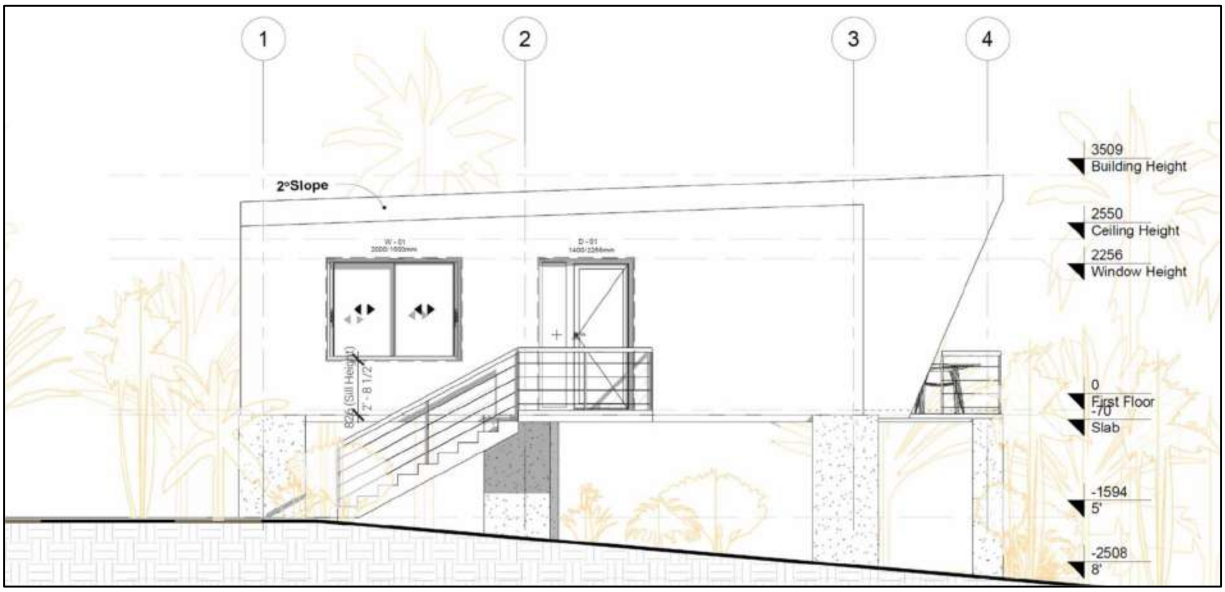
5.2.1 Front Elevation



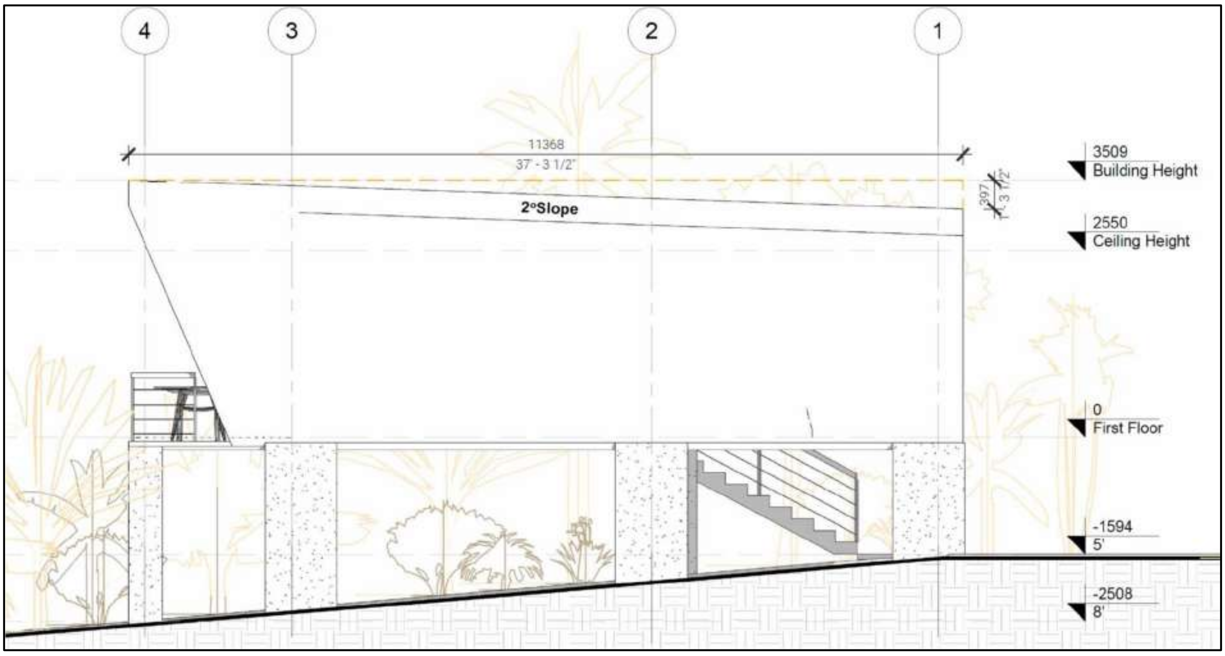
5.2.2 Rear Elevation



5.2.3 Left Elevation

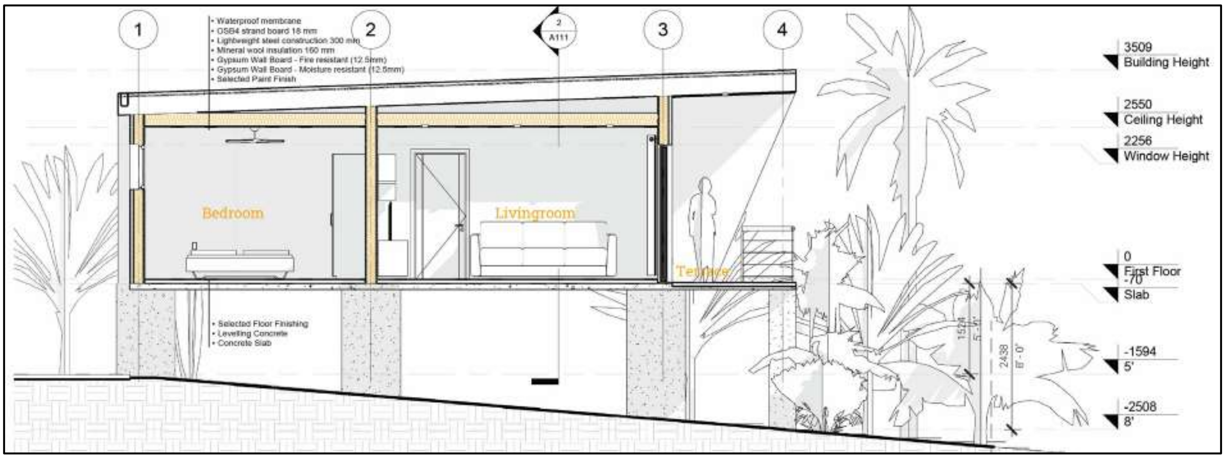


5.2.4 Right Elevation

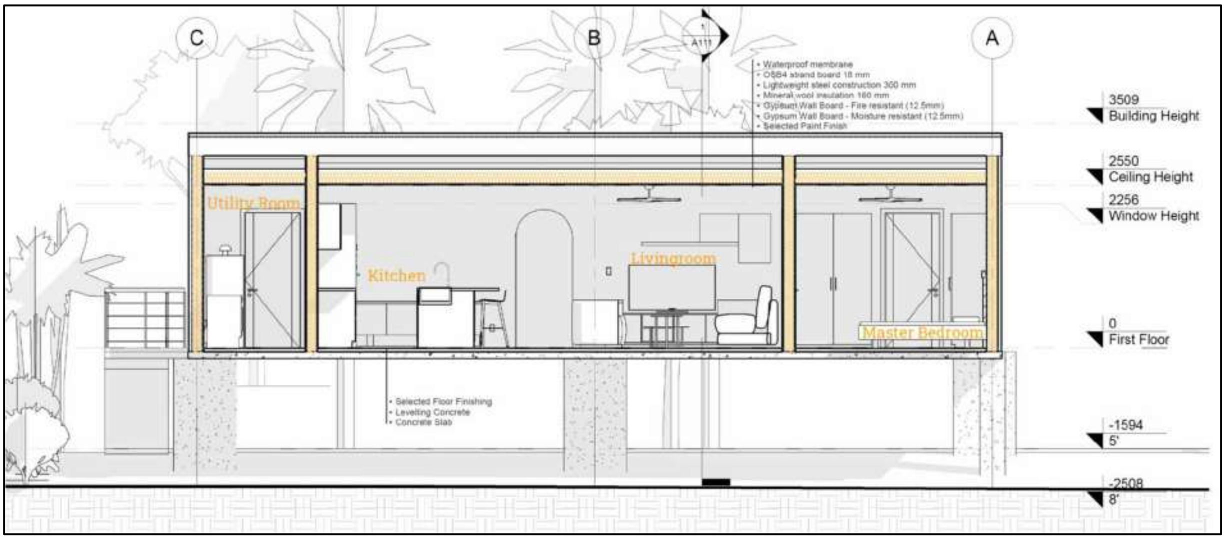


5.3 Section

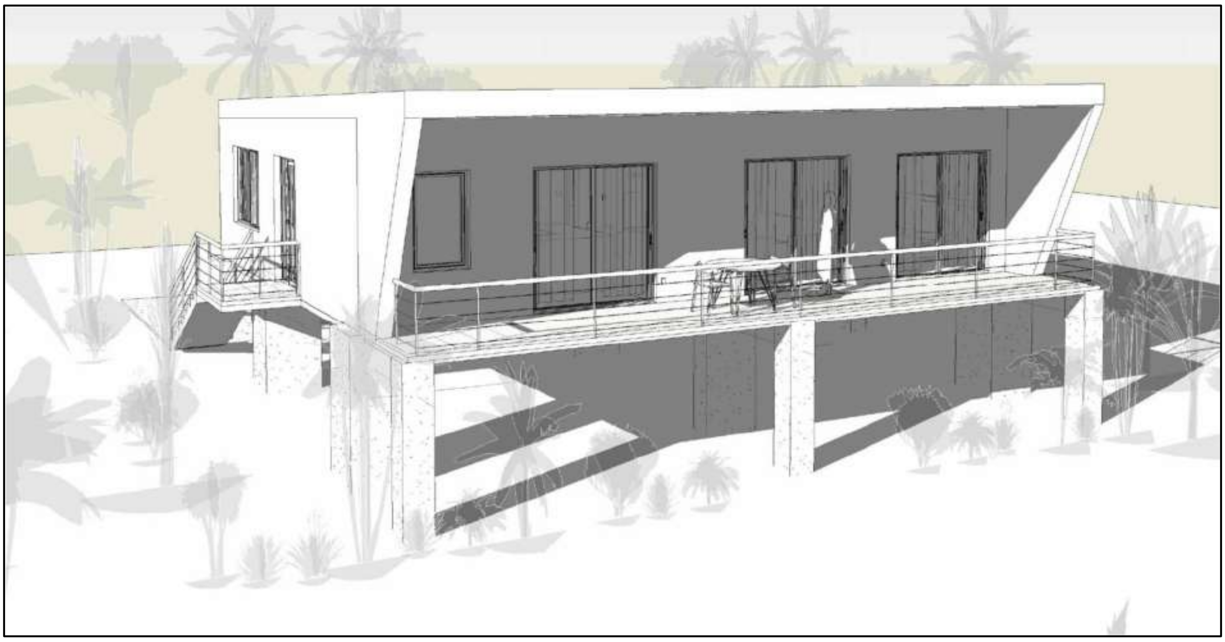
5.3.1 Section 1



5.3.2 Section 2



5.4 3-D



6 Building Code

6.1 Building Codes:

- IBC 2018 — International Building Code
- ASCE 7–10 — American Society of Civil Engineers (Minimum Design loads for buildings & other structures)
- ASCE 7–16 — American Society of Civil Engineers (Minimum Design loads for buildings & other structures)
- AISC 360–10 — American Institute of Steel Construction (Specification for structural steel buildings)
- AISI 2020 — American Iron and Steel Institute (Design of Cold Formed steel & structural members)
- ACI 318–19 — American Concrete Institute (Building code requirements for structural concrete)
- ASTM — American Society of Testing and Material

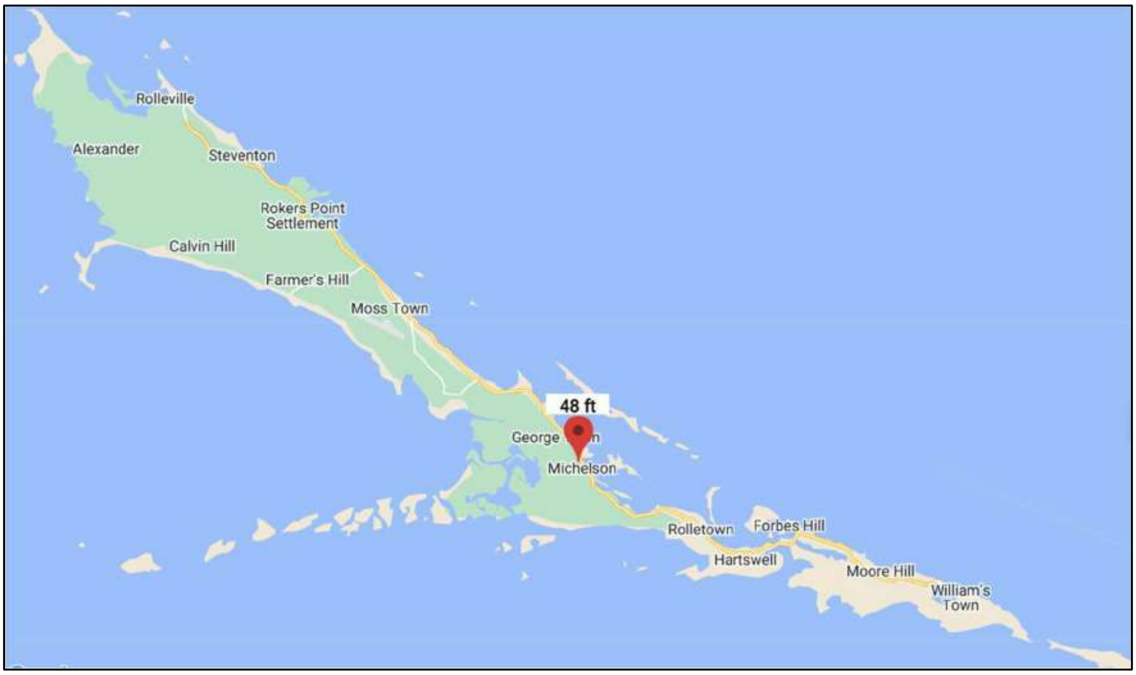
6.2 Design Approach to be used:

All CFS frames will be designed to AISI LRFD CFS design standard.

7 Lateral Design

7.1 Vicinity Map

7.2 Local Map



7.3 Basic Wind Speed

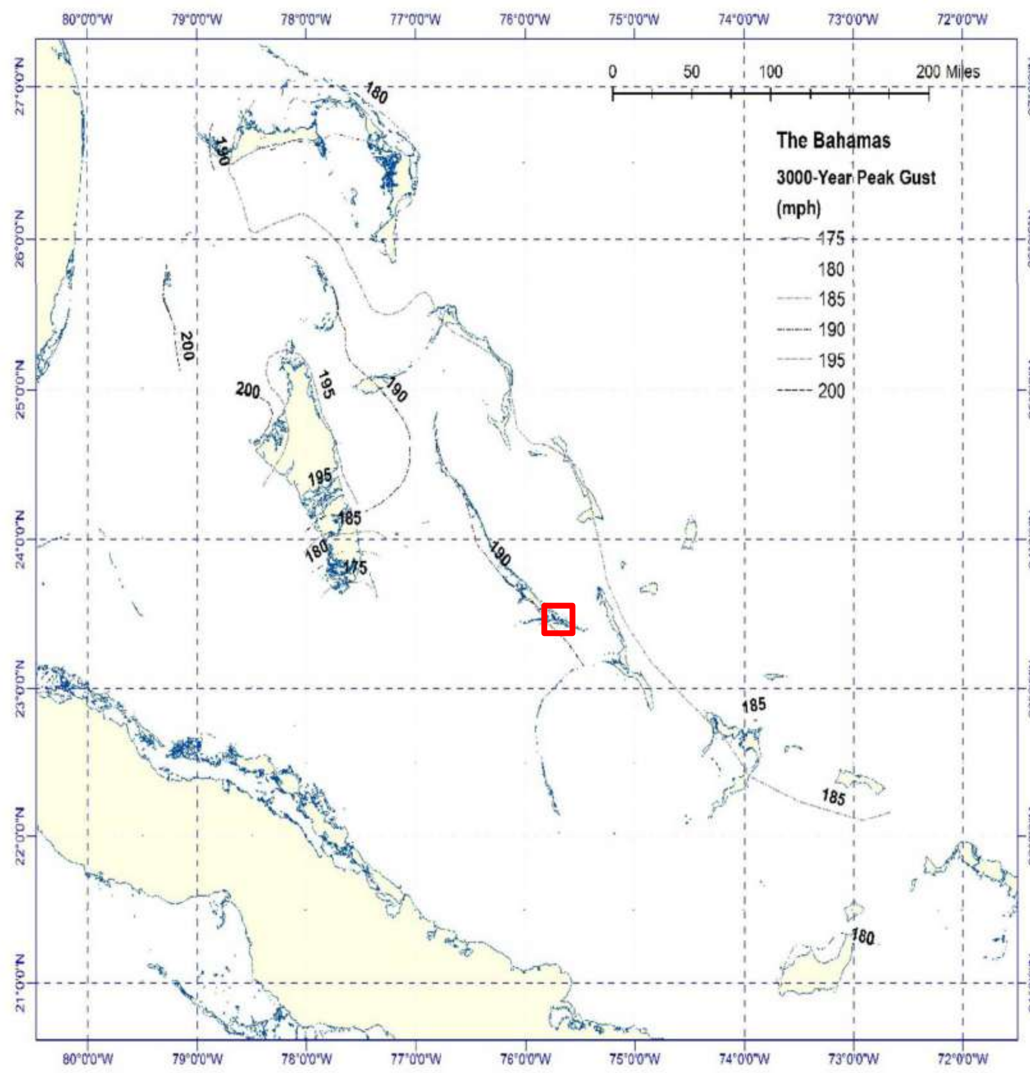


Figure 3-44. Contours of predicted 3,000-year return period wind speeds (mph) at a height of 10 m above flat open terrain for the Bahamas

Basic Ultimate Wind Speed V_{ult} = 200 mph

Basic Nominal Wind Speed V_{asd} = 155 mph

TABLE 1609.3.1
WIND SPEED CONVERSIONS[a, b, c]

V_{ult}	100	110	120	130	140	150	160	170	180	190	200
V_{asd}	78	85	93	101	108	116	124	132	139	147	155

For SI: 1 mile per hour = 0.44 m/s.
a. Linear interpolation is permitted.
b. V_{asd} = nominal design wind speed applicable to methods specified in Exceptions 1 through 5 of Section 1609.1.1.
c. V_{ult} = ultimate design wind speeds determined from Figures 1609A, 1609B, or 1609C.

7.1 Exposure Condition

Exposure Category = D

1609.4.3 Exposure categories.
An exposure category shall be determined in accordance with the following:

Exposure B. For buildings with a mean roof height of less than or equal to 30 feet (9144 mm), Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 1,500 feet (457 m). For buildings with a mean roof height greater than 30 feet (9144 mm), Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance of at least 2,600 feet (792 m) or 20 times the height of the building, whichever is greater.

Exposure C. Exposure C shall apply for all cases where Exposure B or D does not apply.

Exposure D. Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance of at least 5,000 feet (1524 m) or 20 times the height of the building, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 600 feet (183 m) or 20 times the building height, whichever is greater, from an Exposure D condition as defined in the previous sentence.

7.2 Terrain/Surface Roughness Category

Surface roughness = D

1609.4.2 Surface roughness categories.
A ground surface roughness within each 45-degree (0.79 rad) sector shall be determined for a distance upwind of the site as defined in Section 1609.4.3 from the categories defined below, for the purpose of assigning an exposure category as defined in Section 1609.4.3.

Surface Roughness B. Urban and suburban areas, wooded areas or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C. Open terrain with scattered obstructions having heights generally less than 30 feet (9144 mm). This category includes flat open country, and grasslands.

Surface Roughness D. Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats and unbroken ice.

7.3 Risk Category

Risk/Occupancy Category = II

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures designated as essential facilities	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

7.4 Topography Factor

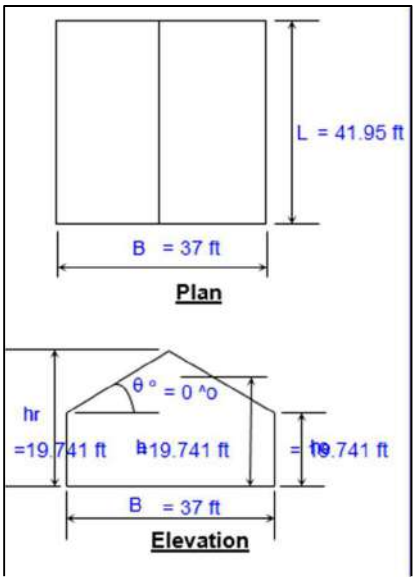
Topography factor (K_{zt}) = 1 (As per clause 26.8.1, ASCE 7-16)

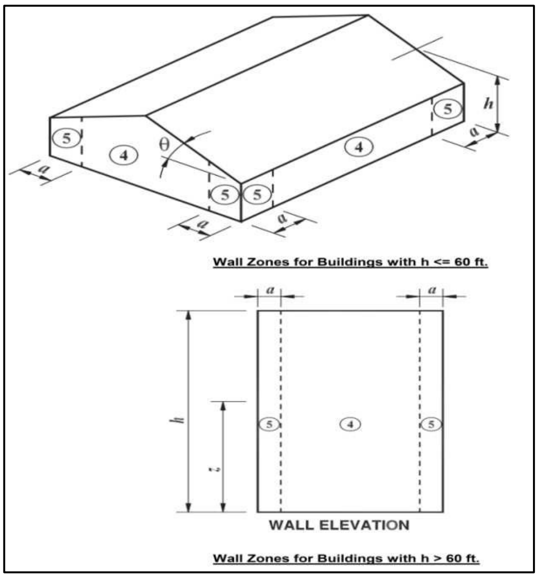
7.5 Gust Factor:

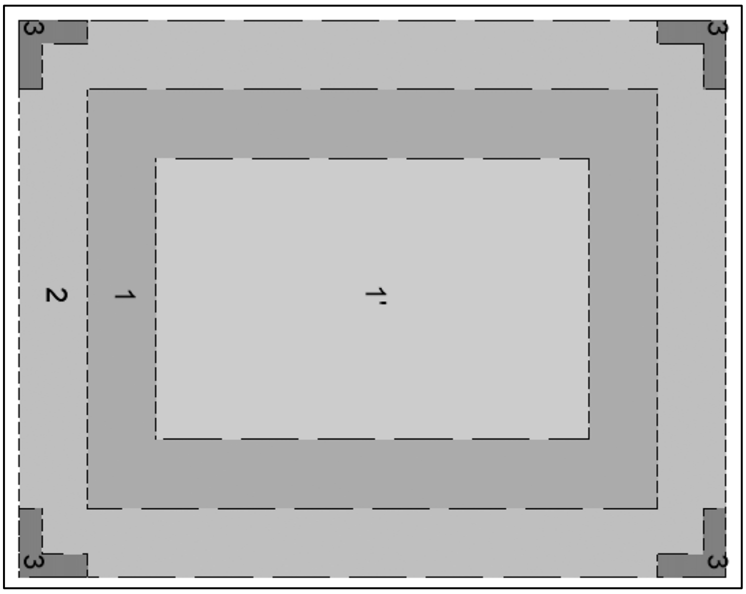
Gust factor = 0.85 (as per clause 26.11.1, ASCE 07-16)

7.6 Design Wind Pressure (Enclosed Building)

Length of building	=	12786 mm	=	41.95 ft
Width of building	=	11278 mm	=	37 ft
Height of Building	=	6017 mm	=	19.741 ft







7.6.1 Component & Cladding Wind Pressure:

Maximum wind pressure, Wall (mid zone, zone 4) = +96.17 / -104.63 psf
Maximum wind pressure, Wall (end zone, zone 5) = +96.17 / -124.62 psf

Maximum wind pressure, Roof (zone 1) = -149.41 psf
Maximum wind pressure, Roof (zone 1’) = -101.58 psf
Maximum wind pressure, Roof (zone 2) = -198.01 psf
Maximum wind pressure, Roof (zone 3) = -247.41 psf

Refer“Appendix – 1. Wind Calculations Wall C&C_Ground floor” for detailed summary.

Refer“Appendix – 2. Wind Calculations Roof C&C_Ground floor” for detailed summary.

7.6.2 MWFRS Wind Pressures (For Lateral analysis and Design):

7.6.2.1 Wind Pressure on Walls:

Wind pressure on Wall (Windward+ Leeward) = 47.03+56.91
=103.94 psf

7.6.2.2 Wind Pressure on Roof:

Maximum wind pressure on Roof (Wind @ Normal)
Zone 3,4 and 5 = +2.54 psf
= -90.18 psf, -87.81 and -57.98 psf respectively

Maximum wind pressure on Roof (Wind @ Parallel)
Zone 3,4,5 and 6 = +2.54 psf
= -88.89 psf, -88.89psf, -56.91psf and -40.92 psf

Refer “Appendix – 3. Wind Calculations MWFRS (DIR)_Ground floor (Normal) ” for detailed summary.

Refer “Appendix – 4. Wind Calculations MWFRS (DIR)_Ground floor (Parallel)” for detailed summary.

8 Seismic Load

8.1 Seismic Location Map:

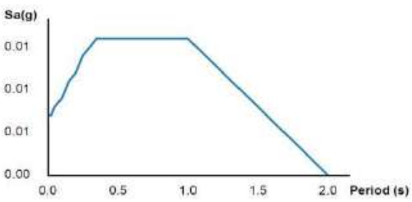
Seismic parameter for Bahamas is not available. So nearest Location Bahamas is considered for now to calculate the parameters.

ATC Hazards by Location

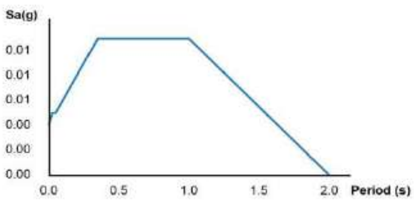
Search Information

Address:	bahamas
Coordinates:	25.03428, -77.39627999999999
Elevation:	29 ft
Timestamp:	2023-07-24T06:46:59.649Z
Hazard Type:	Seismic
Reference Document:	ASCE7-10
Risk Category:	II
Site Class:	D

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.01	MCE_R ground motion (period=0.2s)
S_1	0.012	MCE_R ground motion (period=1.0s)
S_{MS}	0.016	Site-modified spectral acceleration value
S_{M1}	0.028	Site-modified spectral acceleration value
S_{DS}	0.011	Numeric seismic design value at 0.2s SA
S_{D1}	0.019	Numeric seismic design value at 1.0s SA

▾Additional Information

Name	Value	Description
SDC	A	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s
CR_S	0.763	Coefficient of risk (0.2s)
CR_1	0.792	Coefficient of risk (1.0s)
PGA	0.006	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.01	Site modified peak ground acceleration
T_L	null	Long-period transition period (s)
SsRT	0.01	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.013	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.012	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.015	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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8.2 Importance Factor

Importance Factor = 1.0

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads[a]

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

[a]The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it is dependent on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

8.3 Site Class

Site Class = Region D

8.4 Risk Category

Category/Occupancy = II (As per clause 1.5-1, ASCE 7-16)

8.5 Ground Acceleration

Acceleration for short period (Ss) =0.01g (As per ATC Hazard Tool)
Acceleration for 1 second period (S1) = 0.012g (As per ATC Hazard Tool)

8.6 Seismic Site Coefficient

Site Coefficient F_a = 1.6 (As per ATC Hazard Tool)
Site Coefficient F_v = 2.4 (As per ATC hazard Tool)

Table 11.4-1 Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_S.

Table 11.4-2 Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_1.

8.7 Seismic Coefficient

Design Spectral Response Coefficient S_{Ds} =0.011g (As per ATC hazard Tool)

Design Spectral Response Coefficient S_{D1} =0.0192g (As per ATC hazard Tool)

Response modification Factor (R) = 3¼ (Table 12.2-1 ASCE 7-2010)
(Steel ordinary concentrically braced frames)

Overstrength factor (Ω_0) =2 (Table 12.2-1 ASCE 7-2010)
(Steel ordinary concentrically braced frames)

Seismic Design Category =A (Table 11.6-1&2 ASCE 7-2010)

S_{MS} = F_a x S_s
= 1.6 x 0.01g
= 0.016g

S_{M1} = F_v x S_1
= 2.4 x 0.012g
= 0.0288g

S_{DS} = 2/3 x S_{MS}
= 2/3 x 0.016g
= 0.011g

S_{D1} = 2/3 x S_{M1}
= 2/3 x 0.0288g
= 0.0192g

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

Value of S_{DS}	Risk Category I or II or III	Risk Category IV
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.50 \leq S_{DS}$	D	D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

Value of S_{D1}	Risk Category I or II or III	Risk Category IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1}$	D	D

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R[a]	Overstrength Factor, Ω_0[b]	Deflection Amplification Factor, C_d[c]	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category B	C	D[e]	E[e]	F[f]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[g]	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls[g]	14.2	4	2½	4	NL	NL	40[i]	40[i]	40[i]
6. Ordinary precast shear walls[g]	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[g]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[g]	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP

LATERAL CALCULATION

Job: PATRICK DAILY 242201

PART II: Seismic Analysis

Seismic Load Calculations Using Equivalent Lateral Force Procedure per ASCE 7-10 Date: 12-02-2024

Seismic Weight Calculations

External Wall DL =	20	psf
Internal Wall DL =	15	psf
Roof area =	1569.98	sq ft

Ground to Roof level W calculations

Dead load of Roof =	30	psf	
Roof live load =	20	psf	
Height of external wall =	10.21	ft	
Height of Internal wall =	10.21	ft	
Perimeter of external wall =	158.45	ft	
Perimeter of Internal wall =	137.104	ft	
Dead load of wall (per ft2 area) =	33.98	psf	(Perimeter x Height of Wall / Area) x DL
Total Load =	84	psf	(25% Total live load)

Seismic Force Calculations

Site Class =	D		
Seismic Occupancy Category =	II		
Importance factor (I)=	1		
Mapped MCE spectral Response acceleration for short period S_S =	0.010 g		
Mapped MCE spectral Response acceleration for 1 second period S_1 =	0.012 g		
Site Coefficient Fa =	1.6		
Site Coefficient Fv =	2.4	(As per ATC hazard website Fv=2.4 considered)	
MCE spectral Response acceleration for short period SMS = Fa x SS =	0.016 g		
MCE spectral Response acceleration for 1 second period SM1 = Fv x S1 =	0.029 g		
Design spectral Response acceleration for short period SDS = 2/3 x SMS =	0.011 g		
Design spectral Response acceleration for 1 second period SD1 = 2/3 x SM1 =	0.019 g		

Approximate fundamental period calculations

Height of building =	11.51 ft		
C_t =	0.028	From table 12.8-2, ASCE 7-2010	
x =	0.8	From table 12.8-2, ASCE 7-2010	
$T_a = C_t \times h_n^x$ =	0.20 Sec	Approximate fundamental period "T_a"	Using Eq. 12.8-7
T_L =	8.00 Sec	From Fig 22-14, ASCE 7-2010	(Assumed)
Seismic Design Category (SDC) =	A	Table 11.6-1 ASCE 7-2010	
Response Modification Factor R =	3.25	Per table 12.2-1, ASCE 7-2010	
Maximum C_S (If $T_a < T_L$) = S_{D1} / (T x (R / I)) =	0.03	Per Eq 12.8-3, ASCE 7-2010	
Maximum C_S (If $T_a > T_L$) = $S_{D1} \times T_L / (T^2 \times (R / I))$ =	1.21	Per Eq 12.8-4, ASCE 7-2010	
Minimum C_S (If $S_1 < 0.6g$) =	0.01	Per Eq 12.8-5, ASCE 7-2010	
Minimum C_S (If S_1 => 0.6g) = 0.5 S_1 / (R / I) =	0.002	Per Eq 12.8-6, ASCE 7-2010	
Seismic Response Coefficient $C_S = S_{DS}/(R / I)$ =	0.003	Per Eq 12.8-2, ASCE 7-2010	OK
Seismic base shear $V = C_S \times W$ =	0.004 x W	ASCE 7 -10 Section 12.8.3	
Total weight of building (W_{Total}) =	131.9		
Base Shear	0.43	kips	
Over strength Factor (Ω) =	2		

Vertical Shear Force Calculations

Floor	Area of floor (ft^2)	Weight of floor Wx (kips)	Height of story above base hx (ft)	Wxhx	Seismic distribution factor (Wxhx/∑Wihi)	Storey Level Shear Force (kips)	Amplified Shear force at floor level ΩoV (kips)
Roof	1570.0	131.9	10.2	1346.21	1.00	0.43	0.87
Total		**131.85**	**10.21**	**1346.21**	**1.00**	**0.43**	**0.87**

Seismic forces are much lesser than the wind forces. Consequently, the wind forces are considered of more importance and governing for detailed design calculations throughout the document.

Refer "Appendix – 5. Seismic Calculations for Project" for detailed design calculations.

9 Gravity Design Loading

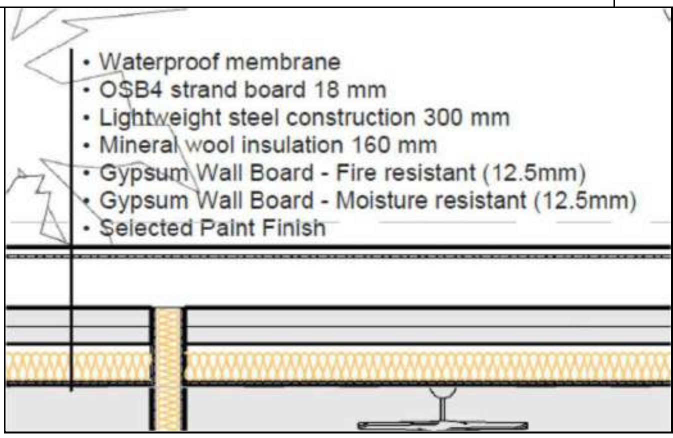
9.1 Roof Load

9.1.1 Roof live Load

Roof Live Load = 20.0 psf (non-accessible)

9.1.2 Roof Dead Load

Roof Load	Load (psf)
Waterproof membrane	0.1
18mm OSB4 Board	2.39
LGS Frame Truss	5.5
Miscellaneous	12.01
Total Roof Dead Load	**20.00**



9.2 Roof Ceiling Load

9.2.1 Roof Ceiling Dead Load

Roof Ceiling Load (Bottom Chord)	Load (psf)
160 mm Mineral Wool Insulation	3.60
12.5 mm Gypsum Moisture Board	2.00
12.5 mm Gypsum Fire Board	2.00
Miscellaneous	2.40
Total Roof Ceiling Dead Load	**10.00**

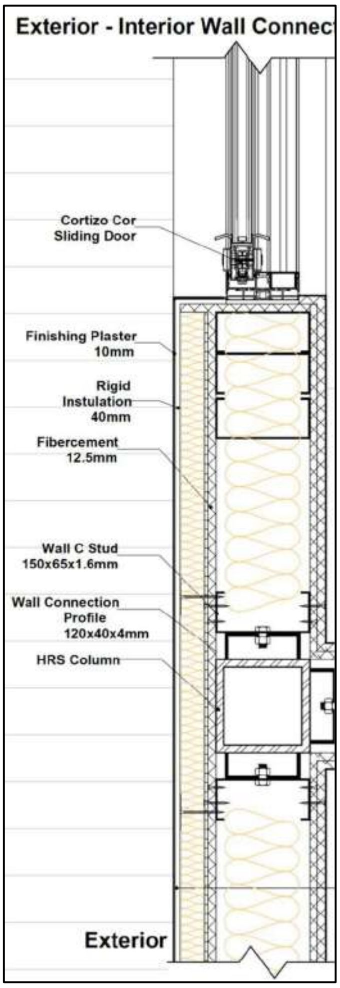
9.3 Wall Load

9.3.1 Exterior Wall Dead Load

Exterior Wall	Load (psf)
150 mm Light Gauge Steel Frame	3.50
150 mm Mineral Wool insulation	3.38
12.5 mm Gypsum Moisture board (Single layer)	2.00
12.5 mm Gypsum Fire board (Single layer)	2.00
12.5 mm Cement board Constructive Coating (Single layer)	3.28

40 mm Mineral Wool insulation	0.90
STO Cement-free Reinforcing coat with mesh and decorative render finish plaster	2.20
Miscellaneous	2.74
Total Exterior Wall Dead Load	**20.00**

Total Exterior Wall Dead Load = 20.00psf



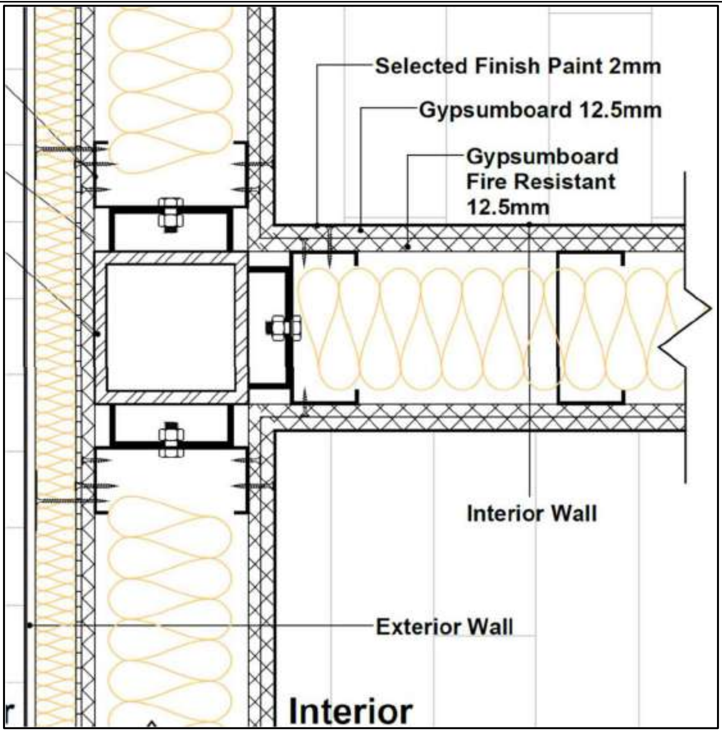
Interior side
- Selected Paint Finish
- Gypsum Wall Board - Moisture resistant (12.5mm)
- Gypsum Wall Board - Fire resistant (12.5mm)
- Light Gauge Steel Frame (150mm)
- Mineral Rockwool (150mm)
- Cementboard (12.5mm)
- Rockwool Insulation Panel 40mm (ETICS)
- STO Cement-free Reinforcing Coat with Mesh
- STO Decorative Render Finish Plaster

Exterior side

9.3.2 Interior Wall Dead Load

Interior Wall	Load (psf)
150 mm Light Gauge Steel Frame	3.50
150 mm Mineral Wool insulation	3.38
12.5 mm Gypsum Moisture board (Double layer)	4.00
12.5mm Gypsum Fire board (Double layer)	4.00
Miscellaneous	0.12
Total Interior Wall Dead Load	**15.00**

Total Interior Wall Dead Load = 15.00psf



Interior side
- Selected Paint Finish
- Gypsum Wall Board - Moisture resistant (12.5mm)
- Gypsum Wall Board - Fire resistant (12.5mm)
- Light Gauge Steel Frame (150mm)
- Mineral Rockwool (150mm)
- Gypsum Wall Board - Fire resistant (12.5mm)
- Gypsum Wall Board - Moisture resistant (12.5mm)
- Selected Paint Finish

Interior side

9.4 Snow Load- NA

10 Roll Former Machine specification.

C.150.50 & C.100.50 Studs can be produced with thicknesses up to 1.2mm (47.24 mil) & 1.5mm (59.05mil).

11 Material specification

11.1 Light gauge cold-formed steel

The walls panels and trusses are all made up of roll-formed cold rolled steel sections fabricated by certified manufacturers in the UK. Section properties in accordance with BS EN 10326:2004. Steel grade S450 GD + Z275.

C 100 -50- 1.2/1.5

C 150 -50- 1.2/1.5

Yield Strength of Light Gauge Steel (f_y) = 450 mPa = 65.27 ksi

Ultimate Strength of Light Gauge Steel (f_u) = 490 mPa = 71.10 ksi

11.2 Concrete

Min. grade of concrete slab is 3000 psi.

Min. grade of concrete piers is 5000 psi.

11.3 Reinforcing Steel (NA)

11.4 Hot Rolled Member

Material Availability of Hot Rolled Steel Sections is as per European/British sections with grade of S355 (A913Gr50) & Plate thickness 1/4" (6mm), 5/16" (8mm), 3/8" (10mm), 1/2" (12mm), 5/8" (15mm) and (13/16") 20mm of grade S355 (A913Gr50).

Yield Strength of Hot Rolled Steel (f_y) = 355 mPa = 51.49 ksi
Ultimate Strength of Hot Rolled Steel (f_u) = 470 mPa = 68.17 ksi

11.5 Connection

- Evolution Screws:

Screw designation	**Nominal Diameter**
# 10	0.189" (4.8 mm)
# 12	0.216" (5.5 mm)

TABLE: Type of Screw/Fastener/Bolt

Screw/Fastener/Bolt Specification	
CFS to CFS Connection	Screw 5.5 mm gauge
CFS to Ancillary Items	Screw 5.5/4.8/4.2 mm gauge
CFS to HRS	Screw 5.5 mm gauge

- Bolts are used as per American standards and their strengths are shown below:

Bolt of Class - 8.8 (A325/ A449)

Yield Strength of Bolt (f_{yb}) = 660mPa = 95.73 ksi
Ultimate Strength of Bolt (f_{ub}) = 830 mPa = 120.38 ksi

Bolt of Class –10.9 (A490/A354BD)

Yield Strength of Bolt (f_{yb}) = 940mPa = 136.33 ksi
Ultimate Strength of Bolt (f_{ub}) = 1040 mPa = 150.84 ksi

Bolt of Class –12.9 (A574)

Yield Strength of Bolt (f_{yb})	= 1100mPa	= 159.54 ksi
Ultimate Strength of Bolt (f_{ub})	= 1220 mPa	= 176.95 ksi

- Resin anchors for CFS/HRS to RC connection.
- Resin anchors for HRS to RC connection-HIT-HY 200 + HAS-U 8.8 HDG M16, M20 or M24.
- Mechanical Anchor for CFS to RC Connection- HUS4-HF 10 h_nom1.

11.6 Soil Data (NA)

12 Load Combination:

ASCE 7-16

(1) : 1.4D
(2) : 1.2D+1.6L+0.5Lr
(3) :1.2D+1.6L+0.5S
(4) :1.2D+1.6L+0.5R
(5) :1.2D+1.6Lr+L
(6) : 1.2D+1.6Lr+0.5W
(7) :1.2D+1.6S+L
(8) : 1.2D+1.6Lr+0.5W
(9) :1.2D+1.6R+L
(10):1.2D+1.6R+0.5W
(11): 1.2D+1L+1W+0.5Lr
(12): 1.2D+1L+1W+0.5S
(13): 1.2D+1L+1W+0.5R
(14): 1.2D+1E+L+0.2S
(15): 0.9D+1W
(16): 0.9D+1E

NOTATION:

D	=	Dead load
L	=	Live load, except roof live load, including any permitted live load reduction
Lr	=	Roof live load including any permitted live load reduction
S	=	Snow load
R	=	Rain load
W	=	Load due to wind pressure
E	=	Combined effect of horizontal and vertical earthquake induced forces as defined in Section 12.4.2 of ASCE 7-16

5.4.5.2 IBC 2012 LRFD, IBC 2015 LRFD, AND IBC 2018 LRFD

LC1	$0.42W_u$
LC2	$1.0G$
LC3	Max (1.0Q, 1.0S)
LC4	$1.2G + 1.6Q$
LC5	$0.9G + 1.0W_u$
LC6	$0.9G + 1.0W_w$
LC7	$0.9G + 1.0W_l$
LC8	$1.2G + 1.0W_w + 0.5$Max (Q,S)
LC9	$1.2G + 1.0W_l + 0.5$Max (Q,S)
LC10	$1.2G + 0.5W_w + 1.6$Max (Q,S)
LC11	$1.2G + 0.5W_w + 1.6$Max (Q,S)
LC12	$1.2G + 1.6S$ (if S>0)
LC13	$1.2G + 1.6 P_e$

5.2 DESIGN LOADS FOR TRUSSES

Loads used in Load combinations are defined as below:

Q	Live Load	=	Roof Live and Ceiling Live
G	Dead load	=	Roof dead and ceiling dead and Ceiling services
S	Snow Load	=	Basic design snow applied to the roof
Wu	Wind up		
Wd	Wind down		
Pt	Truss point load		

13 Criteria and Approach for Design of Framing for Gravity Loads

13.1 Design of Roof framing/Trusses

The structural elements found in roof framing, like roof joists, trusses are designed such that the members take the load coming onto them safely. Roof joist and trusses are designed using light gauge steel section.

13.2 Lintels

Lintels are placed over the openings, and it is subjected to self-weight sheathing load and lateral loads. Lintels are checked in bending, and deflection.

13.3 Load Bearing Wall Studs (NA)

13.4 Non-Load Bearing Wall Studs

These studs are provided at the non-load-bearing wall and shall be designed as a compression member by 'elastic theory of design'. These studs shall be designed for the nominal loads from the joist by considering the joist parallel to the wall and designed for 5 PSF lateral load for interior walls and wind loads for the exterior walls. These studs are made of light gauge structural steel.

13.5 Screw Specification – Self Drilling fasteners

Screw Designation	Nominal Dia.
# 10	0.189” (4.8 mm)
# 12	0.216” (5.5 mm)

13.6 Deflection Criteria

Wall

Walls with brittle finishes: wind load	=	L/240
Walls with flexible finishes: wind Load	=	L/120
Wall with stucco finish	=	L/360

Roof Truss

For non-suspended ceiling: Top chord

Wind Load	=	L/180 mm or 20 mm (0.787”)
Dead	=	L/300 mm or 20 mm (0.787”)
Live or snow	=	L/300 mm or 20 mm (0.787”)

For non-suspended ceiling: Bottom chord

Wind Load	=	L/250 mm or 15 mm (0.591”)
Dead	=	L/300 mm or 12 mm (0.472”)
Live or snow	=	L/300 mm or 12 mm (0.472”)

For suspended ceiling: Top chord & Bottom chord

Wind Load	=	L/180 mm or 30 mm (1.18”)
Dead	=	L/180 mm or 30 mm (1.18”)
Live or snow	=	L/180 mm or 30 mm (1.18”)

Floor

Total load	=	L/240
Live load	=	L/360

Roof

Live load	=	L/360
Wind Load	=	L/360
Total Load	=	L/240
Horizontal Sway Limit	=	L/240

TABLE 1604.3
DEFLECTION LIMITS[a, b, c, h, i]

CONSTRUCTION	L or L_r	S or W^f	$D + L^{d,g}$
Roof members:[a]			
Supporting plaster or stucco ceiling	$l/360$	$l/360$	$l/240$
Supporting nonplaster ceiling	$l/240$	$l/240$	$l/180$
Not supporting ceiling	$l/180$	$l/180$	$l/120$
Floor members	$l/360$	—	$l/240$
Exterior walls:			
With plaster or stucco finishes	—	$l/360$	—
With other brittle finishes	—	$l/240$	—
With flexible finishes	—	$l/120$	—
Interior partitions:[a]			
With plaster or stucco finishes	$l/360$	—	—
With other brittle finishes	$l/240$	—	—
With flexible finishes	$l/120$	—	—
Farm buildings	—	—	$l/180$
Greenhouses	—	—	$l/120$

13.7 CFS Connectors Specification:

All CFS connectors will be Simpson Strong Tie products. There will be a note to specify the use of any alternate connectors that have equal or greater capacity for Simpson. If required, custom connectors will be used.

13.8 HRS framing

Hot Rolled steel frame is used as main frame to take gravity loading from roof, floor, and wall etc. and design as a main frame to resist lateral load.

14 Typical Panel details

14.1 Transportation Limit

Transportation limit (Wall Panel)

Width	-	2.3 m (8.84’)
Length	-	12.0 m (39.37’)

Temporary dummy members to be provided as per requirement for panel handling. Trusses size and span are as per design.

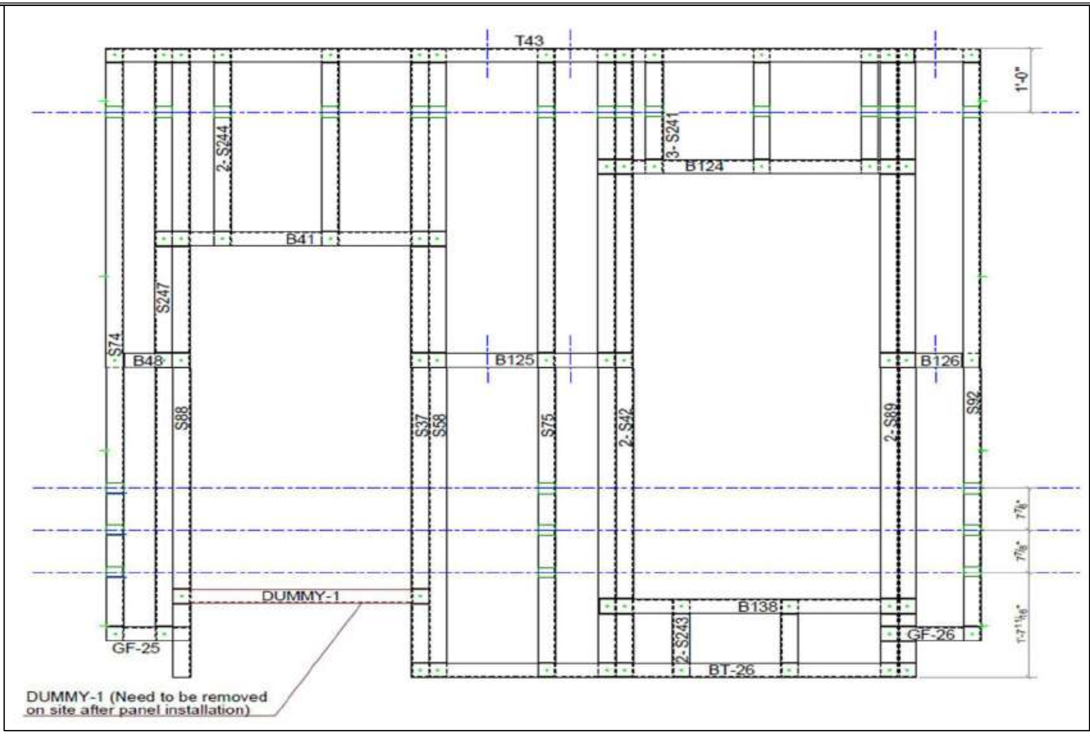
14.2 Service holes and Nogging

Service Holes

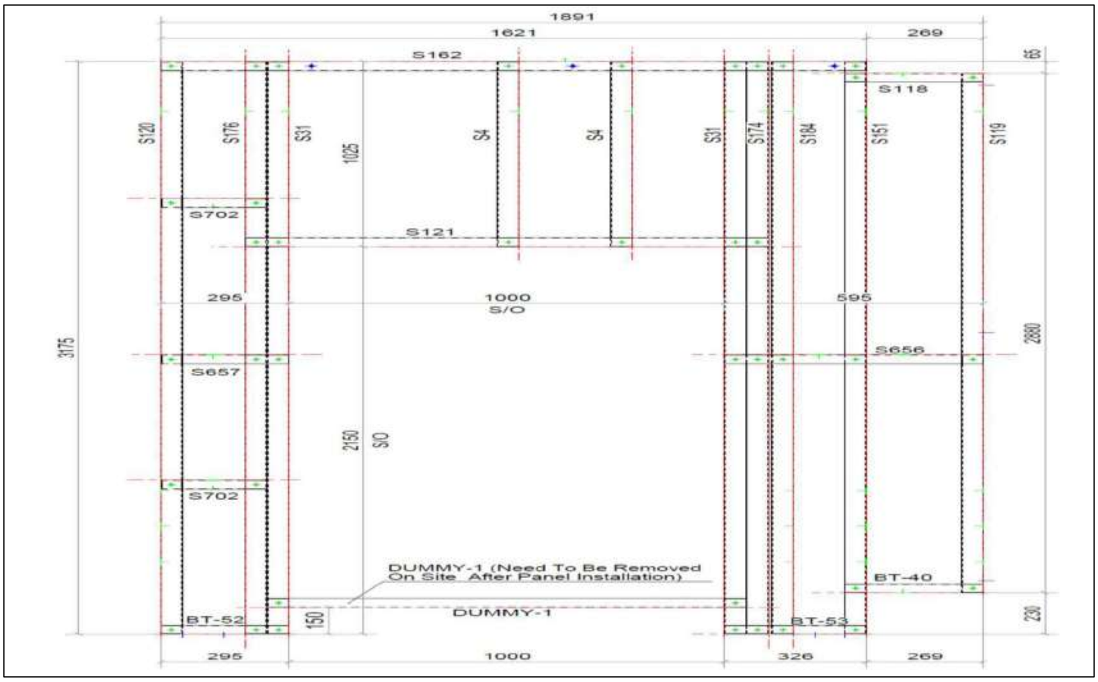
Location of First service hole from Bottom	=	610 mm (24”)
Service Hole Spacing	=	200 mm (7.87”)

Nogging

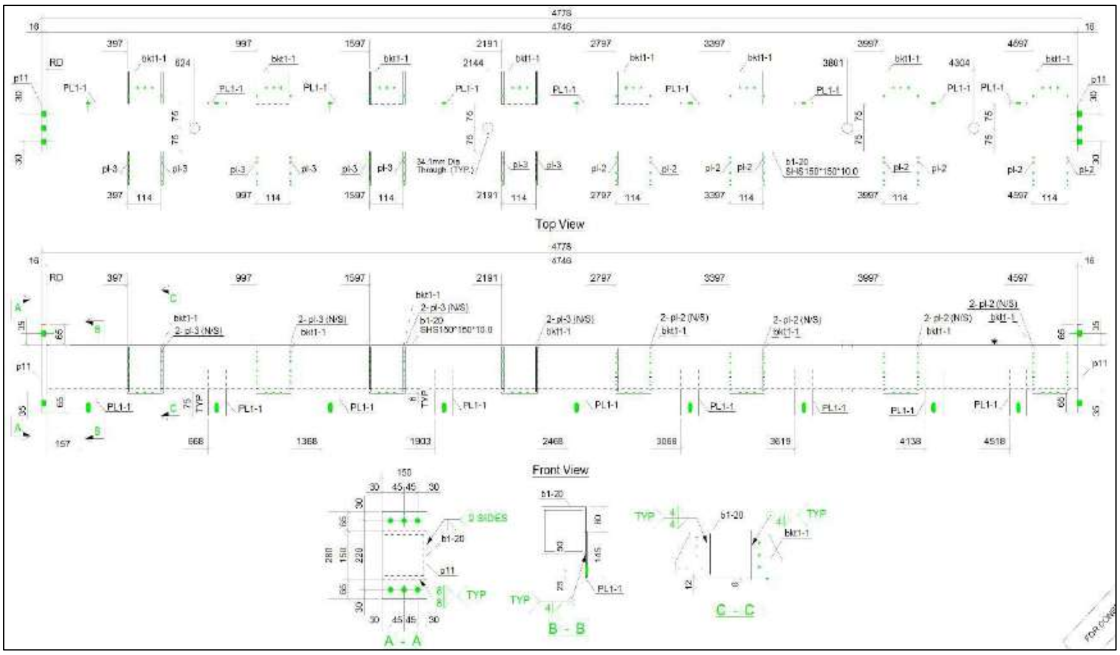
Nogging type	=	Hit & Miss Nogging
Noggins at	=	1.5 m (4.92’) above SSL

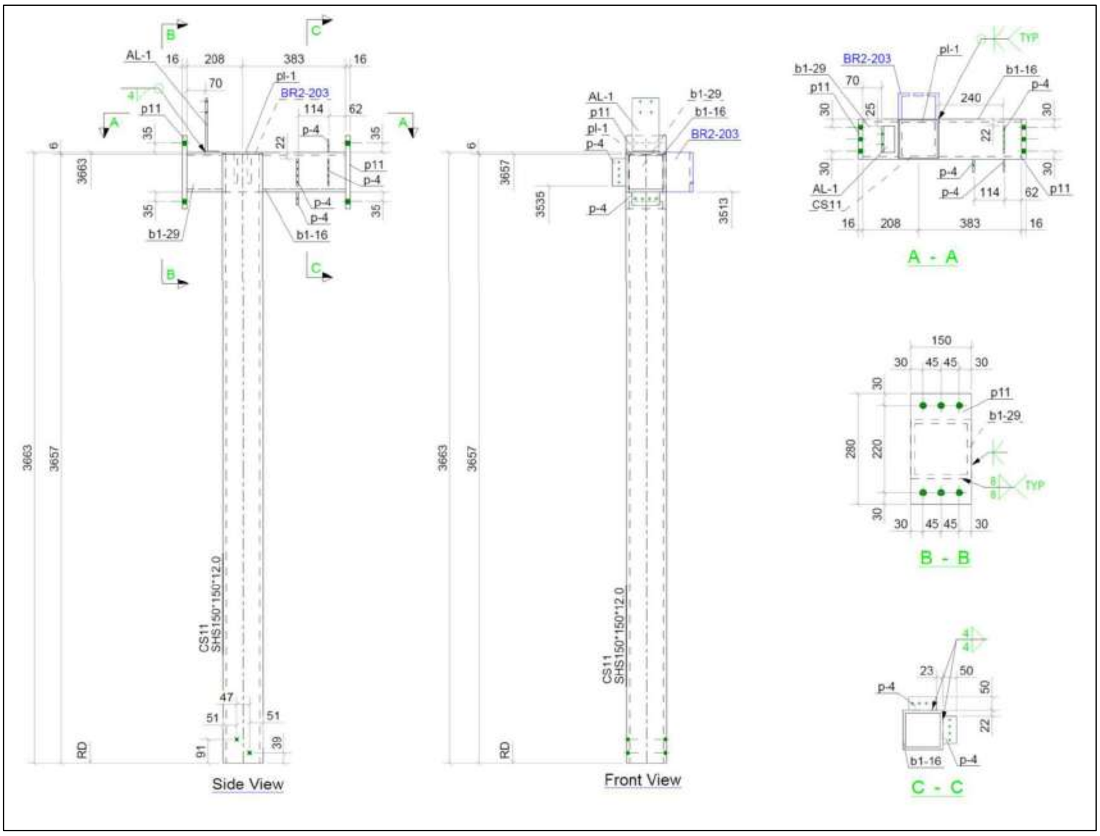


14.3 CFS Shop drawings example:



14.4 HRS fabrication drawings example:





15 Design of CFS and HRS

15.1 Design of Roof Joist – (RJ):

Max Span of Roof Truss	=	19.45’	=	5930 mm
Roof Truss spaced at 2’-0” O.C. Max				
Dead Load on Top chord Roof Truss	=	20 psf	=	0.96 kPa
Live Load on Top chord of Roof Truss	=	20 psf	=	0.96 kPa
Bottom Chord Dead Load	=	10 psf	=	0.48 kPa
Wind Speed	=	200 mph	=	89.408 m/s

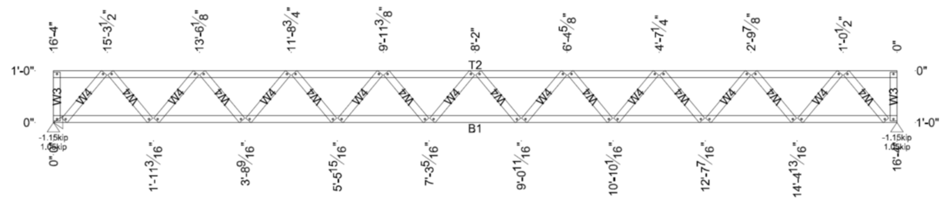
Provide 300 mm deep joists made up of Section C.100.50.15 of grade 450S + Z275 at 2’-0” (600 mm) O.C.

Note: - The section 100S41 – 155 - 350 used in the Framecad Report is equivalent to C.100.50.15 of grade 450S + Z275 sections. Also considering utilization ratio used section is ok.

Refer “Appendix – 6. Design of Roof Joist - RJ” for detailed summary.

B1 100S41-155-350	1	16'-4"	T2 100S41-155-350	1	16'-4"	W3 100S41-155-350	2	11 1/4"	W4 100S41-155-350	18 1'-2 3/8"
FRAMECAD 10g-19mm XDrive	80		Working Sheet: 16' feet			Assembly Weight		89.8lbs		

Powered by FRAMECAD Structure ® Minimum number of fasteners required is 2 per joint



Quantity Required = 1 Mark as RJ2 Engineering Status = 52%

System Name:	FRAMECAD_FT_m	Roof Type:	SHEET	Truss Model:	Default	Wind Speed:	200
Truss Pitch:	0	Truss Spacing:	2'-0"	Design Code:	AISI S100-20 LRFD	Loading Code:	IBC 2018 LRFD
Top Chord Live Load (psf):	20.0	Top Chord Dead Load (psf):	20.0	Bottom Chord Dead Load (psf):	10.0	Number of on-coming Trusses:	0
Envelope:	1'-1/16"h x 16'-4"w						

ides global Dwg: 242201_Patrick Daily roof truss View 1 of 1 Client: Patrick Daily Job No: 242201

15.2 Design of external non load bearing wall (ST-1):

For maximum span and loading

Clear Height	=	10.21’	
DL	=	Wall Load	
	=	(20 x 10.21 x 2)	
	=	408.4 lb	
Load case	=	1.4DL	
	=	1.4 x 408.4	
	=	571.76 lb	
Wind pressure	=	124.62 psf (External Wind Pressure)	
Capacity of stud	=	2032 lb	> 571.76 lb
			Hence Safe….

Provide CFS stud wall made up of section C 150-50-15 of grade 450S + Z275 @ 2’-0” (600 mm) O.C. spacing using Hit & Miss - mid-point noggin @ 1.5m above SSL.

Refer “Appendix - 7A. ST-1 External Wall” for detailed summary.

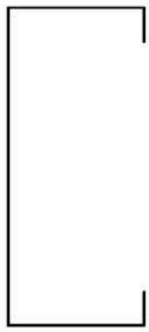
ClarkWestern Building Systems
CW Tech Support: (888) 437-3244
clarkwestern.com

2007 North American Specification LRFD
DATE: 31-01-2024

SECTION DESIGNATION: Single

Section Dimensions:

Web Height =	5.905 in
Top Flange =	1.969 in
Bottom Flange =	1.969 in
Stiffening Lip =	0.590 in
Inside Corner Radius =	0.0590 in
Punchout Dia. =	1.340 in
Design Thickness =	0.0590 in



Steel Properties:

Fy =	65.270 ksi
Fu =	71.100 ksi
Fya =	65.270 ksi

COMBINED AXIAL AND BENDING LOADS

INPUT PARAMETERS
Overall Wall Height = 10.21
Lateral Load = 124.62 psf
Load Factor for Lateral Load = 1.00
Lateral Load multiplied by 0.70 for deflection calculations
Studs Considered Fully Braced for Bending

K-phi (flexure) for Distortional Buckling = 0.00 lb*in/in
K-phi (axial) for Distortional Buckling = 0.00 lb*in/in

MAXIMUM FACTORED AXIAL LOADS (lb)

		SPACING		Maximum
BRACING	12 in	16 in	24 in	KL/r
NONE	2172	1698	770	171
MID Pt	5936	4583	2032	86
THIRD Pt	7548	5798	2551	57
SHEATH 2 SIDES	N/A	N/A	N/A	53
DEFLECTION	L/558	L/419	L/279	

Note: Axial loads for sheathing braced design are based on the North American Standard for Cold-Formed Steel Framing - Wall Stud Design, 2007 Edition with 1/2 inch gypsum sheathing and No. 6 fasteners max 12 inches on center

15.3 Design of Internal load bearing wall (ST-2)

For maximum span and loading,

Wind Pressure	=	5 psf
Clear Height	=	10.21’
Tributary width	=	2’

A. Dead Load Calculations

1.Wall dead load	=	Wall Load x Wall height x Stud Spacing
	=	15 psf x 10.21’ x 2’
	=	306.3 lb
2. Dead load from truss	=	Load x Tributary Width x Stud Spacing
	=	30 psf x 2’ x 2’
	=	120 lb
Total dead load	=	306.3 + 120
	=	426.3 lb

B. Live Load Calculations

1. Live load from truss	=	Load x Tributary Width x Stud Spacing
	=	20 psf x 2’ x 2’
	=	80 lb
Total live load	=	80 lb

C. Load Combination Without Wind

Load case 1	=	1.4DL
	=	1.4 X 426.3
	=	596.82 lb
Load case 2	=	1.2DL + 1.6RLL + 1FLL
	=	1.2 x 426.3 + 1.6 x 80
	=	639.56 lb
Load case 3	=	1.2DL + 1.6FLL + 1RLL
	=	1.2 x 426.3 + 1.0 x 80
	=	591.56 lb

D. Load Combination With Wind

0.5 WL Load case	=	1.2DL + 1.6RLL + 0.5WL
	=	1.2 x 426.3 + 1.6 x 80
	=	639.56 lb
1 WL Load case	=	1.2DL + 1FLL + 0.5RLL + 1WL
	=	1.2 x 426.3 + 0.5 x 80
	=	551.56 lb
Critical load case	=	639.56 lb
Capacity of stud	=	6232 lb > 639.56 lb

Provide CFS wall made up of section C 150-50-12 of grade 450S+Z275 @ 2’-0” (600mm) O.C. spacing using Hit & Miss - mid point nogging @1.5M above SSL.

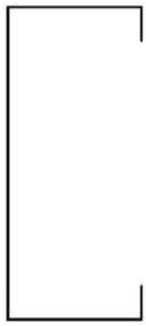
Refer “Appendix - 7B. ST-2 Internal Wall” for detailed summary.

ClarkWestern Building Systems
CW Tech Support: (888) 437-3244
clarkwestern.com

2007 North American Specification LRFD
DATE: 31-01-2024

SECTION DESIGNATION: Single

Section Dimensions:
Web Height = 5.905 in
Top Flange = 1.969 in
Bottom Flange = 1.969 in
Stiffening Lip = 0.429 in
Inside Corner Radius = 0.0590 in
Punchout Dia. = 1.340 in
Design Thickness = 0.0472 in



Steel Properties:
Fy = 65.270 ksi
Fu = 71.100 ksi
Fya = 65.270 ksi

COMBINED AXIAL AND BENDING LOADS

INPUT PARAMETERS
Overall Wall Height = 10.21
Lateral Load = 5.0 psf
Load Factor for Lateral Load = 1.00
Lateral Load multiplied by 0.70 for deflection calculations
Studs Considered Fully Braced for Bending

K-phi (flexure) for Distortional Buckling = 0.00 lb*in/in
K-phi (axial) for Distortional Buckling = 0.00 lb*in/in

MAXIMUM FACTORED AXIAL LOADS (lb)

BRACING	12 in	SPACING 16 in	24 in	Maximum KL/r
NONE	2404	2384	2342	178
MID Pt	6413	6352	6232	89
THIRD Pt	8031	7952	7796	59
SHEATH 2 SIDES	N/A	N/A	N/A	53
DEFLECTION	L/10623	L/7967	L/5311	

Note: Axial loads for sheathing braced design are based on the North American Standard for Cold-Formed Steel Framing - Wall Stud Design, 2007 Edition with 1/2 inch gypsum sheathing and No. 6 fasteners max 12 inches on center

15.4 Design of HRS Framing Using STAAD:

Design loading:

Roof Load

Dead Load	=	30 psf
Live Load	=	20 psf

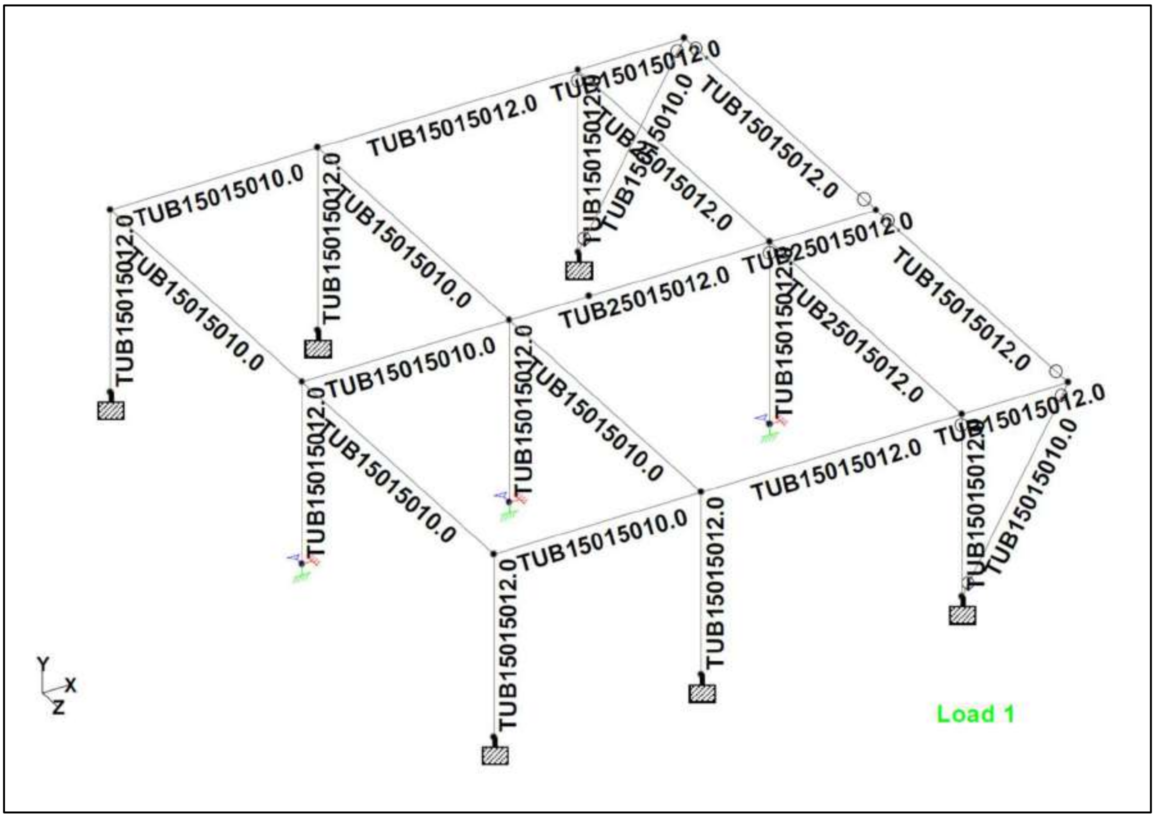
Wall Load

External wall Dead load	=	20 psf
Internal wall Dead load	=	15 psf

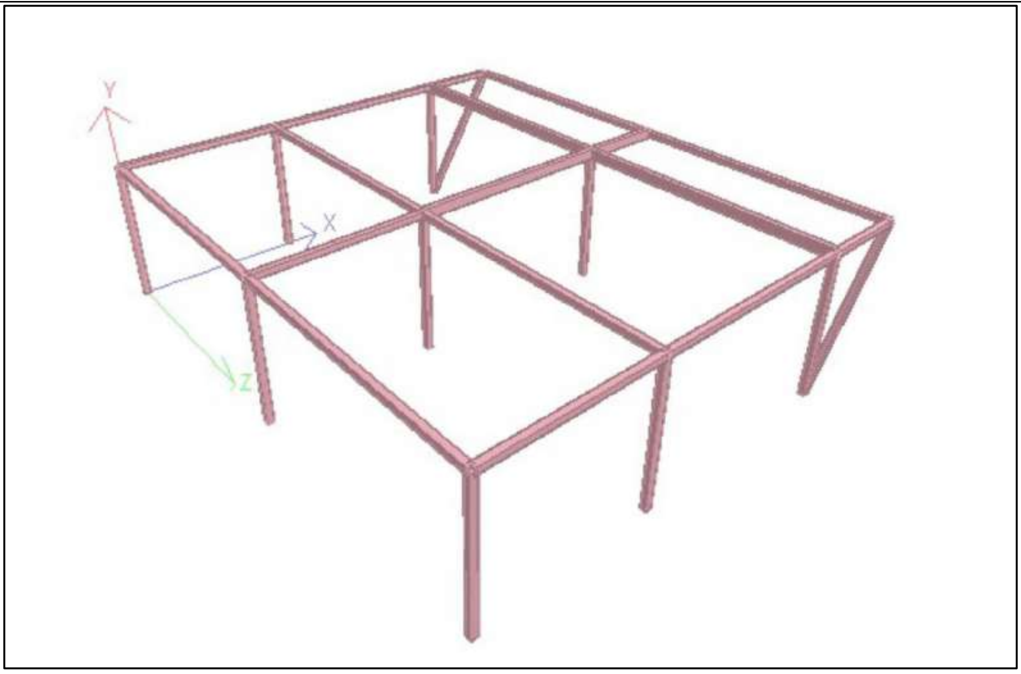
Wind Load:

For lateral load resistance HRS Portal Frame and Strap Bracings are used.

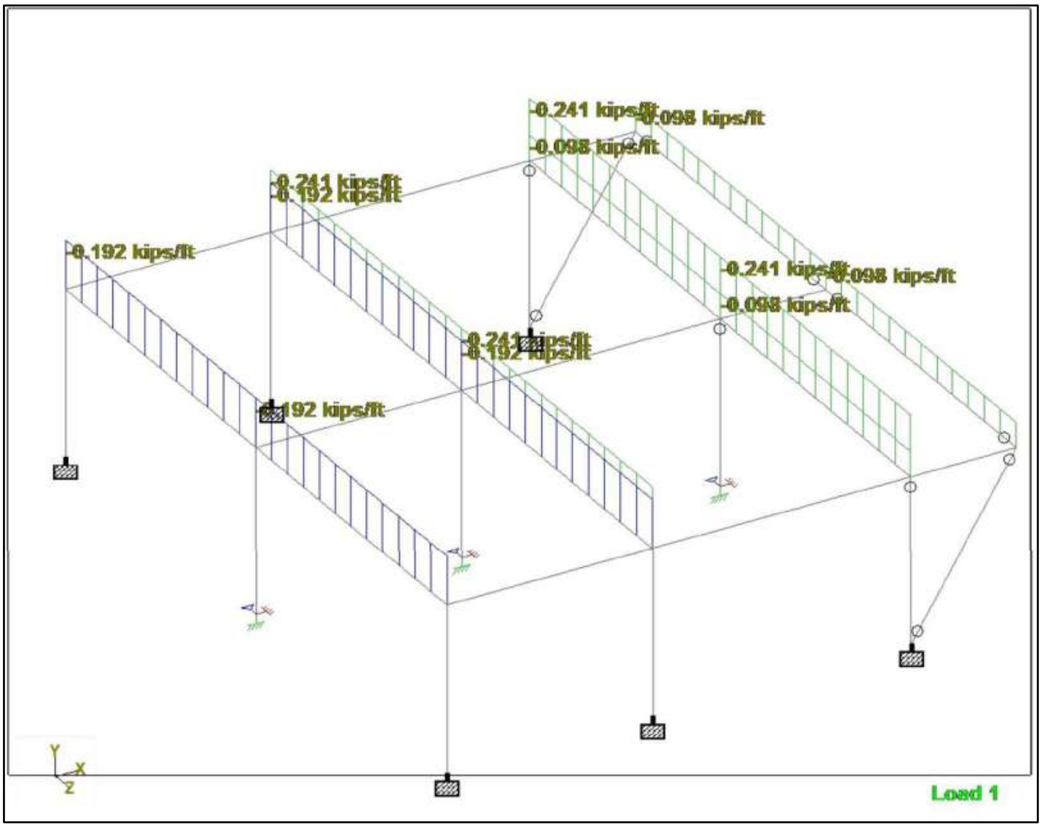
Refer "Appendix – 8. Exuma, Bahamas Lateral Calc._120224" for a detailed summary.



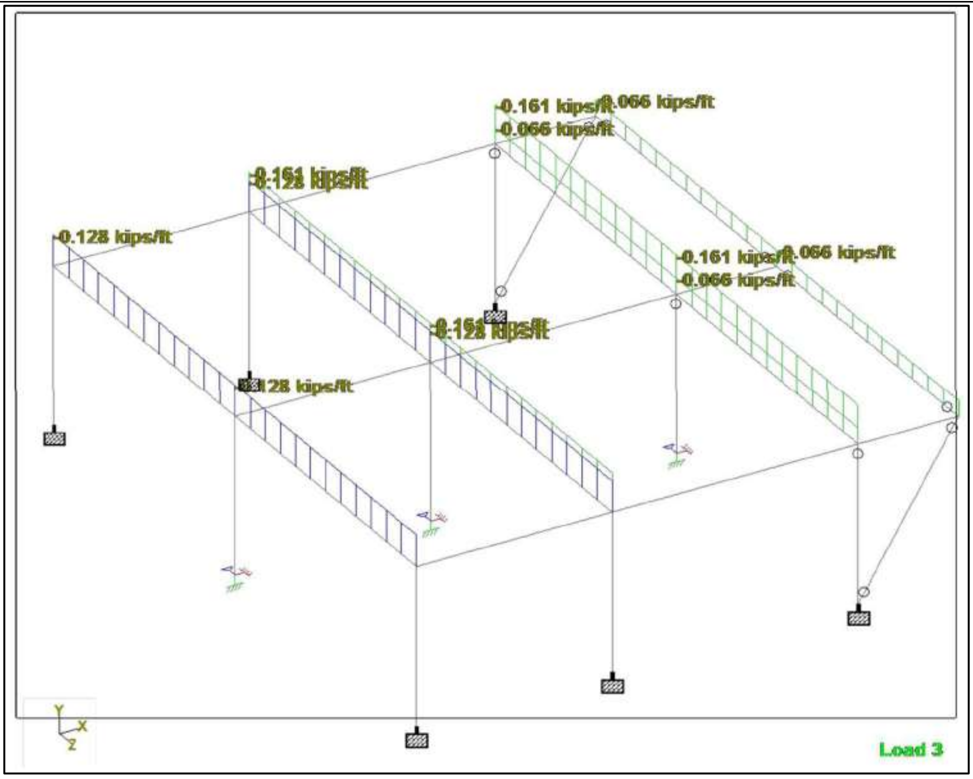
Section Properties



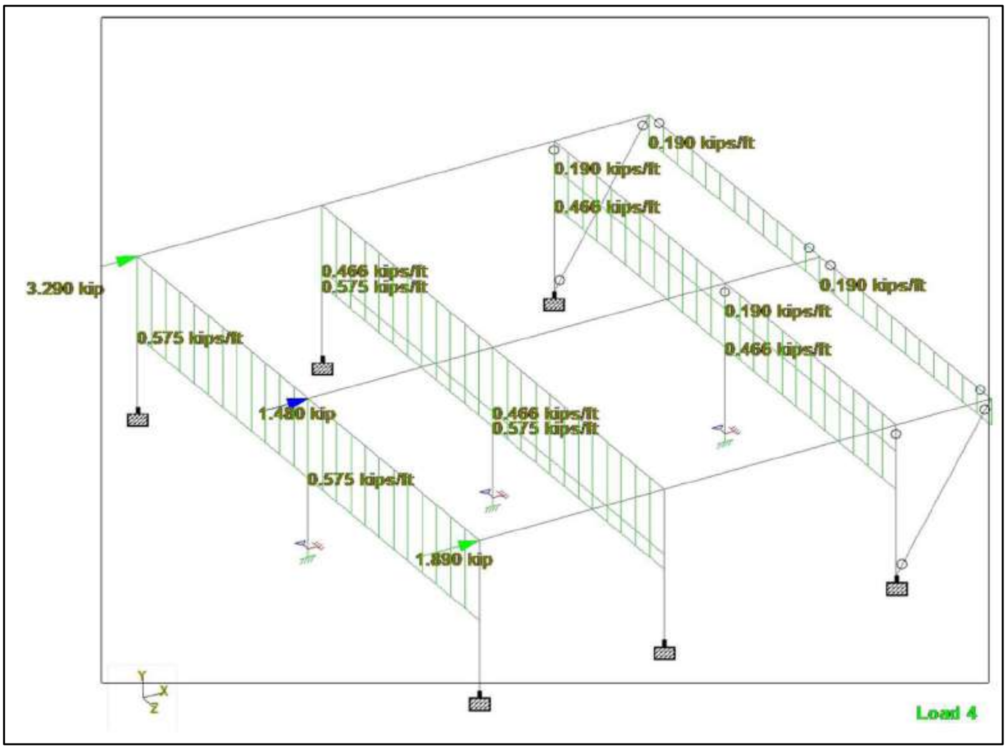
3-D Model



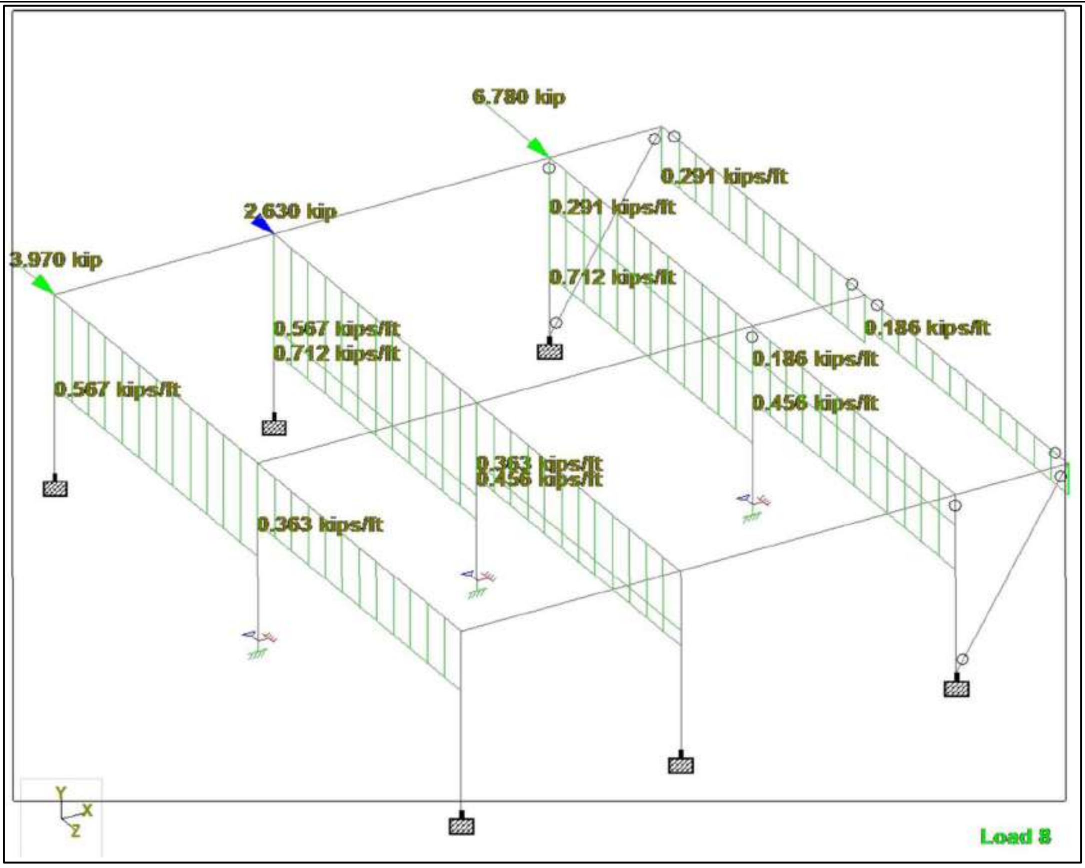
Dead Load



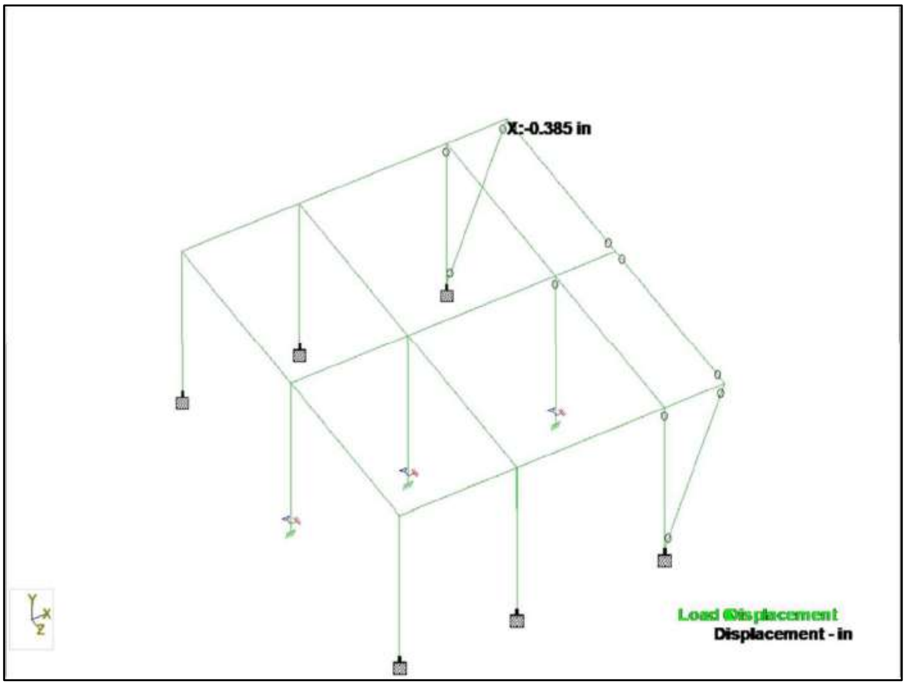
Roof Live Load



Wind Load (X direction)

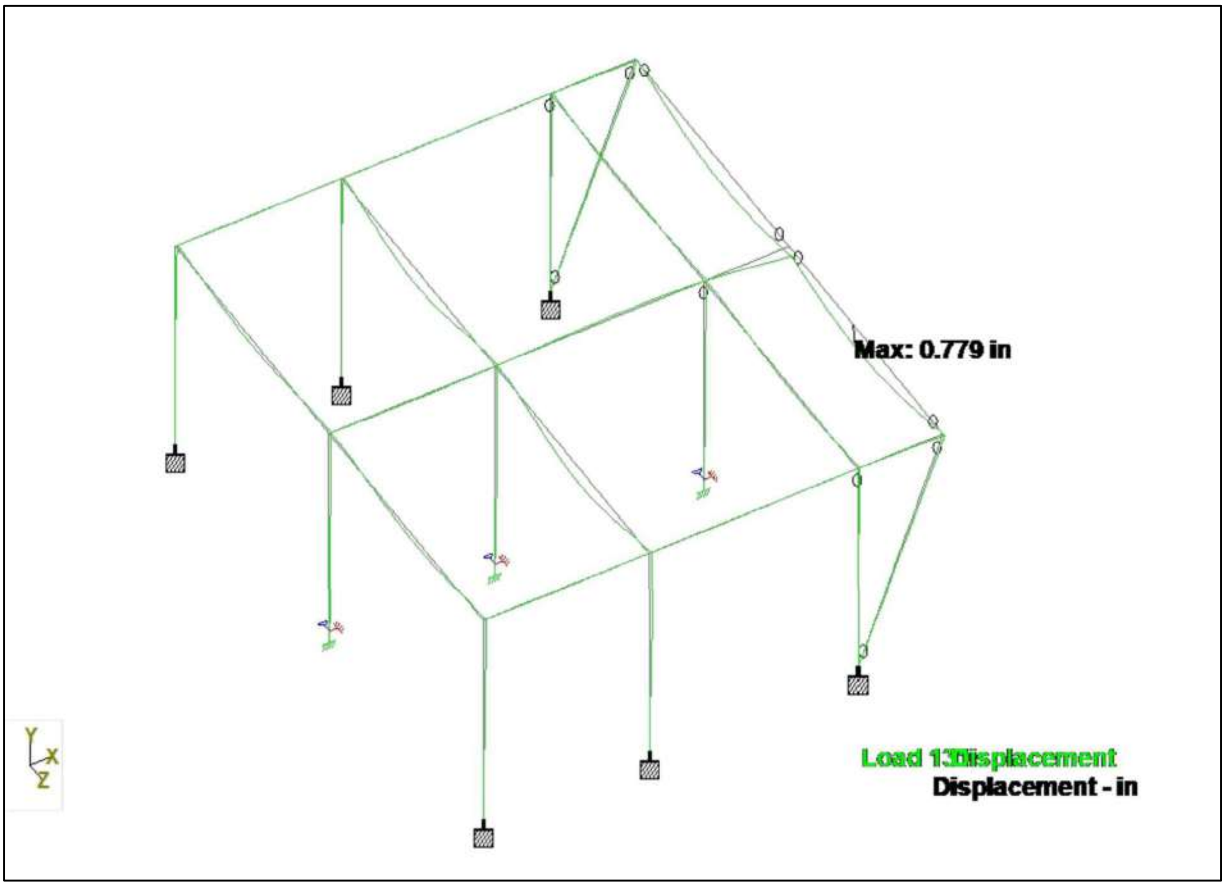


Wind Load (Z direction)



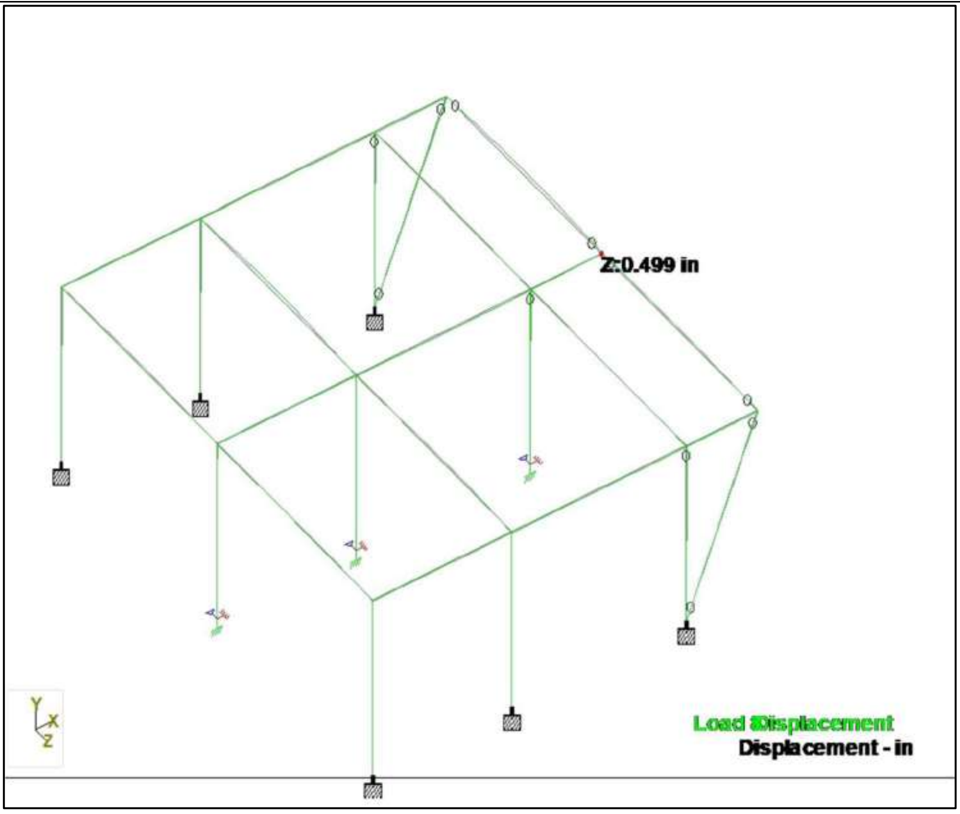
Maximum Lateral Deflection in X Direction

Maximum lateral deflection in X-Direction	=	0.385"
Allowable lateral deflection	=	10.21'/240
	=	0.0425'
	=	0.51"
Utilization Ratio	=	0.385 /0.51
	=	0.75



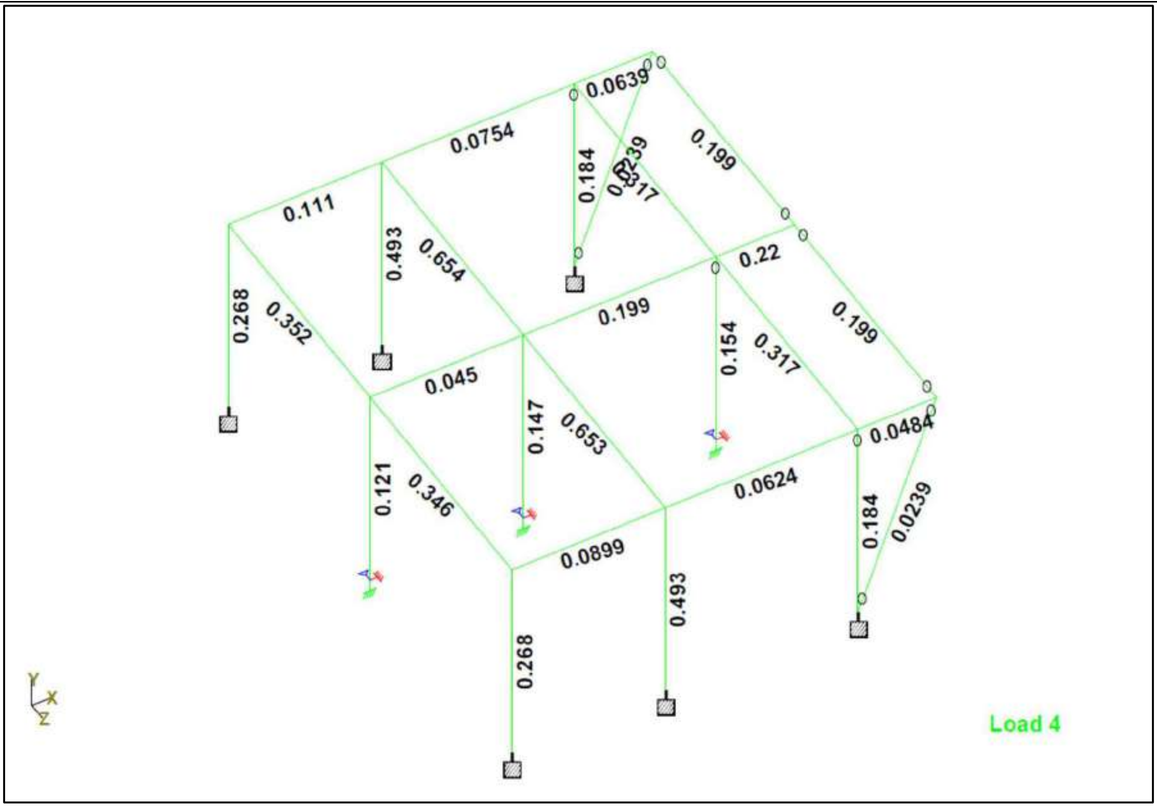
Maximum Deflection in Vertical Direction

Maximum vertical deflection	=	0.779"	
Allowable deflection	=	20.5'/240	
	=	0.0854'	
	=	1.025"	
Utilization Ratio	=	0.779/1.025	
	=	0.76	**….. Hence Ok**



Maximum Lateral Deflection in Z Direction

Maximum deflection	=	0.499”
Allowable lateral deflection	=	10.21’/240
	=	0.0425’
	=	0.51”
Utilization Ratio	=	0.499/0.51
	=	0.98



Utilization Ratio

Refer "Appendix – 8A. STAAD Report HRS Frame Design" for detailed summary.

16 Opening Design

16.1 Door and Window Lintel:

The maximum unsupported length of door Lintel of section size C.150.50.15 is 6.562 ft long.

For 6.562’ unsupported length

Maximum lateral pressure on lintel due to wind load is calculated using components and cladding design method. = 124.62 psf

Maximum height of opening = 7.544’

wall height above opening = 2.67’

Wind load UDL on lintel = (124.62 x (7.544+2.67)/2)/1000

= 0.64 kips/ft

Lateral Load multiplied by 0.7 for deflection calculation:

= 0.64 x 0.7

= 0.45 kips/ft

Moment of Inertia (I) and the bending capacity of member is considered from Section Property:

I = 1372606 mm^4
= 1.59×10^{-4} ft^4

Capacity = 5943171 N.mm
= 4.38 kips-ft

lateral deflection of lintel = $5wl^4/384*(EI)$

= $5x\ (0.45)\ x\ (6.562)^4\ /\ (384x4385941x\ 1.59x10^{-4})$

= 0.01557’

= 0.19“

Allowable limit = span/240

= 6.562/240

= 0.0273’

= 0.328” > 0.19”**....Hence Ok.**

Maximum bending moment at center of lintel = $wl^2/8$

= $0.64\ x\ (6.562)^2/8$

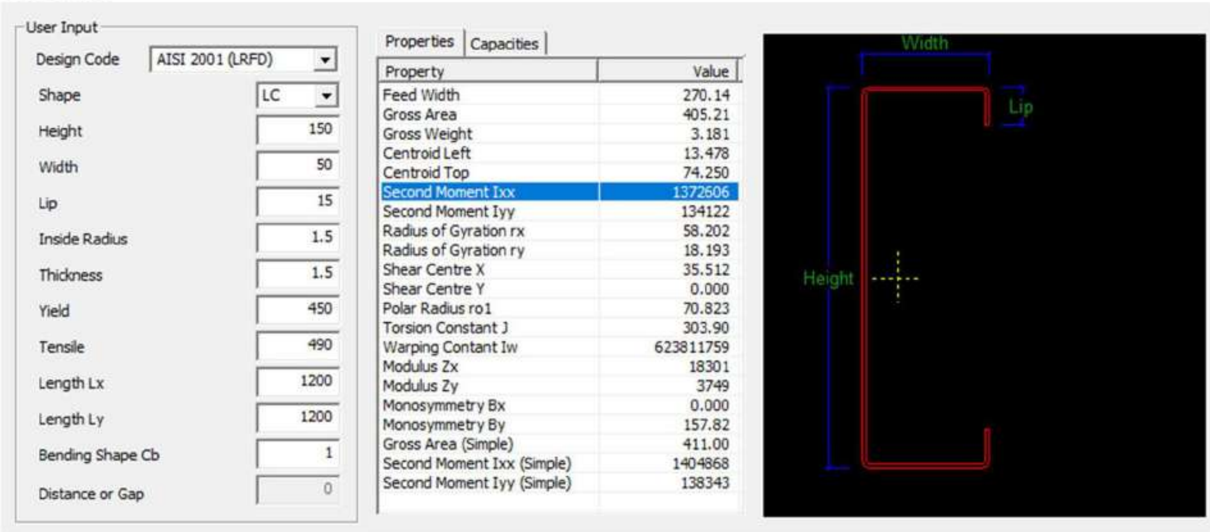
= 3.45 kips-ft

Bending capacity of C-150-50-15 = 4.38 kips-ft > 3.45 kips-ft

....Hence Ok.

Provide Door and Window lintel made up of C-150-50-15 at all locations.

SectionCalc



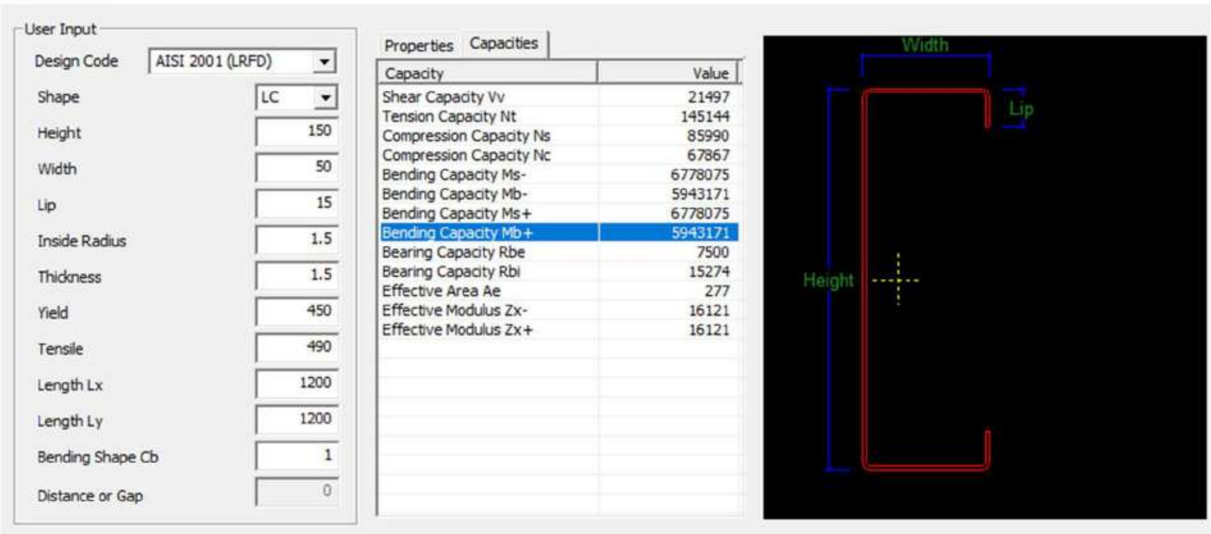
User Input

Design Code	AISI 2001 (LRFD)
Shape	LC
Height	150
Width	50
Lip	15
Inside Radius	1.5
Thickness	1.5
Yield	450
Tensile	490
Length Lx	1200
Length Ly	1200
Bending Shape Cb	1
Distance or Gap	0

Properties | Capacities

Property	Value
Feed Width	270.14
Gross Area	405.21
Gross Weight	3.181
Centroid Left	13.478
Centroid Top	74.250
Second Moment Ixx	1372606
Second Moment Iyy	134122
Radius of Gyration rx	58.202
Radius of Gyration ry	18.193
Shear Centre X	35.512
Shear Centre Y	0.000
Polar Radius ro1	70.823
Torsion Constant J	303.90
Warping Contant Iw	623811759
Modulus Zx	18301
Modulus Zy	3749
Monosymmetry Bx	0.000
Monosymmetry By	157.82
Gross Area (Simple)	411.00
Second Moment Ixx (Simple)	1404868
Second Moment Iyy (Simple)	138343

SectionCalc



User Input

Design Code	AISI 2001 (LRFD)
Shape	LC
Height	150
Width	50
Lip	15
Inside Radius	1.5
Thickness	1.5
Yield	450
Tensile	490
Length Lx	1200
Length Ly	1200
Bending Shape Cb	1
Distance or Gap	0

Properties | Capacities

Capacity	Value
Shear Capacity Vv	21497
Tension Capacity Nt	145144
Compression Capacity Ns	85990
Compression Capacity Nc	67867
Bending Capacity Ms-	6778075
Bending Capacity Mb-	5943171
Bending Capacity Ms+	6778075
Bending Capacity Mb+	5943171
Bearing Capacity Rbe	7500
Bearing Capacity Rbi	15274
Effective Area Ae	277
Effective Modulus Zx-	16121
Effective Modulus Zx+	16121

16.1. Wind Post Design:

Wind post is provided at Center of lintel i.e., 3.608’ (1100 mm)

Wind posts unsupported length 2.67’ (765 mm)

Wind load UDL on lintel = (124.62 x (7.544+2.67)/2)/1000
= 0.64 kips/ft

Point load on HRS Wind Post from Lintel = 0.64 x (3.608+3.608)/2
= 2.31 kips

Moment of Inertia (I) and the bending capacity of member is considered from Section Property:

I = 11731271 mm^4
= 1.35 x 10^{-3} ft^4

Bending Capacity = 49975704 N.mm
= 36.85 kips-ft

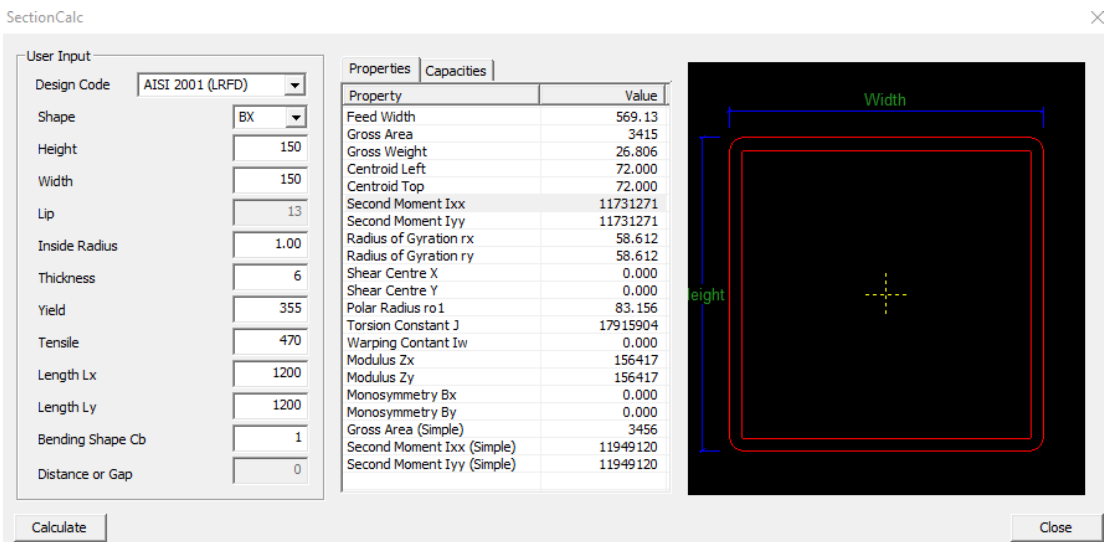
Lateral deflection due to point load
on cantilever wind post = $wl^3/3EI$
= $(2.31) \times (2.67)^3 / (3 \times 4385941 \times 1.35 \times 10^{-3})$
= 0.00247‘
= 0.0297“

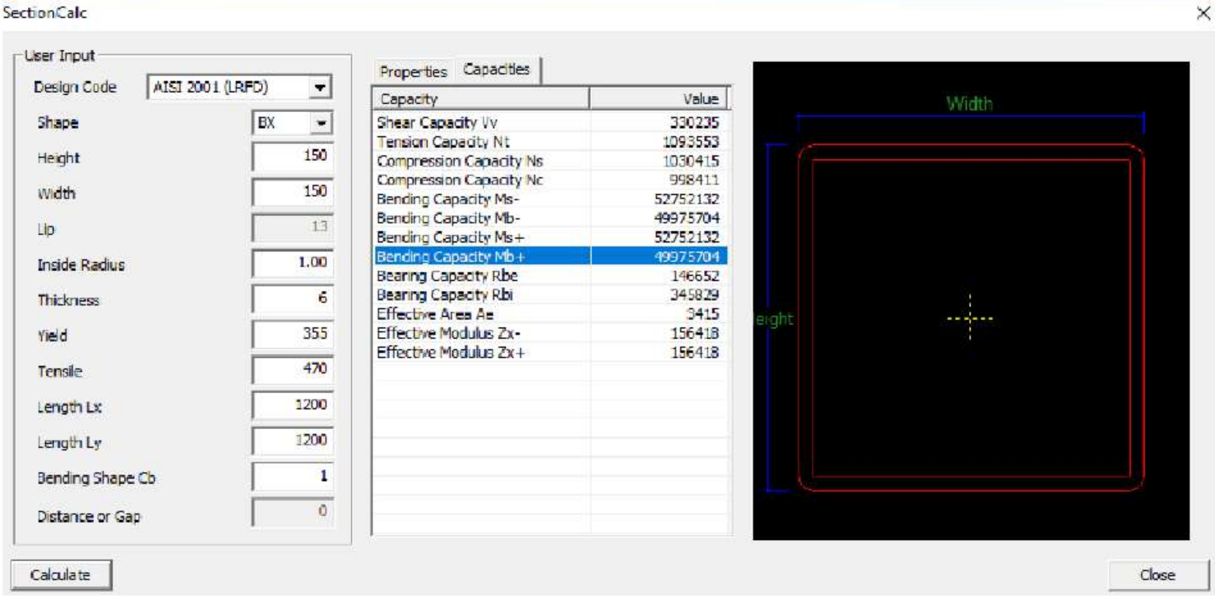
Allowable maximum lateral deflection = 2.67’/180
= 0.015’
= 0.18” >0.0297”….**Hence Ok.**

Maximum bending moment at the
base of wind post = wl
= 2.31 x 2.67
= 6.2 kips-ft

Moment carrying capacity of SHS 150x150x6 = 36.85 kips-ft > 6.2 kips-ft **Hence Ok.**

Provide SHS 150x150x6 as wind posts (1) no. at center of lintel (for opening above 2m).





16.2 Jamb:

For 10.21’ unsupported length

Maximum lateral pressure on jamb due to wind load is calculated using components and cladding design method. = 104.63 psf

Lateral load on jamb due to opening & adjacent stud:

= (104.63 x (4.92)/2)/1000

= 0.258 kips/ft

Lateral Load multiplied by 0.7 for deflection Calculation:

= 0.7 x 0.258

= 0.18 kip/ft

The moment of Inertia (I) and the bending capacity of the member is considered from Section Property:

I = 1372606 mm^4

= 1.59×10^{-4} ft^4

Bending Capacity (M) = 5943171

= 4.38 kips-ft

Maximum deflection of jamb = $5wl^4/384(EI)$

= $5x\ (0.18)\ x\ (10.21)^4\ /\ (384x4385941x\ 1.59x10^{-4})$

= 0.0365’

= 0.44“

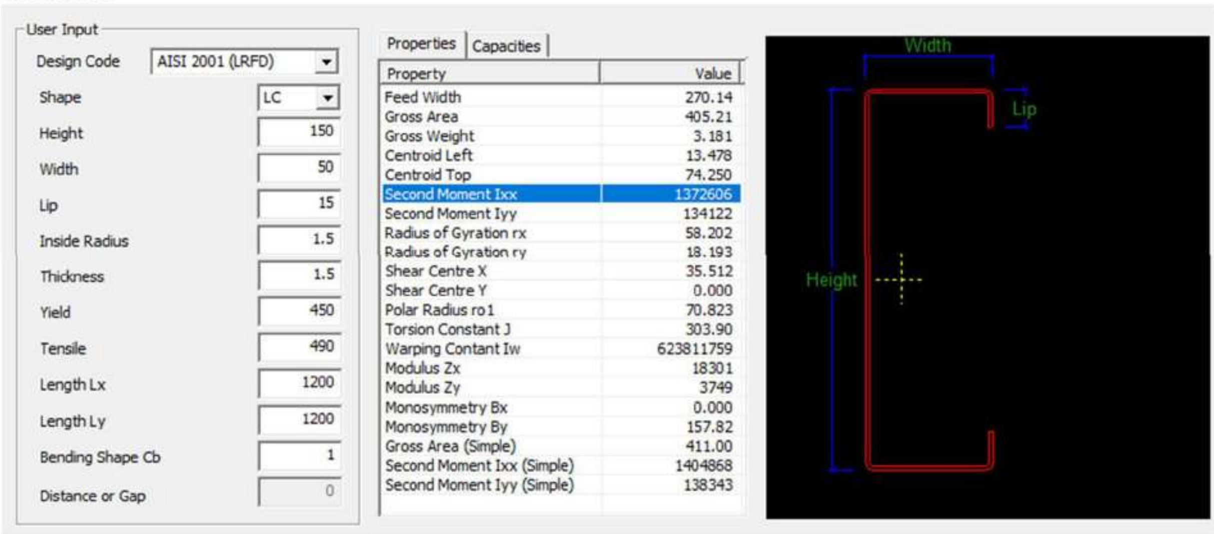
Allowable limit	=	span/240	
	=	10.21’ /240	
	=	0.0425’	
	=	0.51”	> 0.44” **…. Hence Ok.**
Maximum bending moment at center of Jamb	=	$wl^2/8$	
	=	$0.258x\ (10.21)^2/8$	
	=	3.362 kips-ft	
Bending capacity of C-150-50-15	=	4.38 kips-ft	> 3.362 kips-ft

…. Hence Ok.

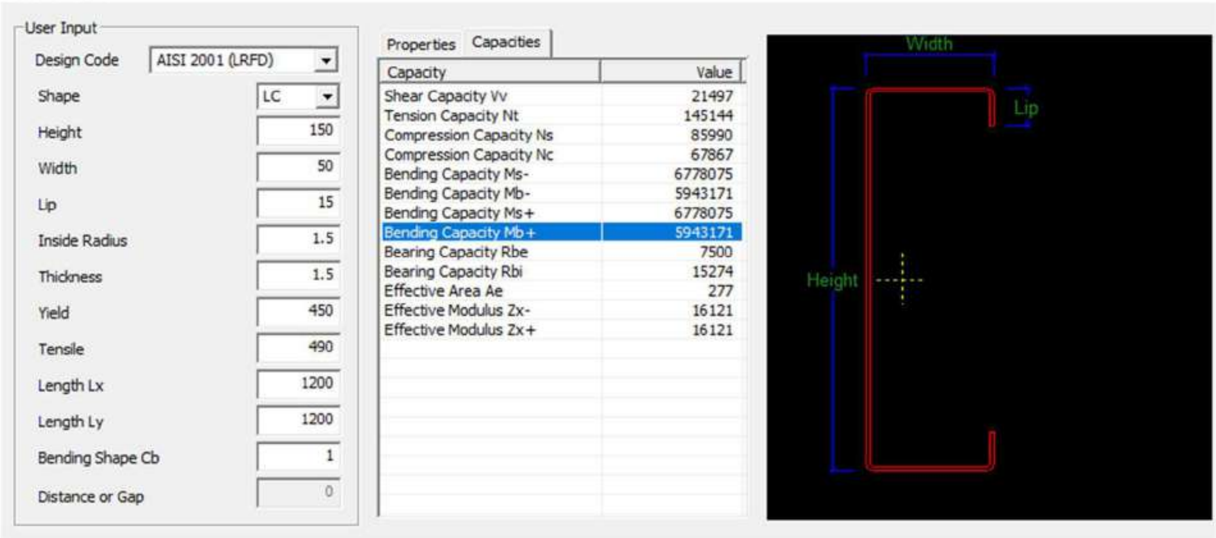
Provide single jamb made up of C-150-50-15 at all location opening.

For opening width more than 4.92’ provide (2) C-150-50-15 at jamb.

SectionCalc



SectionCalc



16.3 Cill Design:

For opening of maximum width 8.528’

For 8.528’ (2600 mm) unsupported length

Maximum lateral pressure on the lintel due to wind load is calculated using the components and cladding design method. = 124.64 psf

consider the maximum height of the opening = 2.5’

wall height below opening = 5.17’

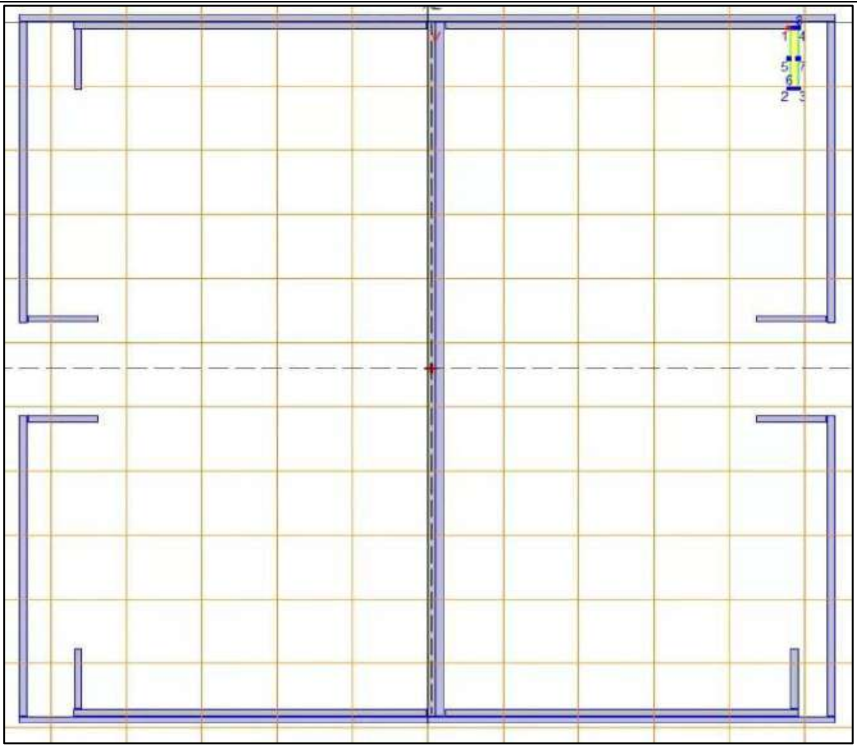
Wind load UDL on Cill = (124.64 x (5.17+2.5)/2)/1000

= 0.48 kips/ft

Lateral Load multiplied by 0.7 for deflection calculation:

= 0.48 x 0.7

= 0.34 kips/ft



The moment of Inertia (I) and the bending capacity of the member is considered from Section Property:

I = 6259142 mm^4
= 7.252×10^{-4} ft^4

Elastic modulus (Z) about U-axis = 81818.86 mm^3

Bending Capacity (M) = Z*σ
= 81818.86 mm^3 X 450 N/mm^2
= 36818487 Nmm
= 27.16 kips-ft

lateral deflection of cill = $5wl^4/384*(EI)$
= $5 \times (0.34) \times (8.528)^4 / (384 \times 4385941 \times 7.252 \times 10^{-4})$
= 0.0074'
= 0.09"

Allowable limit = span/240
= 8.528/240
= 0.0355'
= 0.426" >0.09" **….Hence Ok.**

The maximum bending moment at the Centre of the lintel:
= $wl^2/8$

= (0.48) x $(8.528)^2/8$

= 4.364 kips-ft

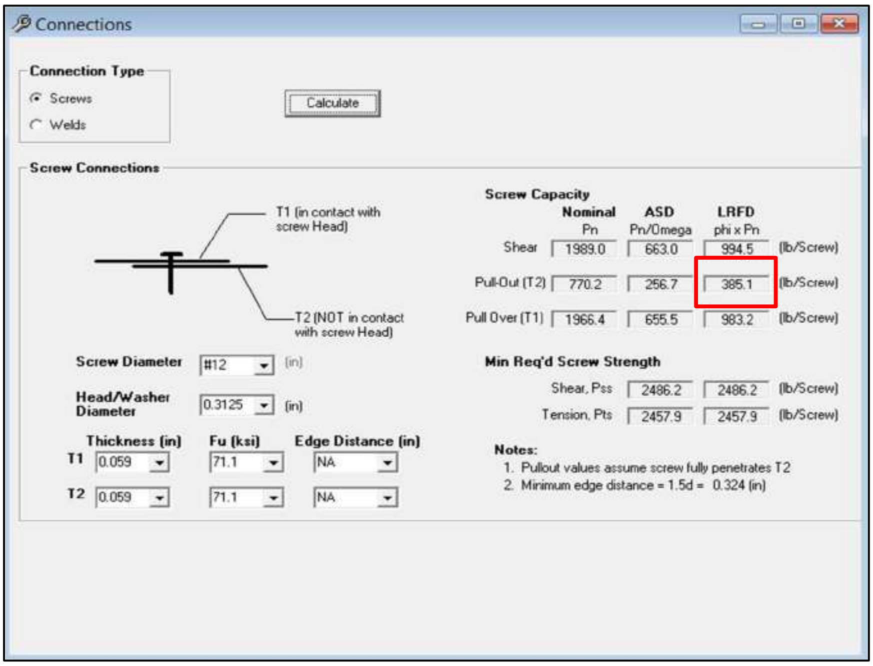
Bending capacity of C-150-50-15 = 27.16 kips-ft > 4.364 kips-ft

…. Hence Ok.

Provide compound window cill made up of (4) C-150-50-15 s/c at all locations for large window greater than 4.92' and window width lower than or equal 4.92' provide (1) C-150-50-15 at cill location.

17 CFS Connections Design

17.1 Typical Stud to Base Track/Top Track Connection:



Using #12 Screw (0.216" dia), min screw capacity in pull out is

Pullout capacity of Screw = 385.1 lb (from appendix 9)

Reaction / connection capacity required from typical Stud to Top or Base track,

R_{stud} = Wind load x Spacing of Stud x Wall height/2

= 124.64 x1.2x (10.21/2)

= 763.54 lb

No. of Screws Required = 763.54 /385.1

= 1.98 no. = 2 no.

Provide (4) screws @ each end of the Stud (2) on each face for stud spacing above 1.20' and for stud spacing below 1.2' spacing provide nominal fixings.

17.2 Jamb to Base Track/Top Track Connection:

Using #12 Screw (0.216" dia), min screw capacity in pull out is

Pullout capacity of Screw = 385.1 lb (from appendix 9)

Reaction / connection capacity required from typical Stud to Top or Base track,

R_{Stud} = Wind load x Spacing of Stud x Wall height/2

= 124.64x (2+6.56)/2 x 10.21/2

= 2723.31 lb

No. of Screws Required = 2723.31/385.1

= 7.1 no. = 8 no.

Provide (8) screws @ each end of the Stud (2) on each face.

Fixings required based upon applied load and fixing capacity, Provide (8) no. fixings per end, (2) no. on each face of the compound jamb at all locations.

17.3 Lintel Connection to Jamb Stud for Large Opening:

Using #12 Screw (0.216" dia), min screw capacity in pull out is

Pullout capacity of Screw = 385.1 lb (from appendix 9)

For maximum width of openings:

Lateral load on jamb due to Window opening & adjacent stud:

= (124.64 x (2.67 + 2.46)/2)/1000

= 0.32 kips/ft

Reaction on jamb due to lintel = 0.32 x 6.562/ 2

= 1.05 kips

OR

Lateral load on jamb due to Door opening & adjacent stud:

= (124.64 x (7.546+2.67)/2)/1000

= 0.64 kips/ft

Reaction on jamb due lintel = 0.64 x 6.562 / 2

= 2.1 kips

For maximum reaction:

No. of Screws Required in pull out:

At Lintel = 2.1 / 0.3851

= 5.45 = 8 no.

Fixings required based upon applied load and fixing capacity, Provide min. (8) no. fixings per end, (2) no. on each face at large openings Greater Than 1.4 m.

17.4 Lintel Connection to Jamb Stud for Small Opening:

Using #12 Screw (0.216” dia), min screw capacity in pull out is

Pullout capacity of Screw	=	385.1 lb	(from appendix 9)	
For maximum width of opening				
UDL on window lintel	=	(124.64 x (7.4 +2.67)/2)/1000		
	=	0.627 kips/ft		
Reaction on jamb due to lintel	=	0.627 x 3.28/ 2		
	=	1.03 kips		
For maximum reaction:				
No. of Screws Required in pull out:				
At Lintel	=	1.03 / 0.3851		
	=	2.67	=	4 no.

Provide (4) screws @ each end of the Stud (2) on each face.

Fixings required based upon applied load and fixing capacity, Provide (4) no. fixings per end, (2) no. on each face at small openings Upto 1 m.

17.5 Compound Cill to Jamb Stud for Large Opening:

Using #12 Screw (0.216” dia), min screw capacity in pull out is

Pullout capacity of Screw	=	385.1 lb	(from appendix 9)	
UDL on window Cill	=	(124.64 x (5.17 + 2.46)/2)/1000		
	=	0.476 kips/ft		
Reaction on jamb due to Cill	=	0.476 x 8.528 / 2		
	=	2.03 kips		
For maximum reaction:				
No. of Screws Required in pull out:				
At Lintel	=	2.03 / 0.3851		
	=	5.271	=	8 no.

Provide (8) screws @ each end of the Stud (4) on each face.

Fixings required based upon applied load and fixing capacity, Provide (8) no. fixings per end, (4) no. on each face at Large Openings Greater Than 2 m.

17.6 Cill to Jamb Stud for Small Opening:

Using #12 Screw (0.216” dia), min screw capacity in pull out is

Pullout capacity of Screw	=	385.1 lb	(from appendix 09)
UDL on window Cill	=	(124 x (2.46 + 5.16)/2)/1000	
	=	0.475 kips/ft	
Reaction on jamb due to Cill	=	0.475 x 4.92 / 2	
	=	1.17 kips	

For maximum reaction:

No. of Screws Required in pull out:

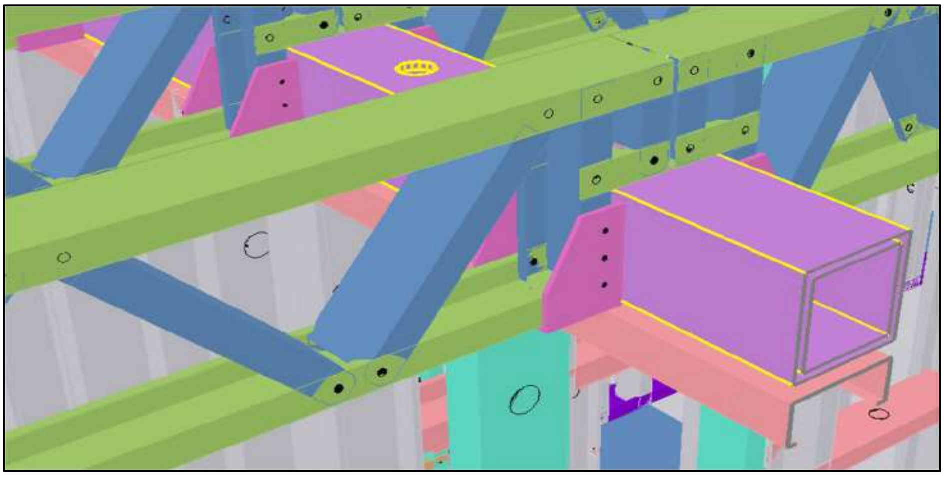
At Lintel	=	1.17 / 0.3851		
	=	3.038	=	4 no.

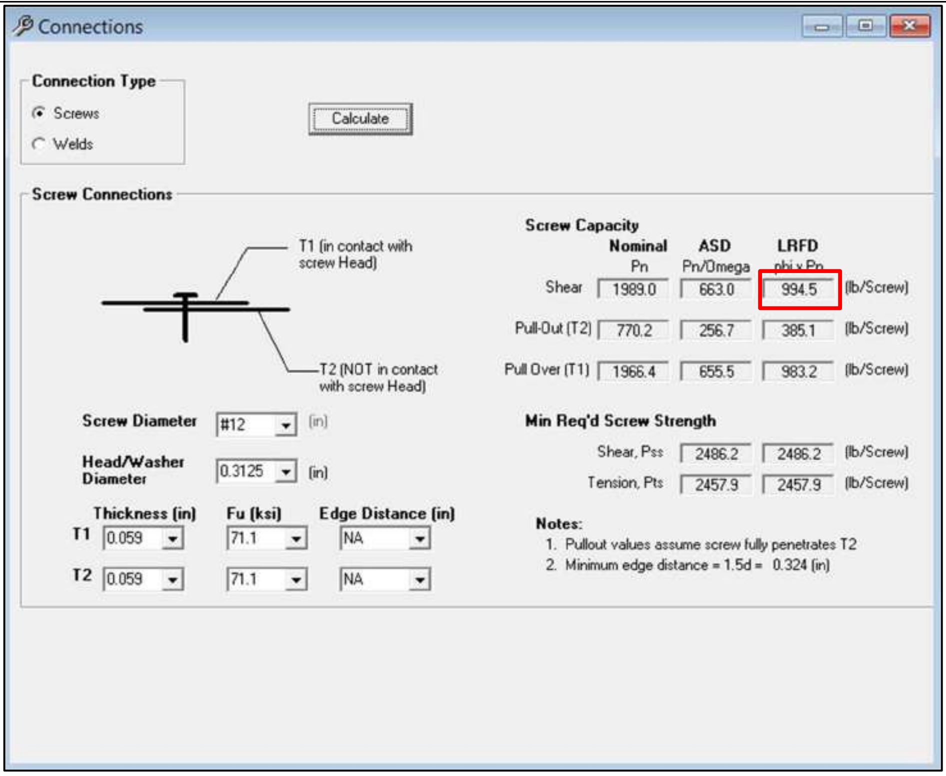
Provide (4) screws @ each end of the Stud (2) on each face.

Fixings required based upon applied load and fixing capacity, Provide (4) no. fixings per end, (2) no. on each face at Small Opening Upto 1.5 m.

17.7 Joist to HRS:

For joist to HRS connection use 6 mm thick bracket as shown in below snap.





Using #12 Screw (0.216” dia), min of screw capacity in shear

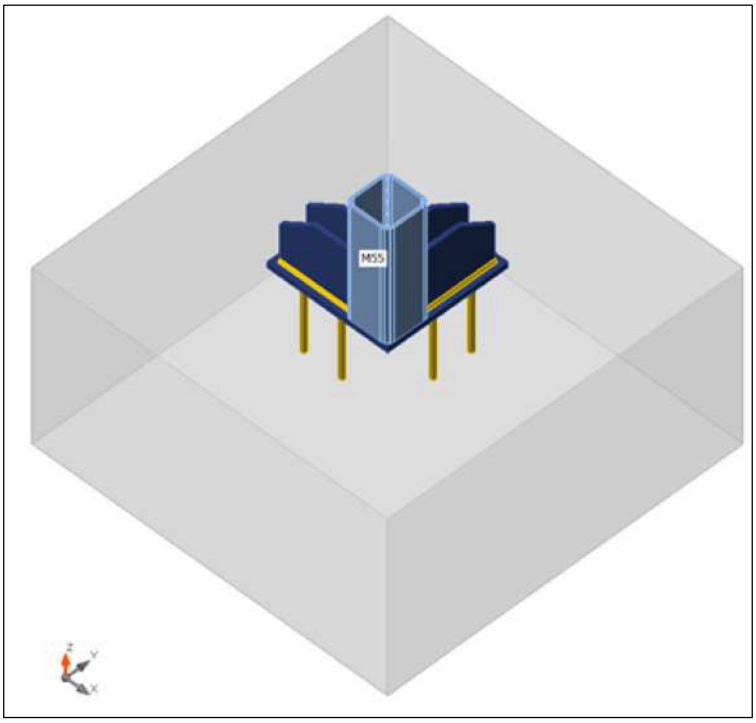
Shear capacity of screw	=	994.5 lb	
From “7B. Design of Roof Joist - RJ”			
Maximum factored Shear Force on joist end	=	1.153 kips	=5.16 kN
No. of Screws Required	=	1.153/0.9945	
	=	1.16	=3 no.

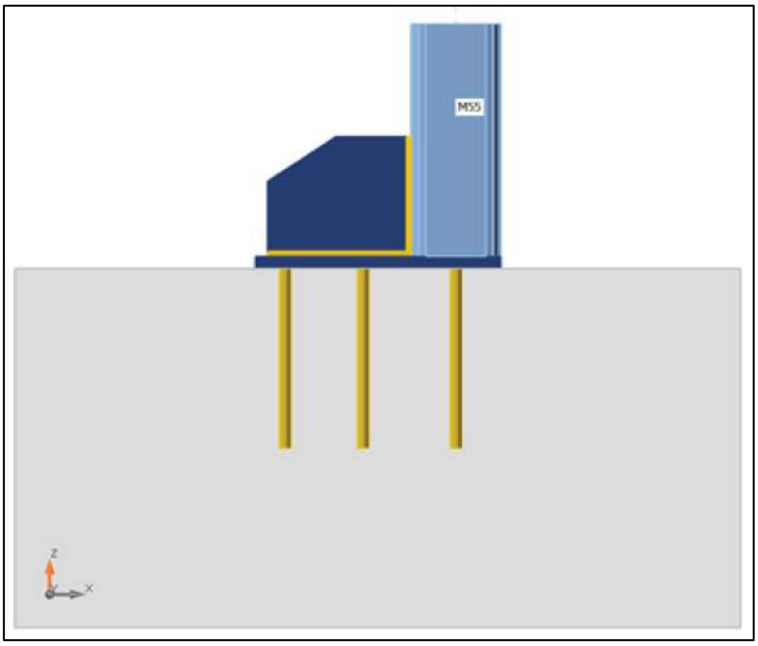
Provide (6) no. screws for each bracket, (3) no. of screws on each face of bracket.

18 HRS Connections Design

18.1 Design of L Shape Base Plate for SHS150x150 Grid A and C:

L Type Base Plate for 150 x150 is as shown below:

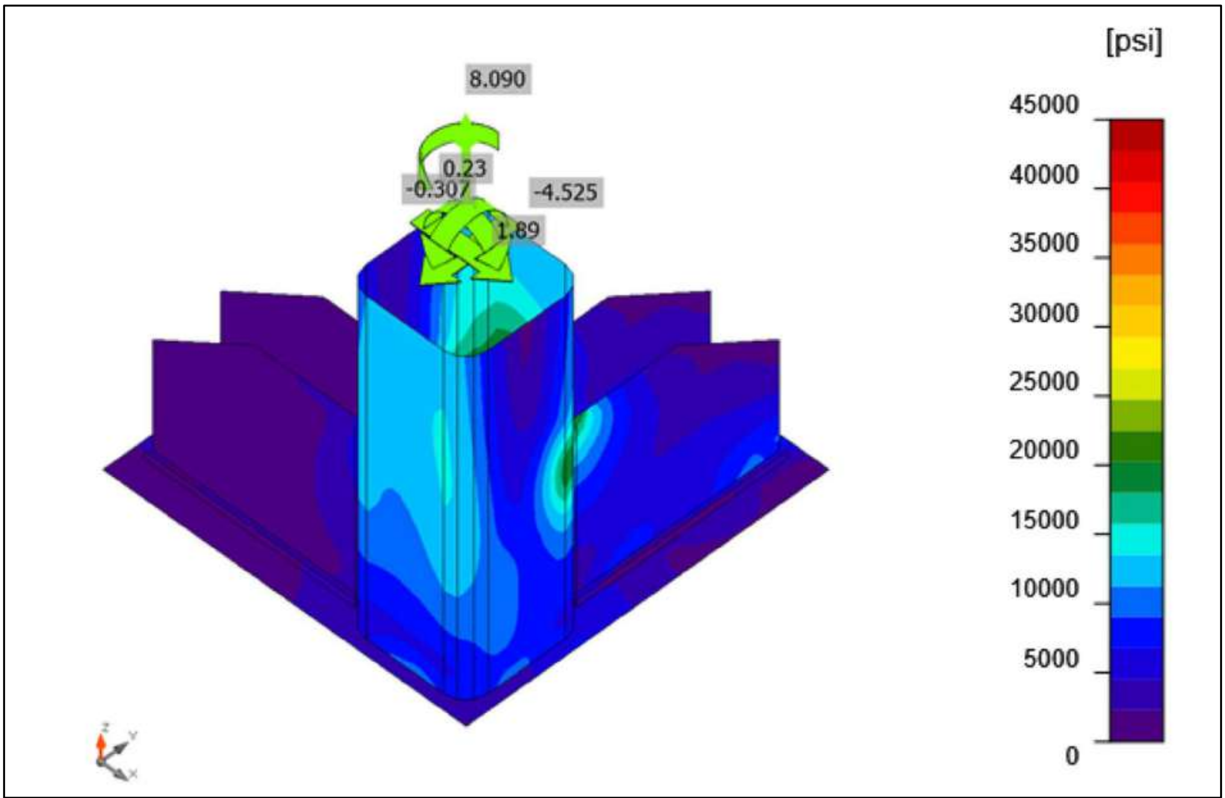




Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE8	M55 / End	8.090	-4.525	-0.307	0.23	1.89	-21.62
LE10	M55 / End	5.971	-1.941	-2.068	0.08	7.37	-11.39

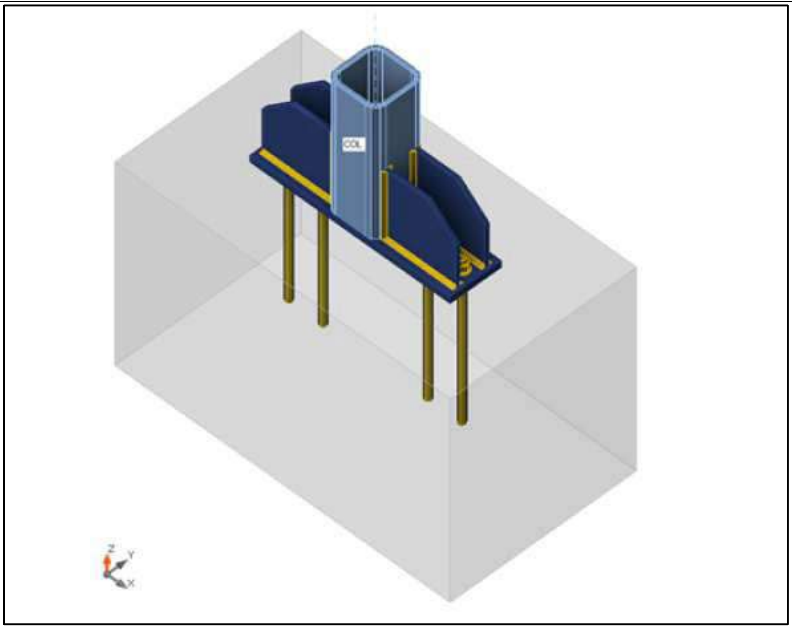
Stress Check: -

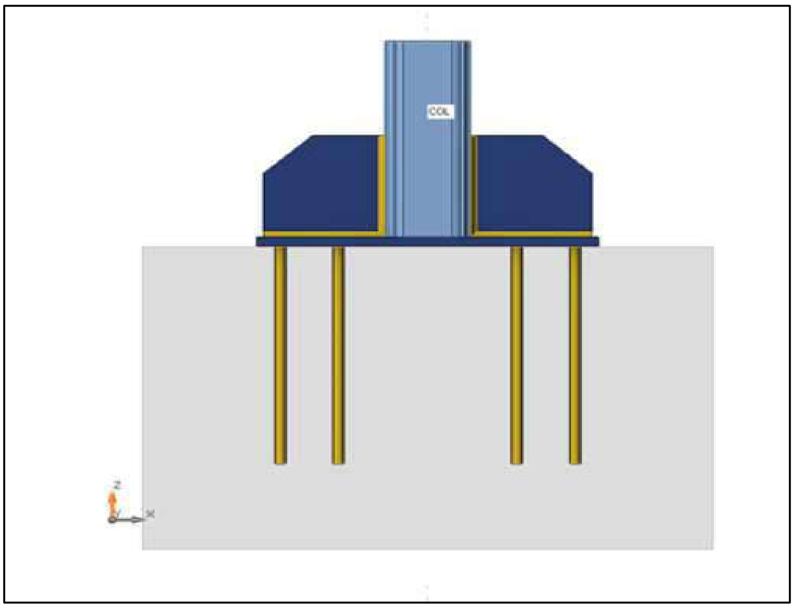


Provide L shape Base plate of thickness 13/16" (20 mm) using Butt weld to connect with column and provide 9/16" (15 mm) Stiffener plates using 3/8" (10 mm) double fillet weld.

Refer to "Appendix - 10. L Shape Base Plate for SHS150x150 Grid A and C" for a detailed summary.

18.2 Design of Straight Base Plate for 150x150 Columns Grid B:

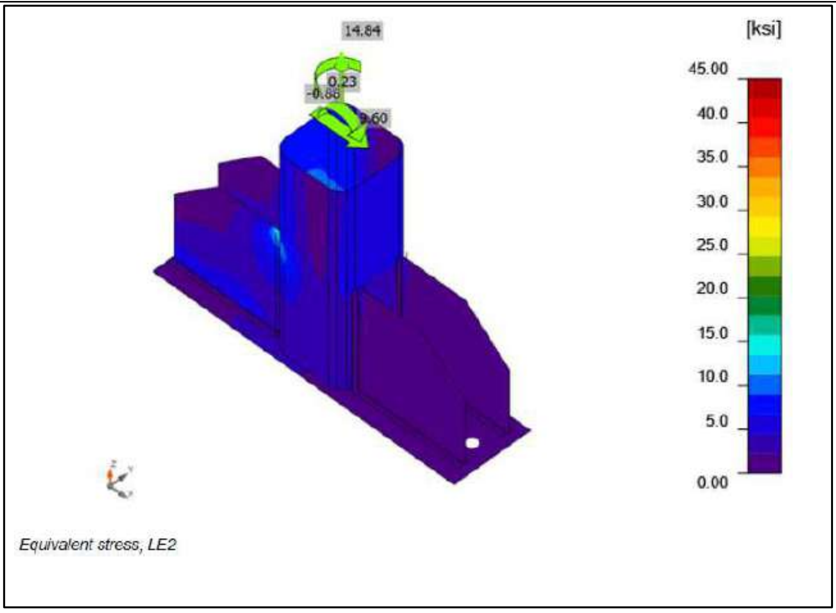




Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	COL / End	21.55	0.00	-0.06	0.08	0.69	0.00
LE2	COL / End	14.84	0.00	-0.88	0.23	9.60	0.00

Stress Check: -

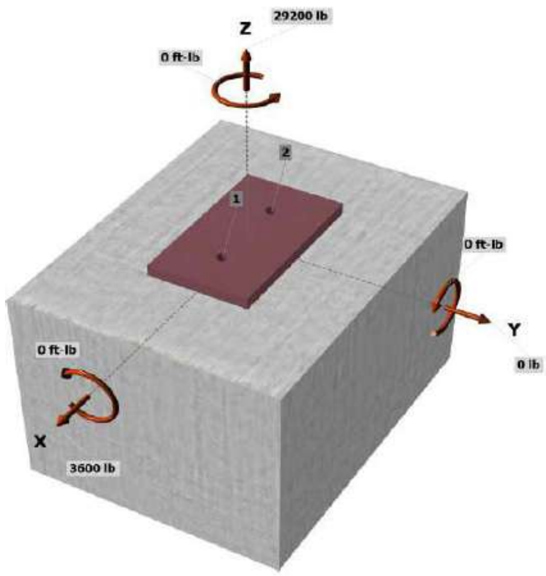


Provide Straight Base plate of thickness 13/16” (20 mm) using Butt weld to connect with column and provide 9/16” (15 mm) Stiffener plates using 3/8” (10 mm) double fillet weld.

Refer to “Appendix - 11. Straight Base Plate for 150x150x12 Column Grid B” for a detailed summary.

18.3 Anchor Design for L shape and Straight Base Plate for 150x150 Columns:

Design of anchor for worst case load from L shape and straight base plate.



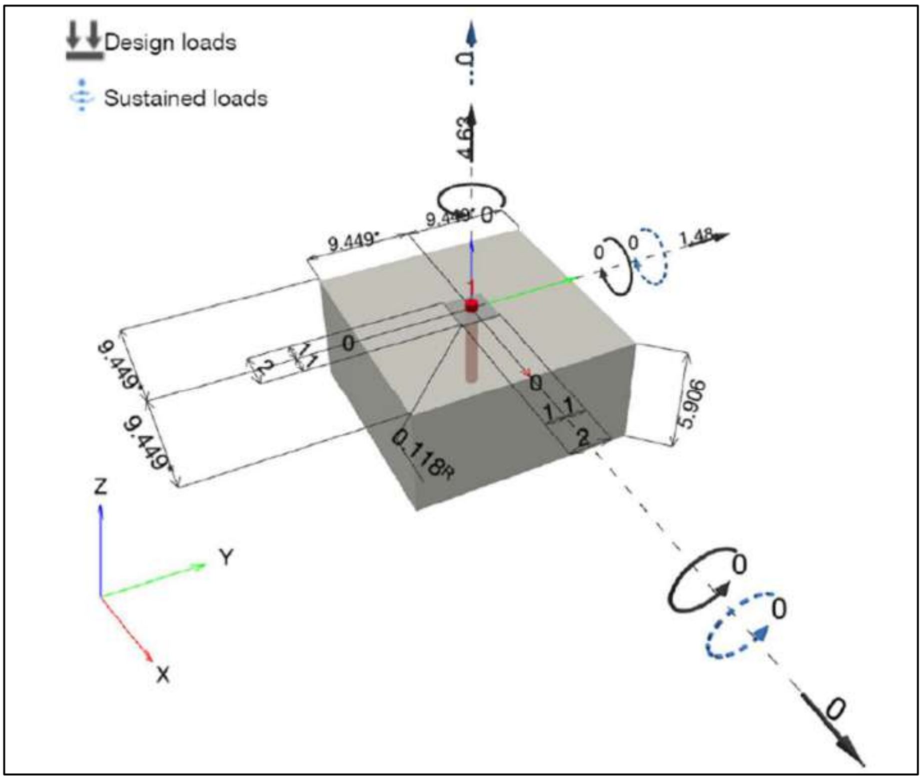
Provide (4) SET-3G™ - SET-3G w/ #6 A615 Gr. 60 Rebar (20 mm diameter) anchor in L shape and Straight base plate with 16.96 in (431 mm) effective embedment depth and 8.0 in (200mm) edge distance from concrete edge.

Min. Tension and Shear anchor reinforcement in concrete to be design by RC designer.

Refer to "Appendix - 12. Anchor Design for L shape and straight base plate" for a detailed summary.

18.4 Design of Anchor for Hold-down:

The anchor design for the hold-down is as shown below:



Factored forces on the members are as follows:

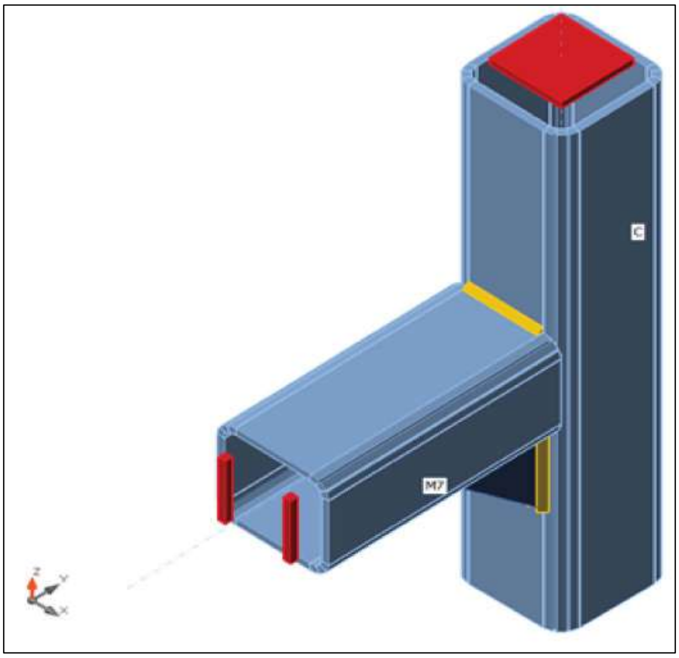
Case	Description	Forces [kip] / Moments [ft.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 4.630; V_x = 0.000; V_y = 1.480; M_x = 0.000; M_y = 0.000; M_z = 0.000;	no	99

Provide HIT-HY 200 + HAS-U 8.8 HDG (1) M16 anchor to connect Hold-down and concrete, with 4.331" (110 mm) effective embedment depth and C20/25 cracked concrete at minimum 9.449" (240 mm) edge distance.

Refer to "Appendix - 13. Design of Anchor for hold-down" for a detailed summary.

18.5 SHS 150x150 Roof beam to SHS 150x150 column Connection:

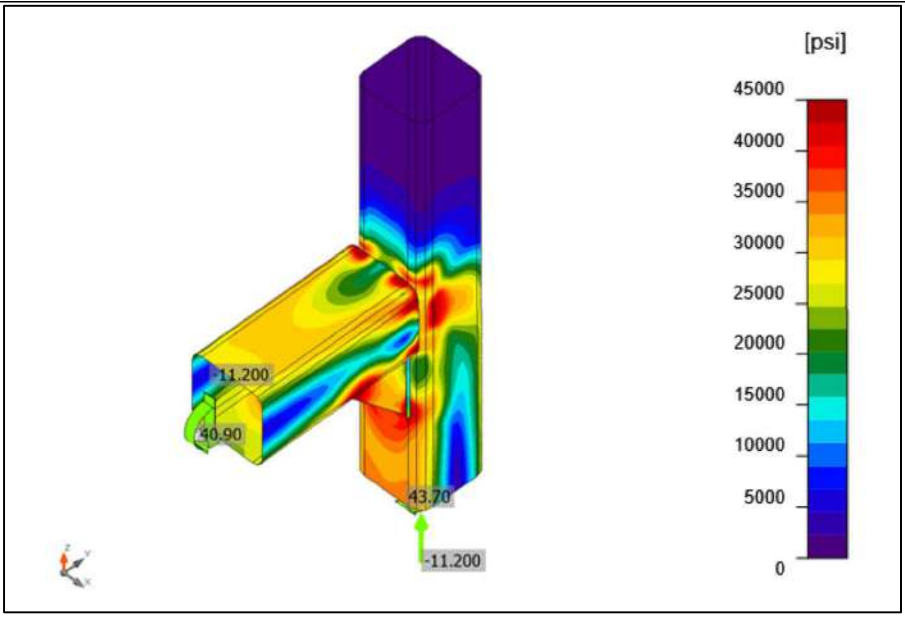
SHS 150 Roof beam to SHS 150 Column Connection is as shown below:



Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	C / End	-11.200	0.000	0.000	0.00	43.70	0.00
	M7 / End	0.000	0.000	-11.200	0.00	40.90	0.00
	C / Begin	0.000	0.000	0.000	0.00	0.00	0.00

Stress Check : -

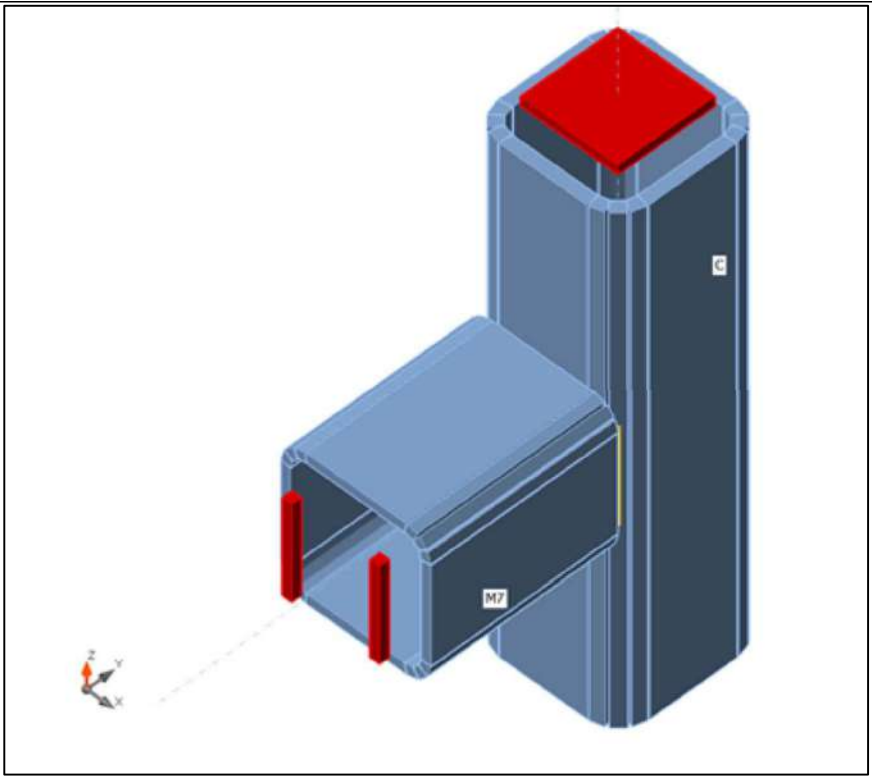


Provide (2) underside Stiffener plates of thickness 12 mm (1/2”) using 5/16” (8 mm) all-around Fillet Weld to connect the Stiffeners and provide fillet weld @ top & bottom to connect SHS beam with the SHS column and butt weld on both sides of SHS beams.

Refer “Appendix - 14. SHS 150x150 Roof beam to SHS 150x150 column (With Stiffener)” for detail summary.

18.6 SHS 150x150 Roof beam to SHS 150x150 column Connection:

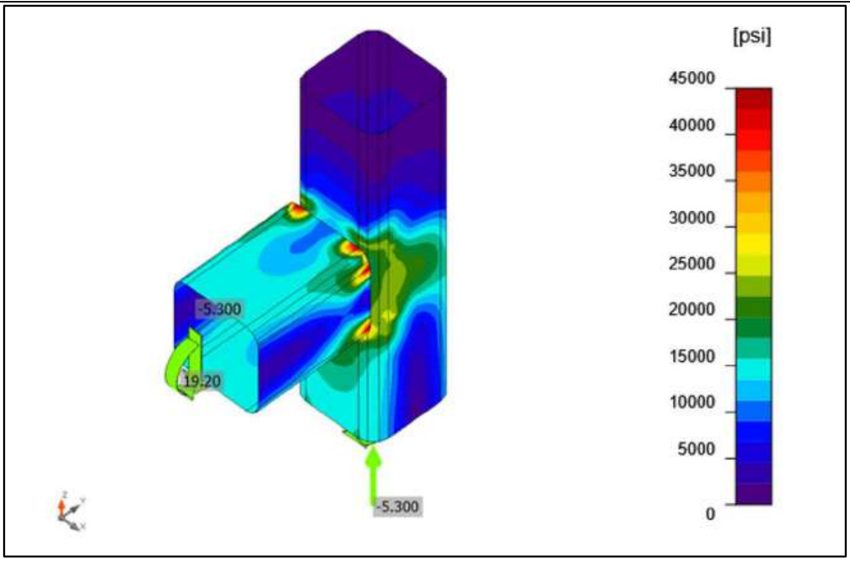
SHS 150 Roof beam to SHS 150 Column Connection is as shown below:



Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	C / End	-5.300	0.000	0.000	0.00	20.50	0.00
	M7 / End	0.000	0.000	-5.300	0.00	19.20	0.00
	C / Begin	0.000	0.000	0.000	0.00	0.00	0.00

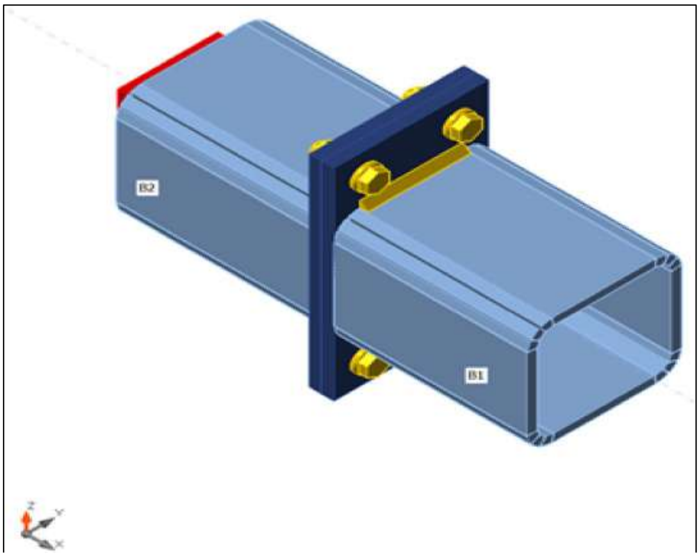
Stress Check : -



Refer "Appendix - 15. SHS 150x150 Roof beam to SHS 150x150 column (WithOut Stiffener)" for detail summary.

18.7 Vertical Splice Connection @ roof Beam:

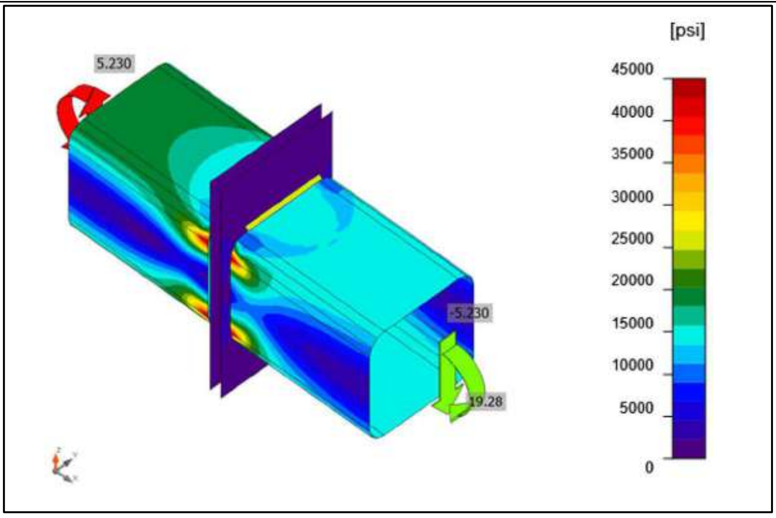
Vertical splice Connection for SHS 150 Beam at roof level is as shown below:



Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	B1 / End	0.000	-11.200	0.000	0.00	0.00	-40.91
	B2 / End	0.000	11.200	0.000	0.00	0.00	-40.91

Stress Check : -

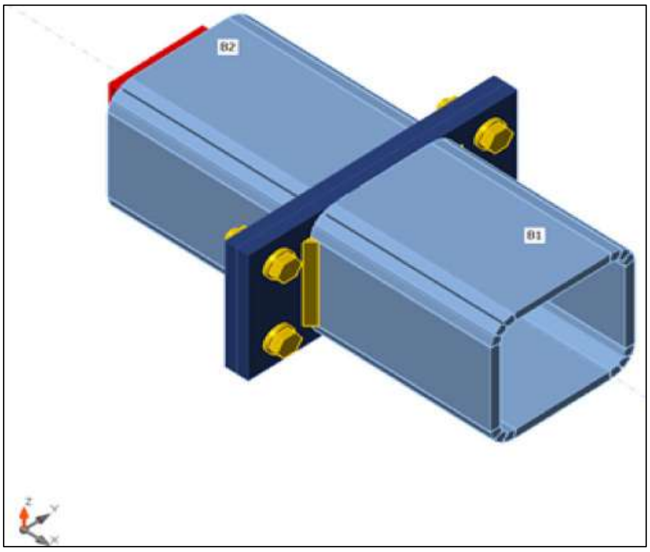


Provide (2) End plates of thickness 15 mm (9/16”) with (4) 5/8” dia. bolts of grade A325 (M16 bolts of grade 8.8) using 5/16” (8 mm) fillet weld on top and bottom and butt weld on both sides of plates.

Refer “Appendix - 16. Vertical Splice Connection @ roof Beam” for detail summary

18.8 Horizontal Splice Connection @ Roof:

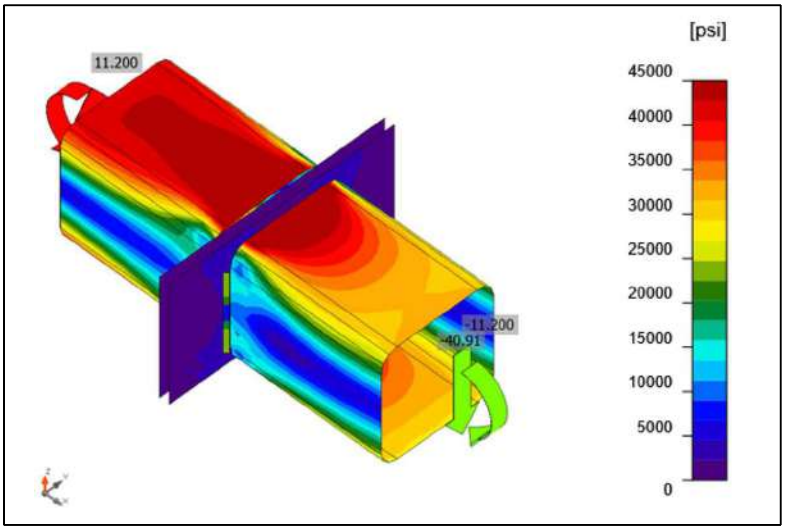
Horizontal splice Connection for SHS 150 Beam at roof level is as shown below:



Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	B1 / End	0.000	-11.200	0.000	0.00	0.00	-40.91
	B2 / End	0.000	11.200	0.000	0.00	0.00	-40.91

Stress Check : -

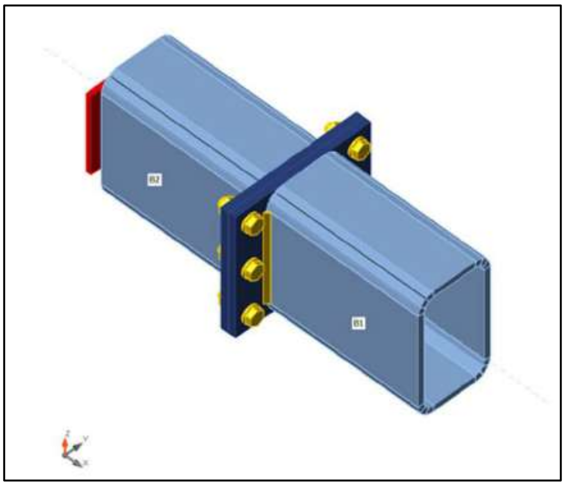


Provide (2) End plates of thickness 15 mm (9/16") with (4) 5/8" dia. bolts of grade A325 (M16 bolts of grade 8.8) using 8 mm (5/16") fillet weld on both sides and butt weld on top and bottom of plates.

Refer "Appendix - 17. Horizontal Splice Connection @ Roof" for detail summary.

18.9 Horizontal Splice Connection @ Roof:

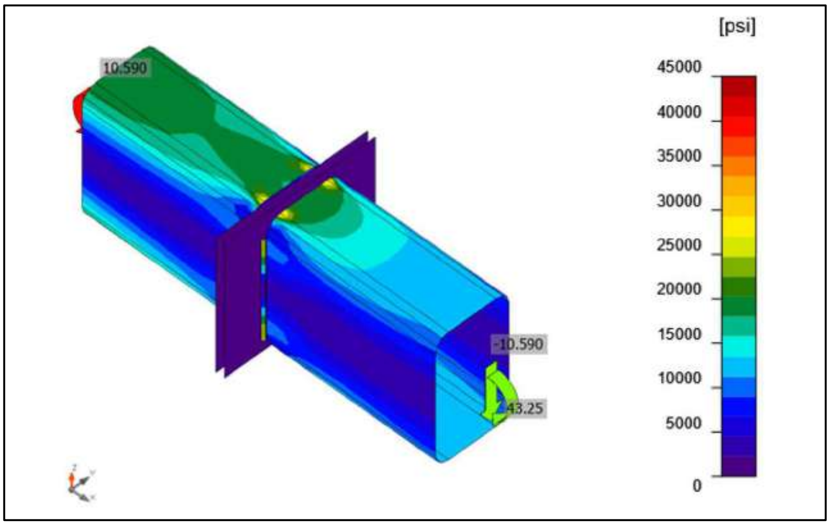
Horizontal splice Connection for SHS 250 Beam at roof level is as shown elow:



Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	B1 / End	0.000	0.000	-10.590	0.00	43.25	0.00
	B2 / End	0.000	0.000	10.590	0.00	43.25	0.00

Stress Check : -

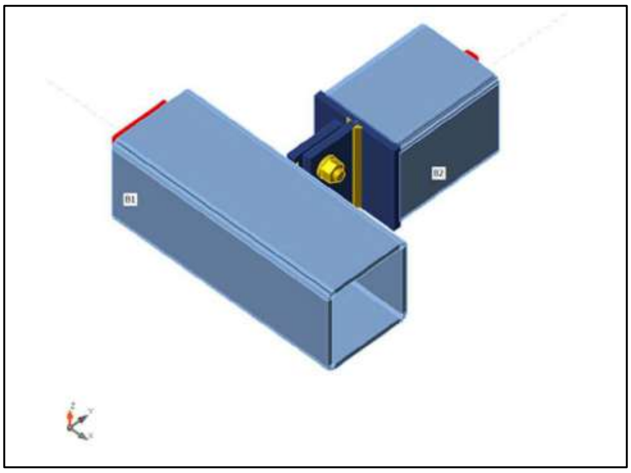


Provide (2) End plates of thickness 15 mm (9/16”) with (6) 5/8” dia. bolts of grade A325 (M16 bolts of grade 8.8) using 8 mm (5/16”) fillet weld on both sides and butt weld on top and bottom of plates.

Refer “Appendix - 18. Horizontal Splice Connection @ Roof” for detail summary.

18.10 Beam to Beam Fin Connection:

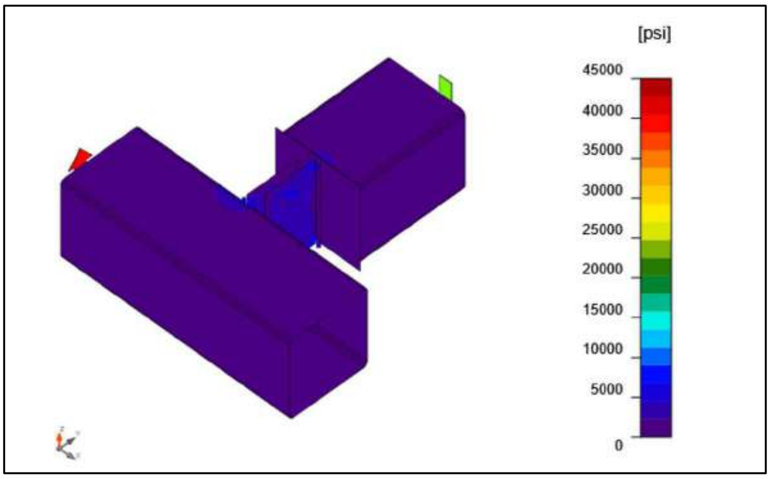
Beam to Beam Fin Connection is as shown below:



Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	B1 / Begin	0.000	0.000	-2.700	0.00	0.00	0.00
	B1 / End	0.000	0.000	0.000	0.00	0.00	0.00
	B2 / End	0.000	0.000	2.700	0.00	0.00	0.00

Stress Check : -

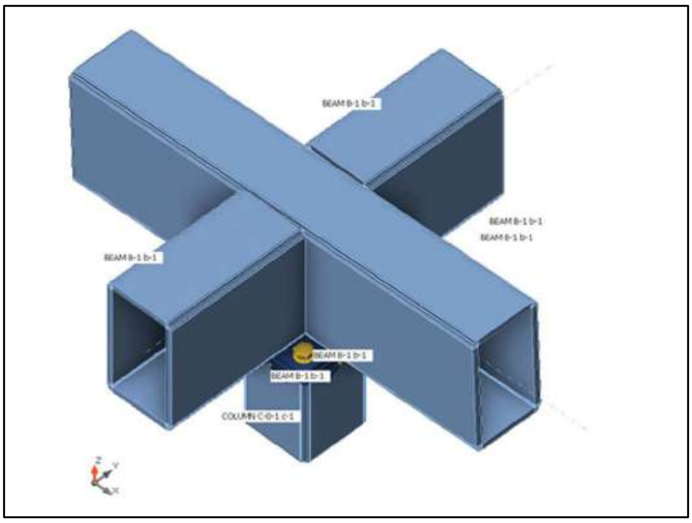


Provide (2) end plates of thickness 12 mm (1/2") with (2) 5/8" dia. bolts of grade A325 (M16 bolts of grade 8.8) to connect plates. provide 5/16" (8 mm) Fillet Weld to connect.

Refer "Appendix – 19. Beam to beam fin plate connection" for detail summary.

18.11 Beam To Column Connection @ Roof:

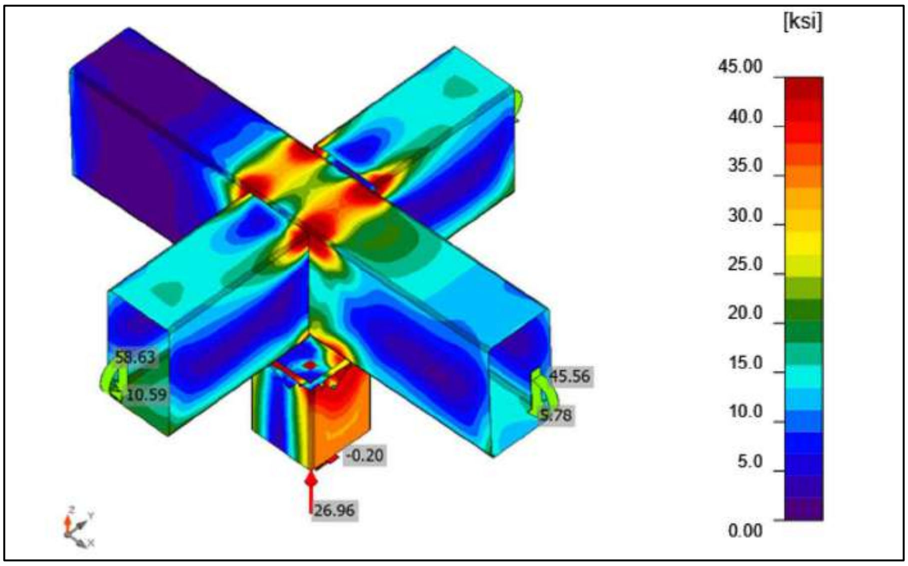
Beam To Column Connection for SHS 250 Beam & 150 Column at Centre is as shown below:



Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	BEAM B-1 b-1 / End	0.00	0.00	0.00	0.00	0.00	0.00
	BEAM B-1 b-1 / End	0.00	0.00	10.59	0.00	-58.63	0.00
	BEAM B-1 b-1 / Begin	0.00	0.00	10.59	0.00	58.63	0.00
	COLUMN C-0-1 c-1 / Begin	26.96	0.00	-0.20	0.00	45.56	0.00
	BEAM B-1 b-1 / Begin	0.00	0.00	5.78	0.00	45.56	0.00

Stress Check : -

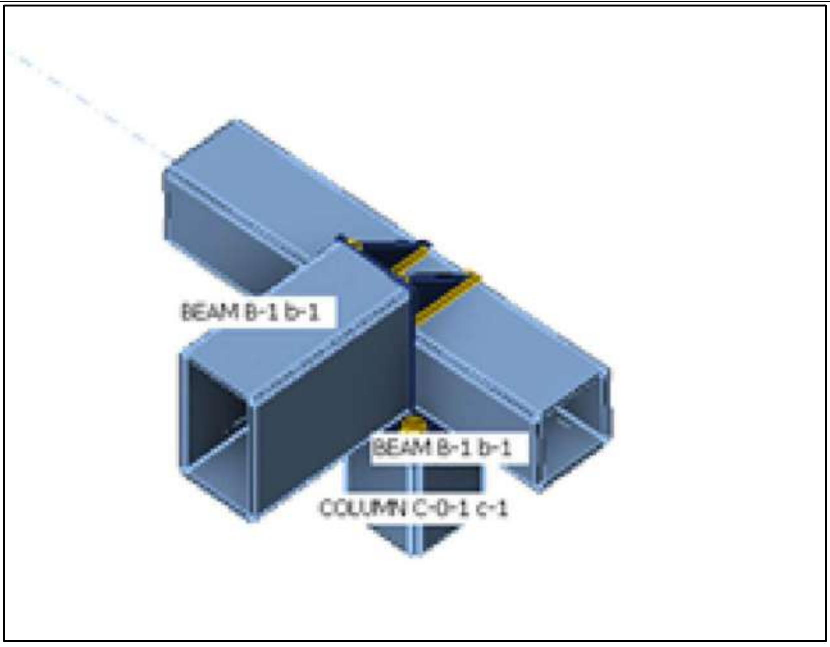


Provide (2) seat plates of thickness 15 mm (9/16”) with (4) 3/4” dia. bolts of grade A325 (M20 bolts of grade 8.8) using 8 mm (5/16”) fillet weld for stiffener of thickness 12 mm (1/2”) & butt weld Beam & Column.

Refer “Appendix - 20. Beam To Column Connection @ Roof” for detail summary.

18.12 Beam To Column Connection @ Roof:

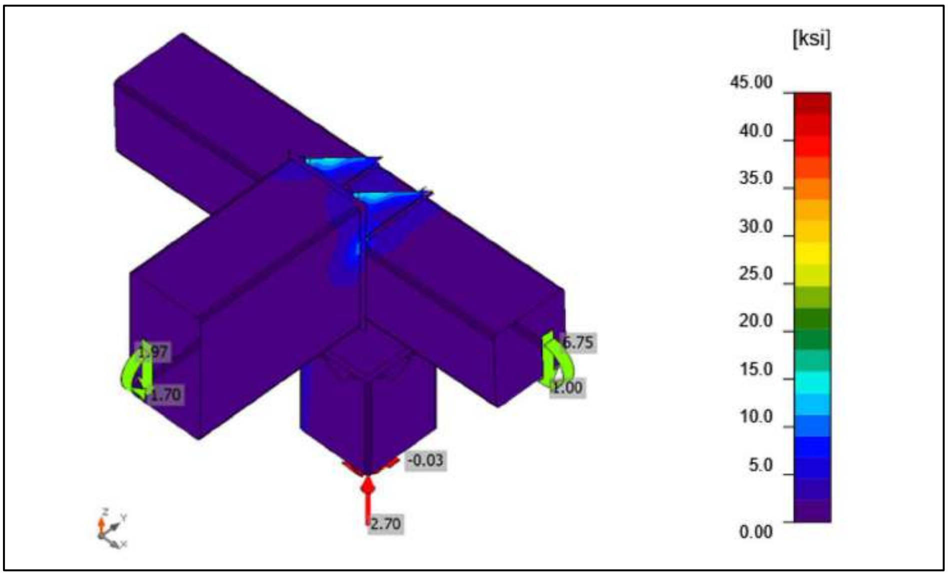
Simply Supported Connection for SHS 150 Beam & SHS 150 Column at end is as shown below:



Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	BEAM B-1 b-1 / Begin	0.00	0.00	1.00	0.00	6.75	0.00
	COLUMN C-0-1 c-1 / Begin	2.70	0.00	-0.03	0.00	-1.12	-1.97
	BEAM B-1 b-1 / Begin	0.00	0.00	1.70	0.00	1.97	0.00

Stress Check : -

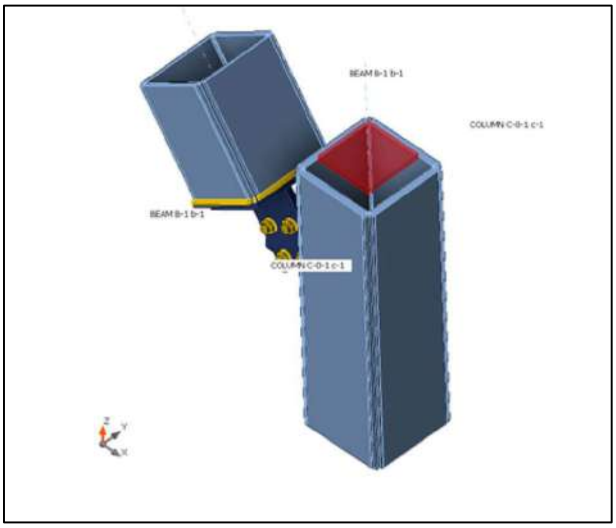


Provide (2) seat plates of thickness 15 mm (9/16") with (4) 3/4" dia. bolts of grade A325 (M20 bolts of grade 8.8) using 8 mm (5/16") fillet weld for stiffener of thickness 12 mm (1/2") & butt weld Beam & Column.

Refer "Appendix - 21. Beam To Column Connection @ Roof" for detail summary.

18.13 Incline Column Connection @ Roof:

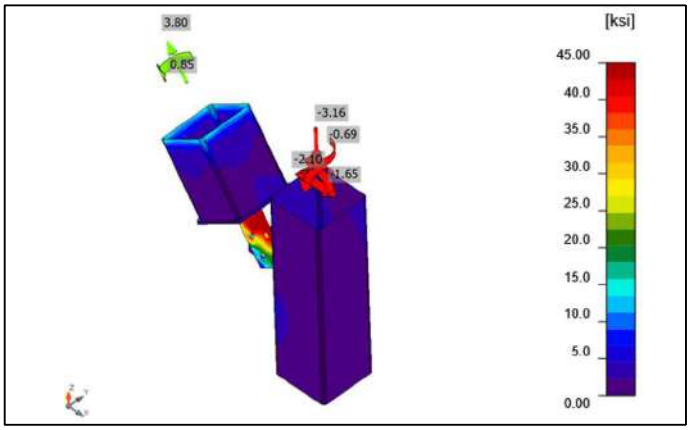
Incline Column Connection for SHS 150 Column To SHS 150 Column at base level is as shown below:



Factored forces on the members are as follows:

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	COLUMN C-0-1 c-1 / End	-3.16	0.00	-2.10	-0.69	-1.65	-0.47
	BEAM B-1 b-1 / End	3.80	0.00	0.00	0.85	0.00	0.00

Stress Check : -

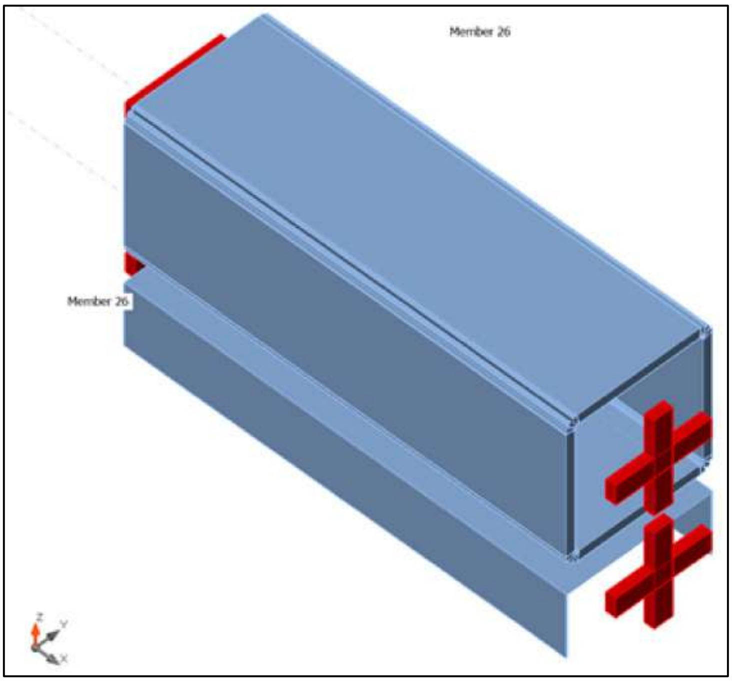


Provide (2) fin plates of thickness 10 mm (3/8”) with (4) 1/2” dia. bolts of grade A325 (M12 bolts of grade 8.8) using 6 mm (1/4”) fillet weld.

Refer “Appendix - 22. Incline Column Connection @ Base” for detail summary.

18.14 Wall top track to HRS Connection:

Wall top track to HRS connection is shown below:



Factored forces on the members are as follows-

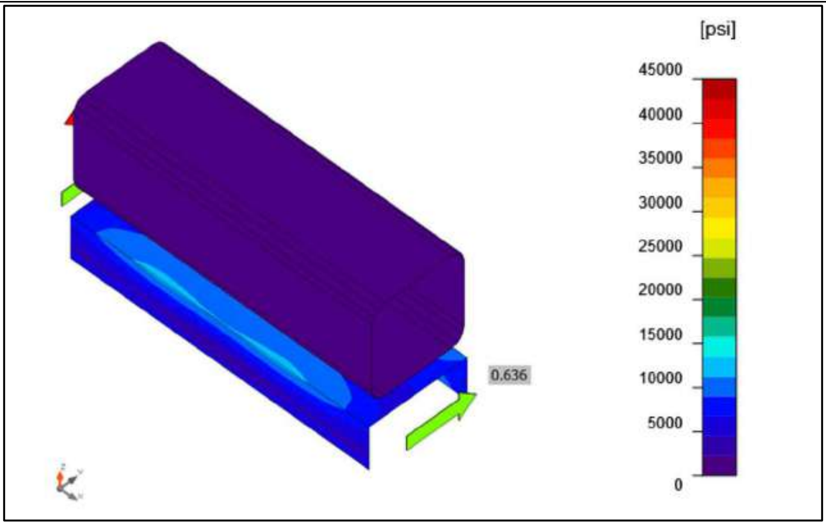
Wall top track to HRS connection at max 2’-0” (600 mm) spacing.

Force at each connection = (124.64 x (2+2)/2 x (10.21/2))/1000

= 1.272 kips

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE2	Member 26 / Begin	0.000	1.272	0.000	0.00	0.00	0.00
	Member 26 / End	0.000	0.000	0.000	0.00	0.00	0.00
	Member 27 / Begin	0.000	0.000	0.636	0.00	0.00	0.00
	Member 27 / End	0.000	0.000	0.636	0.00	0.00	0.00

Stress Check:

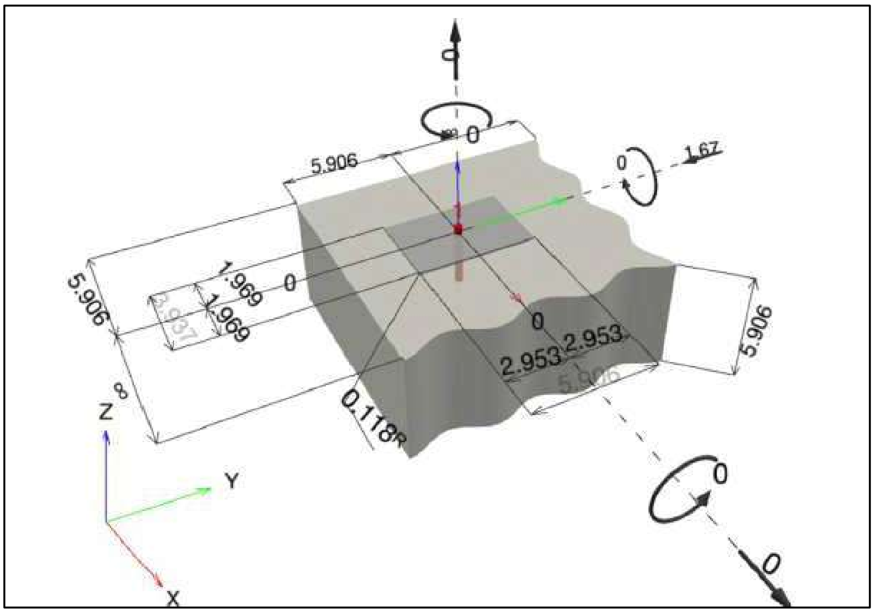


Provide plate of thickness 1/4” (6 mm) and (1) M12 bolts of grade 8.8 (A325) used with 3/16” (4 mm) fillet weld.

Refer “Appendix - 23. wall Top Track to HRS Plate Connection” for detail summary

18.15 Base track to concrete Connection:

Base track to concrete connection is as follows:



For maximum width of opening as shown below:

For critical loaction

Lateral load on jamb due to adjacent stud = Wind load x Tributary x Stud Height/2

	=	(124.64 x 5.24/2x 10.21/2)/1000
Maximum Reaction on end	=	1.67 kips
Lateral load on Typical stud	=	Wind load x Tributary x Stud Height/2
	=	124.64 x2x (10.21/2)
Maximum Reaction on end	=	1.272 kips

Factored forces on the members are as follows-

1.1 Load combination

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Fire	Max. Util. Anchor [%]
1	Combination 1	N = 0.000; V_x = 0.000; V_y = -1.670; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	no	79

Provide HUS4-HF 10h_nom3 (3/8”) dia screw anchor with minimum 3.819” (97 mm) embedment depth at each wall stud location to connect basetrack with cracked concrete of grade C20/25 at minimum 5.906”(150 mm) edge distance.

Refer “Appendix - 24. Base track to concrete connection” for detailed summary.

Appendix- 1

WIND LOADING ANALYSIS - Wall Components and Cladding

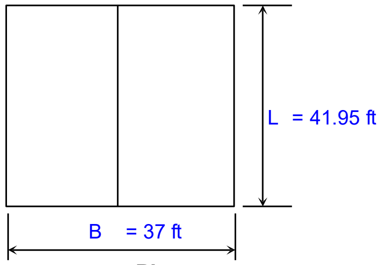
Per ASCE 7-10 / 7-16 Code for Buildings of Any Height

Using Part 1 & 3: Analytical Procedure (Section 30.4 & 30.6 for 2010) & (Section 30.3 & 30.5 for 2016)

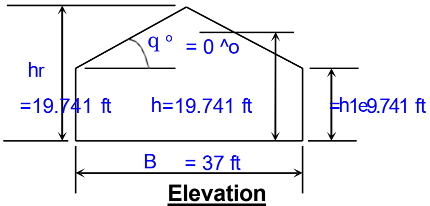
Project Name:	Exuma, Bahamas	23.02		
Project Number:	242201	Calc. by :	SK	Checked by: MS

Input Data:

ASCE 7 Code =	2016	
Wind Speed, V =	200	mph (Wind Map, Figure 26.5-1A-D)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	D	(Sect. 26.7)
Ridge Height, hr =	19.74	ft. (hr >= he)
Eave Height, he =	19.74	ft. (he <= hr)
Building Width =	37.00	ft. (Normal to Building Ridge)
Building Length =	41.95	ft. (Parallel to Building Ridge)
Roof Type =		(Gable or Monoslope or Hip or Flat)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.2 & Figure 26.13-1)
Hurricane Region?	N	
Component Name =	Wall	(Girt, Siding, Wall, or Fastener)
Effective Area (Wall), Ae =	23.02	ft.^2 (Area Tributary to C&C)
Parapet ? =	N	
Height of Parapet =	0	ft.
Effective Area (Parapet), Ape =	0	ft.^2 (Area Tributary to C&C)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	0.00	deg.
Mean Roof Ht., h =	19.74	ft. (h = he, for roof angle <=10 deg.)

Wall External Pressure Coefficients, GCp:

GCp Zone 4 Pos. =	0.84	(Fig. 30.3-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Pos. =	0.84	(Fig. 30.3-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 4 Neg. =	-0.93	(Fig. 30.3-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Neg. =	-1.14	(Fig. 30.3-1, GCp is reduced by 10% for roof angle <=10 deg.)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 26.13-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

Parapet Wall External Pressure Coefficients, GCp:

	Windward	Leeward	
GCp Zone 4 Pos. =	N.A.	N.A.	(Fig. 30.3-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Pos. =	N.A.	N.A.	(Fig. 30.3-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 4 Neg. =	N.A.	N.A.	(Fig. 30.3-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Neg. =	N.A.	N.A.	(Fig. 30.3-1, GCp is reduced by 10% for roof angle <=10 deg.)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/a) , If z > 15 then: Kz = 2.01*(z/zg)^(2/a) (Table 26.10-1)

	α =	11.50	(Table 26.11-1)
	zg =	700	(Table 26.11-1)
For Wall	Kh =	1.08	(Kh = Kz evaluated at z = h)
For Parapet	Kh =	N.A.	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 26.10.2, Eq. 26.10-1)

For Wall	qh =	94.06	psf	qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)
For Parapet	qp =	N.A.	psf	qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = hp)

Design Net External Wind Pressures (Sect. 30.3 & 30.5):

For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)

For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)

where: q = qz for windward walls, q = qh for leeward walls and side walls

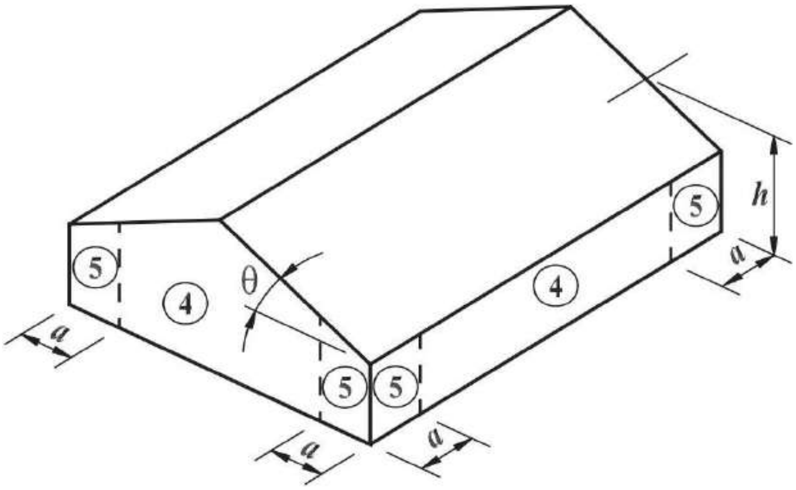
qi = qh for all walls (conservatively assumed per Sect. 30.5)

Wind Load Tabulation for Wall Components & Cladding							
Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Wall	0	1.08	94.06	96.17	-104.63	96.17	-124.62
	15.00	1.08	94.06	96.17	-104.63	96.17	-124.62
For z = hr:	19.74	1.08	94.06	96.17	-104.63	96.17	-124.62
For z = he:	19.74	1.08	94.06	96.17	-104.63	96.17	-124.62
For z = h:	19.74	1.08	94.06	96.17	-104.63	96.17	-124.62

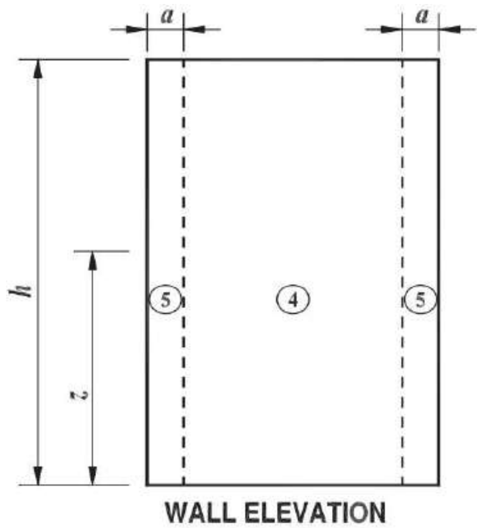
Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 4		Zone 5	
Parapet wall calculation				(+)	(-)	(+)	(-)
	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
For Windward parapet z = hp:	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (end zones), 'a' = 3.70 ft.
3. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
4. References : a. ASCE 7-10 & ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02"

Wall Components and Cladding:



Wall Zones for Buildings with h <= 60 ft.



Wall Zones for Buildings with h > 60 ft.

WIND LOADING ANALYSIS - Roof Components and Cladding

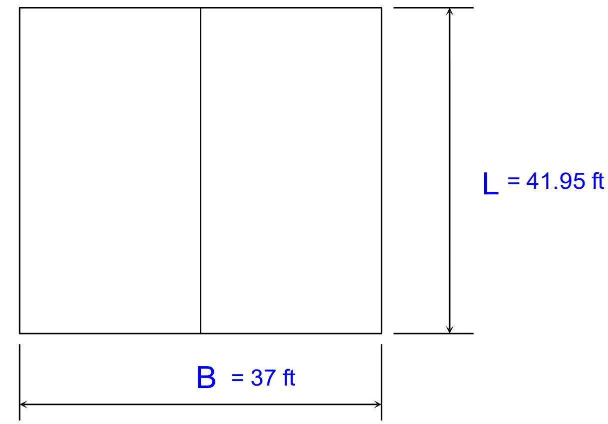
Per ASCE 7-16 / 7-10 Code for Bldgs. of Any Height with Gable Roof θ <= 45° or Monoslope Roof θ <= 3°

Using Part 1 & 3: Analytical Procedure (Section 30.4 & 30.6)

Project Name:	Patric Daily	Subject:			
Project Number:	242201	Calc. by :	SK	Checked by:	MS

Input Data:

ASCE 7 Code =	2016	
Wind Speed, V =	200	mph (Wind Map, Figure 26.5-1A-D)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	D	(Sect. 26.7)
Ridge Height, hr =	19.74	ft. (hr >= he)
Eave Height, he =	19.74	ft. (he <= hr)
Building Width =	37.00	ft. (Normal to Building Ridge)
Building Length =	41.95	ft. (Parallel to Building Ridge)
Roof Type =	Flat	(Gable or Monoslope or Hip or Flat)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.2 & Figure 26.13-1)
Hurricane Region?	N	
Component Name =	Joist/Truss	(Purlin, Joist/Truss, Decking, or Fastener)
Effective Area, Ae =	51	ft.^2 (Area Tributary to C&C)
Overhangs? (Y/N)	N	(if used, overhangs on all sides)



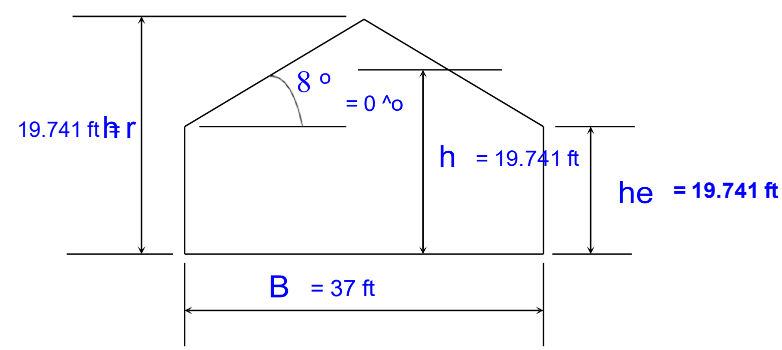
Plan

Resulting Parameters and Coefficients:

Roof Angle, θ =	0.00	deg.
Mean Roof Ht., h =	19.74	ft. (h = he, for roof angle <=10 deg.)

Roof External Pressure Coefficients, GCp:

GCp Zone 1-3 Pos. =	0.23	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)
GCp Zone 1 Neg. =	-1.41	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)
GCp Zone 2 Neg. =	-1.93	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)
GCp Zone 3 Neg. =	-2.45	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)
GCp Zone 1' Neg. =	-0.90	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)
GCp Zone 2e Neg. =	N.A.	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)
GCp Zone 2n Neg. =	N.A.	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)
GCp Zone 2r Neg. =	N.A.	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)
GCp Zone 3e Neg. =	N.A.	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)
GCp Zone 3r Neg. =	N.A.	(Fig. 30.3-2A to 30.3-2I and Fig. 30.3-5A to 30.3-5B)



Elevation

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 26.13-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/a) , If z > 15 then: Kz = 2.01*(z/zg)^(2/a) (Table 26.10-1)

α =	11.50	(Table 26.11-1)
zg =	700	(Table 26.11-1)
Kh =	1.08	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 26.10.2, Eq. 26.10-1)

qh = 94.06 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

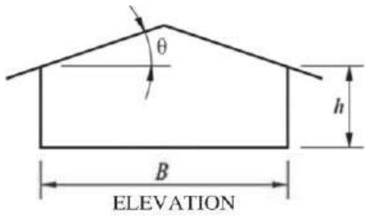
Design Net External Wind Pressures (Sect. 30.3 & 30.5):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qh for roof
qi = qh for all walls (conservatively assumed per Sect. 30.5)

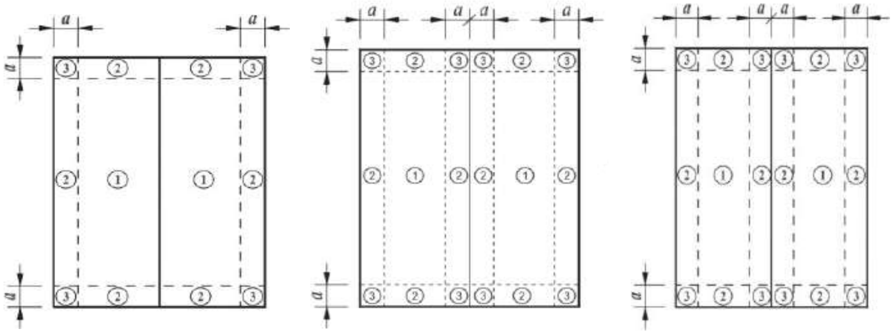
Wind Load Tabulation for Roof Components & Cladding

Component	z	Kh	qh	**p = Net Design Pressures (psf)**									
	(ft.)		(psf)	Zone 1,2,3 (+)	Zone 1 (-)	Zone 2 (-)	Zone 3 (-)	Zone 1' (-)	Zone 2e (-)	Zone 2n (-)	Zone 2r (-)	Zone 3e (-)	Zone 3r (-)
Joist/Truss	0	1.08	94.06	38.49	-149.41	-198.01	-247.41	-101.58	N.A.	N.A.	N.A.	N.A.	N.A.
	15.00	1.08	94.06	38.49	-149.41	-198.01	-247.41	-101.58	N.A.	N.A.	N.A.	N.A.	N.A.
For z = hr:	19.74	1.08	94.06	38.49	-149.41	-198.01	-247.41	-101.58	N.A.	N.A.	N.A.	N.A.	N.A.
For z = he:	19.74	1.08	94.06	38.49	-149.41	-198.01	-247.41	-101.58	N.A.	N.A.	N.A.	N.A.	N.A.
For z = h:	19.74	1.08	94.06	38.49	-149.41	-198.01	-247.41	-101.58	N.A.	N.A.	N.A.	N.A.	N.A.

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 2 (edge), 'a' = 3.70 ft.
3. Width of Zone 3 (corner), 'a' = 3.70 ft.
4. For monoslope roofs with 8 <= 3 degrees, use Fig. 30.4-2A (2010) & use Fig. 30.3-2A (2016) for 'GCp' values with 'qh'. >16PSF
5. For buildings with h > 60' and 8 > 10 degrees, use Fig. 30.6-1 (2010) for 'GCpi' values with 'qh'.
6. For all buildings with overhangs, use 'GCp' values per Chapter. 30.
7. If a parapet >= 3' in height is provided around perimeter of roof with 8 <= 10 degrees, Zone 3 shall be treated as Zone 2.
8. Per Code Section 30, the minimum wind load for C&C shall not be less than 16 psf.
9. References : a. ASCE 7-10 & ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures".

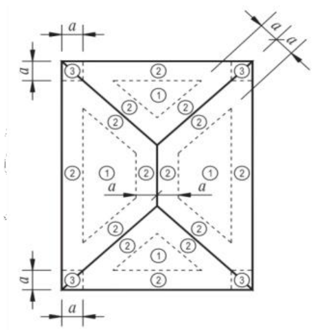
Roof Components and Cladding: 2010





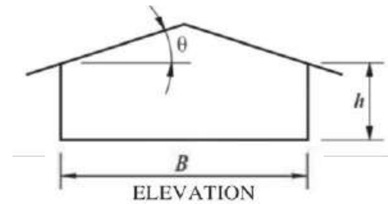
8 <= 7 deg. | 7 deg. < 8 <= 27 deg. | 27 deg. < 8 <= 45 deg.

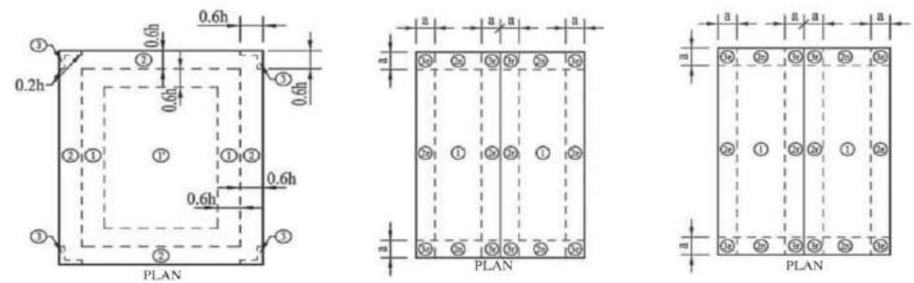
Roof Zones for Buildings with h <= 60 ft.
(for Gable Roofs <= 45° and Monoslope Roofs <= 3°)



Roof Zones for Buildings with h <= 60 ft.
(for Hip Roofs 7° < θ ≤ 45°)

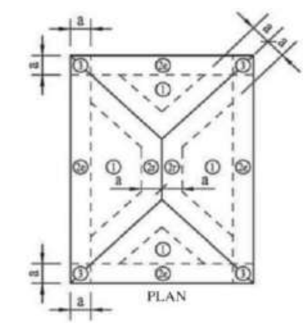
Roof Components and Cladding: 2016

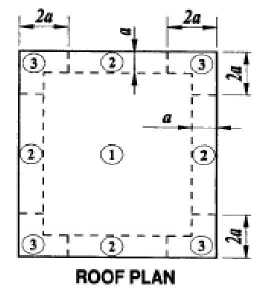




8 <= 7 deg. | 7 deg. < 8 <= 27 deg. | 27 deg. < 8 <= 45 deg.

Roof Zones for Buildings with h <= 60 ft.
(for Gable Roofs <= 45° and Monoslope Roofs <= 3°)

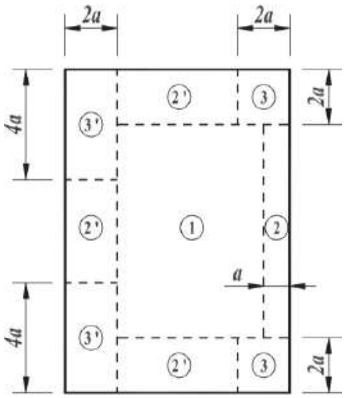




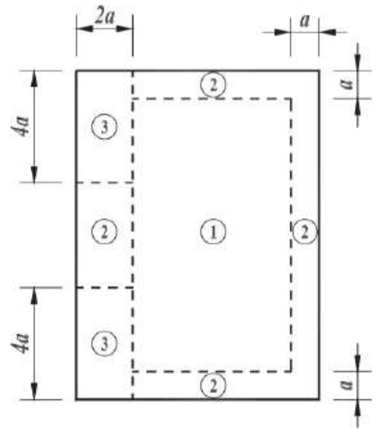
Roof Zones for Buildings with h <= 60 ft.
(for Hip Roofs 7° < θ ≤ 45°)

Roof Zones for Buildings with h > 60 ft.

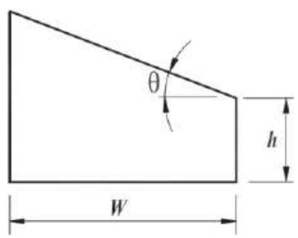
Roof Components and Cladding: 2010



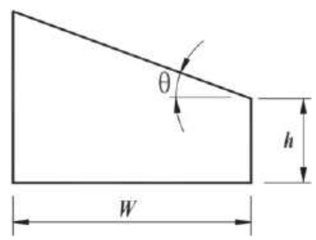
Plan



Plan



Elevation

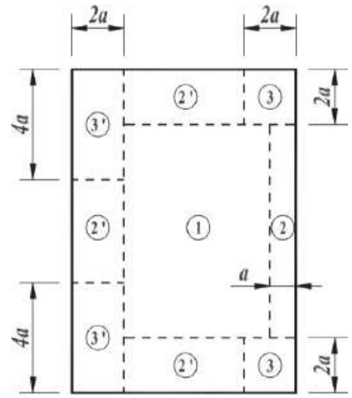


Elevation

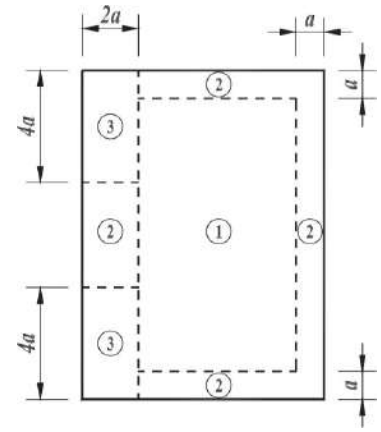
Roof Zones for Buildings with h <= 60 ft.
Monoslope Roofs, 3° < θ ≤ 10°

Roof Zones for Buildings with h <= 60 ft.
Monoslope Roofs, 10° < θ ≤ 30°

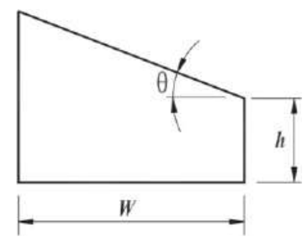
Roof Components and Cladding: 2016



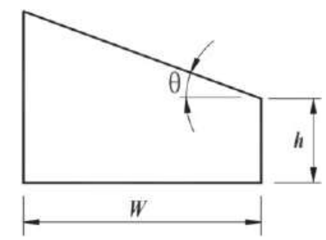
Plan



Plan



Elevation



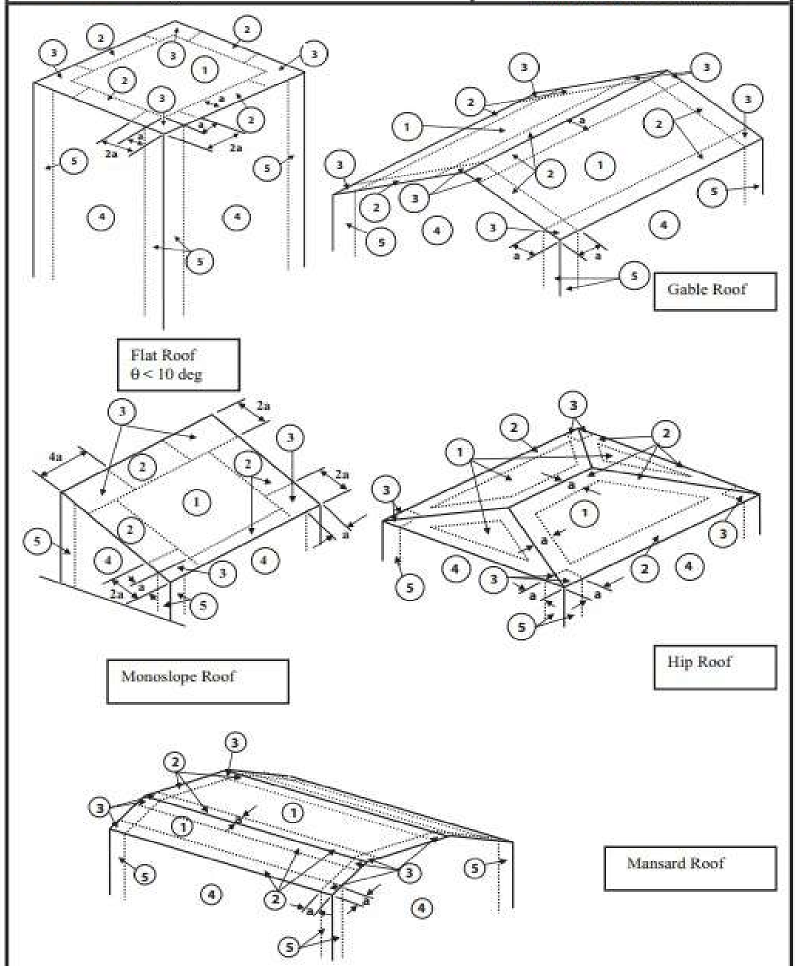
Elevation

Roof Zones for Buildings with h <= 60 ft.
Monoslope Roofs, 3° < θ ≤ 10°

Roof Zones for Buildings with h <= 60 ft.
Monoslope Roofs, 10° < θ ≤ 30°

Roof Components and Cladding: 2010

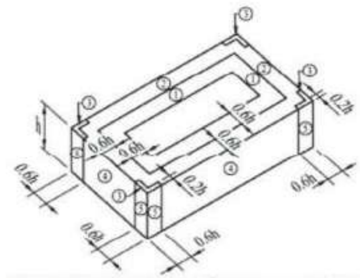
Components and Cladding – Part 4		h ≤ 160 ft.
Table 30.7-2	C & C Zones	C&C
Enclosed Buildings		Wall and Roof Pressures



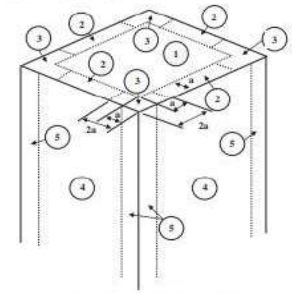
Roof Components and Cladding: 2016

Table 30.6-2 Components and Cladding, Part 4 [$h \le$ 160 ft ($h \le$ 48.8 m)]: C&C Zones for Enclosed Buildings—C&C Wall and Roof Pressures

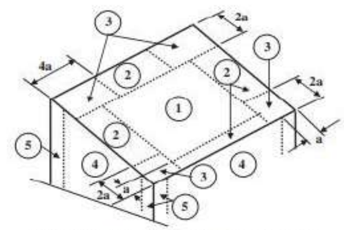
Parameters for Application of C&C Wall and Roof Pressures



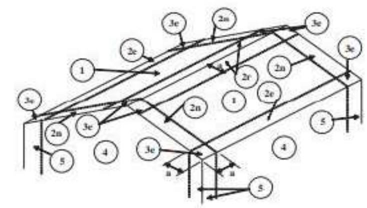
Flat/Hip/Gable/Mansard Roof: $\theta \le 7$ degrees; $h \le 60$ ft



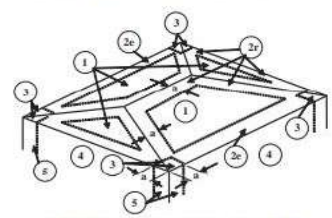
Flat Roof: $\theta \le 7$ degrees; $h > 60$ ft



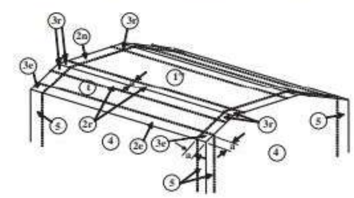
Monoslope Roof: $7 < \theta \le 30$ degrees; $h \le 160$ ft



Gable Roof: $7 < \theta \le 45$ degrees; $h \le 160$ ft



Hip Roof: $7 \le \theta \le 45$ degrees; $h \le 160$ ft



Mansard Roof: $7 \le \theta \le 45$ degrees; $h \le 160$ ft (See Note 2)

Notation

a = 10% of the least horizontal dimension but not less than 3 ft (0.9 m).
h = Mean roof height, in ft (m).
V = Basic wind speed, in mph (m/s).

Notes

1. See Section 30.6.1.1 for tabulated wall and roof pressure from Table 30.6-2 and Fig. 30.4-1, as applicable.
2. For mansard roofs, apply roof pressures on sloped surfaces as tabulated for sloped surfaces of gable roofs; apply roof pressure on flat surfaces ($\theta < 7°$) as tabulated for flat roofs.

Appendix- 3

WIND LOADING ANALYSIS - Main Wind-Force Resisting System

Per ASCE 7-16 Code for Enclosed or Partially Enclosed Buildings

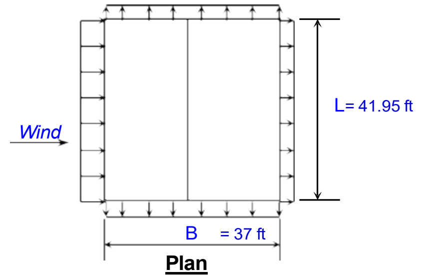
Using Method 2: Analytical Procedure (Section 27) for Buildings of Any Height

Note: This program assumes buildings are a maximum of 500 feet tall.

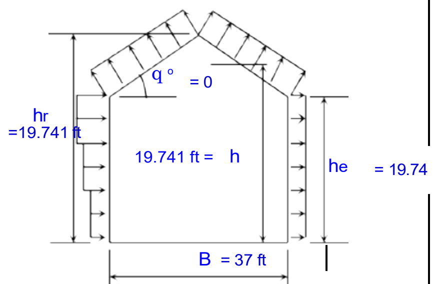
Project Name:	Exuma, Bahamas	Subject:			
Project Number:	242201	Calc. by :	SK	Checked by:	MS

Input Data:

Wind Direction =	Normal	(Normal or Parallel to building ridge)
Wind Speed, V =	200	mph (Wind Map, Figure 26.5-1A-D)
Bldg. Classification =	II	(Table 1.5-1 Risk Cat.)
Exposure Category =	D	(Sect. 26.7)
Ridge Height, hr =	19.74	ft. (hr >= he)
Eave Height, he =	19.74	ft. (he <= hr)
Building Width =	37.00	ft. (Normal to Building Ridge)
Building Length =	41.95	ft. (Parallel to Building Ridge)
Roof Type =	Flat	(Gable or Monoslope or Hip or Flat)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Table 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Enclosed? (Y/N)	Y	(Sect 26.2 & Figure 26.13-1)
Hurricane Region?	N	
Damping Ratio, b =	0.050	(Suggested Range = 0.010-0.070)
		(Suggested Range = 0.020-0.035) &
Period Coef., Ct =	0.0350	(Assume: T = Ct*h^(3/4) , and f = 1/T)
Parapet ? =	N	
Height of Parapet =	0	ft.



Plan



Elevation

L = 37 ft.
B = 41.95 ft.

Resulting Parameters and Coefficients:

Roof Angle, q =	0.00	deg.		
Mean Roof Ht., h =	19.74	ft. (h = he, for roof angle <=10 deg.)		
Windward Wall Cp =	0.80	(Fig. 27.3-1)		
Leeward Wall Cp =	-0.50	(Fig. 27.3-1)		
Side Walls Cp =	-0.70	(Fig. 27.3-1)		
Roof Cp (zone #1) =	-0.92	-0.18	(Fig. 27.3-1)	(zone #1 for 0 to h/2)
Roof Cp (zone #2) =	-0.89	-0.18	(Fig. 27.3-1)	(zone #2 for h/2 to h)
Roof Cp (zone #3) =	-0.51	-0.18	(Fig. 27.3-1)	(zone #3 for h to 2*h)
Roof Cp (zone #4) =	N.A.	N.A.	(Fig. 27.3-1)	(zone #4 for > 2*h)
+GCpi Coef. =	0.18	(Table 26.13 (positive internal pressure)		
-GCpi Coef. =	-0.18	(Table 26.13 (negative internal pressure)		

If z <= 15 then: Kz = 2.01*(15/zg)^(2/a) , If z > 15 then: Kz = 2.01*(z/zg)^(2/a) (Table 26.10-1)

a =	11.50	zg =	700	(Table 26.11-1)
For Wall Kh =	1.08	(Kh = Kz evaluated at z = h)		
For Parapet Kh =	N.A.	(Kh = Kz evaluated at z = hp)		

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2*I (Eq. 26.10-1)

qh =	94.06	psf	qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)	
Ratio h/L =	0.534	freq., f =	3.051 hz.	(f >= 1, Rigid structure)
Gust Factor, G =	0.850	(Sect. 26.11)		
For Parapet qp =	N.A.	psf	qp = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = hp)	

Design Net External Wind Pressures (Sect. 27.3):

p = qz*G*Cp - qi*(+/-GCpi) for windward wall (psf), where: qi =qh (Eq. 27.3-1)
p = qh*G*Cp - qi*(+/-GCpi) for leeward wall, sidewalls, and roof (psf), where: qi = qh (Eq. 27.3-1)

Normal to Ridge Wind Load Tabulation for MWFRS - Buildings of Any Height

Surface	z (ft.)	Kz	qz (psf)	GCp	p = Net Design Press. (psf) (w/ +GCpi)	(w/ -GCpi)
Windward Wall "A"	0	1.03	89.67	0.80	44.05	77.91
	15.00	1.03	89.67	0.80	44.05	77.91
For z = hr:	19.74	1.08	94.06	0.80	47.03	80.89
For z = he:	19.74	1.08	94.06	0.80	47.03	80.89
For z = h:	19.74	1.08	94.06	0.80	47.03	80.89
Leeward Wall "B"	All	-	-	-0.50	-56.91	-23.04
Side Walls "C"	All	-	-	-0.70	-72.90	-39.03
Side Walls "D"	All	-	-	-0.70	-72.90	-39.03
Roof (zone #1) Case. 1	-	-	-	-0.92	-90.18	-56.32
Roof (zone #1) Case. 2	-	-	-	-0.18	-31.32	2.54
Roof (zone #2) Case. 1	-	-	-	-0.89	-87.81	-53.95
Roof (zone #2) Case. 2	-	-	-	-0.18	-31.32	2.54
Roof (zone #3) Case. 1	-	-	-	-0.92	-90.18	-56.32
Roof (zone #3) Case. 2	-	-	-	-0.18	-31.32	2.54
Roof (zone #4) Case. 1	-	-	-	-0.89	-87.81	-53.95
Roof (zone #4) Case. 2	-	-	-	-0.18	-31.32	2.54
Roof (zone #5) Case. 1	-	-	-	-0.51	-57.98	-24.12
Roof (zone #5) Case. 2	-	-	-	-0.18	-31.32	2.54
Roof (zone #6) Case. 1					#VALUE!	#VALUE!
Roof (zone #6) Case. 2					#VALUE!	#VALUE!

Surface	z (ft.)	Kz	qp (psf)	GCpn	p = Net Design Press. (psf) (w/ +GCpi)	(w/ -GCpi)
Parapet wall calculation						
	N.A.	1.08	N.A.		N.A.	N.A.
	N.A.	1.08	N.A.		N.A.	N.A.

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Per Code Section 27.1.5, the minimum wind load for MWFRS shall not be less than 16 psf.
3. Roof zone #1 is applied for windward face.
4. Roof zone #2 is applied for leeward face.
5. Roof zone #3 is applied for horizontal distance of 0 to h/2 from windward edge.
6. Roof zone #4 is applied for horizontal distance of h/2 to h from windward edge.
7. Roof zone #5 is applied for horizontal distance of h to 2*h from windward edge.
8. Roof zone #6 is applied for horizontal distance of > 2*h from windward edge.
9. Case 1 is for +ve GCpi (+0.18).
10. Case 2 is for -ve GCpi (-0.18).

Wall Output	Zone	Windward 'A'	Leeward 'B'	Side wall 'C'	Sidewall 'D'	A+B
Cond. =Normal to Ridge Line	Case 1 +Gcpi	47.03	-56.91	-72.90	-72.90	103.93
	Case 2 -Gcpi	80.89	-23.04	-39.03	-39.03	103.93

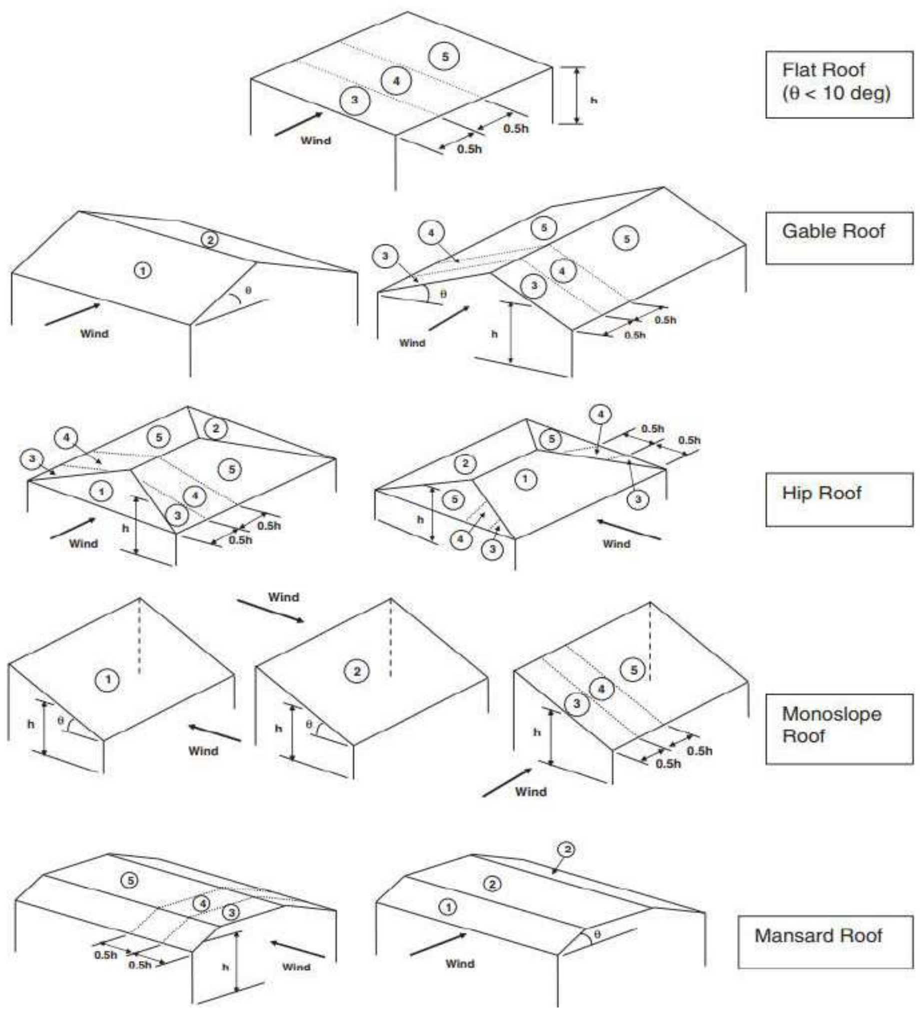
Notes: "A+B" column value is abslute value.(without considering +/- sign)

Roof Output	Zone	1	2	3	4	5	6	Max.
Cond. =Normal to Ridge Line	Case 1 +Gcpi	N.A.	N.A.	-90.18	-87.81	-57.98	#VALUE!	90.18
	Case 2 -Gcpi	N.A.	N.A.	2.54	2.54	2.54	#VALUE!	2.54

Notes: "Max." column value is abslute value.(without considering +/- sign)

Table 27.5-2 (*Continued*). Main Wind Force Resisting System, Part 2 [$h \leq 160$ ft ($h \leq 48.8$ m)]: Enclosed Simple Diaphragm Buildings—Wind Pressures—Roofs

Parameters for Application of Roof Pressures



Determination of Gust Effect Factor, G:

Is Building Flexible? No f >=1 Hz.

1: Simplified Method for Rigid Building

G = 0.850

Parameters Used in Both Item #2 and Item #3 Calculations (from Table 26.9-1):

a^ =	0.087	
b^ =	1.07	
a(bar) =	0.111	
b(bar) =	0.80	
c =	0.15	
l =	650	ft.
e(bar) =	0.125	
z(min) =	7	ft.

Calculated Parameters Used in Both Rigid and/or Flexible Building Calculations:

z(bar) =	11.84	= 0.6*h , but not < z(min) , ft. Table 26.9-1
Iz(bar) =	0.178	= c*(33/z(bar))^(1/6) , Eq. 26.9-7
Lz(bar) =	571.86	= *l**(z(bar)/33)^(e(bar)) , Eq. 26.9-9
gq =	3.4	(3.4, per Sect. 26.9.4)
gv =	3.4	(3.4, per Sect. 26.9.4)
gr =	4.447	= (2*(LN(3600*f)))^(1/2)+0.577/(2*LN(3600*f))^(1/2) , Eq. 26.9-11
Q =	0.931	= (1/(1+0.63*((B+h)/Lz(bar))^0.63))^(1/2) , Eq. 26.9-8

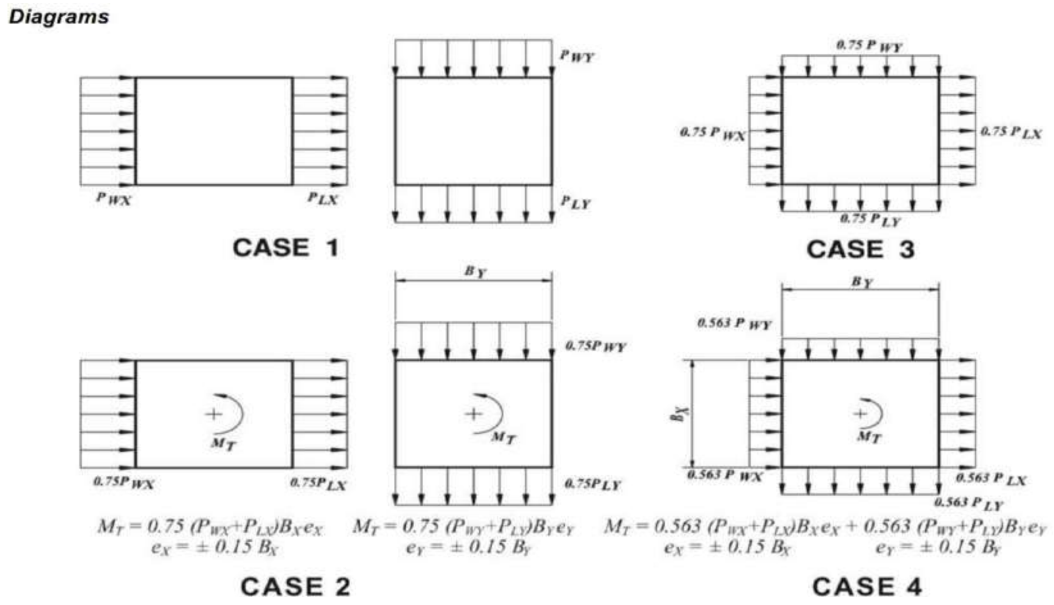
2: Calculation of G for Rigid Building

G = 0.892 = 0.925*((1+1.7*gq*Iz(bar)*Q)/(1+1.7*gv*Iz(bar))) , Eq. 26.9-6

3: Calculation of Gf for Flexible Building

b =	0.050	Damping Ratio
Ct =	0.035	Period Coefficient
T =	0.328	= Ct*h^(3/4) , sec. (Approximate fundamental period)
f =	3.051	= 1/T , Hz. (Natural Frequency)
V(fps) =	N.A.	= V(mph)*(88/60) , ft./sec.
V(bar,zbar) =	N.A.	= b(bar)*(z(bar)/33)^(a(bar))*V*(88/60) , ft./sec. , Eq. 26.9-16
N1 =	N.A.	= f*Lz(bar)/(V(bar,zbar)) , Eq. 26.9-14
Rn =	N.A.	= 7.47*N1/(1+10.3*N1)^(5/3) , Eq. 26.9-13
hh =	N.A.	= 4.6*f*h/(V(bar,zbar))
Rh =	N.A.	= (1/hh)-1/(2*hh^2)*(1-e^(-2*hh)) for hh>0, or = 1 for hh=0 ,Eq. 26.9-15a, b
hb =	N.A.	= 4.6*f*B/(V(bar,zbar))
RB =	N.A.	= (1/hb)-1/(2*hb^2)*(1-e^(-2*hb)) for hb>0, or = 1 for hb=0,Eq. 26.9-15a, b
hd =	N.A.	= 15.4*f*L/(V(bar,zbar))
RL =	N.A.	= (1/hd)-1/(2*hd^2)*(1-e^(-2*hd)) for hd>0, or = 1 for hd=0 ,Eq. 26.9-15a, b
R =	N.A.	= ((1/b)*Rn*Rh*RB*(0.53+0.47*RL))^(1/2) , Eq. 26.9-12
Gf =	N.A.	= 0.925*(1+1.7*Iz(bar)*(gq^2*Q^2+gr^2*R^2)^(1/2))/(1+1.7*gv*Iz(bar)) ,
Use: G =	0.850	Eq. 26.9-10

Figure 6-9 - Design Wind Load Cases of MWFRS for Buildings of All Heights



Case 1: Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2: Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3: Wind pressure as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4: Wind pressure as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:
1. Design wind pressures for windward (Pw) and leeward (PL) faces shall be determined in accordance with the provisions of Section 27.4.1 and 27.4.2 as applicable for buildings of all heights.
2. Above diagrams show plan views of building.
3. Notation:
   - P_{wx}, P_{wy} = Windward face pressure acting in the X, Y principal axis, respectively.
   - P_{Lx}, P_{Ly} = Leeward face pressure acting in the X, Y principal axis, respectively.
   - e (e_x, e_y) = Eccentricity for the X, Y principal axis of the structure, respectively.
   - M_T = Torsional moment per unit height acting about a vertical axis of the building.

Appendix- 4

WIND LOADING ANALYSIS - Main Wind-Force Resisting System

Per ASCE 7-16 Code for Enclosed or Partially Enclosed Buildings

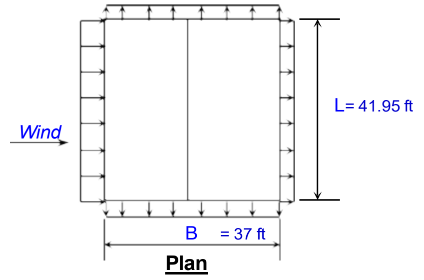
Using Method 2: Analytical Procedure (Section 27) for Buildings of Any Height

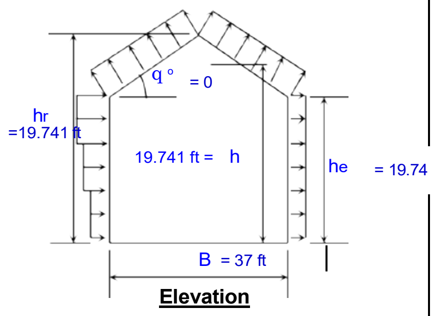
Note: This program assumes buildings are a maximum of 500 feet tall.

Project Name:	Exuma, Bahamas	Subject:			
Project Number:	242201	Calc. by :	SK	Checked by:	MS

Input Data:

Wind Direction =	Parallel	(Normal or Parallel to building ridge)
Wind Speed, V =	200	mph (Wind Map, Figure 26.5-1A-D)
Bldg. Classification =	II	(Table 1.5-1 Risk Cat.)
Exposure Category =	D	(Sect. 26.7)
Ridge Height, hr =	19.74	ft. (hr >= he)
Eave Height, he =	19.74	ft. (he <= hr)
Building Width =	37.00	ft. (Normal to Building Ridge)
Building Length =	41.95	ft. (Parallel to Building Ridge)
Roof Type =	Flat	(Gable or Monoslope or Hip or Flat)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Table 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Enclosed? (Y/N)	Y	(Sect 26.2 & Figure 26.13-1)
Hurricane Region?	N	
Damping Ratio, b =	0.050	(Suggested Range = 0.010-0.070)
		(Suggested Range = 0.020-0.035) &
Period Coef., Ct =	0.0350	(Assume: T = Ct*h^(3/4) , and f = 1/T)
Parapet ? =	N	
Height of Parapet =	0	ft.





L = 41.95 ft.
B = 37 ft.

Resulting Parameters and Coefficients:

Roof Angle, q =	0.00	deg.		
Mean Roof Ht., h =	19.74	ft. (h = he, for roof angle <=10 deg.)		
Windward Wall Cp =	0.80	(Fig. 27.3-1)		
Leeward Wall Cp =	-0.47	(Fig. 27.3-1)		
Side Walls Cp =	-0.70	(Fig. 27.3-1)		
Roof Cp (zone #1) =	-0.90	-0.18	(Fig. 27.3-1)	(zone #1 for 0 to h/2)
Roof Cp (zone #2) =	-0.90	-0.18	(Fig. 27.3-1)	(zone #2 for h/2 to h)
Roof Cp (zone #3) =	-0.50	-0.18	(Fig. 27.3-1)	(zone #3 for h to 2*h)
Roof Cp (zone #4) =	-0.30	-0.18	(Fig. 27.3-1)	(zone #4 for > 2*h)
+GCpi Coef. =	0.18	(Table 26.13 (positive internal pressure)		
-GCpi Coef. =	-0.18	(Table 26.13 (negative internal pressure)		

If z <= 15 then: Kz = 2.01*(15/zg)^(2/a) , If z > 15 then: Kz = 2.01*(z/zg)^(2/a) (Table 26.10-1)

a =	11.50	zg =	700	(Table 26.11-1)
For Wall Kh =	1.08	(Kh = Kz evaluated at z = h)		
For Parapet Kh =	N.A.	(Kh = Kz evaluated at z = hp)		

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2*I (Eq. 26.10-1)

qh =	94.06	psf	qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)		
Ratio h/L =	0.471	freq., f =	3.051	hz.	(f >= 1, Rigid structure)
Gust Factor, G =	0.850	(Sect. 26.11)			
For Parapet qp =	N.A.	psf	qp = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = hp)		

Design Net External Wind Pressures (Sect. 27.3):

p = qz*G*Cp - qi*(+/-GCpi) for windward wall (psf), where: qi =qh (Eq. 27.3-1)
p = qh*G*Cp - qi*(+/-GCpi) for leeward wall, sidewalls, and roof (psf), where: qi = qh (Eq. 27.3-1)

Parallel to Ridge Wind Load Tabulation for MWFRS - Buildings of Any Height

Surface	z (ft.)	Kz	qz (psf)	GCp	p = Net Design Press. (psf) (w/ +GCpi)	(w/ -GCpi)
Windward Wall "A"	0	1.03	89.67	0.80	44.05	77.91
	15.00	1.03	89.67	0.80	44.05	77.91
For z = hr:	19.74	1.08	94.06	0.80	47.03	80.89
For z = he:	19.74	1.08	94.06	0.80	47.03	80.89
For z = h:	19.74	1.08	94.06	0.80	47.03	80.89
Leeward Wall "B"	All	-	-	-0.47	-54.77	-20.91
Side Walls "C"	All	-	-	-0.70	-72.90	-39.03
Side Walls "D"	All	-	-	-0.70	-72.90	-39.03
Roof (zone #1) Case. 1	-	-	-	N.A.	-	-
Roof (zone #1) Case. 2	-	-	-	N.A.	-	-
Roof (zone #2) Case. 1	-	-	-	N.A.	-	-
Roof (zone #2) Case. 2	-	-	-	N.A.	-	-
Roof (zone #3) Case. 1	-	-	-	-0.90	-88.89	-55.02
Roof (zone #3) Case. 2	-	-	-	-0.18	-31.32	2.54
Roof (zone #4) Case. 1	-	-	-	-0.90	-88.89	-55.02
Roof (zone #4) Case. 2	-	-	-	-0.18	-31.32	2.54
Roof (zone #5) Case. 1	-	-	-	-0.50	-56.91	-23.04
Roof (zone #5) Case. 2	-	-	-	-0.18	-31.32	2.54
Roof (zone #6) Case. 1	-	-	-	-0.30	-40.92	-7.05
Roof (zone #6) Case. 2	-	-	-	-0.18	-31.32	2.54

Surface	z (ft.)	Kz	qp (psf)	GCpn	p = Net Design Press. (psf) (w/ +GCpi)	(w/ -GCpi)
Parapet wall calculation						
	N.A.	1.08	N.A.		N.A.	N.A.
	N.A.	1.08	N.A.		N.A.	N.A.

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Per Code Section 27.1.5, the minimum wind load for MWFRS shall not be less than 16 psf.
3. Roof zone #1 is applied for windward face.
4. Roof zone #2 is applied for leeward face.
5. Roof zone #3 is applied for horizontal distance of 0 to h/2 from windward edge.
6. Roof zone #4 is applied for horizontal distance of h/2 to h from windward edge.
7. Roof zone #5 is applied for horizontal distance of h to 2*h from windward edge.
8. Roof zone #6 is applied for horizontal distance of > 2*h from windward edge.
9. Case 1 is for +ve GCpi (+0.18).
10. Case 2 is for -ve GCpi (-0.18).

Wall Output Cond. =Parallel to Ridge Line	Zone	Windward 'A'	Leeward 'B'	Side wall 'C'	Sidewall 'D'	A+B
	Case 1 +Gcpi	47.03	-54.77	-72.90	-72.90	101.80
	Case 2 -Gcpi	80.89	-20.91	-39.03	-39.03	101.80

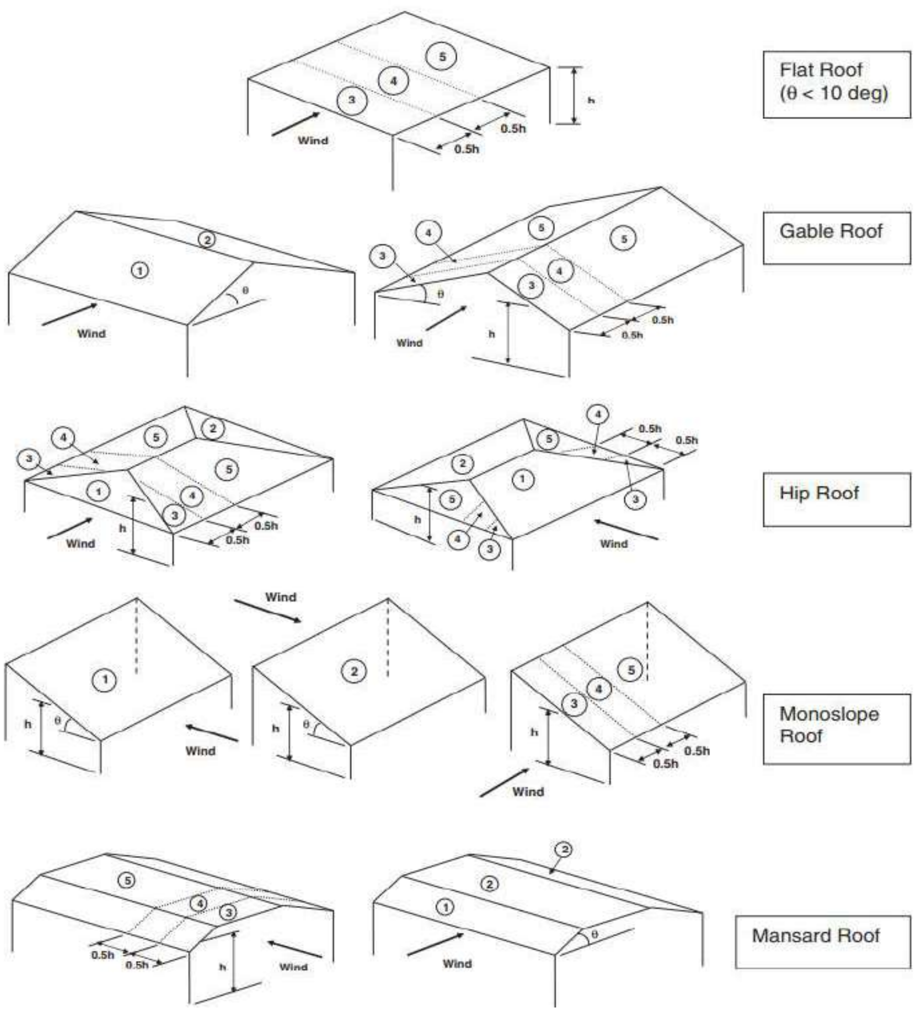
Notes: "A+B" column value is abslute value.(without considering +/- sign)

Roof Output Cond. =Parallel to Ridge Line	Zone	1	2	3	4	5	6	Max.
	Case 1 +Gcpi	N.A.	N.A.	-88.89	-88.89	-56.91	-40.92	88.89
	Case 2 -Gcpi	N.A.	N.A.	2.54	2.54	2.54	2.54	2.54

Notes: "Max." column value is abslute value.(without considering +/- sign)

Table 27.5-2 (*Continued*). Main Wind Force Resisting System, Part 2 [$h \leq 160$ ft ($h \leq 48.8$ m)]: Enclosed Simple Diaphragm Buildings—Wind Pressures—Roofs

Parameters for Application of Roof Pressures



Determination of Gust Effect Factor, G:

Is Building Flexible? No   f >=1 Hz.

1: Simplified Method for Rigid Building

G = 0.850

Parameters Used in Both Item #2 and Item #3 Calculations (from Table 26.9-1):

a^ =	0.087	
b^ =	1.07	
a(bar) =	0.111	
b(bar) =	0.80	
c =	0.15	
l =	650	ft.
e(bar) =	0.125	
z(min) =	7	ft.

Calculated Parameters Used in Both Rigid and/or Flexible Building Calculations:

z(bar) =	11.84	= 0.6*h , but not < z(min) , ft. Table 26.9-1
Iz(bar) =	0.178	= c*(33/z(bar))^(1/6) , Eq. 26.9-7
Lz(bar) =	571.86	= *l**(z(bar)/33)^(e(bar)) , Eq. 26.9-9
gq =	3.4	(3.4, per Sect. 26.9.4)
gv =	3.4	(3.4, per Sect. 26.9.4)
gr =	4.447	= (2*(LN(3600*f)))^(1/2)+0.577/(2*LN(3600*f))^(1/2) , Eq. 26.9-11
Q =	0.934	= (1/(1+0.63*((B+h)/Lz(bar))^0.63))^(1/2) , Eq. 26.9-8

2: Calculation of G for Rigid Building

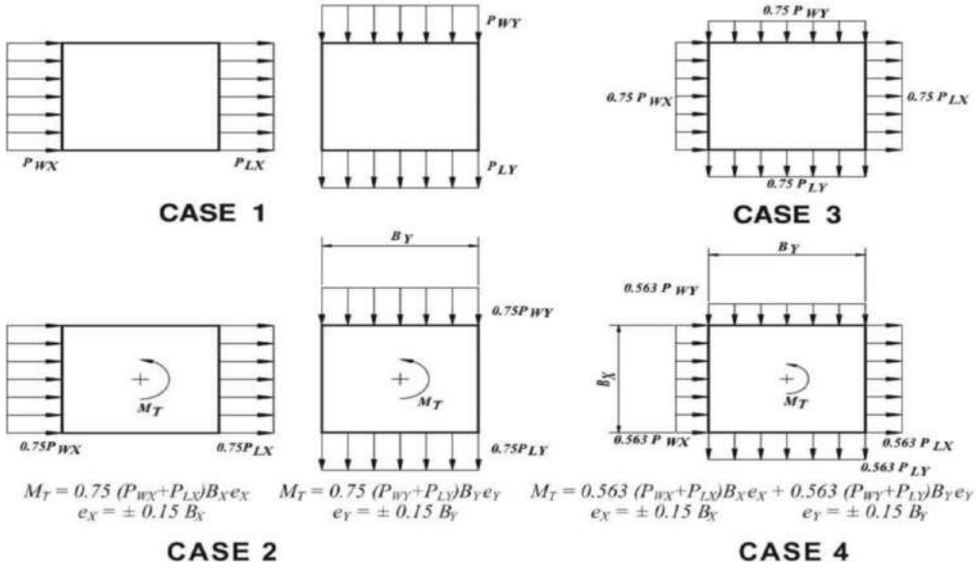
G = 0.894 = 0.925*((1+1.7*gq*Iz(bar)*Q)/(1+1.7*gv*Iz(bar))) , Eq. 26.9-6

3: Calculation of Gf for Flexible Building

b =	0.050	Damping Ratio
Ct =	0.035	Period Coefficient
T =	0.328	= Ct*h^(3/4) , sec. (Approximate fundamental period)
f =	3.051	= 1/T , Hz. (Natural Frequency)
V(fps) =	N.A.	= V(mph)*(88/60) , ft./sec.
V(bar,zbar) =	N.A.	= b(bar)*(z(bar)/33)^(a(bar))*V*(88/60) , ft./sec. , Eq. 26.9-16
N1 =	N.A.	= f*Lz(bar)/(V(bar,zbar)) , Eq. 26.9-14
Rn =	N.A.	= 7.47*N1/(1+10.3*N1)^(5/3) , Eq. 26.9-13
hh =	N.A.	= 4.6*f*h/(V(bar,zbar))
Rh =	N.A.	= (1/hh)-1/(2*hh^2)*(1-e^(-2*hh)) for hh>0, or = 1 for hh=0 ,Eq. 26.9-15a, b
hb =	N.A.	= 4.6*f*B/(V(bar,zbar))
RB =	N.A.	= (1/hb)-1/(2*hb^2)*(1-e^(-2*hb)) for hb>0, or = 1 for hb=0,Eq. 26.9-15a, b
hd =	N.A.	= 15.4*f*L/(V(bar,zbar))
RL =	N.A.	= (1/hd)-1/(2*hd^2)*(1-e^(-2*hd)) for hd>0, or = 1 for hd=0 ,Eq. 26.9-15a, b
R =	N.A.	= ((1/b)*Rn*Rh*RB*(0.53+0.47*RL))^(1/2) , Eq. 26.9-12
Gf =	N.A.	= 0.925*(1+1.7*Iz(bar)*(gq^2*Q^2+gr^2*R^2)^(1/2))/(1+1.7*gv*Iz(bar)) ,
Use: G =	0.850	Eq. 26.9-10

Figure 6-9 - Design Wind Load Cases of MWFRS for Buildings of All Heights

Diagrams



Case 1: Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2: Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3: Wind pressure as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4: Wind pressure as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes: 1. Design wind pressures for windward (Pw) and leeward (PL) faces shall be determined in accordance with the provisions of Section 27.4.1 and 27.4.2 as applicable for buildings of all heights.

2. Above diagrams show plan views of building.

3. Notation:

P_{wx}, P_{wy} = Windward face pressure acting in the X, Y principal axis, respectively.

P_{Lx}, P_{Ly} = Leeward face pressure acting in the X, Y principal axis, respectively.

e (e_x, e_y) = Eccentricity for the X, Y principal axis of the structure, respectively.

M_T = Torsional moment per unit height acting about a vertical axis of the building.

Appendix- 5

Job: EXUMA, BAHAMAS 242201

PART II: Seismic Analysis

Seismic Load Calculations Using Equivalent Lateral Force Procedure per ASCE 7-10 **Date:** 12-02-2024

Seismic Weight Calculations

External Wall DL = 20 psf
Internal Wall DL = 15 psf
Roof area = 1569.98 sq ft

Ground to Roof level W calculations

Dead load of Roof = 30 psf
Roof live load = 20 psf
Height of external wall = 10.21 ft
Height of Internal wall = 10.21 ft
Perimeter of external wall = 158.45 ft
Perimeter of Internal wall = 137.104 ft
Dead load of wall (per ft2 area) = 33.98 psf (Perimeter x Height of Wall / Area) x DL
Total Load = 84 psf (25% Total live load)

Seismic Force Calculations

Site Class = D
Seismic Occupancy Category = II
Importance factor (I)= 1
Mapped MCE spectral Response acceleration for short period S_S = 0.010 g
Mapped MCE spectral Response acceleration for 1 second period S_1 = 0.012 g

Site Coefficient Fa = 1.6
Site Coefficient Fv = 2.4 (As per ATC hazard website Fv=2.4 considered)

MCE spectral Response acceleration for short period SMS = Fa x SS = 0.016 g
MCE spectral Response acceleration for 1 second period SM1 = Fv x S1 = 0.029 g
Design spectral Response acceleration for short period SDS = 2/3 x SMS = 0.011 g
Design spectral Response acceleration for 1 second period SD1 = 2/3 x SM1 = 0.019 g

Approximate fundamental period calculations

Height of building = 11.51 ft
C_t = 0.028 From table 12.8-2, ASCE 7-2010
x = 0.8 From table 12.8-2, ASCE 7-2010

$T_a = C_t \times h_n^x$ = 0.20 Sec Approximate fundamental period "T_a" Using Eq. 12.8-7
T_L = 8.00 Sec From Fig 22-14, ASCE 7-2010 (Assumed)
Seismic Design Category (SDC) = A Table 11.6-1 ASCE 7-2010
Response Modification Factor R = 3.25 Per table 12.2-1, ASCE 7-2010
Maximum C_S (If $T_a < T_L$) = S_{D1} / (T x (R / I)) = 0.03 Per Eq 12.8-3, ASCE 7-2010
Maximum C_S (If $T_a > T_L$) = $S_{D1} \times T_L$ / (T^2 x (R / I)) = 1.21 Per Eq 12.8-4, ASCE 7-2010
Minimum C_S (If S_1 < 0.6g) = 0.01 Per Eq 12.8-5, ASCE 7-2010
Minimum C_S (If S_1 => 0.6g) = 0.5 S_1 / (R / I) = 0.002 Per Eq 12.8-6, ASCE 7-2010

Seismic Response Coefficient $C_S = S_{DS}$/(R / I) = 0.003 Per Eq 12.8-2, ASCE 7-2010 OK
Seismic base shear V = C_S x W = 0.004 x W ASCE 7 -10 Section 12.8.3
Total weight of building (W_{Total}) = 131.9
Base Shear 0.43 kips

Over strength Factor (Ω) = 2

Vertical Shear Force Calculations

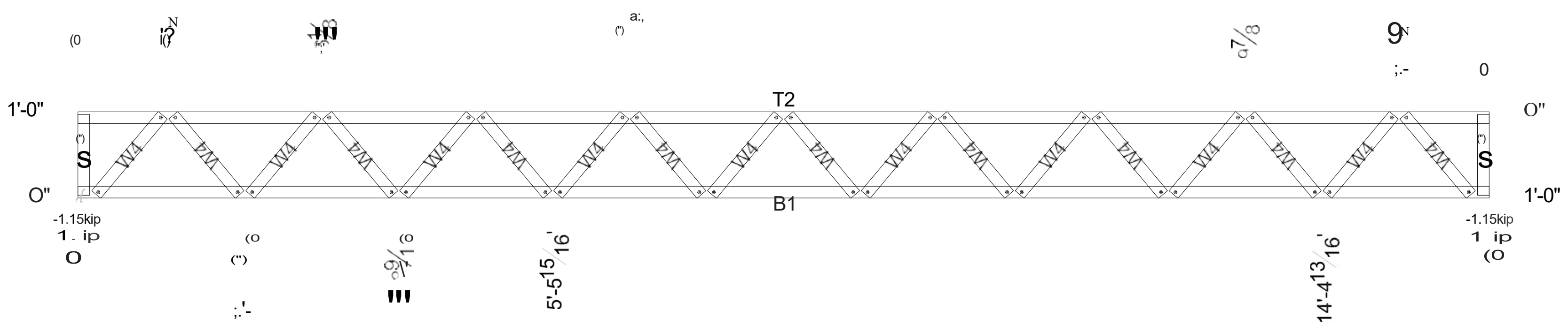
Floor	Area of floor (ft^2)	Weight of floor Wx (kips)	Height of story above base hx (ft)	Wxhx	Seismic distribution factor (Wxhx/∑Wihi)	Storey Level Shear Force (kips)	Amplified Shear force at floor level Ω_0V (kips)
Roof	1570.0	131.9	10.2	1346.21	1.00	0.43	0.87
Total		**131.85**	**10.21**	**1346.21**	**1.00**	**0.43**	**0.87**

B1 100S41-155-350 16'-4"	T2 100S41-155-350 16'-4"	W3 100S41-155-350 2 111/4"	W4 100S41-155-350 18 1'-2 3/8"
FRAMECAD 10g-19mm XDrive 80	Working Sheet: 16' feet	Assembly Weight 89.81bs	

Powered by FRAMECAD Structure®

Minimum number oft asteners required is 2 per joint

Appendix- 6



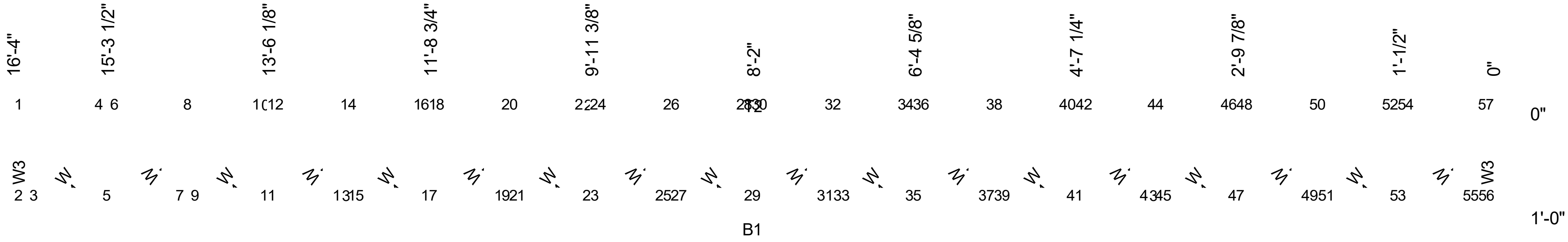
Quantity Required = 1 Mark as RJ2 Engineering Status = 52%

System Name	FRAMECAD FT m	Roof Type:	SHEET	Truss Model:	Default	Wind Speed:	200
Truss Pitch:	- - 0	Truss Spacing:	2'-0"	Design Code:	AISI S100-20 LRFD	Loading Code:	IBC 2018 LRFD
Top Chord Live Load (psf):	20.0	Top Chord Dead Load (psf):	20.0	Bottom Chord Dead Load(psf):	10.0	Number of on-coming Trusses:	0
Envelope:	1'-1/16"h x 16'-4"w						

ides global Dwg: 242201_Exuma, Bahamas roof truss View 1 of 1 Client: f– ·ck Daily Job

No: 242201

B1	100S41-155-350	1	16'-4"	T2	100S41-155-350	1	16'-4"	W3	100S41-155-350	2	11 1/4"	W4	100S41-155-350	18	1'-2 3/8"
Assembly Weight			89.8lbs	FRAMECAD 10g-19mm XDrive		80									



Bearing -1153 lbf
Uplift 1045 lbf

Bearing -1153 lbf
Uplift 1045 lbf

Quantity Required = 1 Mark as RJ2 Engineering Status = Passed

Minimum number of fasteners required is 2 per joint

LOADS & DESIGN FACTORS

WIND Factors

Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00

SNOW Factors

No Snow Load Apply

Wind Pressure Factors

Wup Bottom Chord	0.2
Wup Top Chord	0.9
Wdown Bottom Chord	0.2
Wdown Top Chord	0.2

DESIGN LOADING

Load Cases:	Type
G	Gravity(Dead)
Q	Roof Live
P	Concentrated Live
S	Snow
Wu	Wind Up
Wd	Wind Down

FASTENERS

Type:	Name	Qty
Main:	FRAMECAD 10g-19mm XDrive (SJ)	2
Sub:	FRAMECAD 10g-16mm Flathead(SJ)	Refer DWG

F# denotes total number fasteners in each member at joint.

Provide plate or reinforcing member where number of screw in a flange exceeds 2. or where screw spacing requirement can not be met.

Min Screws Spacing: 3 x Diameter.

Min Edge Distance: 2.2d for End Edge, or 1.5d for Side Edge.

BRACING

Provide Chord Restraints at specified spacing

Provide Rail Restraints at ends & over supports

Roof System Bracing to be Specified by Engineer

Member Effective Lengths for Design

Type	Kx	Lx	Ky	Ly	Kt	Lt
Chords	1.00	Panel	1.00	Rest.	1.00	Min(Lx,Ly)
Web	1.0	Panel	1.0	Panel	1.0	Panel

' Panel 'denotes distance between panel points (joints)

' Rest. 'denotes lateral restraint spacing

REINFORCING PLATES & MEMBERS

Notes: Plates Fixed on Both Sides,
Reinforcing Chord member may Substitute Plate.

Fasteners: FRAMECAD 10-g 16m Flathead

Members:Provide 2 fasteners at each end & each crossing member and at maximum 300mm(12") centres along member length.

Plates:Provide 2 fasteners at plate edges & each crossing member and at maximum 50mm(2") centres along member length.

Node: Plate:

MAXIMUM MEMBER AXIAL FORCES AND CRITICAL STRUCTURAL DESIGN INDEX

(All member actions are factored as per specified load combination)

Top Chord

Member	Com. lb	Ten. lb	CSI	LC
1-4	0	0	21%	(4)
4-10	-2148	1942	36%	(4)
10-16	-3708	3357	42%	(23)
16-22	-4744	4296	48%	(24)
22-28	-5262	4766	51%	(25)
28-34	-5360	4865	51%	(26)
34-40	-5100	4630	48%	(27)
40-46	-4322	3924	42%	(28)
46-52	-3034	2756	36%	(4)
52-57	-1137	1036	26%	(4)

Bottom Chord

Member	Com. lb	Ten. lb	CSI	LC
2-7	-1036	1137	52%	(4)
7-13	-2756	3034	33%	(5)
13-19	-3924	4322	39%	(5)
19-25	-4630	5100	42%	(5)
25-31	-4865	5360	44%	(5)
31-37	-4766	5262	43%	(5)
37-43	-4296	4744	39%	(5)
43-49	-3357	3708	33%	(5)
49-55	-1942	2148	52%	(4)
55-56	0	0	46%	(4)

Web

Member	Com. lb	Ten. lb	CSI	LC
2-1	0	0	0%	(0)
3-4	-1774	1617	16%	(4)
6-7	-1413	1578	13%	(5)
9-10	-1382	1269	12%	(4)
12-13	-938	1052	8%	(5)
15-16	-959	885	9%	(4)
18-19	-580	657	5%	(5)
21-22	-557	521	5%	(4)
24-25	-213	353	2%	(5)
27-28	-247	154	2%	(26)
30-31	-247	154	2%	(25)
33-34	-213	353	2%	(5)
36-37	-557	521	5%	(4)
39-40	-580	657	5%	(5)
42-43	-959	885	9%	(4)
45-46	-938	1052	8%	(5)
48-49	-1382	1269	12%	(4)
51-52	-1413	1578	13%	(5)
54-55	-1774	1617	16%	(4)
56-57	-43	70	0%	(5)

SPECIAL LOADING

Suspended Ceilings

No manually added Point Load applied

There are 0 on-coming locations on Girder.

On-coming trusses are.

Please refer to engineering report for further information.

DESIGN LOAD COMBINATIONS

LC1	(0.42Wu)	27%
LC2	(G)	24%
LC3	(max(Q,S))	15%
LC4	(1.2G+1.6Q)	52%
LC5	(0.9G+1.0Wu)	44%
LC6	(0.9G+1.0Ww)	6%
LC7	(0.9G+1.0Wl)	6%
LC8	(1.2G+1.0Ww+0.5max(Q,S))	31%
LC9	(1.2G+1.0Wl+0.5max(Q,S))	31%
LC10	(1.2G+0.5Ww+1.6max(Q,S))	50%
LC11	(1.2G+0.5Wl+1.6max(Q,S))	50%
LC12	(1.2G+1.6S)	29%
LC13-LC29	(1.2G+1.6P)	51%

DEFLECTION

LC	Max in	Node ID	Criteria in	Critical in	CSI
LC1	0.1902	28	Span/180,0.79	0.79	17.6%
LC2	0.1577	29	Span/180,0.79	0.79	14.6%
LC3	0.0964	28	Span/180,0.79	0.79	8.9%

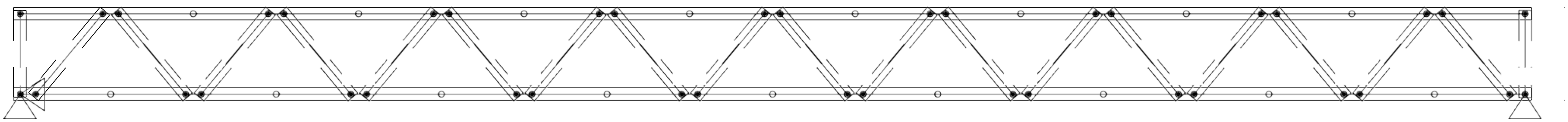
Framing System Name: FRAMECAD_FT_m	Top Chord Restraint : 3'-11 1/4"	Truss Spacing : 2'-0"	Roof Load Type: SHEET	Bottom Chord Dead Load (psf): 10.0	Top Chord Dead Load (psf): 20.0
Loading Code: IBC 2018 LRFD	Horizontal Chord Restraint : N/A	Shuttered Conditions: No	Ground Snow Load (psf): 0.0	Bottom Chord Live Load (psf): 0.0	Top Chord Live Load (psf): 20.0
Design Code: AISI S100-20 LRFD	Bottom Chord Restraint : 3'-11 1/4"		Wind speed (mph): 200	Bottom Chord Service Load (psf): 0.0	Concentrated Live Load (lb) : 247

Company: Bauhu

Dwg Name: 242201_Exuma, Bahamas roof truss MS Sheet 1 of 1

Project: Exuma, Bahamas

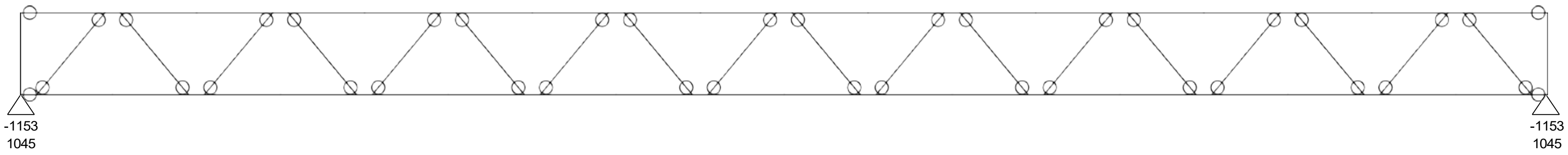
Job Number:242201



System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are the maximum of all load cases
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Summary for Truss RJ2
Engineering Status = 52%

LC1 (0.42Wu)	27%	LC2 (G)	24%	LC3 (max(Q, S))	15%	LC4 (1.2G + 1.6Q)	52%
LC5 (0.9G + 1.0Wu)	44%	LC6 (0.9G + 1.0Ww)	6%	LC7 (0.9G + 1.0Wl)	6%	LC8 (1.2G + 1.0Ww + 0.5max(Q, S))	31%
LC9 (1.2G + 1.0Wl + 0.5max(Q, S))	31%	LC10 (1.2G + 0.5Ww + 1.6max(Q, S))	50%	LC11 (1.2G + 0.5Wl + 1.6max(Q, S))	50%	LC12 (1.2G + 1.6S)	29%
LC13 (1.2G + 1.6P)	38%	LC14 (1.2G + 1.6P)	39%	LC15 (1.2G + 1.6P)	37%	LC16 (1.2G + 1.6P)	41%
LC17 (1.2G + 1.6P)	42%	LC18 (1.2G + 1.6P)	41%	LC19 (1.2G + 1.6P)	37%	LC20 (1.2G + 1.6P)	39%
LC21 (1.2G + 1.6P)	38%	LC22 (1.2G + 1.6P)	40%	LC23 (1.2G + 1.6P)	42%	LC24 (1.2G + 1.6P)	48%
LC25 (1.2G + 1.6P)	51%	LC26 (1.2G + 1.6P)	51%	LC27 (1.2G + 1.6P)	48%	LC28 (1.2G + 1.6P)	42%
LC29 (1.2G + 1.6P)	40%						

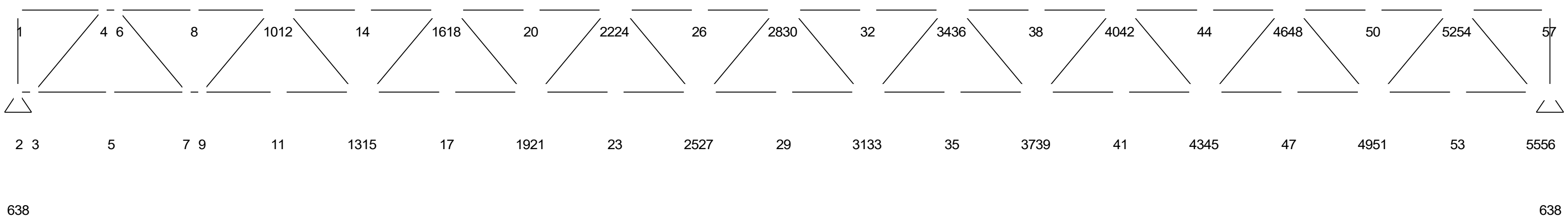
truss MS	Bahamas

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



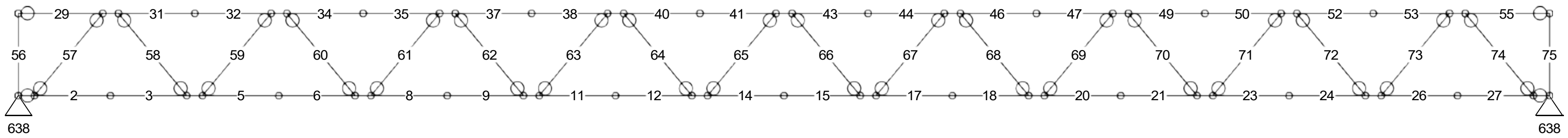
Node Diagram for Truss RJ2
Engineering Status = 52%

LC1 (0.42Wu)   Maximum for this Load Case = 27%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Member Diagram for Truss RJ2
Engineering Status = 52%

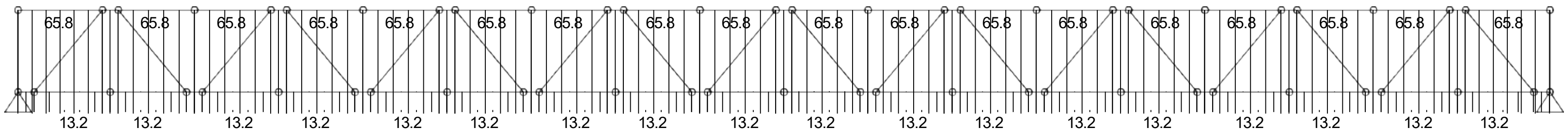
LC1 (0.42Wu) Maximum for this Load Case = 27%

System Name: FRAMECAD_FT_m
Roof Type: SHEET
Wind Speed: 200
Terrain Factor Kz 1.00
Importance Factor I 1.00
Topography Factor Kzt 1.00
Truss Spacing: 2'-0"
Top Chord Restraints: 3'-11 1/4"
Bottom Chord Restraints: 3'-11 1/4"
Design Code: AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load: 20.0psf
Top Chord Dead Load: 20.0psf
Bottom Chord Live Load: 0.0psf
Bottom Chord Dead Load: 10.0psf
Bottom Chord Services Load: 0.0psf
Top Chord Ground Snow Load: 0.0psf
Suspended Ceilings
Loading Code: IBC 2018 LRFD

FRAMECAD



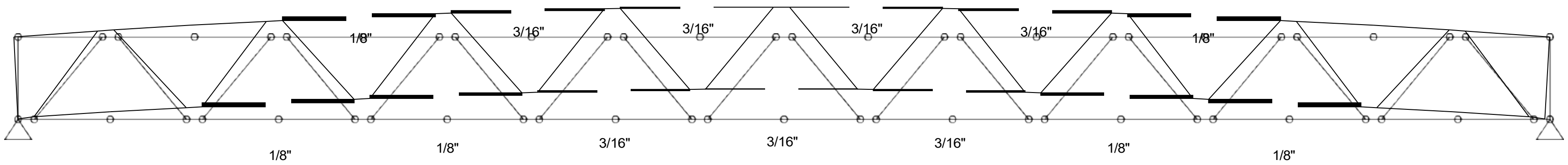
Load Diagram for Truss RJ2
Engineering Status = 52%

LC1 (0.42Wu) Maximum for this Load Case = 27%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



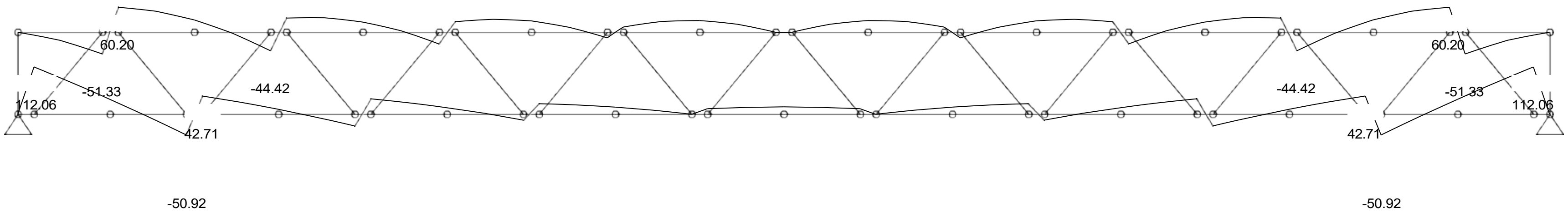
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC1 (0.42Wu) Maximum for this Load Case = 27%

System Name:	FRAMECAD_FT_m	
Roof Type:	SHEET	
Wind Speed:	200	
Terrain Factor Kz	1.00	
Importance Factor I	1.00	
Topography Factor Kzt	1.00	
Truss Spacing:	2'-0"	
Top Chord Restraints:	3'-11 1/4"	
Bottom Chord Restraints:	3'-11 1/4"	
Design Code:	AISI S100-20 LRFD	

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



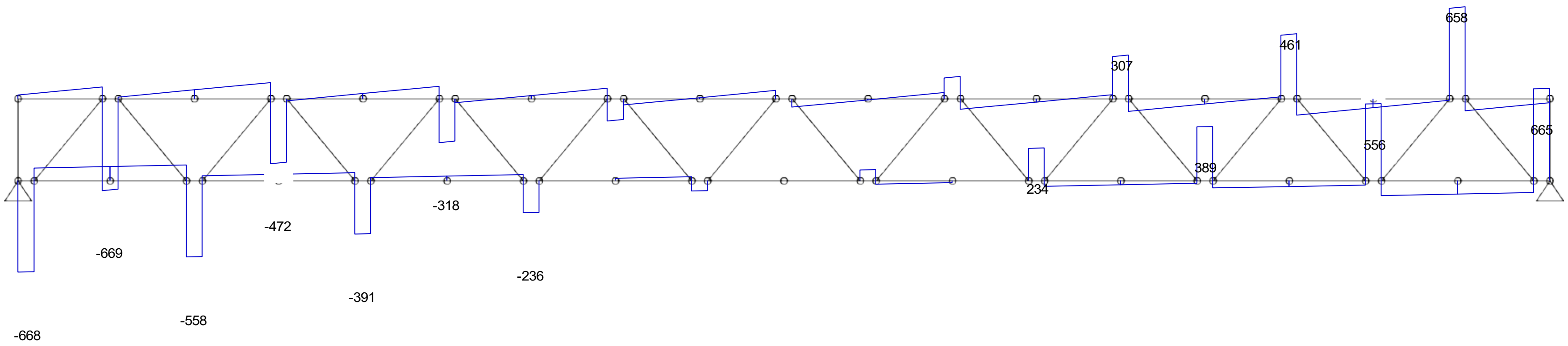
Bending Diagram for Truss RJ2
Engineering Status = 52%

LC1 (0.42Wu) Maximum for this Load Case = 27%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

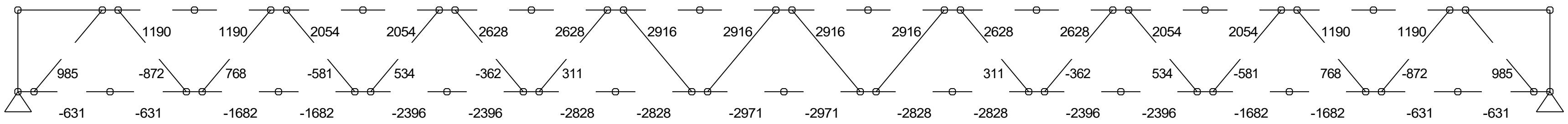
Engineering Status = 52%

LC1 (0.42Wu) Maximum for this Load Case = 27%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



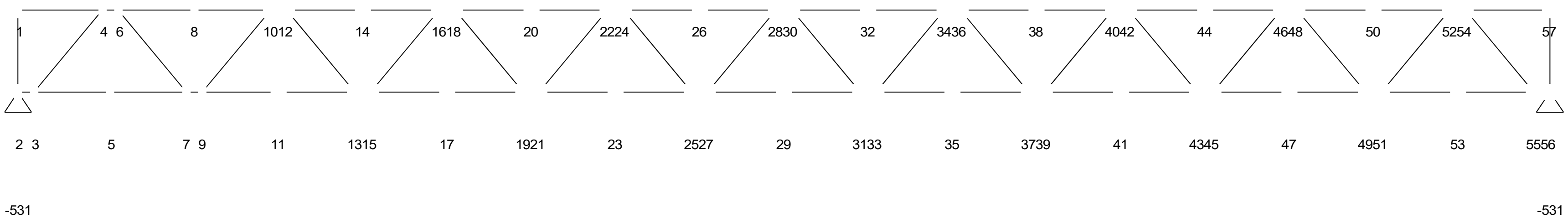
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC1 (0.42Wu)   Maximum for this Load Case = 27%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

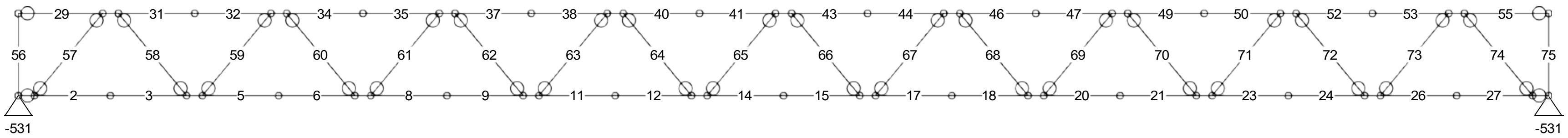
LC2 (G)   Maximum for this Load Case = 24%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

FRAMECAD



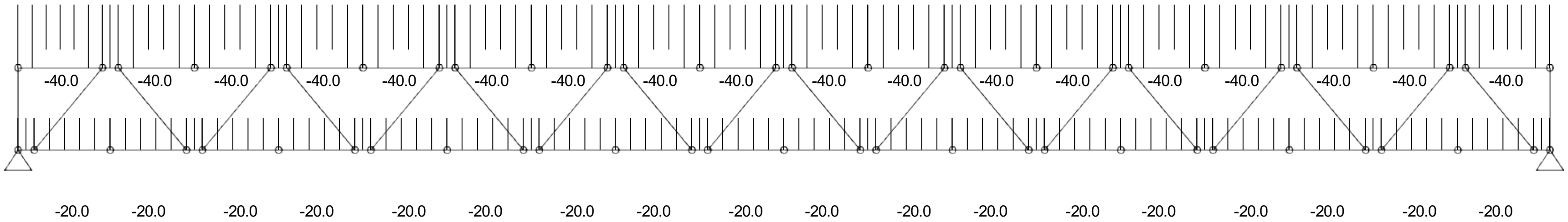
Member Diagram for Truss RJ2
Engineering Status = 52%

LC2 (G) Maximum for this Load Case = 24%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



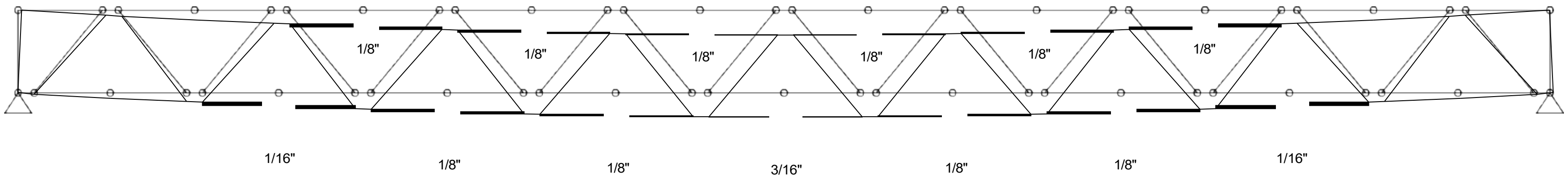
Load Diagram for Truss RJ2
Engineering Status = 52%

LC2 (G) Maximum for this Load Case = 24%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



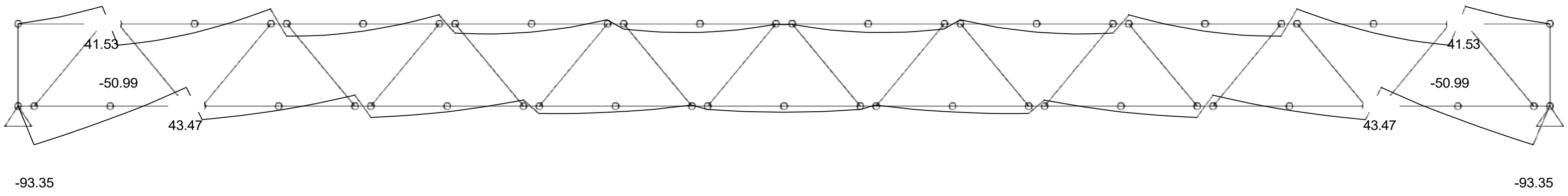
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC2 (G) Maximum for this Load Case = 24%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2
Engineering Status = 52%

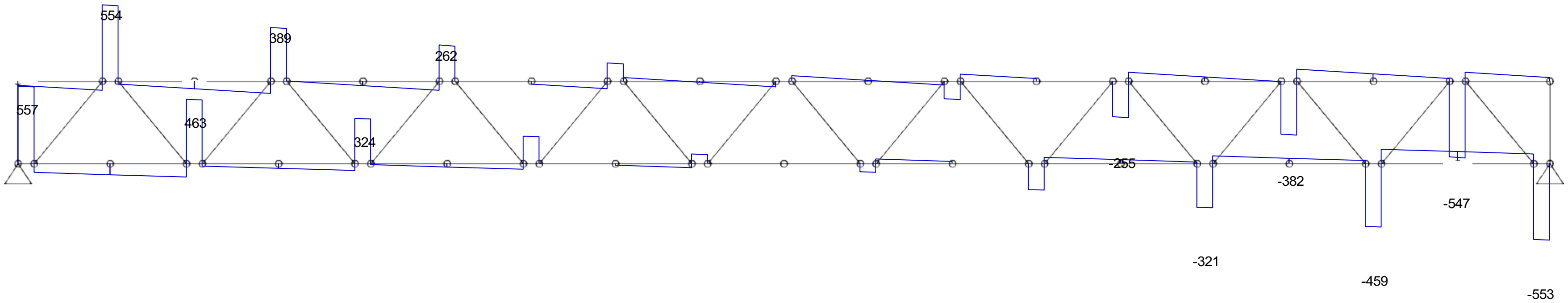
~~LC2 (G) Maximum for this Load Case = 24%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

FRAMECAD



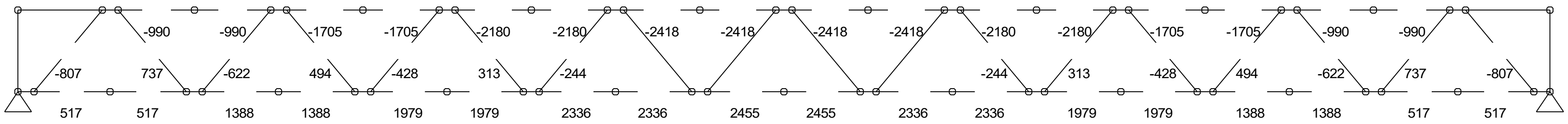
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC2 (G) Maximum for this Load Case = 24%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Axial Diagram for Truss RJ2
Engineering Status = 52%

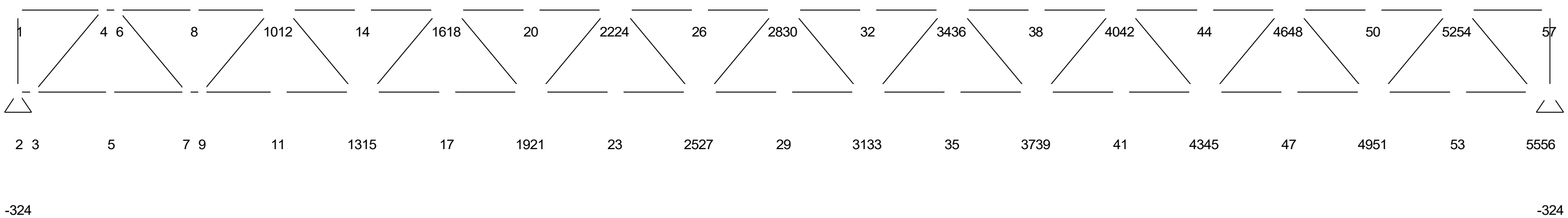
LC2 (G) Maximum for this Load Case = 24%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

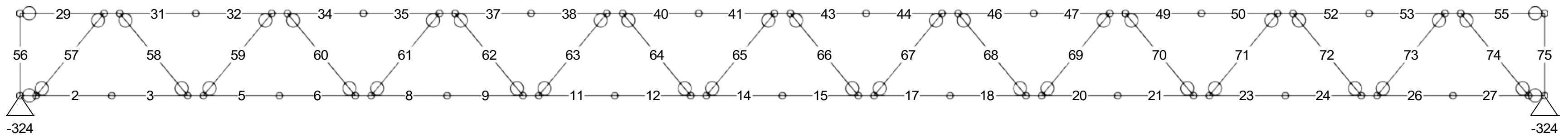
LC3 (max(Q, S)) Maximum for this Load Case = 15%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



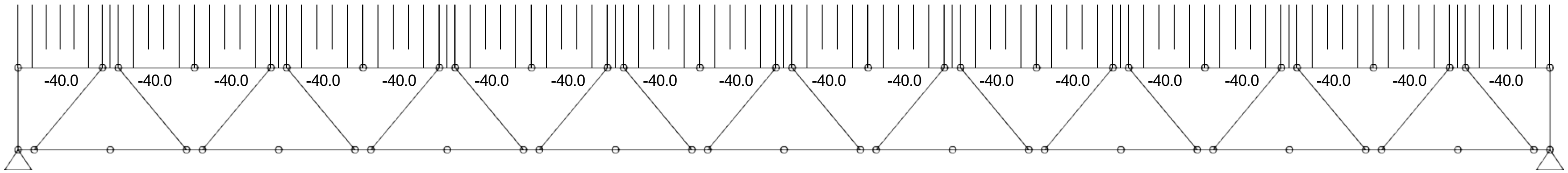
Member Diagram for Truss RJ2
Engineering Status = 52%

LC3 (max(Q, S)) Maximum for this Load Case = 15%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Load Diagram for Truss RJ2
Engineering Status = 52%

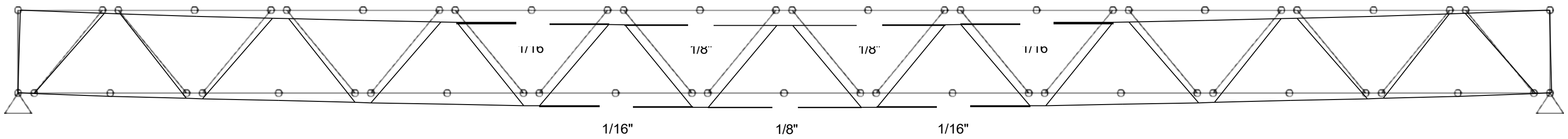
LC3 (max(Q, S)) Maximum for this Load Case = 15%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



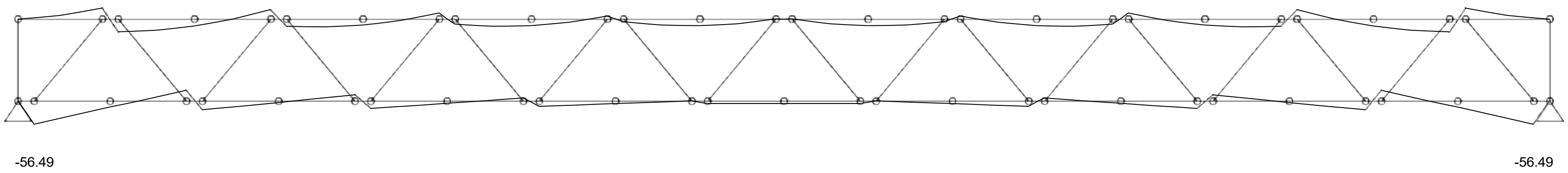
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC3 (max(Q, S)) Maximum for this Load Case = 15%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



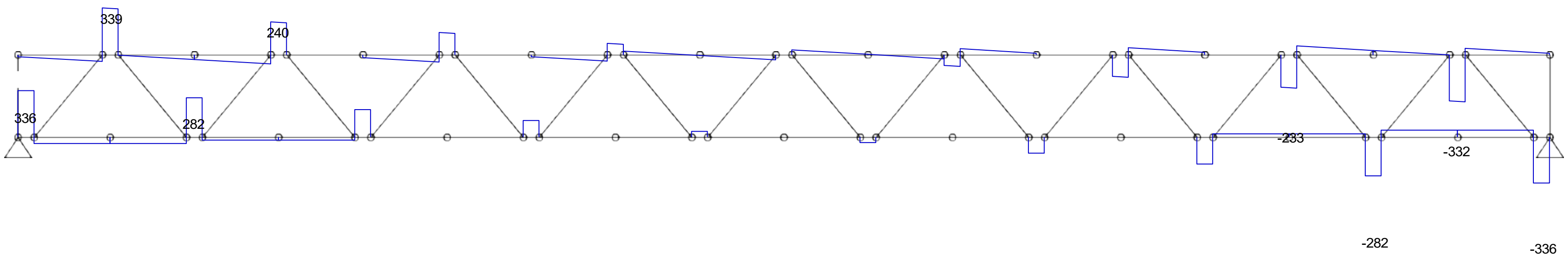
Bending Diagram for Truss RJ2
Engineering Status = 52%

~~LC3 (max(Q, S)) Maximum for this Load Case = 15%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2
Engineering Status = 52%

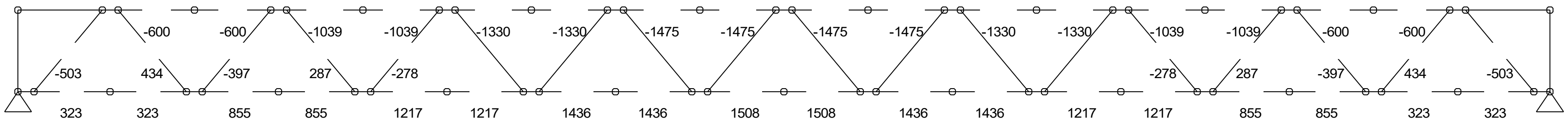
LC3 (max(Q, S)) Maximum for this Load Case = 15%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



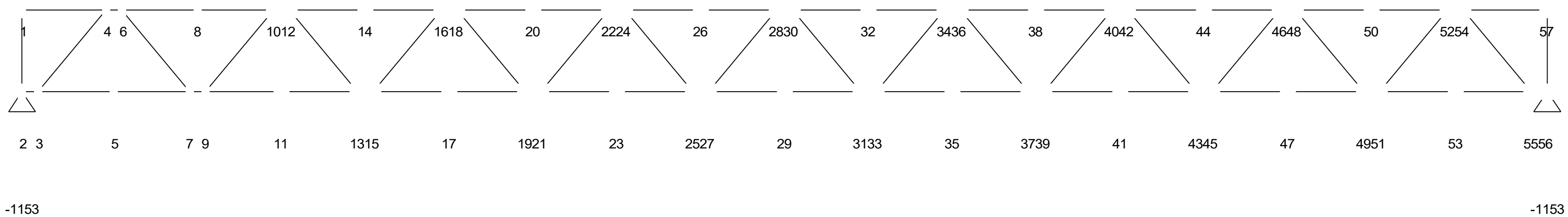
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC3 (max(Q, S)) Maximum for this Load Case = 15%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

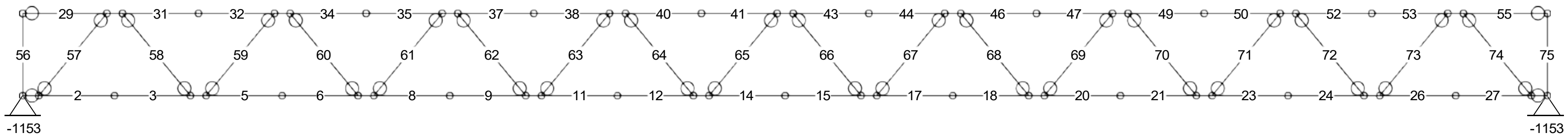
LC4 (1.2G + 1.6Q)   Maximum for this Load Case = 52%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



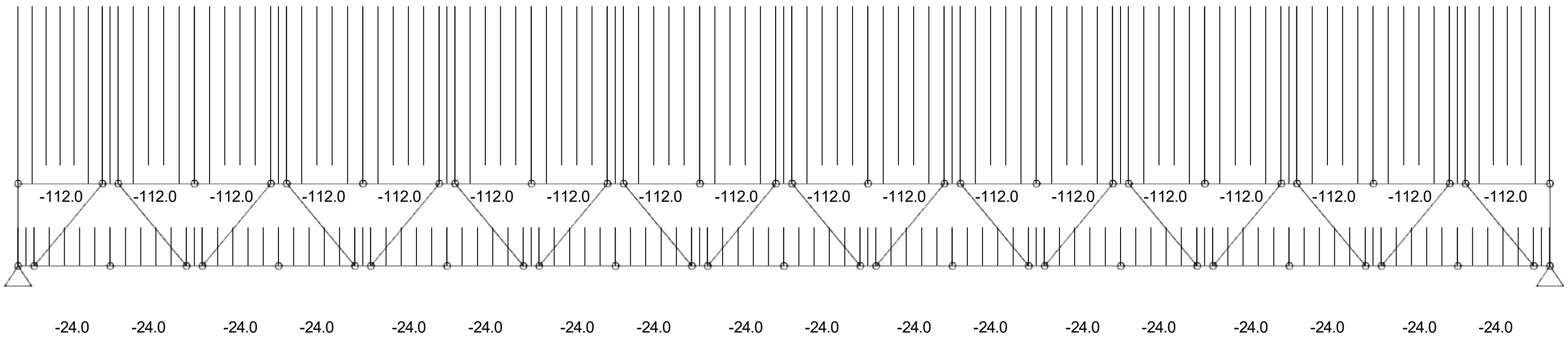
Member Diagram for Truss RJ2
Engineering Status = 52%

LC4 (1.2G + 1.6Q) Maximum for this Load Case = 52%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Load Diagram for Truss RJ2
Engineering Status = 52%

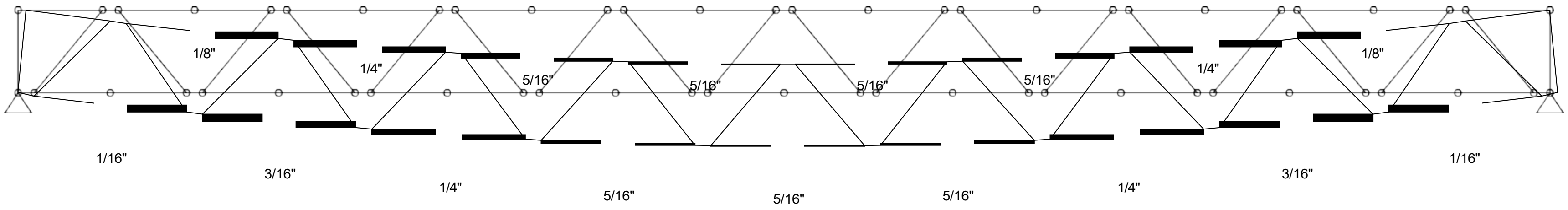
LC4 (1.2G + 1.6Q) Maximum for this Load Case = 52%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



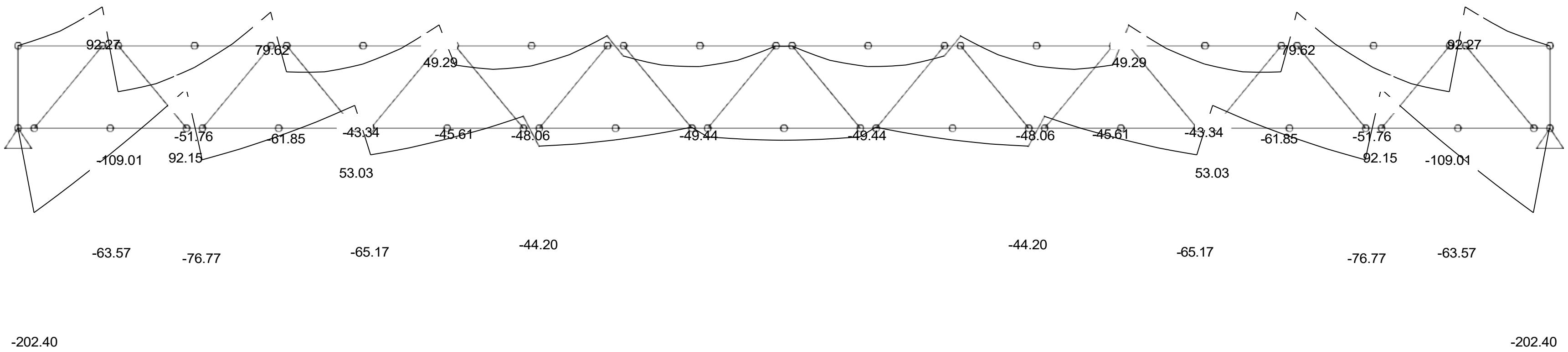
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC4 (1.2G + 1.6Q)   Maximum for this Load Case = 52%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

Engineering Status = 52%

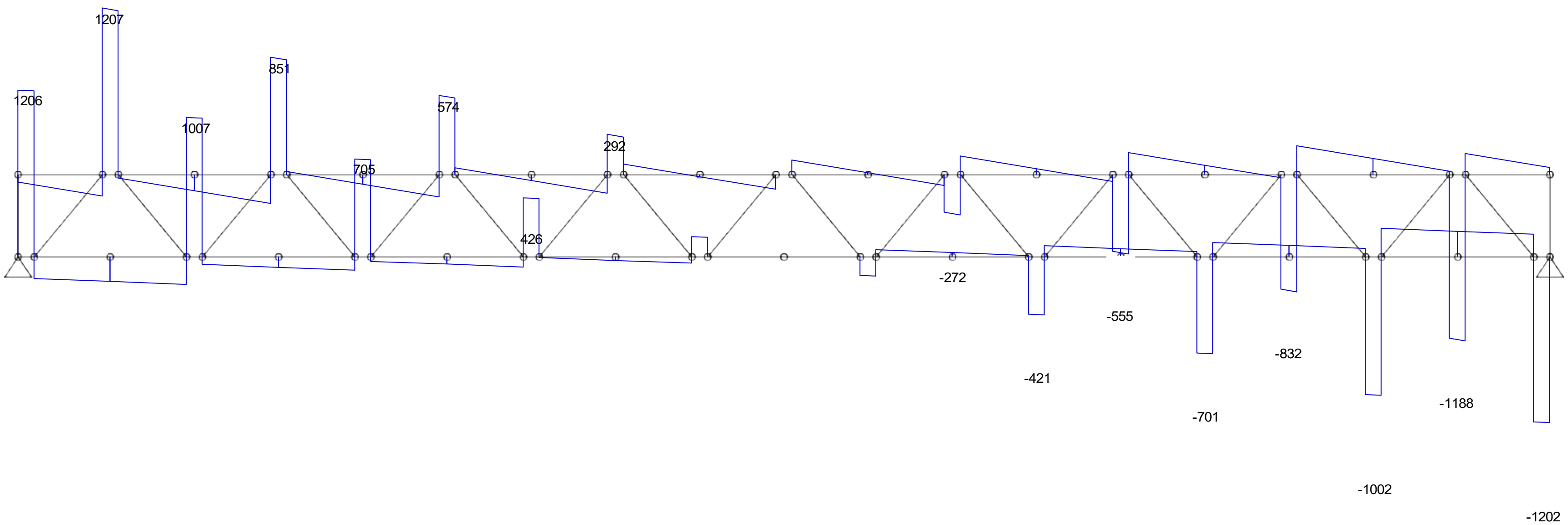
LC4 (1.2G + 1.6Q)   Maximum for this Load Case = 52%

Company: Bauhu   Dwg Name: 242201_Exuma, Bahamas roof truss MS Sheet 27 of 1   Project: Exuma, Bahamas   Job Number:242201

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

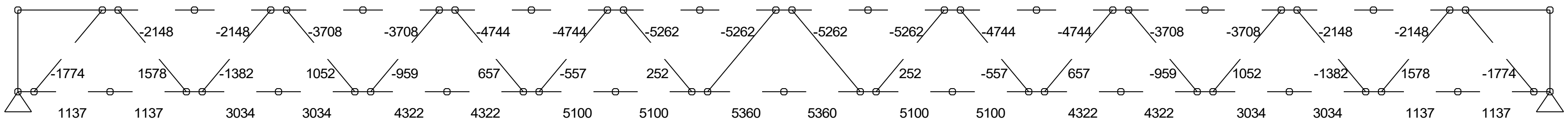


Shear Diagram for Truss RJ2
Engineering Status = 52%

LC4 (1.2G + 1.6Q) Maximum for this Load Case = 52%

System Name:	FRAMECAD_FT_m	Top Chord Live Load:	20.0psf
Roof Type:	SHEET	Top Chord Dead Load:	20.0psf
Wind Speed:	200	Bottom Chord Live Load:	0.0psf
Terrain Factor Kz	1.00	Bottom Chord Dead Load:	10.0psf
Importance Factor I	1.00	Bottom Chord Services Load:	0.0psf
Topography Factor Kzt	1.00	Top Chord Ground Snow Load:	0.0psf
Truss Spacing:	2'-0"	Suspended Ceilings	
Top Chord Restraints:	3'-11 1/4"	Loading Code:	IBC 2018 LRFD
Bottom Chord Restraints:	3'-11 1/4"		
Design Code:	AISI S100-20 LRFD		

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb



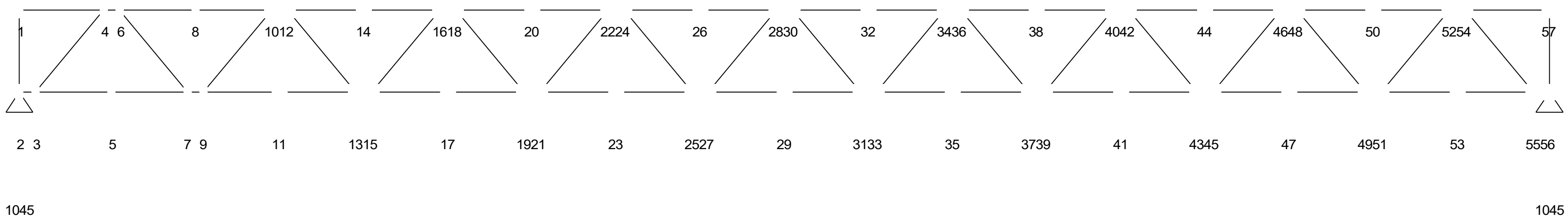
Axial Diagram for Truss RJ2
Engineering Status = 52%

~~LC4 (1.2G + 1.6Q) Maximum for this Load Case = 52%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

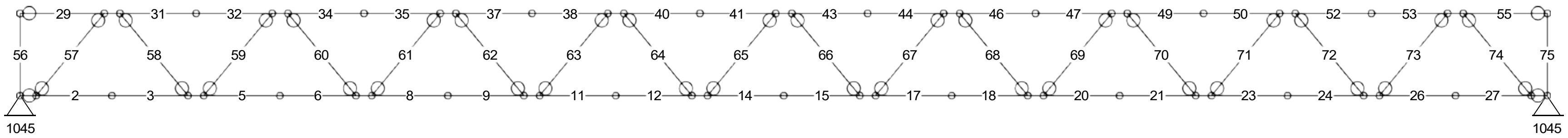
LC5 (0.9G + 1.0Wu)   Maximum for this Load Case = 44%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



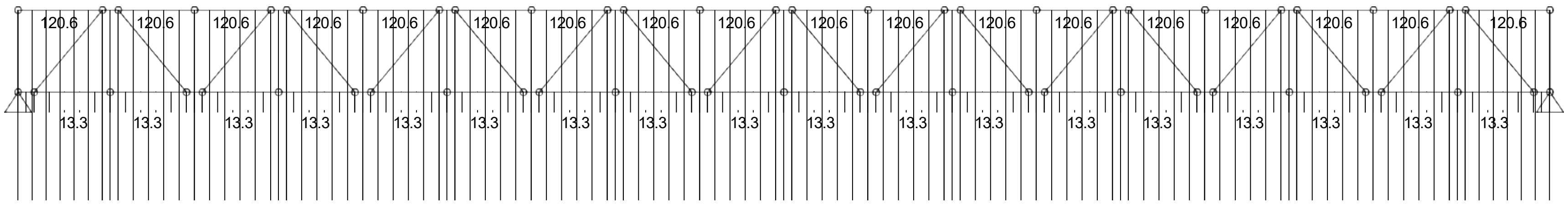
Member Diagram for Truss RJ2
Engineering Status = 52%

LC5 (0.9G + 1.0Wu) Maximum for this Load Case = 44%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



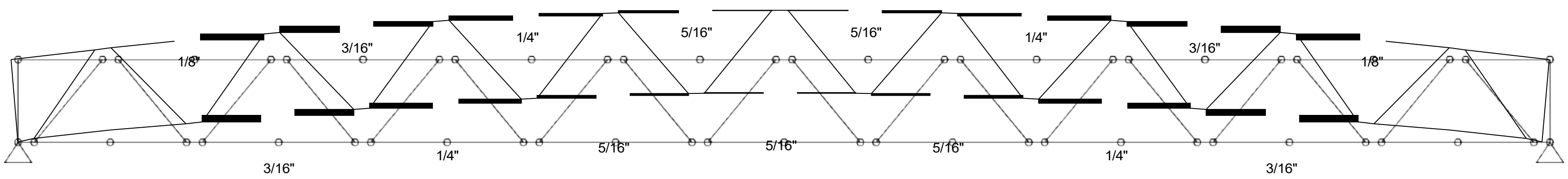
Load Diagram for Truss RJ2
Engineering Status = 52%

LC5 (0.9G + 1.0Wu)   Maximum for this Load Case = 44%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



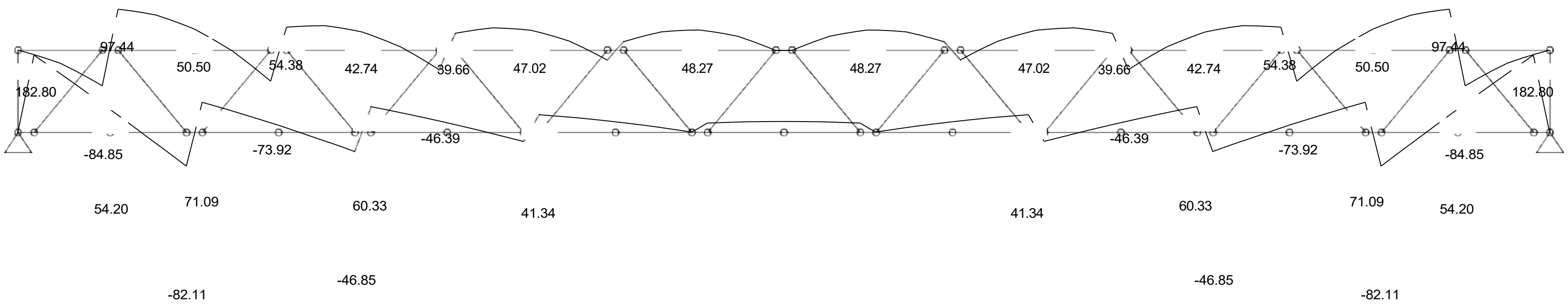
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC5 (0.9G + 1.0Wu)   Maximum for this Load Case = 44%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2
Engineering Status = 52%

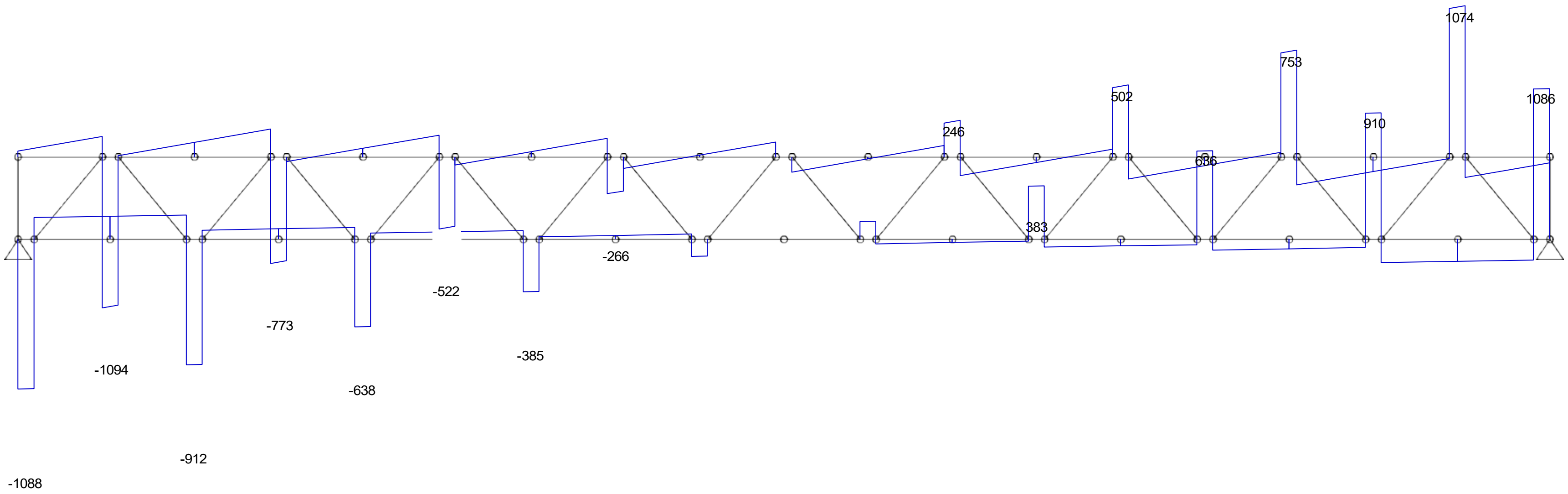
LC5 (0.9G + 1.0Wu)   Maximum for this Load Case = 44%

Company: Bauhu   Dwg Name: 242201_Exuma, Bahamas roof truss MS   Sheet 34 of 1   Project: Exuma, Bahamas   Job Number:242201

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



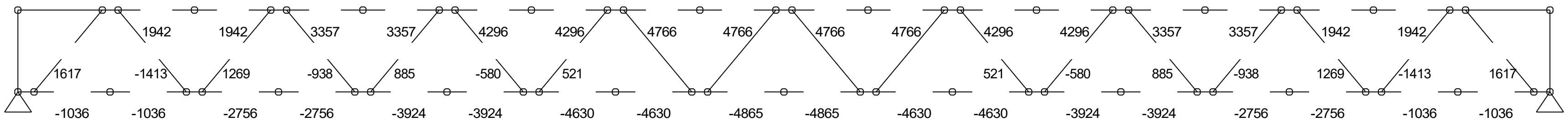
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC5 (0.9G + 1.0Wu)   Maximum for this Load Case = 44%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



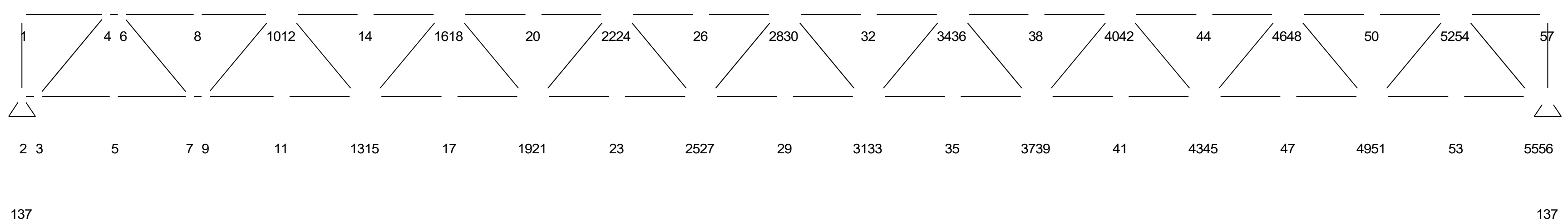
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC5 (0.9G + 1.0Wu) Maximum for this Load Case = 44%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

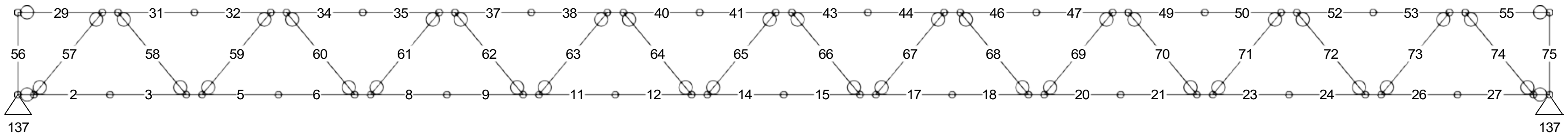
LC6 (0.9G + 1.0Ww)   Maximum for this Load Case = 6%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



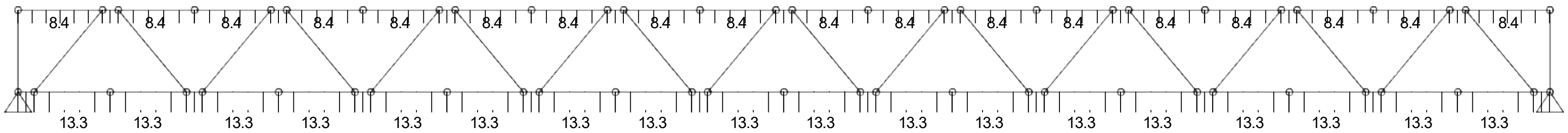
Member Diagram for Truss RJ2
Engineering Status = 52%

~~LC6 (0.9G + 1.0Ww) Maximum for this Load Case = 6%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



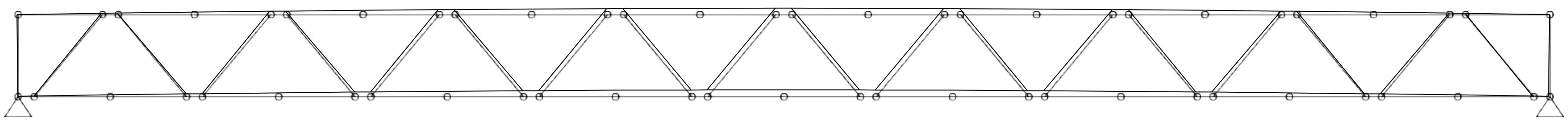
Load Diagram for Truss RJ2
Engineering Status = 52%

LC6 (0.9G + 1.0Ww) Maximum for this Load Case = 6%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



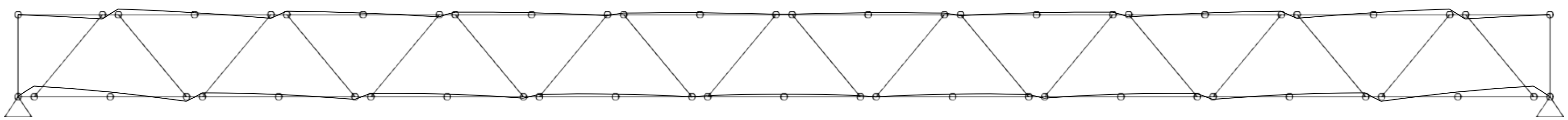
Deflection Diagram for Truss RJ2
Engineering Status = 52%

~~LC6 (0.9G + 1.0Ww) Maximum for this Load Case = 6%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



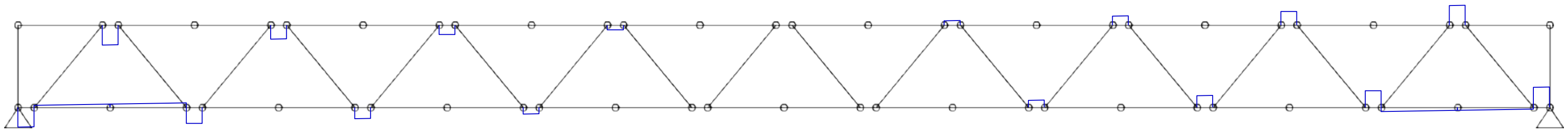
Bending Diagram for Truss RJ2
Engineering Status = 52%

~~LC6 (0.9G + 1.0Ww) Maximum for this Load Case = 6%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



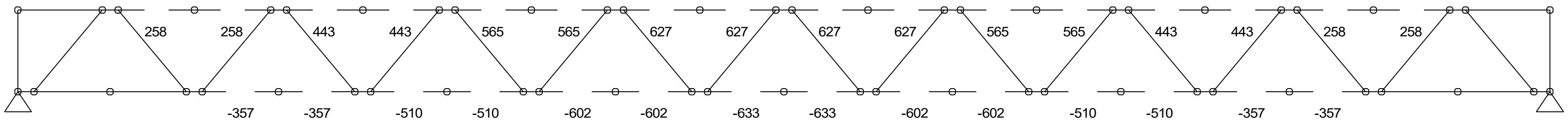
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC6 (0.9G + 1.0Ww)   Maximum for this Load Case = 6%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



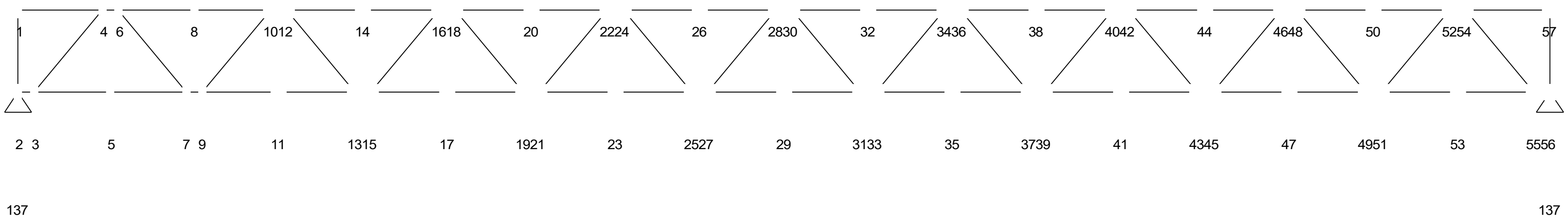
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC6 (0.9G + 1.0Ww) Maximum for this Load Case = 6%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

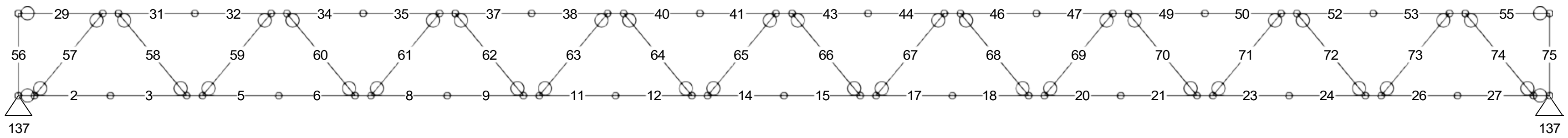
LC7 (0.9G + 1.0Wl) Maximum for this Load Case = 6%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



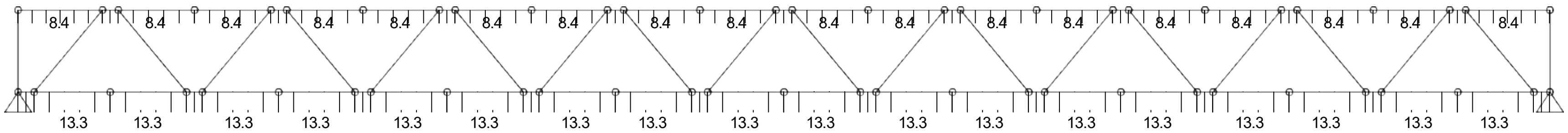
Member Diagram for Truss RJ2
Engineering Status = 52%

LC7 (0.9G + 1.0WI) Maximum for this Load Case = 6%

System Name: FRAMECAD_FT_m
Roof Type: SHEET
Wind Speed: 200
Terrain Factor Kz 1.00
Importance Factor I 1.00
Topography Factor Kzt 1.00
Truss Spacing: 2'-0"
Top Chord Restraints: 3'-11 1/4"
Bottom Chord Restraints: 3'-11 1/4"
Design Code: AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load: 20.0psf
Top Chord Dead Load: 20.0psf
Bottom Chord Live Load: 0.0psf
Bottom Chord Dead Load: 10.0psf
Bottom Chord Services Load: 0.0psf
Top Chord Ground Snow Load: 0.0psf
Suspended Ceilings
Loading Code: IBC 2018 LRFD



Load Diagram for Truss RJ2
Engineering Status = 52%

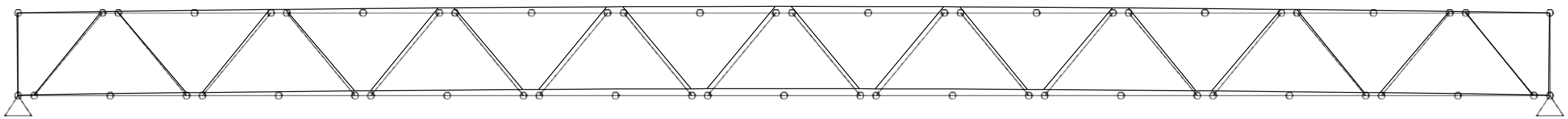
LC7 (0.9G + 1.0WI) Maximum for this Load Case = 6%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



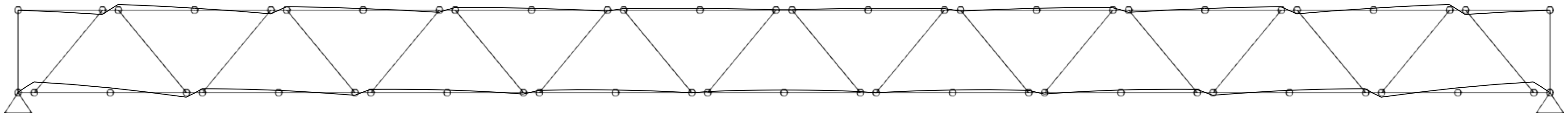
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC7 (0.9G + 1.0WI) Maximum for this Load Case = 6%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2
Engineering Status = 52%

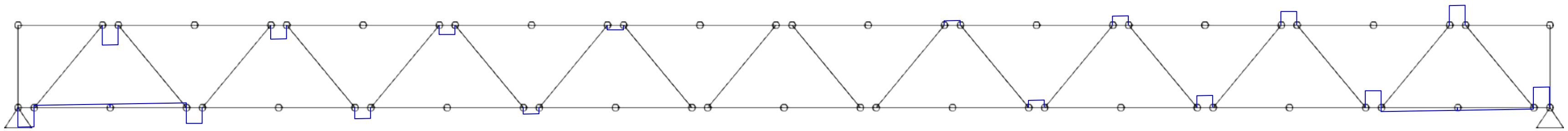
LC7 (0.9G + 1.0WI) Maximum for this Load Case = 6%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



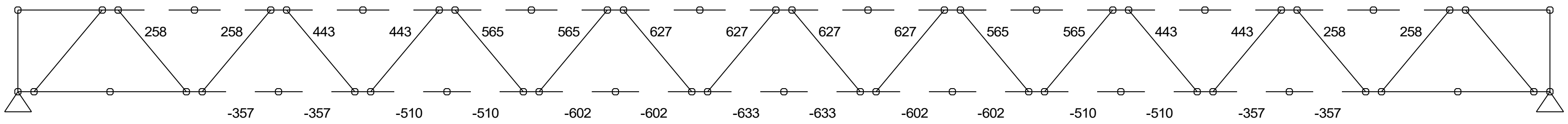
Shear Diagram for Truss RJ2
Engineering Status = 52%

~~LC7 (0.9G + 1.0WI) Maximum for this Load Case = 6%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Axial Diagram for Truss RJ2
Engineering Status = 52%

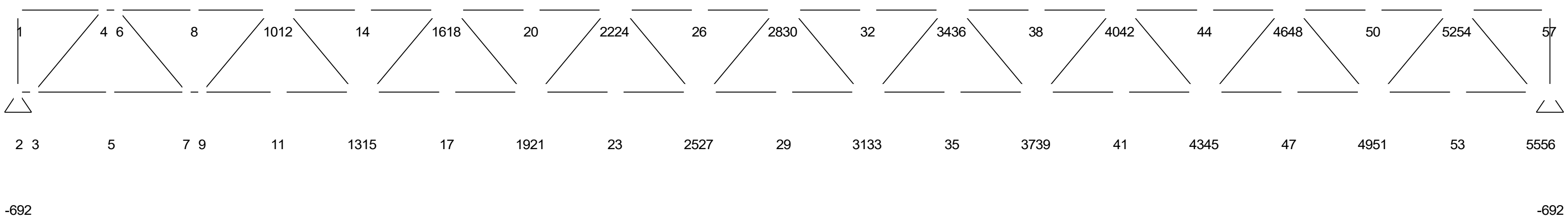
LC7 (0.9G + 1.0WI) Maximum for this Load Case = 6%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

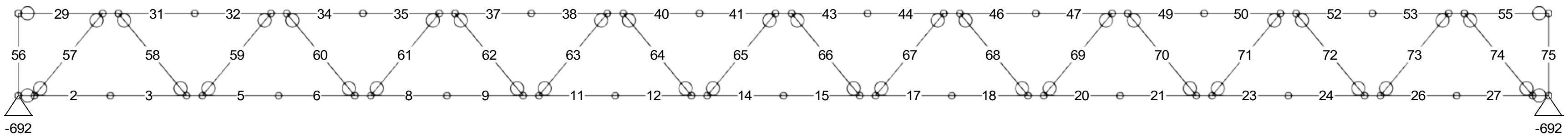
LC8 (1.2G + 1.0Ww + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

FRAMECAD



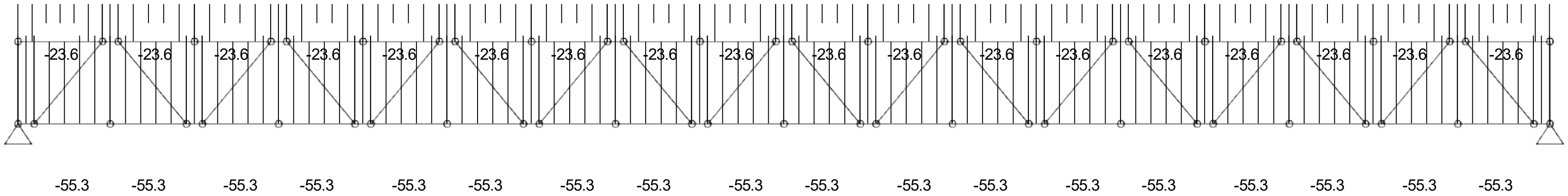
Member Diagram for Truss RJ2
Engineering Status = 52%

~~LC8 (1.2G + 1.0Ww + 0.5max(Q, S)) Maximum for this Load Case = 31%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



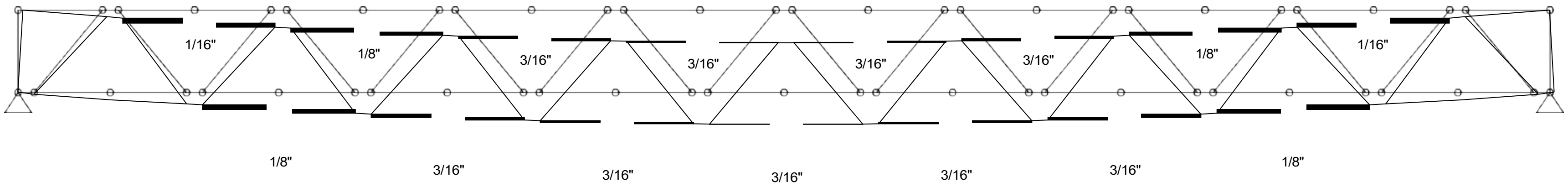
Load Diagram for Truss RJ2
Engineering Status = 52%

LC8 (1.2G + 1.0Ww + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



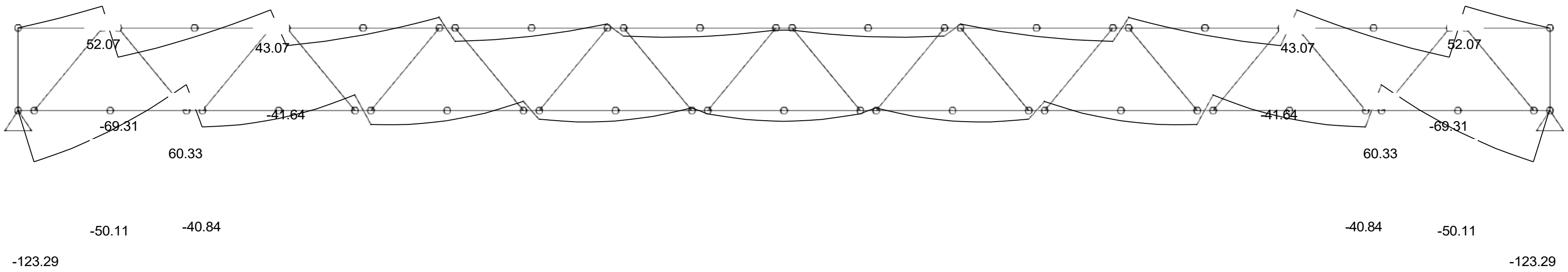
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC8 (1.2G + 1.0Ww + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

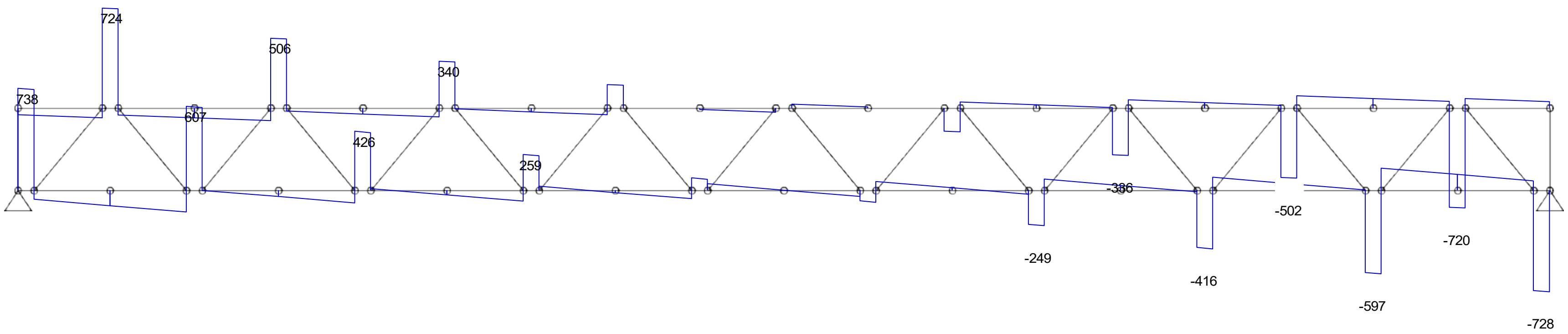
Engineering Status = 52%

LC8 (1.2G + 1.0Ww + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name: FRAMECAD_FT_m
Roof Type: SHEET
Wind Speed: 200
Terrain Factor Kz 1.00
Importance Factor I 1.00
Topography Factor Kzt 1.00
Truss Spacing: 2'-0"
Top Chord Restraints: 3'-11 1/4"
Bottom Chord Restraints: 3'-11 1/4"
Design Code: AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load: 20.0psf
Top Chord Dead Load: 20.0psf
Bottom Chord Live Load: 0.0psf
Bottom Chord Dead Load: 10.0psf
Bottom Chord Services Load: 0.0psf
Top Chord Ground Snow Load: 0.0psf
Suspended Ceilings
Loading Code: IBC 2018 LRFD



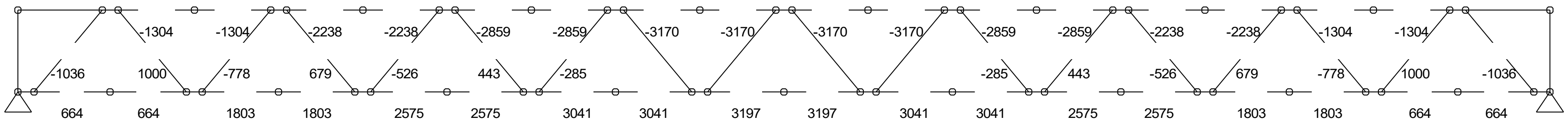
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC8 (1.2G + 1.0Ww + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Axial Diagram for Truss RJ2
Engineering Status = 52%

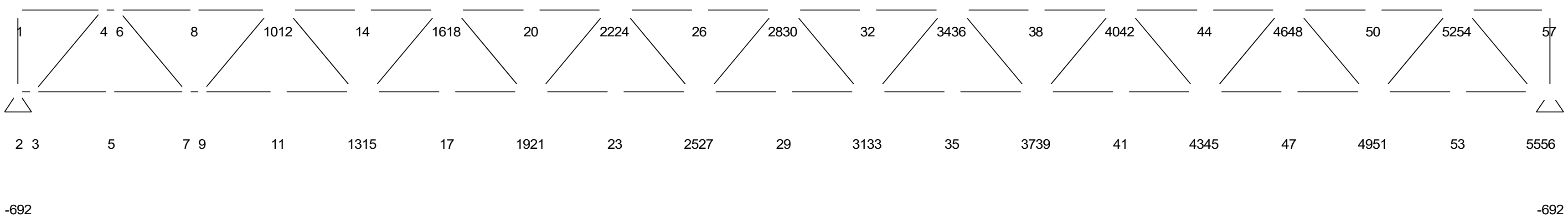
~~LC8 (1.2G + 1.0Ww + 0.5max(Q, S)) Maximum for this Load Case = 31%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

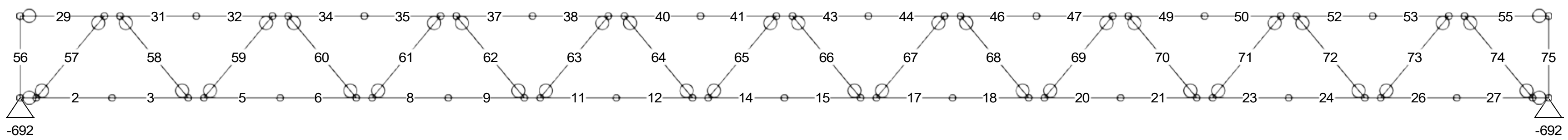
LC9 (1.2G + 1.0Wl + 0.5max(Q, S))   Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Member Diagram for Truss RJ2
Engineering Status = 52%

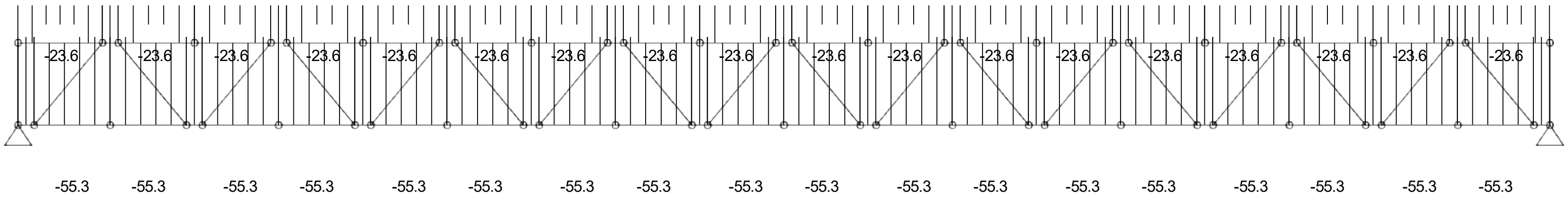
LC9 (1.2G + 1.0Wl + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



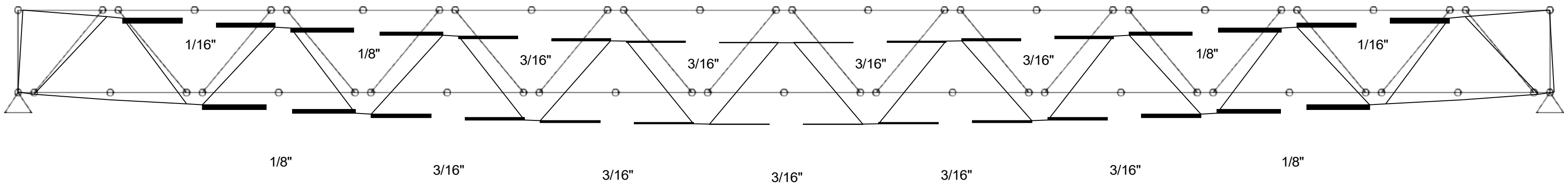
Load Diagram for Truss RJ2
Engineering Status = 52%

LC9 (1.2G + 1.0Wl + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Deflection Diagram for Truss RJ2
Engineering Status = 52%

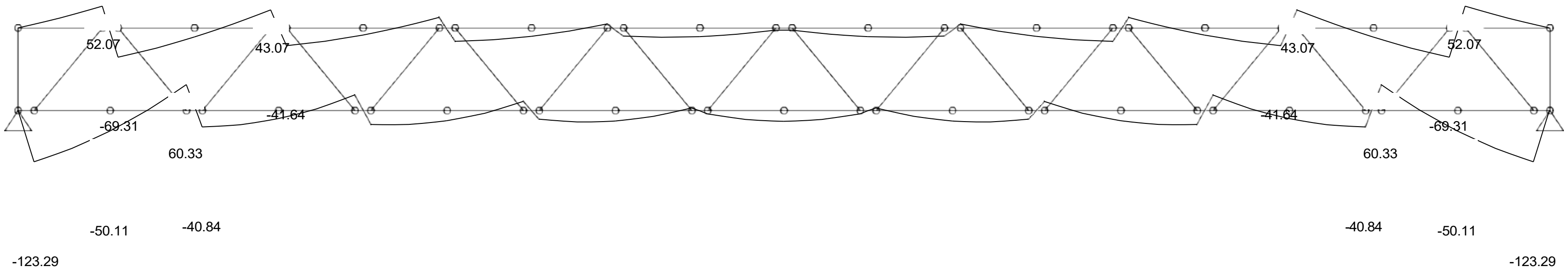
LC9 (1.2G + 1.0Wl + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

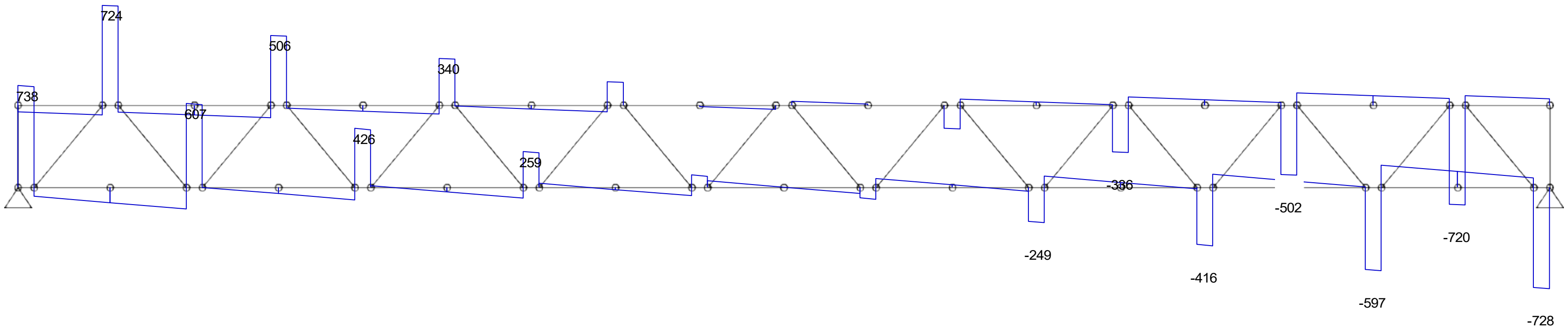
Engineering Status = 52%

LC9 (1.2G + 1.0Wl + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



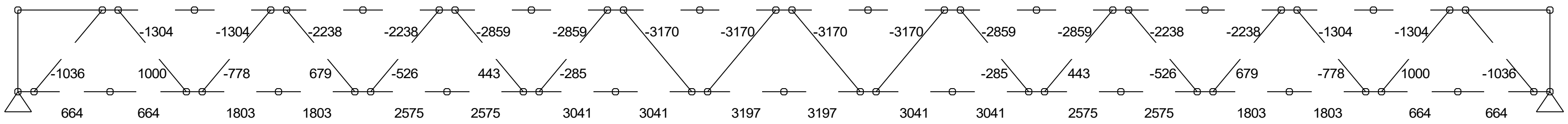
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC9 (1.2G + 1.0WI + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



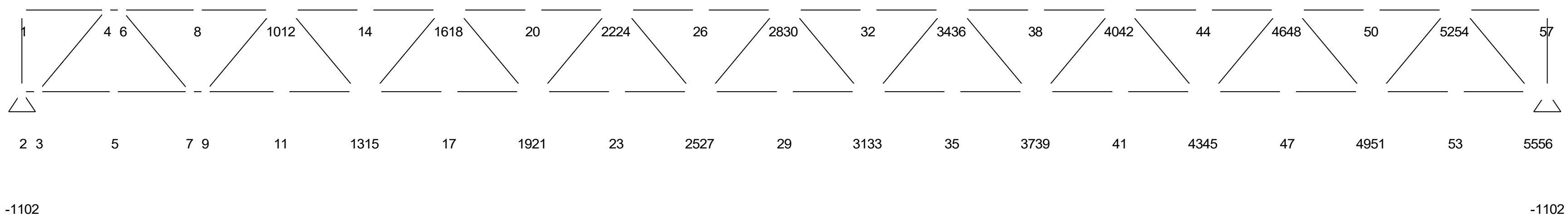
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC9 (1.2G + 1.0Wl + 0.5max(Q, S)) Maximum for this Load Case = 31%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

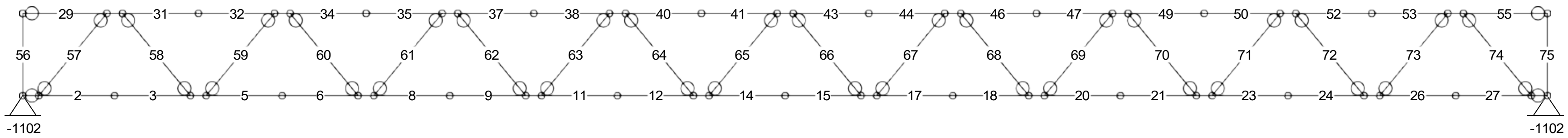
LC10 (1.2G + 0.5Ww + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

FRAMECAD



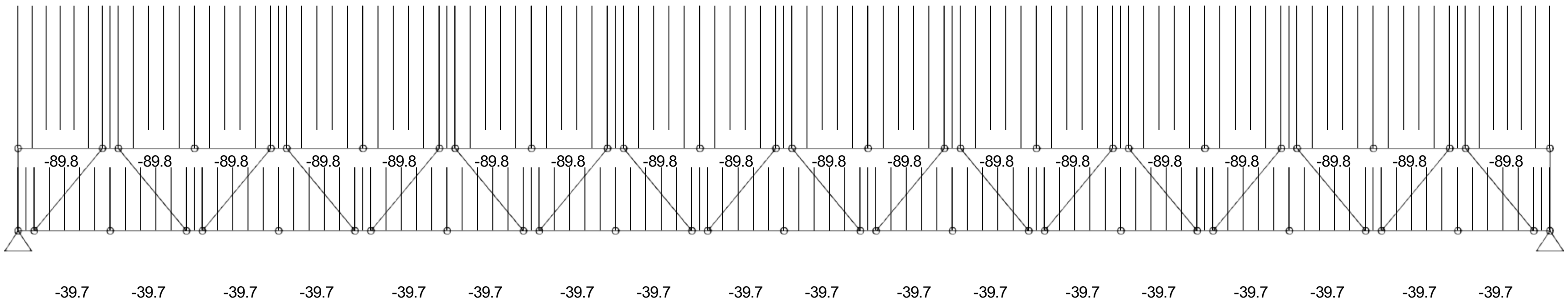
Member Diagram for Truss RJ2
Engineering Status = 52%

LC10 (1.2G + 0.5Ww + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

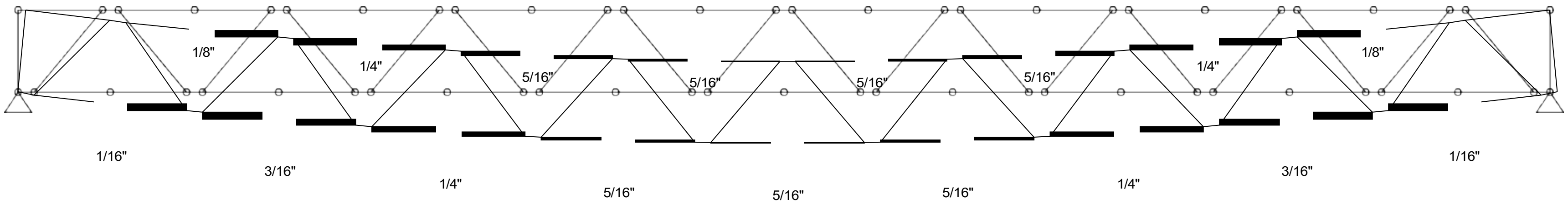


Load Diagram for Truss RJ2
Engineering Status = 52%

LC10 (1.2G + 0.5Ww + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m	Top Chord Live Load:	20.0psf
Roof Type:	SHEET	Top Chord Dead Load:	20.0psf
Wind Speed:	200	Bottom Chord Live Load:	0.0psf
Terrain Factor Kz	1.00	Bottom Chord Dead Load:	10.0psf
Importance Factor I	1.00	Bottom Chord Services Load:	0.0psf
Topography Factor Kzt	1.00	Top Chord Ground Snow Load:	0.0psf
Truss Spacing:	2'-0"	Suspended Ceilings	
Top Chord Restraints:	3'-11 1/4"	Loading Code:	IBC 2018 LRFD
Bottom Chord Restraints:	3'-11 1/4"		
Design Code:	AISI S100-20 LRFD		

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x



Deflection Diagram for Truss RJ2
Engineering Status = 52%

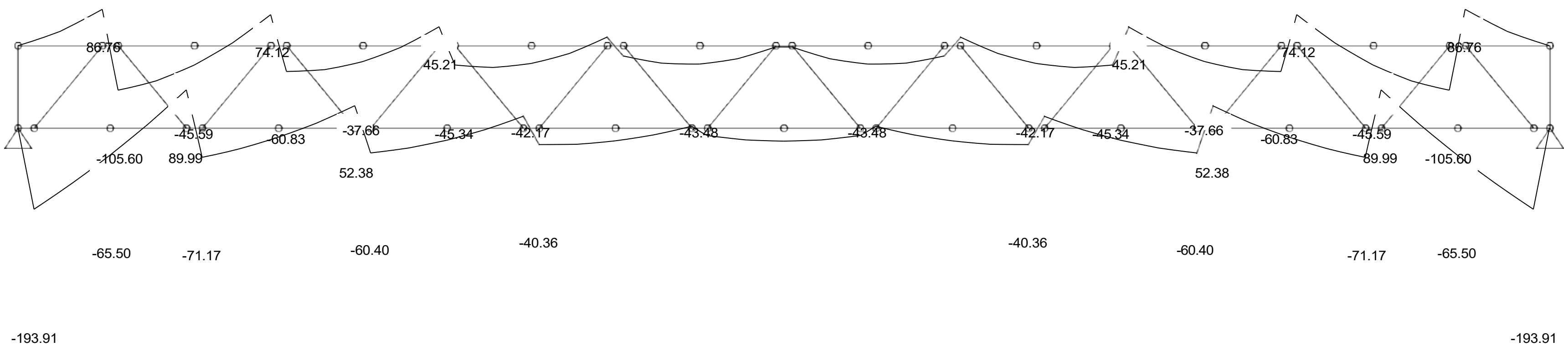
LC10 (1.2G + 0.5Ww + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

Engineering Status = 52%

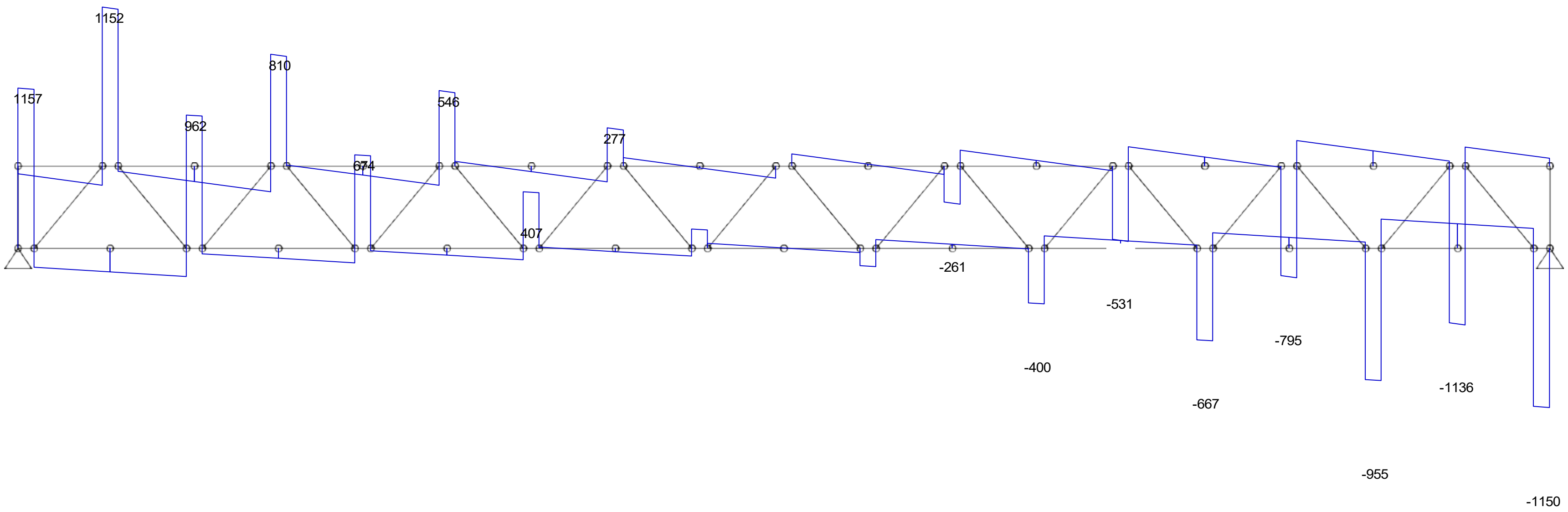
LC10 (1.2G + 0.5Ww + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2
Engineering Status = 52%

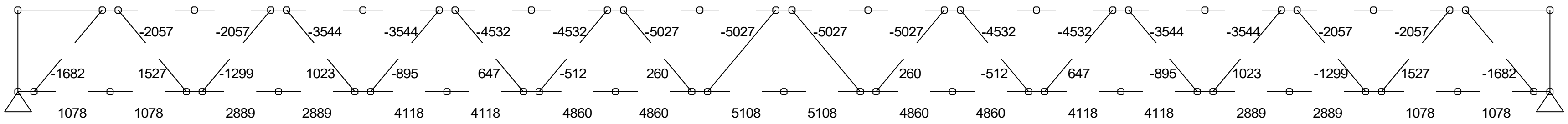
LC10 (1.2G + 0.5Ww + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



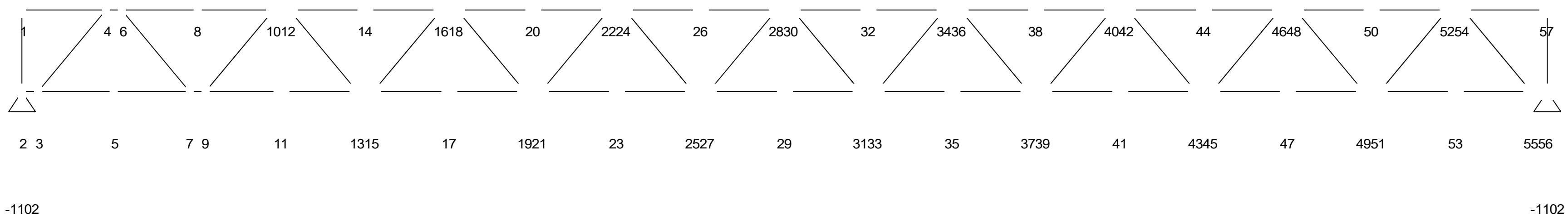
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC10 (1.2G + 0.5Ww + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



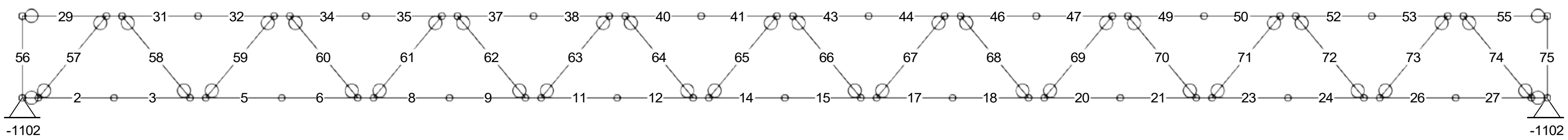
Node Diagram for Truss RJ2
Engineering Status = 52%

LC11 (1.2G + 0.5Wl + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



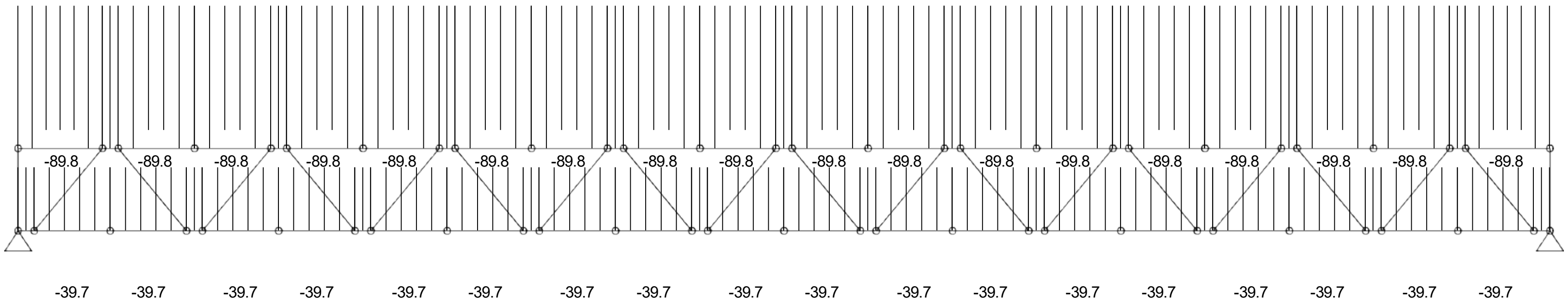
Member Diagram for Truss RJ2
Engineering Status = 52%

LC11 (1.2G + 0.5Wl + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Load Diagram for Truss RJ2
Engineering Status = 52%

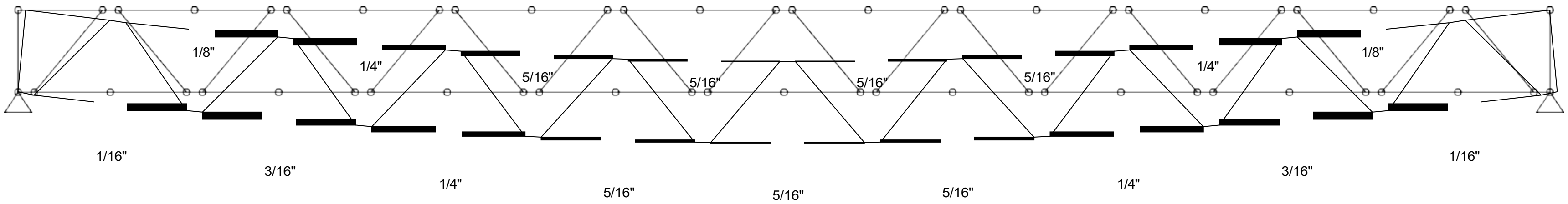
LC11 (1.2G + 0.5Wl + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Deflection Diagram for Truss RJ2
Engineering Status = 52%

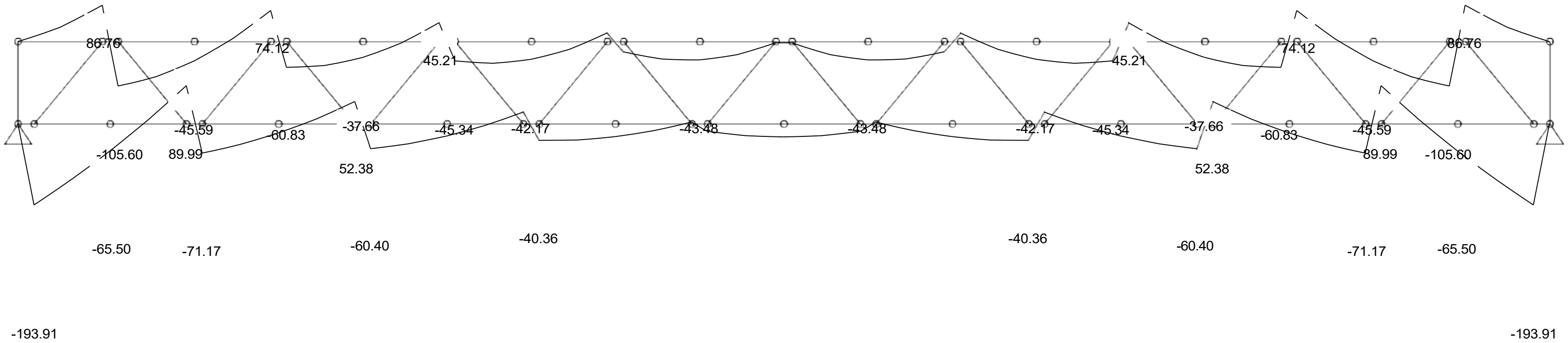
LC11 (1.2G + 0.5Wl + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

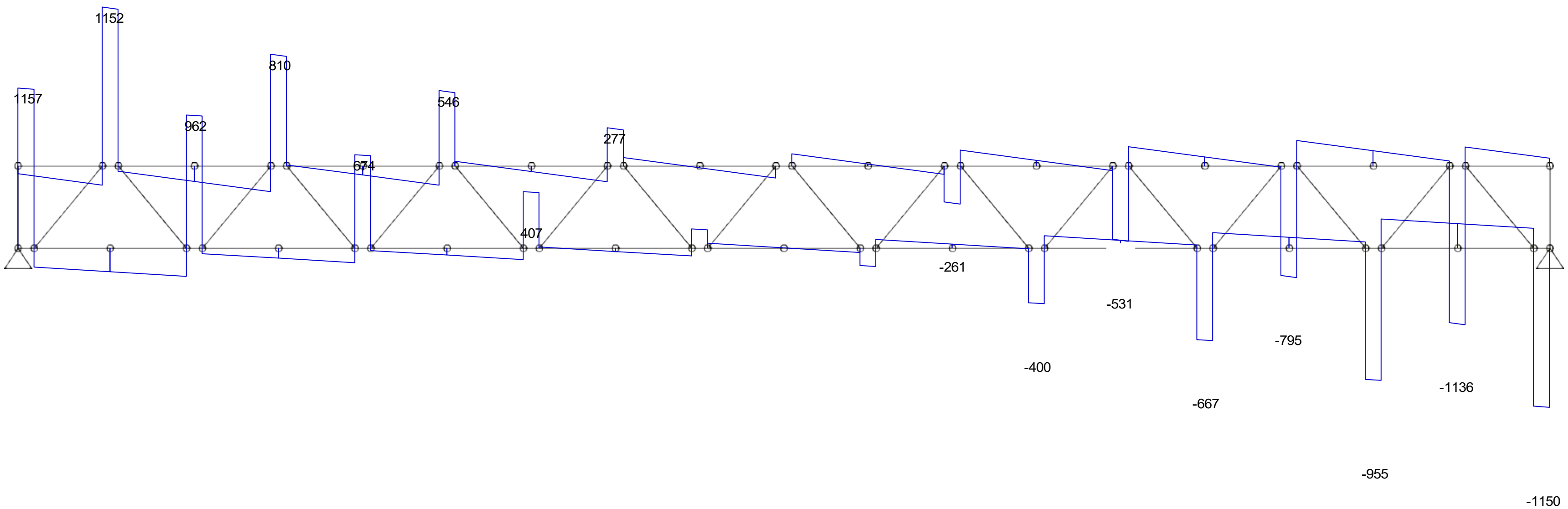
Engineering Status = 52%

LC11 (1.2G + 0.5Wl + 1.6max(Q, S))   Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



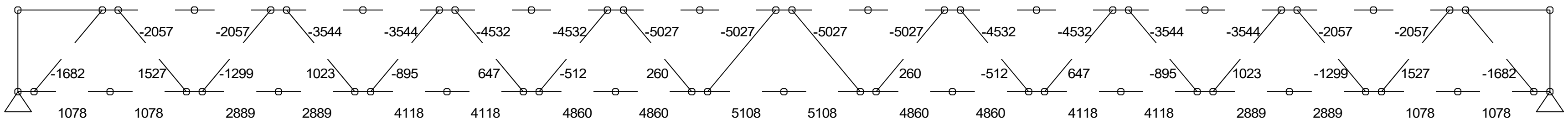
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC11 (1.2G + 0.5Wl + 1.6max(Q, S))   Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



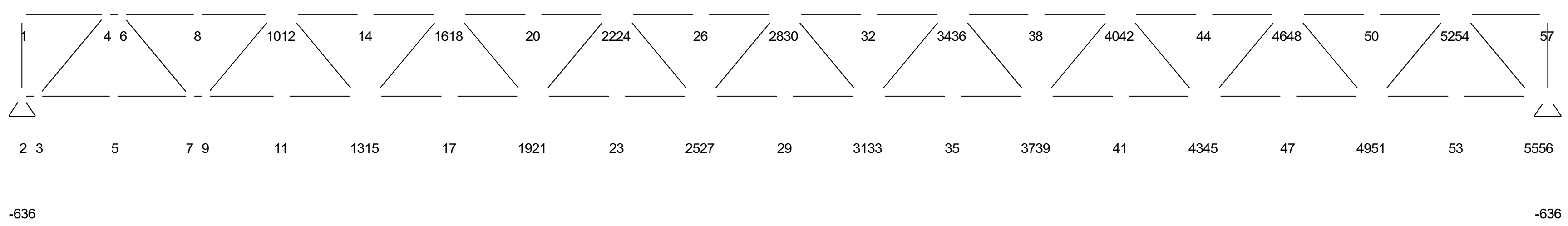
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC11 (1.2G + 0.5Wl + 1.6max(Q, S)) Maximum for this Load Case = 50%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

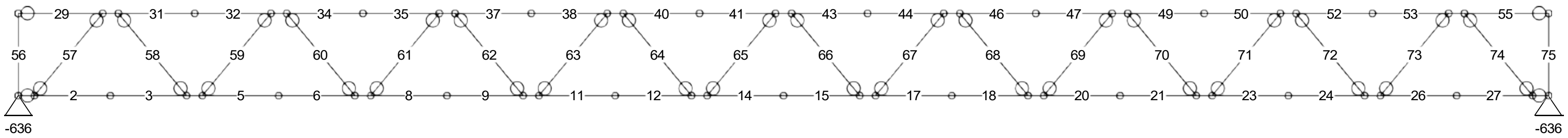
LC12 (1.2G + 1.6S)   Maximum for this Load Case = 29%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



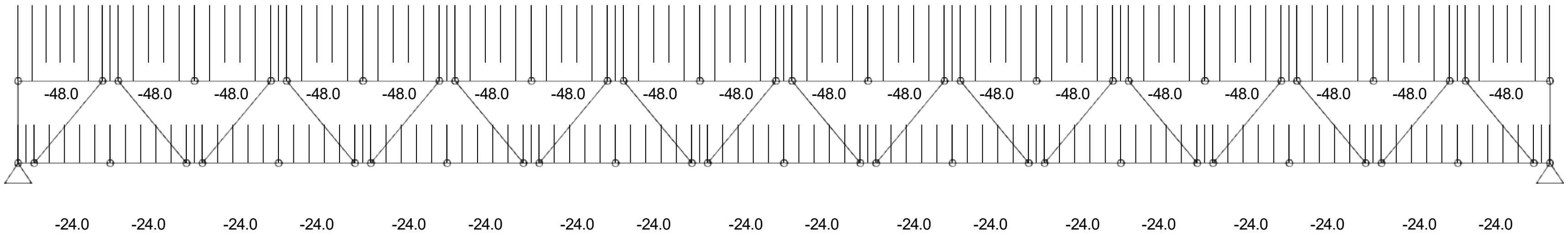
Member Diagram for Truss RJ2
Engineering Status = 52%

LC12 (1.2G + 1.6S) Maximum for this Load Case = 29%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Load Diagram for Truss RJ2
Engineering Status = 52%

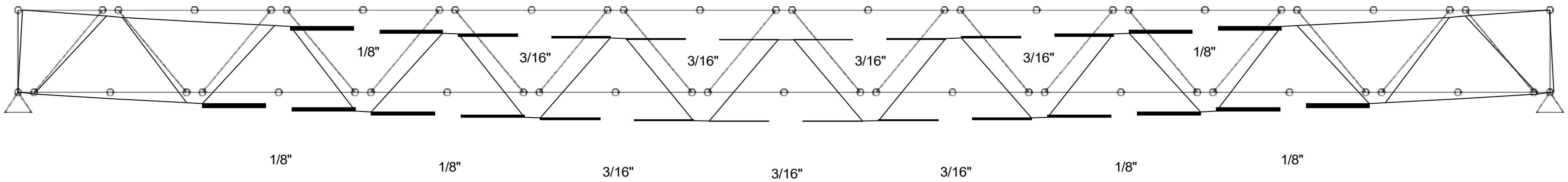
LC12 (1.2G + 1.6S) Maximum for this Load Case = 29%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



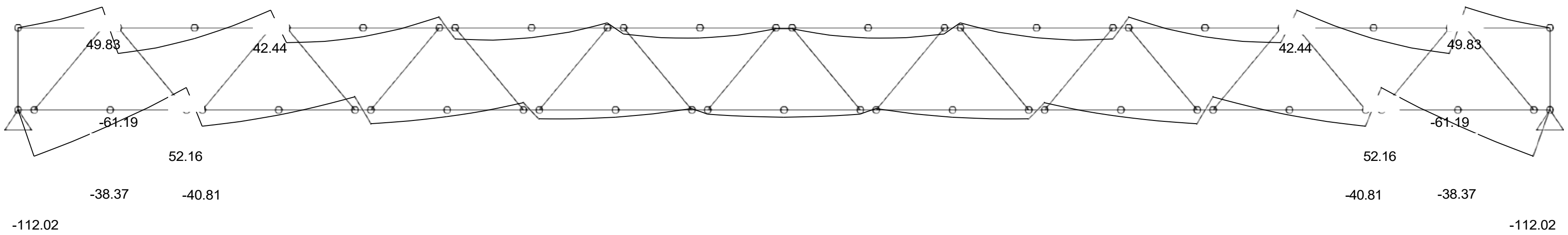
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC12 (1.2G + 1.6S)   Maximum for this Load Case = 29%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



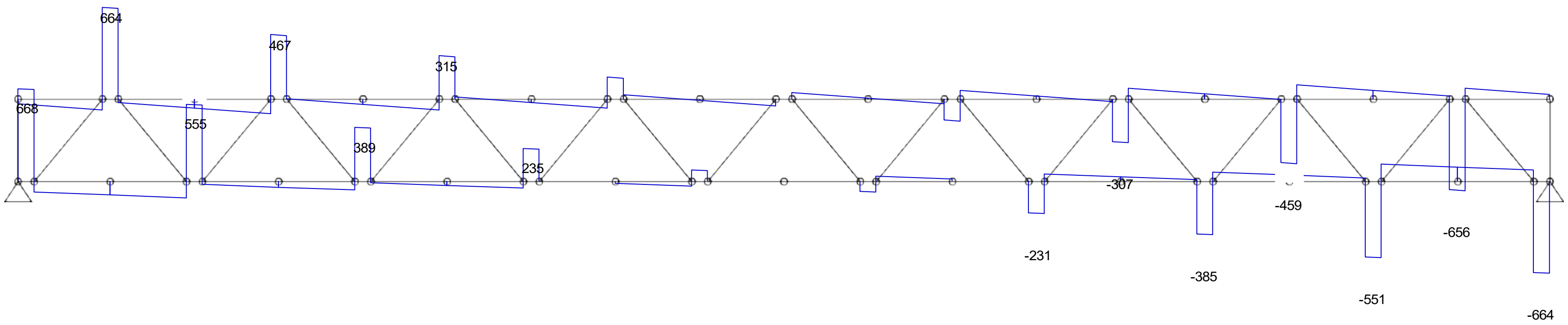
~~Bending Diagram for Truss RJ2~~
Engineering Status = 52%

LC12 (1.2G + 1.6S)   Maximum for this Load Case = 29%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



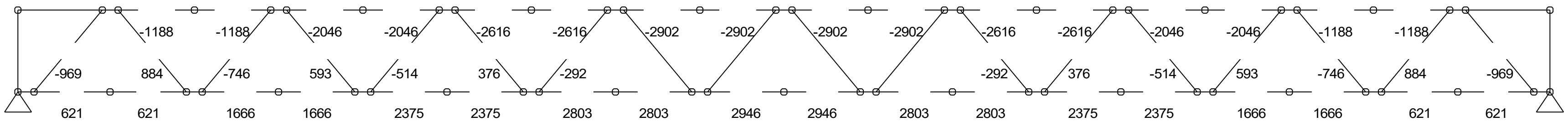
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC12 (1.2G + 1.6S)   Maximum for this Load Case = 29%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



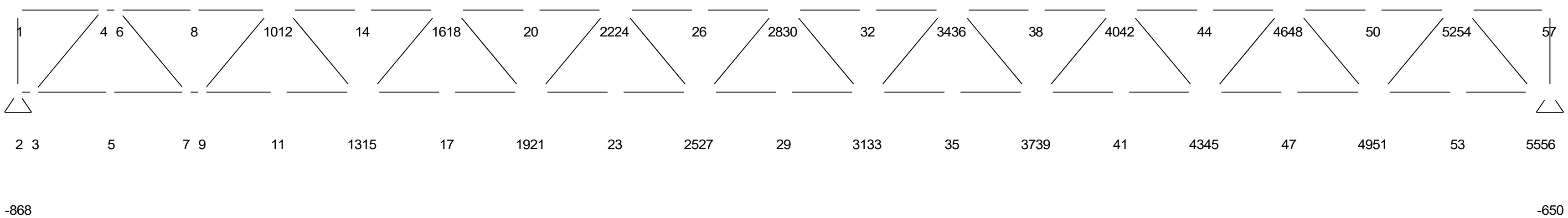
Axial Diagram for Truss RJ2
Engineering Status = 52%

~~LC12 (1.2G + 1.6S) Maximum for this Load Case = 29%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



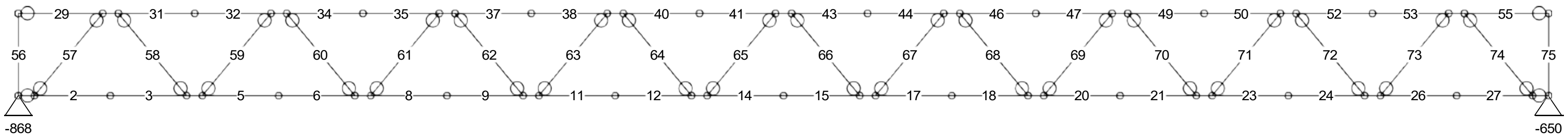
Node Diagram for Truss RJ2
Engineering Status = 52%

LC13 (1.2G + 1.6P)   Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



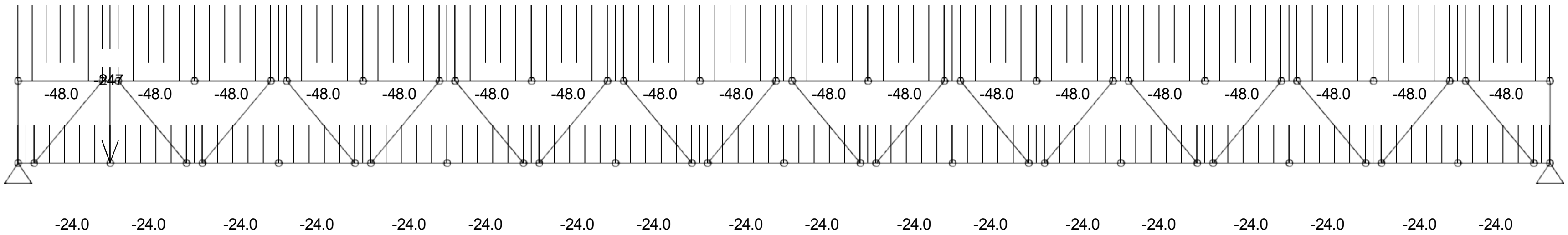
Member Diagram for Truss RJ2
Engineering Status = 52%

~~LC13 (1.2G + 1.6P) Maximum for this Load Case = 38%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Load Diagram for Truss RJ2

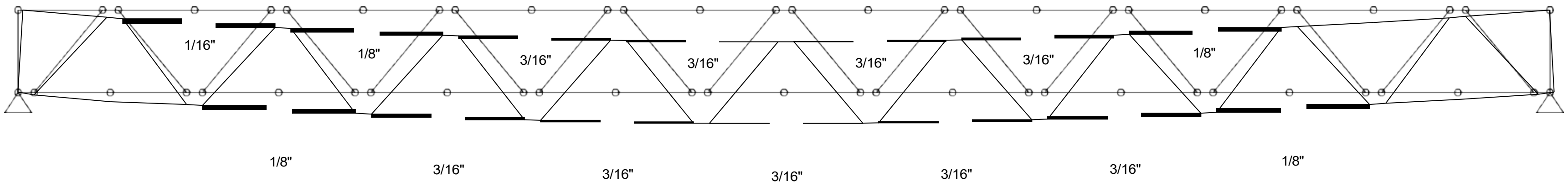
Engineering Status = 52%

LC13 (1.2G + 1.6P) Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



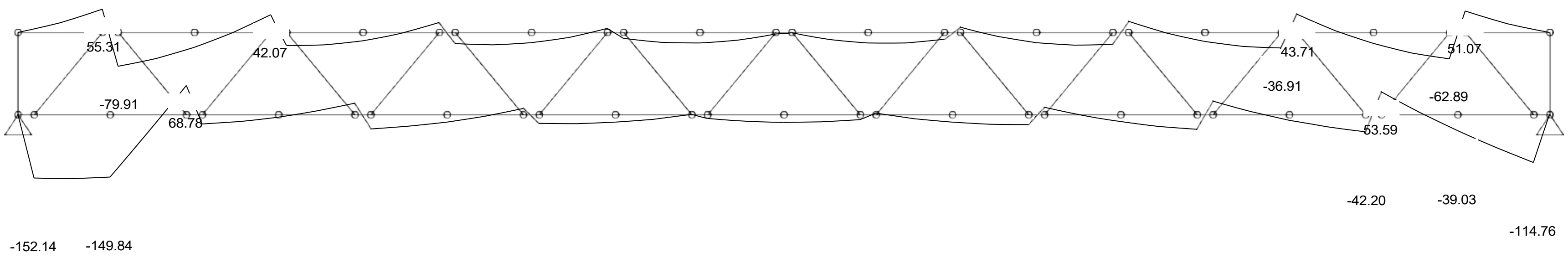
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC13 (1.2G + 1.6P)   Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



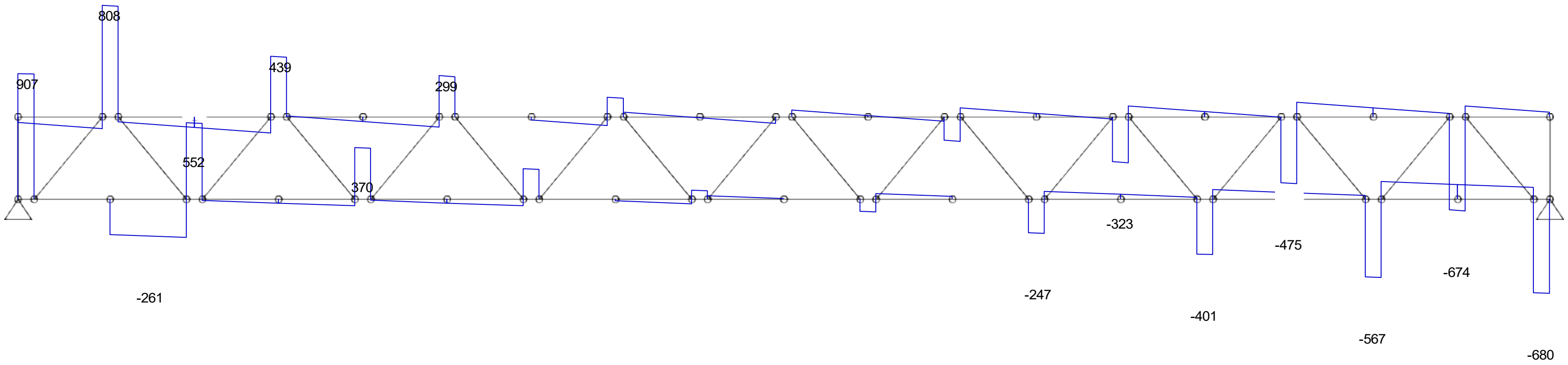
Bending Diagram for Truss RJ2
Engineering Status = 52%

LC13 (1.2G + 1.6P) Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

Engineering Status = 52%

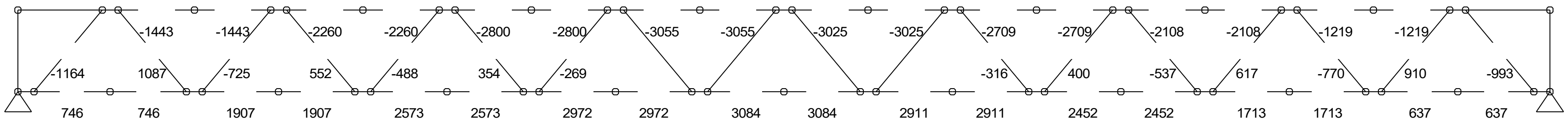
LC13 (1.2G + 1.6P)   Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



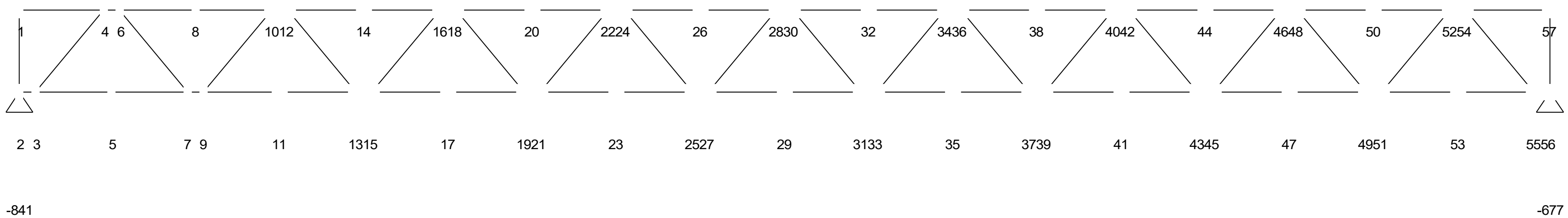
Axial Diagram for Truss RJ2
Engineering Status = 52%

~~LC13 (1.2G + 1.6P) Maximum for this Load Case = 38%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



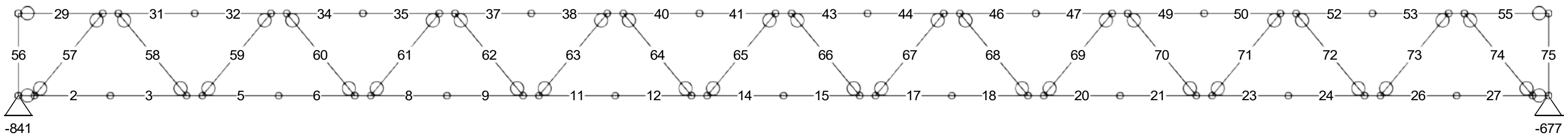
Node Diagram for Truss RJ2
Engineering Status = 52%

LC14 (1.2G + 1.6P)   Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



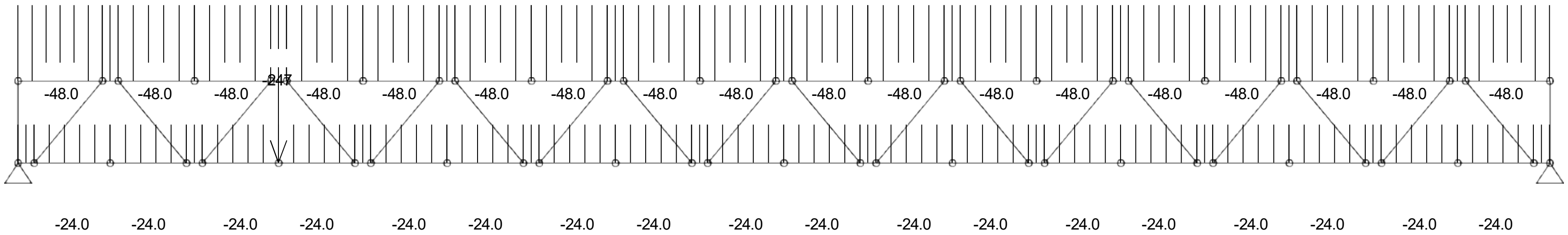
Member Diagram for Truss RJ2
Engineering Status = 52%

LC14 (1.2G + 1.6P) Maximum for this Load Case = 39%

System Name: FRAMECAD_FT_m
Roof Type: SHEET
Wind Speed: 200
Terrain Factor Kz 1.00
Importance Factor I 1.00
Topography Factor Kzt 1.00
Truss Spacing: 2'-0"
Top Chord Restraints: 3'-11 1/4"
Bottom Chord Restraints: 3'-11 1/4"
Design Code: AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load: 20.0psf
Top Chord Dead Load: 20.0psf
Bottom Chord Live Load: 0.0psf
Bottom Chord Dead Load: 10.0psf
Bottom Chord Services Load: 0.0psf
Top Chord Ground Snow Load: 0.0psf
Suspended Ceilings
Loading Code: IBC 2018 LRFD



Load Diagram for Truss RJ2
Engineering Status = 52%

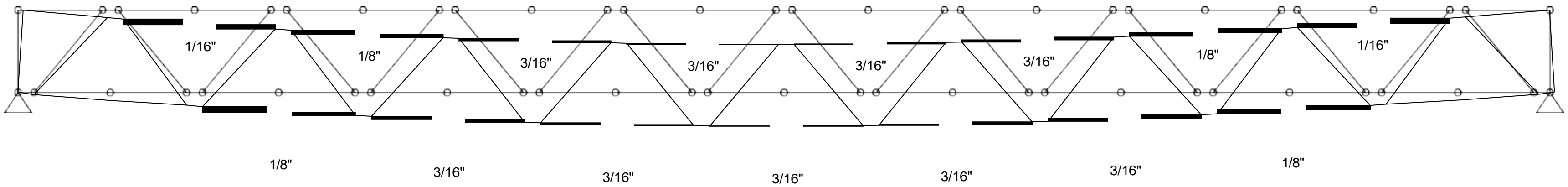
LC14 (1.2G + 1.6P) Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

FRAMECAD



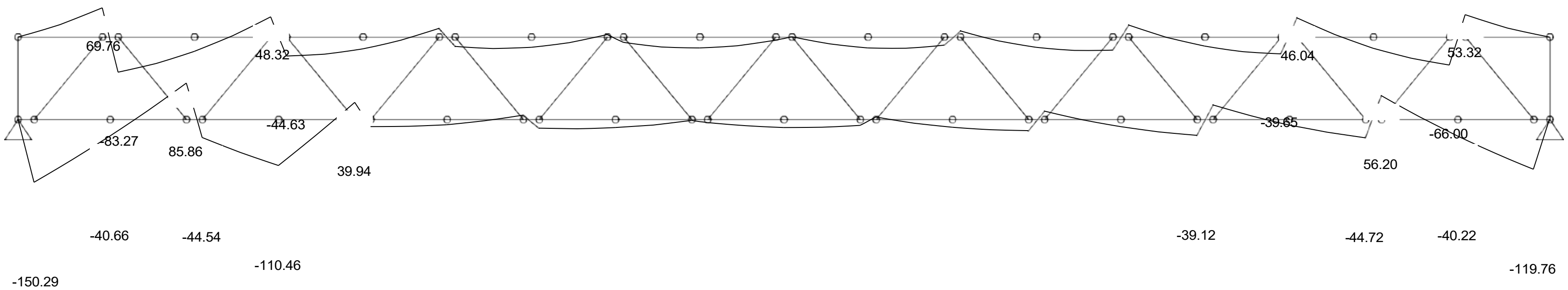
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC14 (1.2G + 1.6P) Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

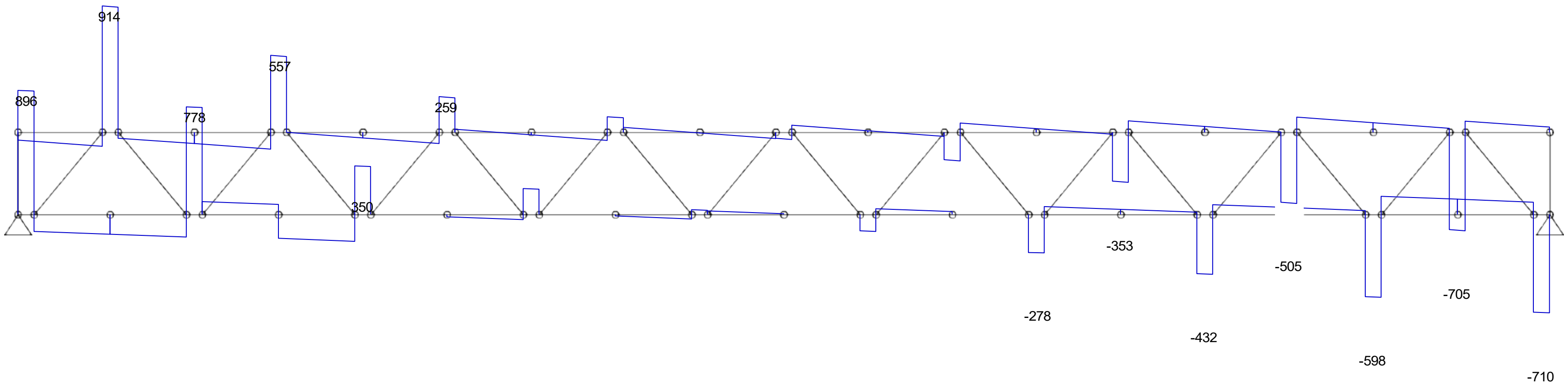
Engineering Status = 52%

LC14 (1.2G + 1.6P)   Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

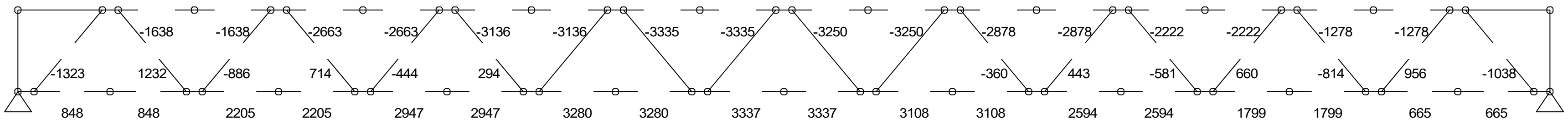
Engineering Status = 52%

LC14 (1.2G + 1.6P)   Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



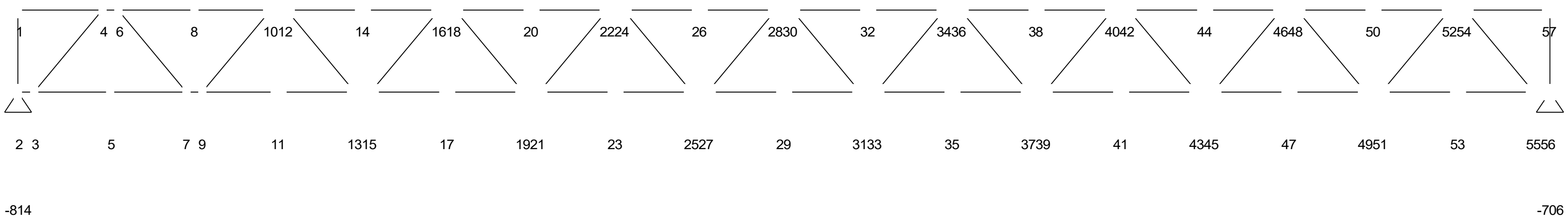
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC14 (1.2G + 1.6P) Maximum for this Load Case = 39%

System Name: FRAMECAD_FT_m
Roof Type: SHEET
Wind Speed: 200
Terrain Factor Kz 1.00
Importance Factor I 1.00
Topography Factor Kzt 1.00
Truss Spacing: 2'-0"
Top Chord Restraints: 3'-11 1/4"
Bottom Chord Restraints: 3'-11 1/4"
Design Code: AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load: 20.0psf
Top Chord Dead Load: 20.0psf
Bottom Chord Live Load: 0.0psf
Bottom Chord Dead Load: 10.0psf
Bottom Chord Services Load: 0.0psf
Top Chord Ground Snow Load: 0.0psf
Suspended Ceilings
Loading Code: IBC 2018 LRFD



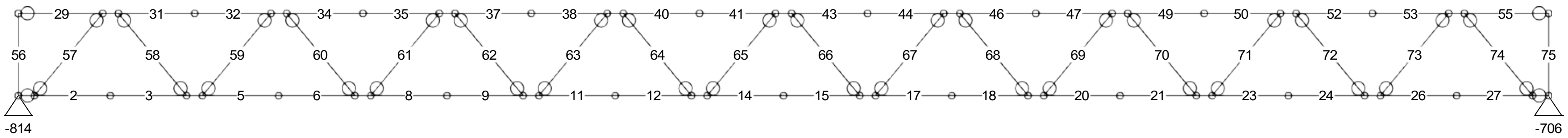
Node Diagram for Truss RJ2
Engineering Status = 52%

LC15 (1.2G + 1.6P)   Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



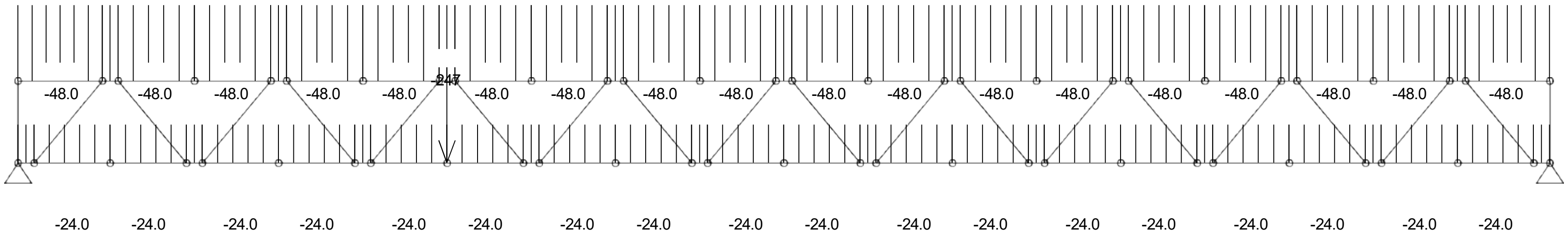
Member Diagram for Truss RJ2
Engineering Status = 52%

LC15 (1.2G + 1.6P) Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



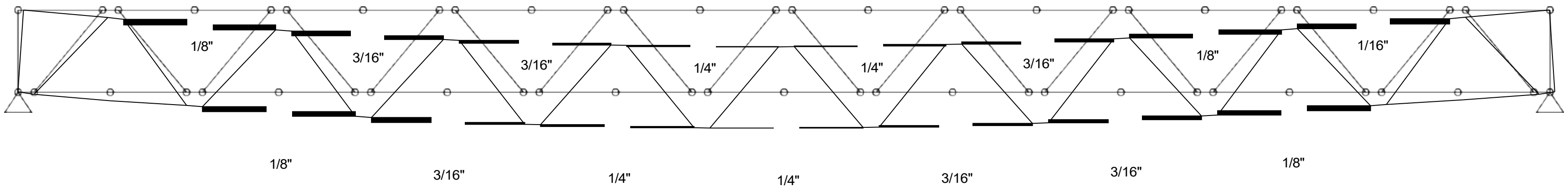
Load Diagram for Truss RJ2
Engineering Status = 52%

LC15 (1.2G + 1.6P) Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Deflection Diagram for Truss RJ2
Engineering Status = 52%

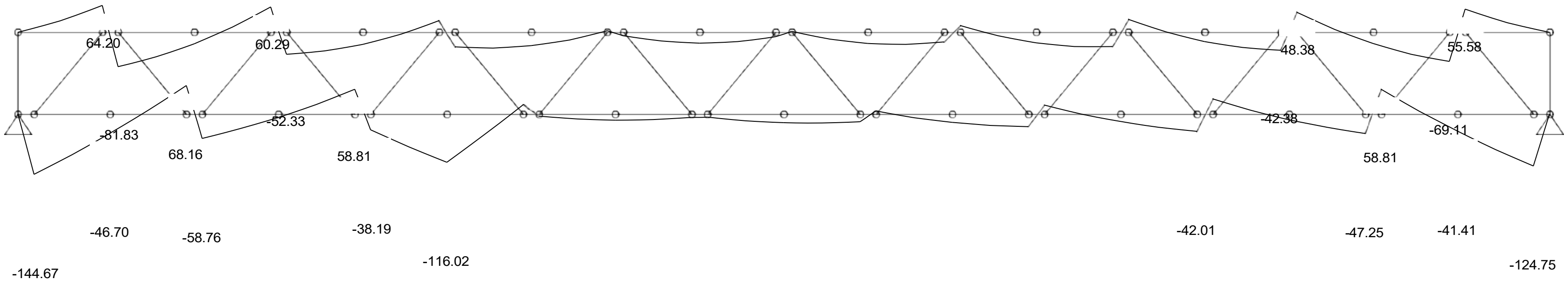
LC15 (1.2G + 1.6P)   Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD®

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

Engineering Status = 52%

LC15 (1.2G + 1.6P)   Maximum for this Load Case = 37%

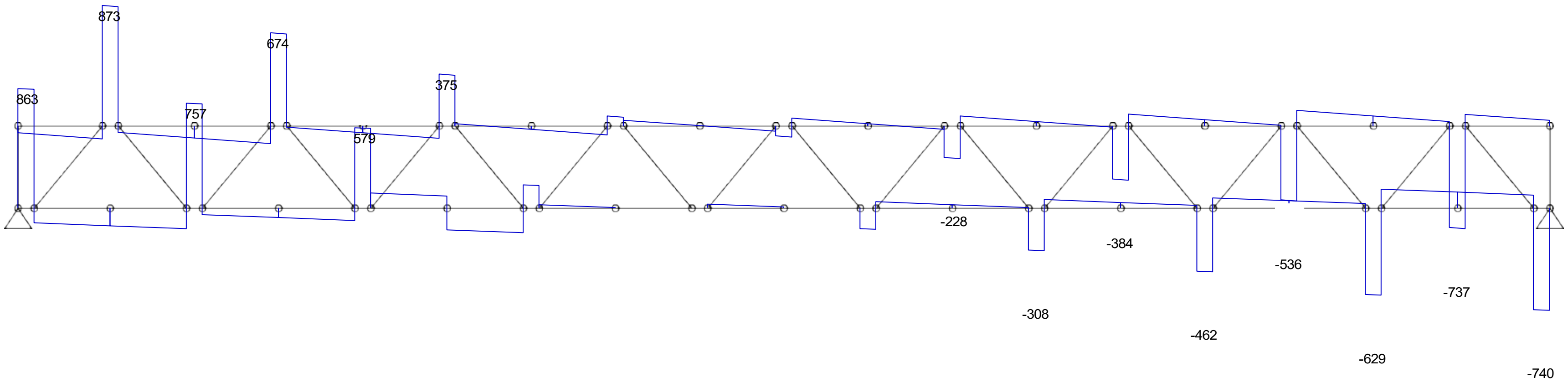
Company: Bauhu   Dwg Name: 242201_Exuma, Bahamas roof truss MS   Sheet 104 of 1   Project: Exuma, Bahamas   Job Number:242201

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

FRAMECAD



Shear Diagram for Truss RJ2
Engineering Status = 52%

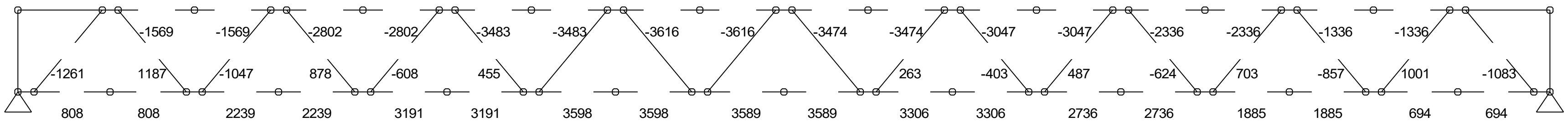
LC15 (1.2G + 1.6P)   Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD®

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



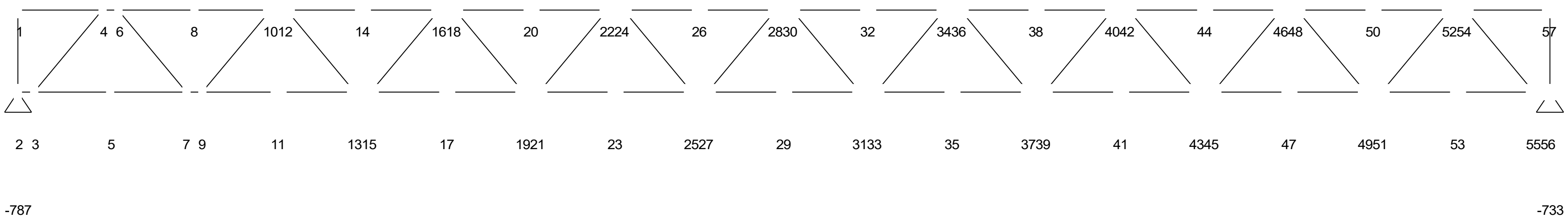
Axial Diagram for Truss RJ2
Engineering Status = 52%

~~LC15 (1.2G + 1.6P) Maximum for this Load Case = 37%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

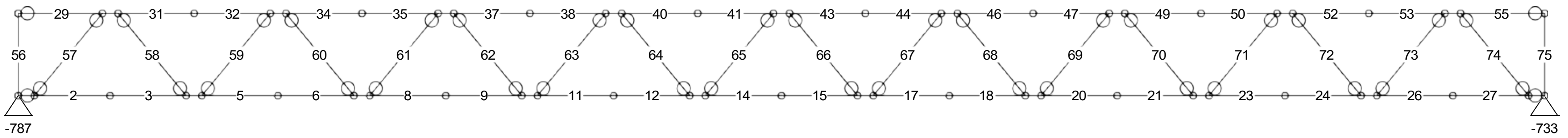
LC16 (1.2G + 1.6P)   Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



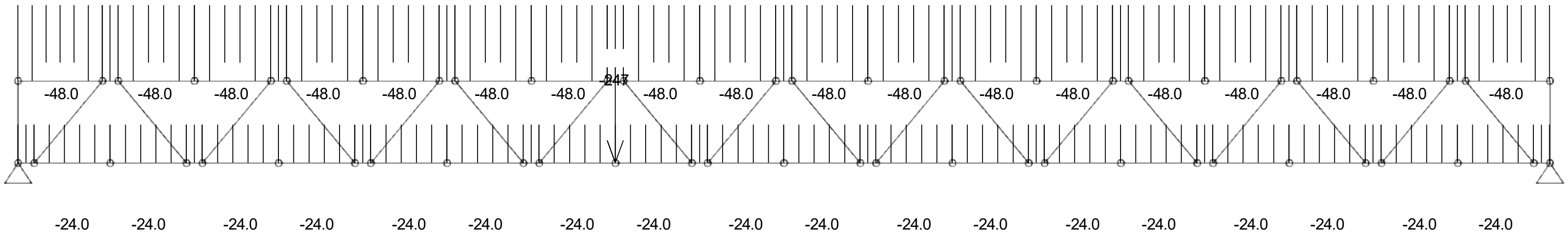
Member Diagram for Truss RJ2
Engineering Status = 52%

LC16 (1.2G + 1.6P) Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



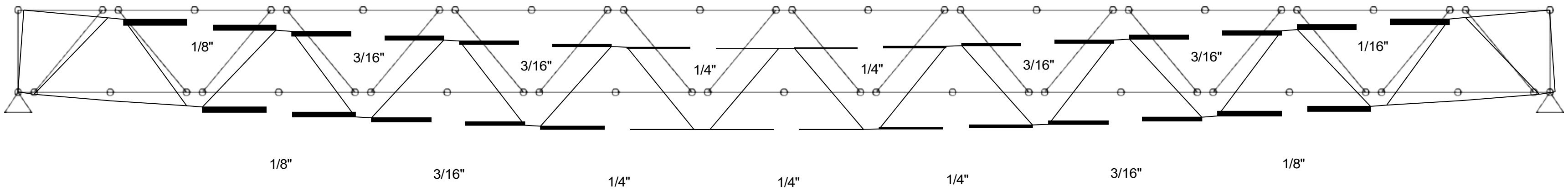
Load Diagram for Truss RJ2
Engineering Status = 52%

LC16 (1.2G + 1.6P) Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



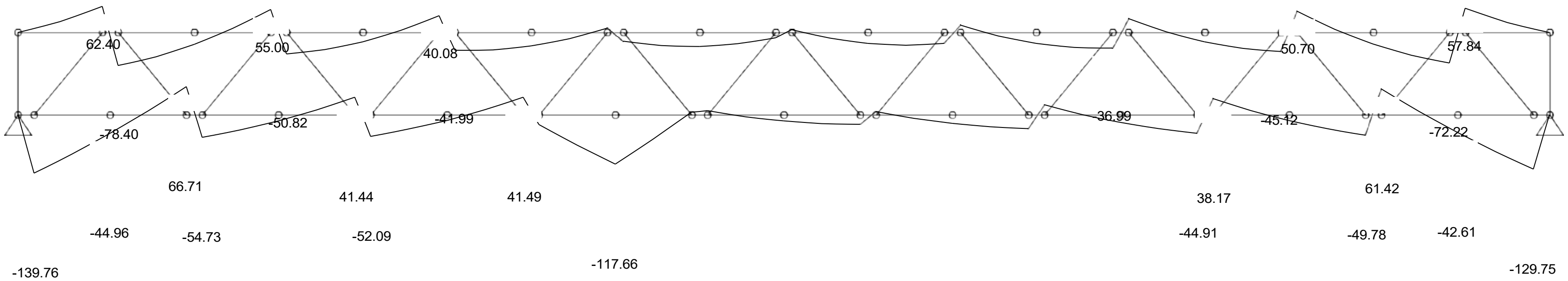
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC16 (1.2G + 1.6P) Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

Engineering Status = 52%

LC16 (1.2G + 1.6P)   Maximum for this Load Case = 41%

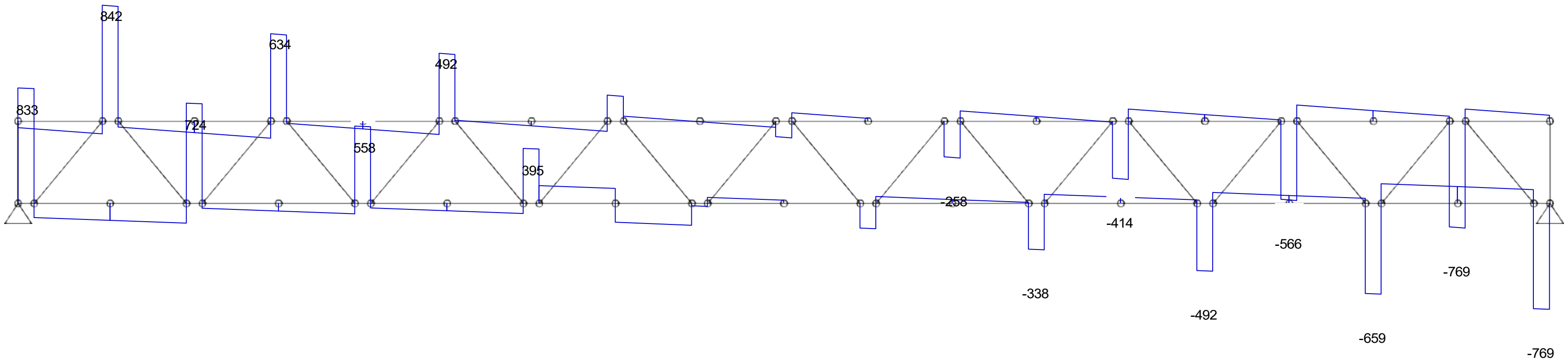
Company: Bauhu   Dwg Name: 242201_Exuma, Bahamas roof truss MS Sheet 111 of 1   Project: Exuma, Bahamas   Job Number:242201

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

FRAMECAD



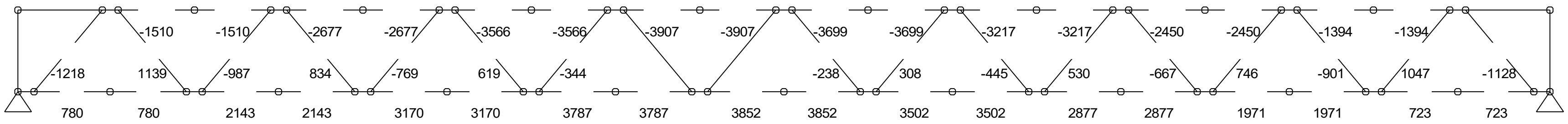
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC16 (1.2G + 1.6P) Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



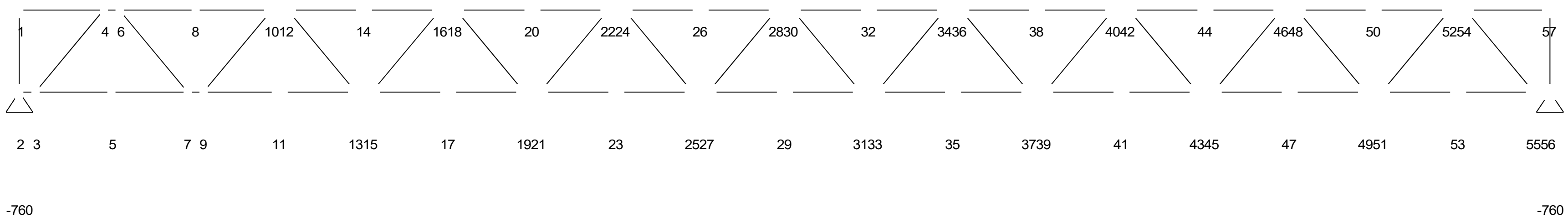
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC16 (1.2G + 1.6P) Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

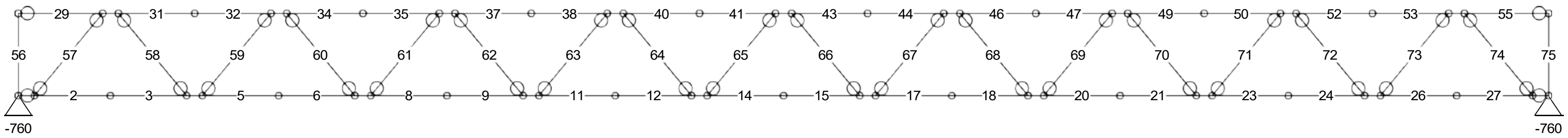
LC17 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



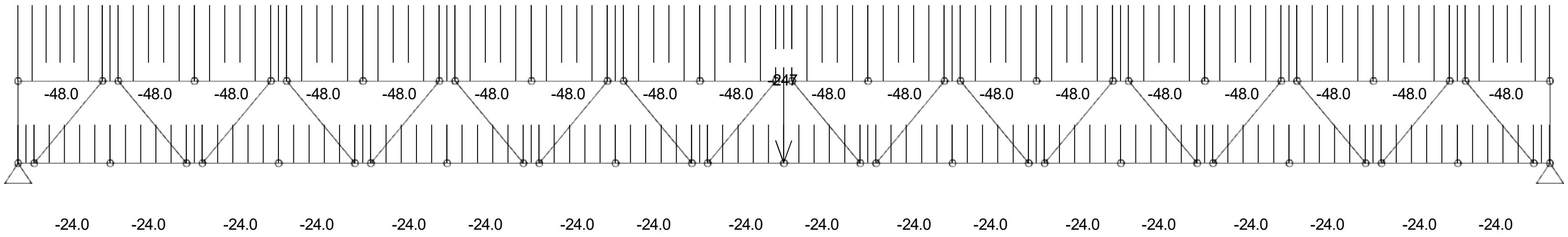
Member Diagram for Truss RJ2
Engineering Status = 52%

LC17 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



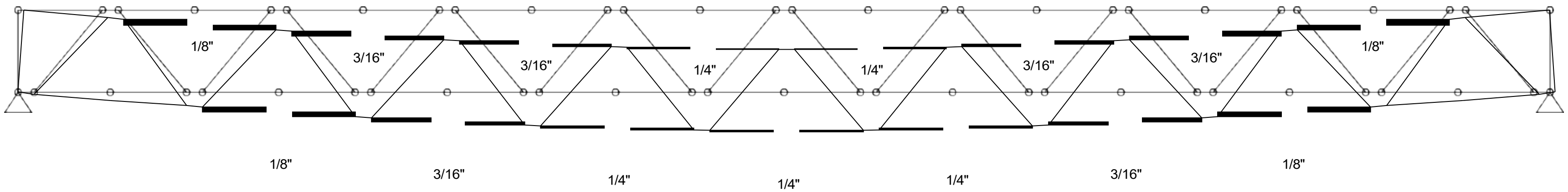
Load Diagram for Truss RJ2
Engineering Status = 52%

LC17 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



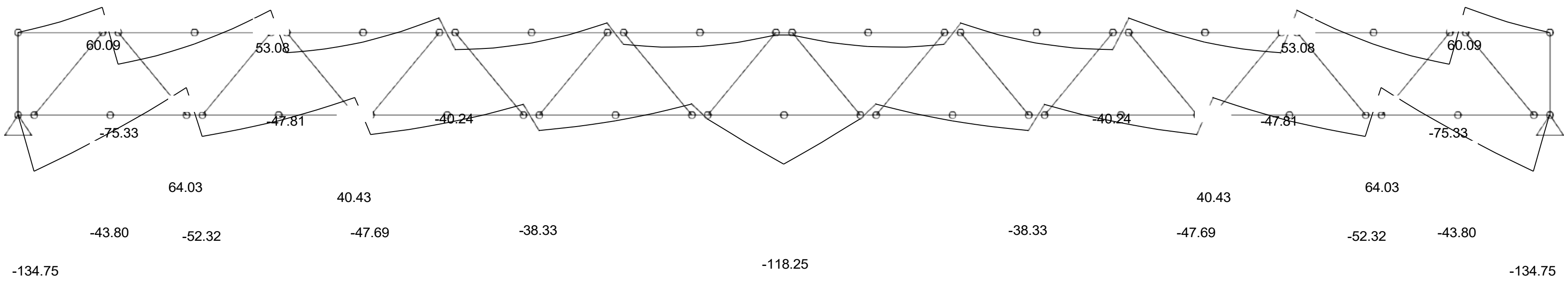
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC17 (1.2G + 1.6P)   Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

Engineering Status = 52%

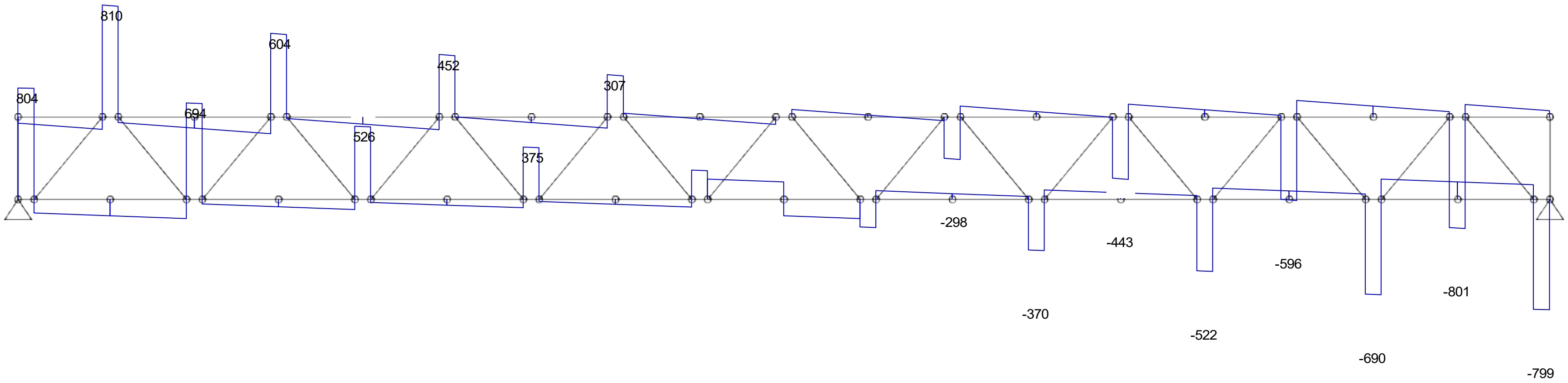
LC17 (1.2G + 1.6P) Maximum for this Load Case = 42%

Company: Bauhu | Dwg Name: 242201_Exuma, Bahamas roof truss MS Sheet 118 of 1 | Project: Exuma, Bahamas | Job Number:242201

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

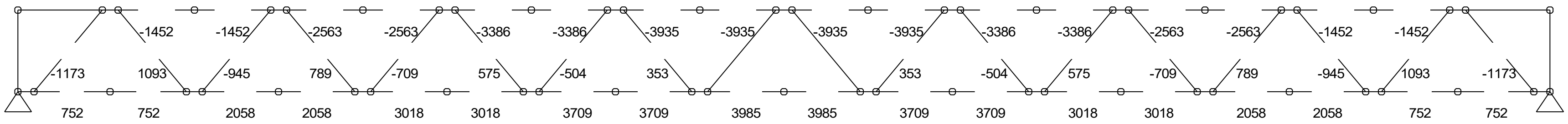
Engineering Status = 52%

LC17 (1.2G + 1.6P)   Maximum for this Load Case = 42%

System Name: FRAMECAD_FT_m
Roof Type: SHEET
Wind Speed: 200
Terrain Factor Kz 1.00
Importance Factor I 1.00
Topography Factor Kzt 1.00
Truss Spacing: 2'-0"
Top Chord Restraints: 3'-11 1/4"
Bottom Chord Restraints: 3'-11 1/4"
Design Code: AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load: 20.0psf
Top Chord Dead Load: 20.0psf
Bottom Chord Live Load: 0.0psf
Bottom Chord Dead Load: 10.0psf
Bottom Chord Services Load: 0.0psf
Top Chord Ground Snow Load: 0.0psf
Suspended Ceilings
Loading Code: IBC 2018 LRFD



Axial Diagram for Truss RJ2
Engineering Status = 52%

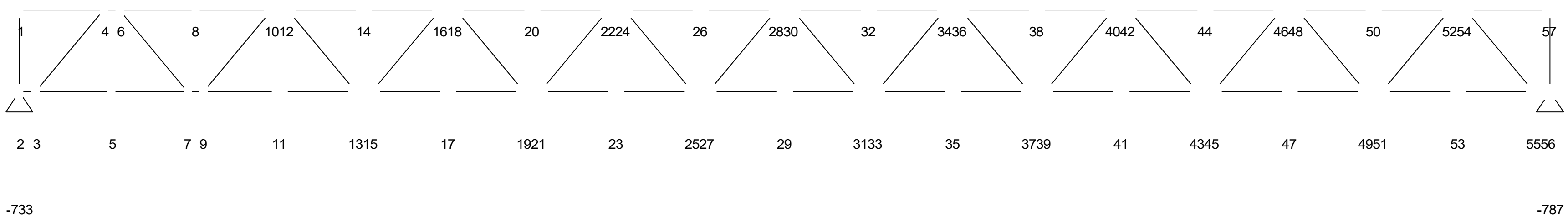
LC17 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



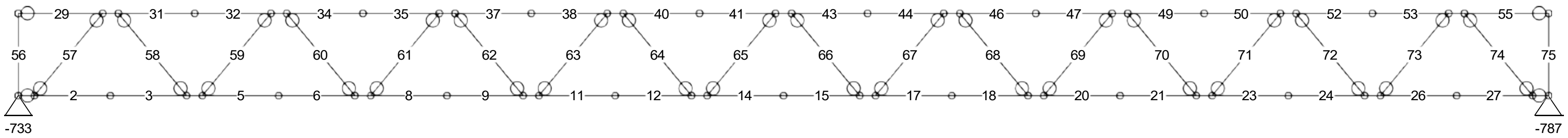
Node Diagram for Truss RJ2
Engineering Status = 52%

LC18 (1.2G + 1.6P)   Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



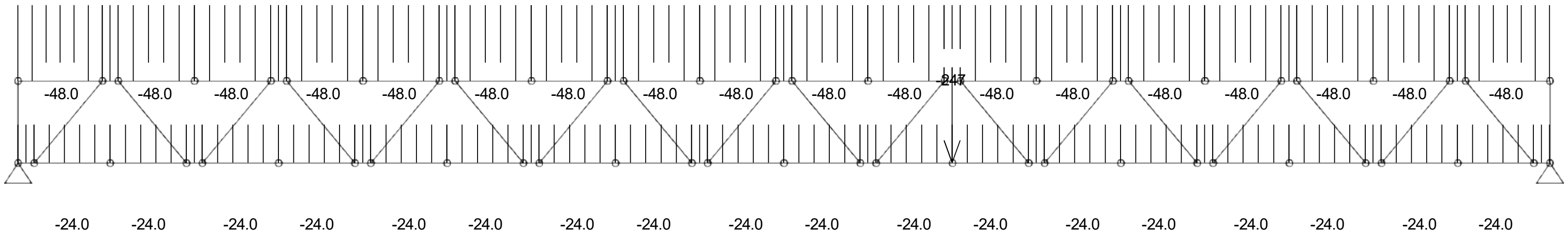
Member Diagram for Truss RJ2
Engineering Status = 52%

LC18 (1.2G + 1.6P) Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



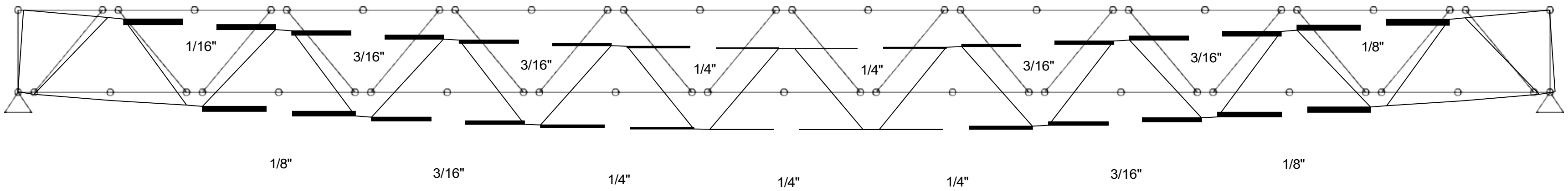
Load Diagram for Truss RJ2
Engineering Status = 52%

LC18 (1.2G + 1.6P) Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



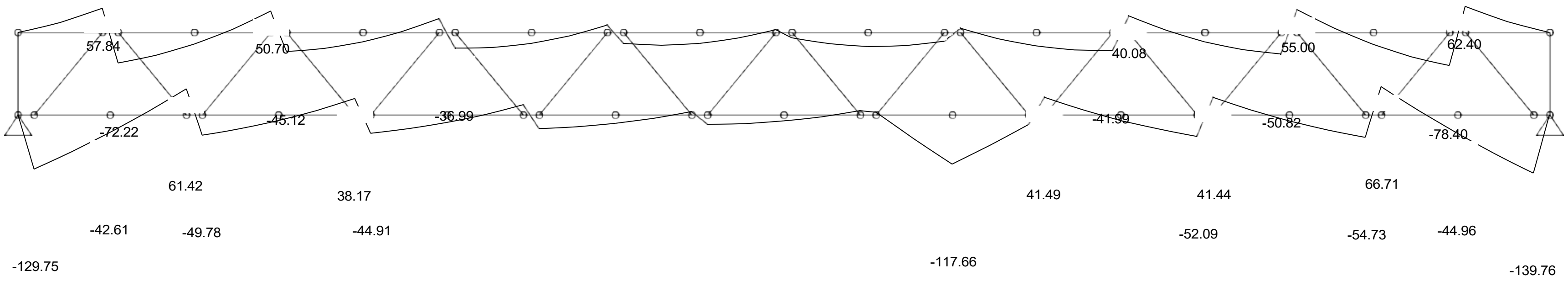
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC18 (1.2G + 1.6P)   Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



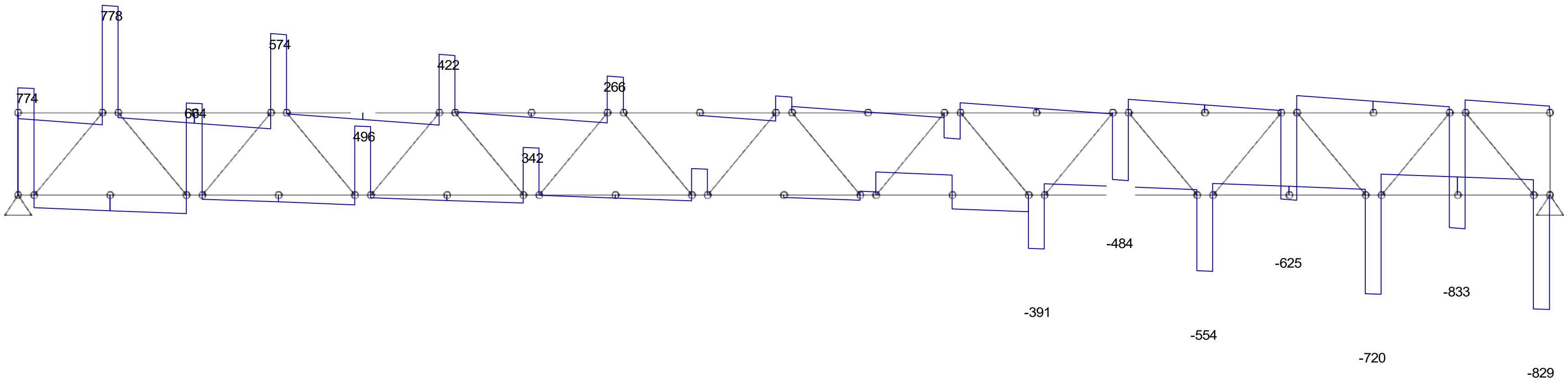
Bending Diagram for Truss RJ2

Engineering Status = 52%

LC18 (1.2G + 1.6P)   Maximum for this Load Case = 41%

Company: Bauhu   Dwg Name: 242201_Exuma, Bahamas roof truss MS Sheet 125 of 1   Project: Exuma, Bahamas   Job Number:242201

System Name:	FRAMECAD_FT_m	All force values are shown in lb Only values over 200 lb are shown	Top Chord Live Load:	20.0psf
Roof Type:	SHEET		Top Chord Dead Load:	20.0psf
Wind Speed:	200		Bottom Chord Live Load:	0.0psf
Terrain Factor Kz	1.00		Bottom Chord Dead Load:	10.0psf
Importance Factor I	1.00		Bottom Chord Services Load:	0.0psf
Topography Factor Kzt	1.00		Top Chord Ground Snow Load:	0.0psf
Truss Spacing:	2'-0"		Suspended Ceilings	
Top Chord Restraints:	3'-11 1/4"		Loading Code:	IBC 2018 LRFD
Bottom Chord Restraints:	3'-11 1/4"			
Design Code:	AISI S100-20 LRFD			



Shear Diagram for Truss RJ2

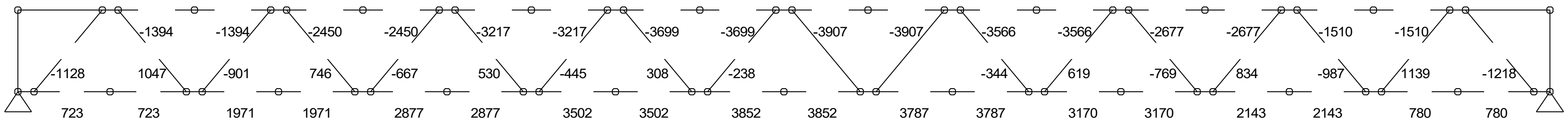
Engineering Status = 52%

LC18 (1.2G + 1.6P)   Maximum for this Load Case = 41%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Axial Diagram for Truss RJ2
Engineering Status = 52%

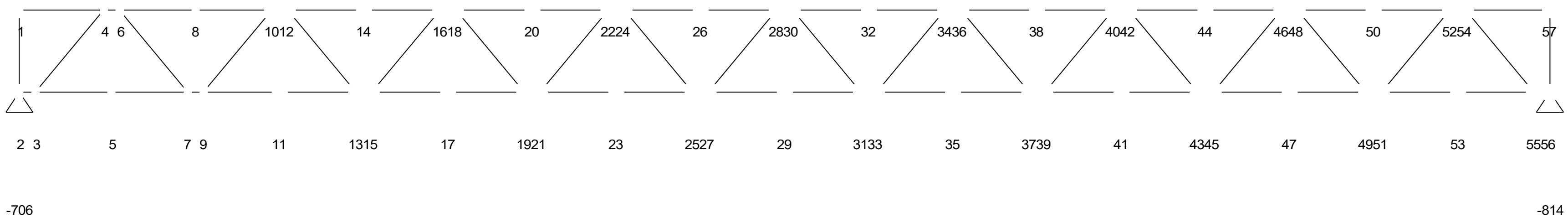
~~LC18 (1.2G + 1.6P) Maximum for this Load Case = 41%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

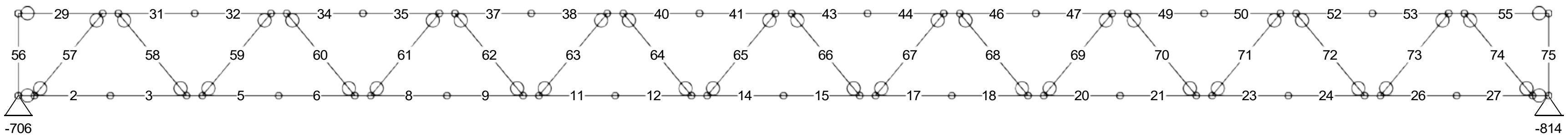
LC19 (1.2G + 1.6P)   Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



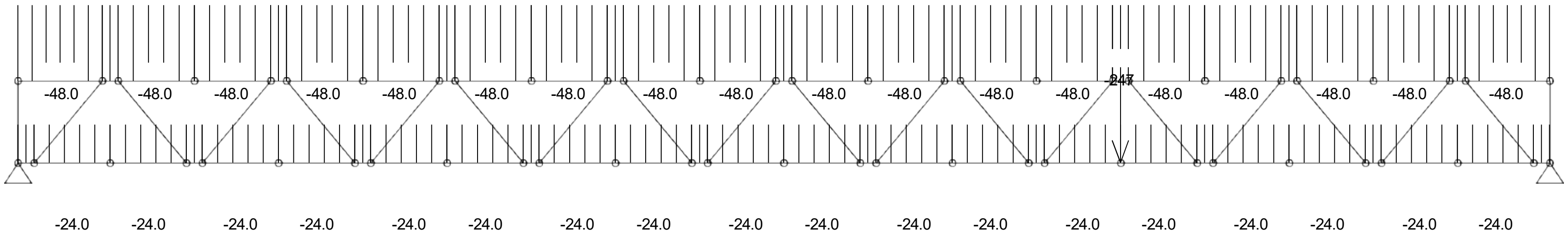
Member Diagram for Truss RJ2
Engineering Status = 52%

LC19 (1.2G + 1.6P) Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Load Diagram for Truss RJ2
Engineering Status = 52%

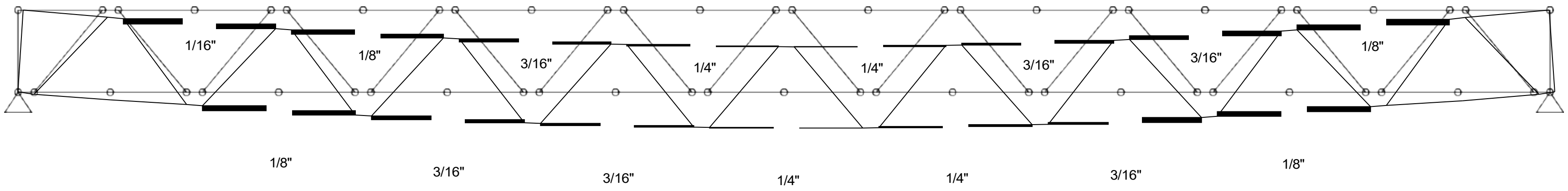
LC19 (1.2G + 1.6P) Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD®

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



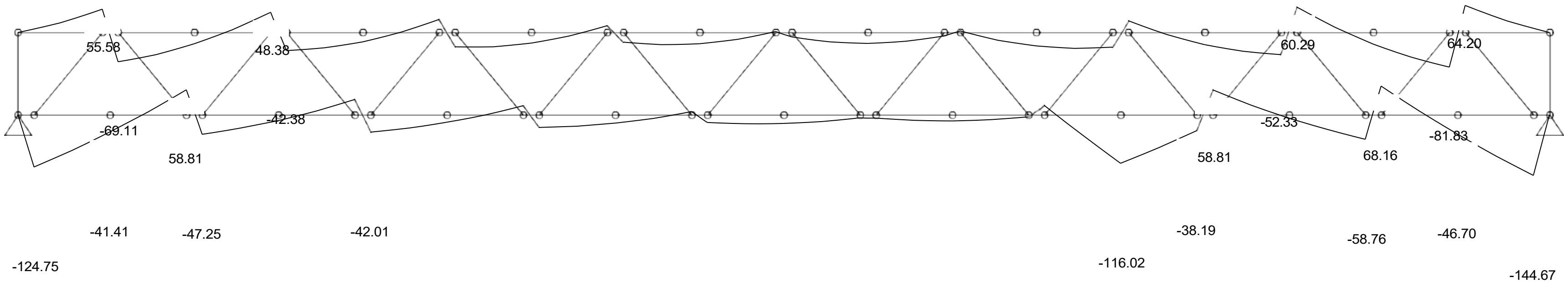
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC19 (1.2G + 1.6P)   Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

Engineering Status = 52%

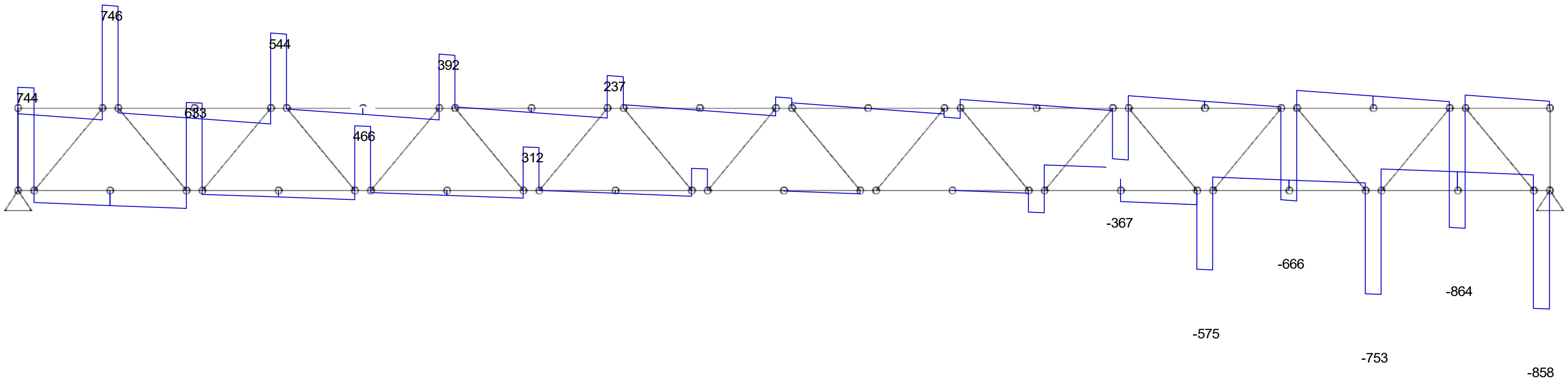
LC19 (1.2G + 1.6P)   Maximum for this Load Case = 37%

Company: Bauhu   Dwg Name: 242201_Exuma, Bahamas roof truss MS   Sheet 132 of 1   Project: Exuma, Bahamas   Job Number:242201

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

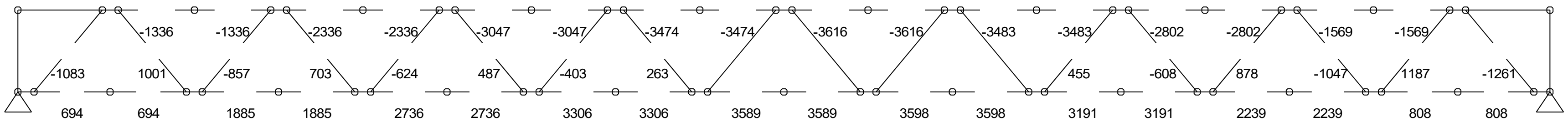
Engineering Status = 52%

LC19 (1.2G + 1.6P)   Maximum for this Load Case = 37%

System Name: FRAMECAD_FT_m
Roof Type: SHEET
Wind Speed: 200
Terrain Factor Kz 1.00
Importance Factor I 1.00
Topography Factor Kzt 1.00
Truss Spacing: 2'-0"
Top Chord Restraints: 3'-11 1/4"
Bottom Chord Restraints: 3'-11 1/4"
Design Code: AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load: 20.0psf
Top Chord Dead Load: 20.0psf
Bottom Chord Live Load: 0.0psf
Bottom Chord Dead Load: 10.0psf
Bottom Chord Services Load: 0.0psf
Top Chord Ground Snow Load: 0.0psf
Suspended Ceilings
Loading Code: IBC 2018 LRFD



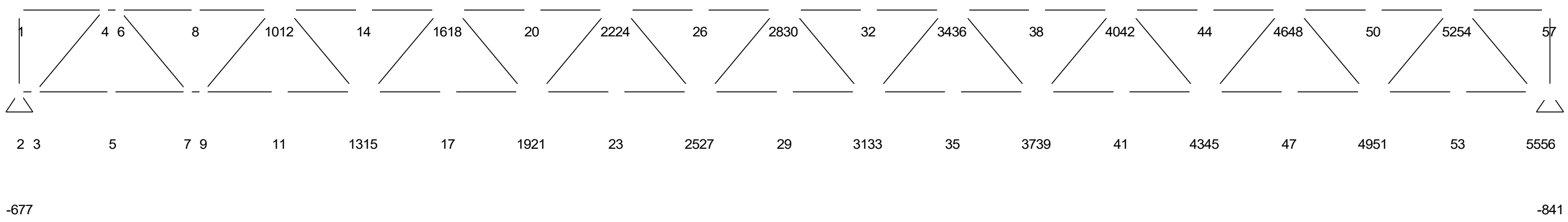
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC19 (1.2G + 1.6P) Maximum for this Load Case = 37%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



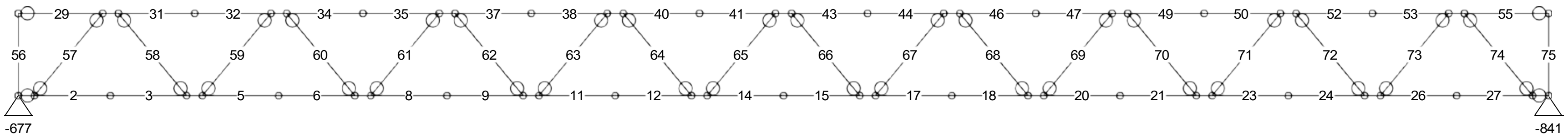
Node Diagram for Truss RJ2
Engineering Status = 52%

LC20 (1.2G + 1.6P)   Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Member Diagram for Truss RJ2
Engineering Status = 52%

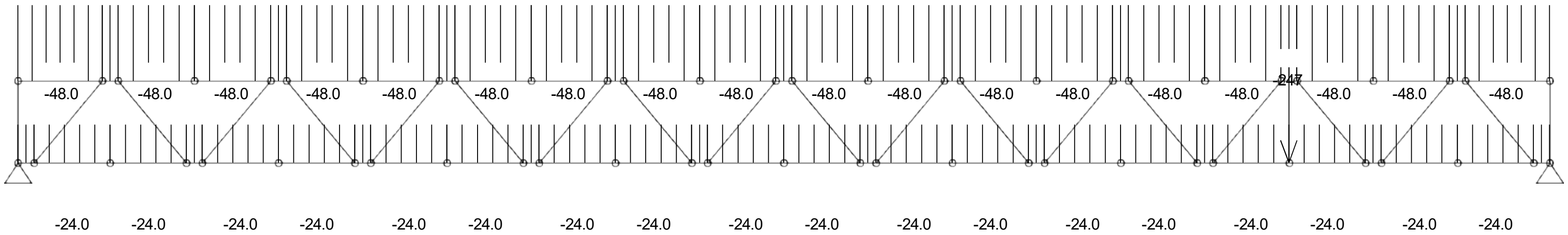
LC20 (1.2G + 1.6P) Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

FRAMECAD



Load Diagram for Truss RJ2
Engineering Status = 52%

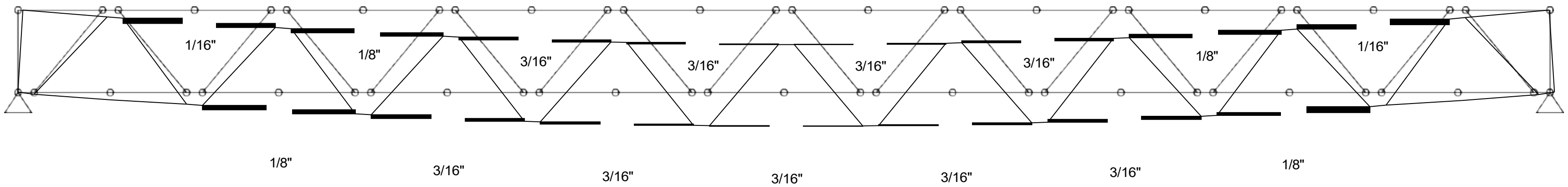
LC20 (1.2G + 1.6P) Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD®

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



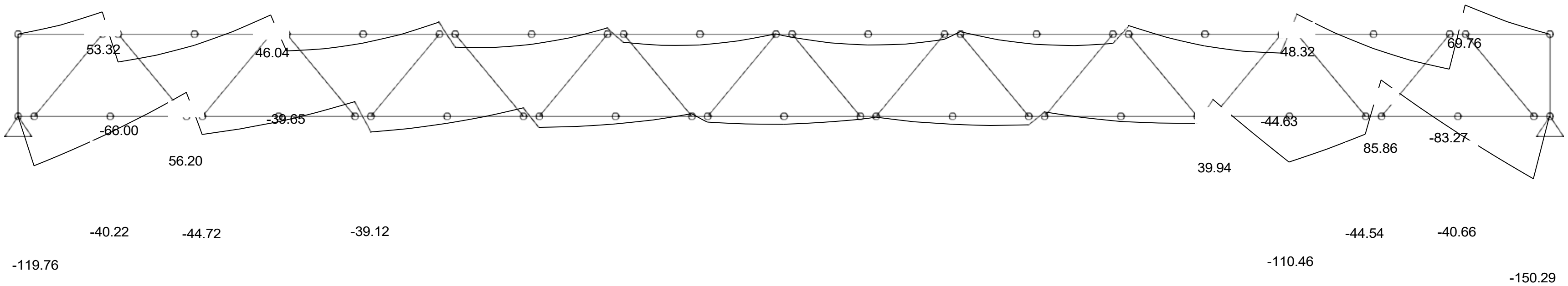
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC20 (1.2G + 1.6P)   Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

Engineering Status = 52%

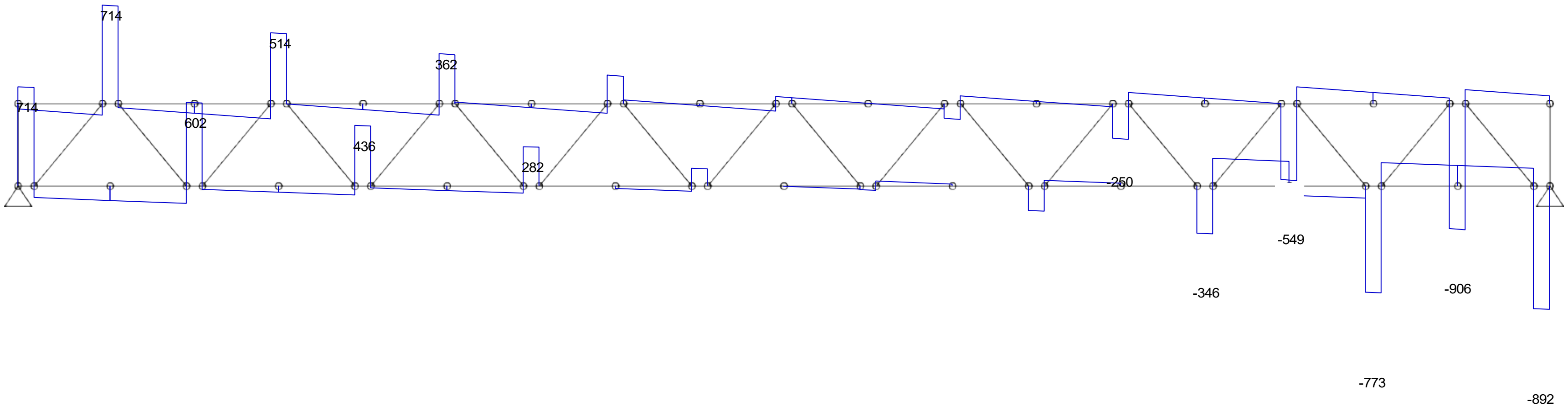
LC20 (1.2G + 1.6P)   Maximum for this Load Case = 39%

Company: Bauhu   Dwg Name: 242201_Exuma, Bahamas roof truss MS Sheet 139 of 1   Project: Exuma, Bahamas   Job Number:242201

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



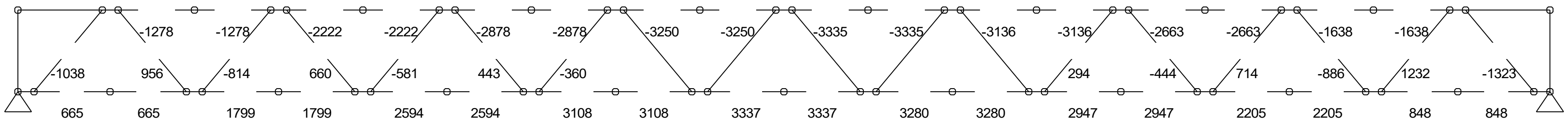
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC20 (1.2G + 1.6P)   Maximum for this Load Case = 39%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



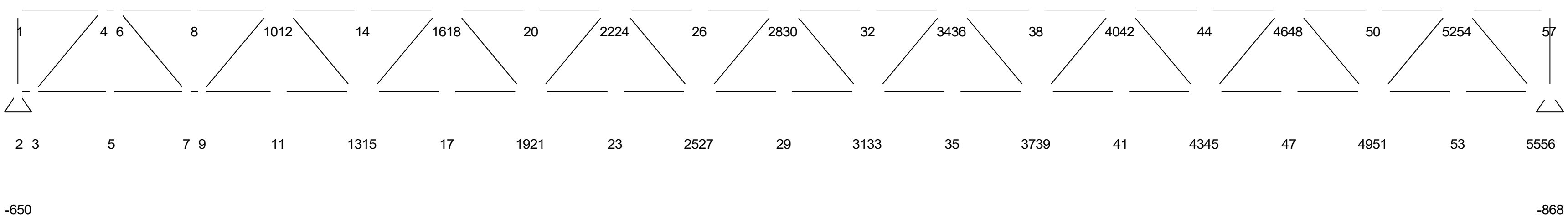
Axial Diagram for Truss RJ2
Engineering Status = 52%

~~LC20 (1.2G + 1.6P) Maximum for this Load Case = 39%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



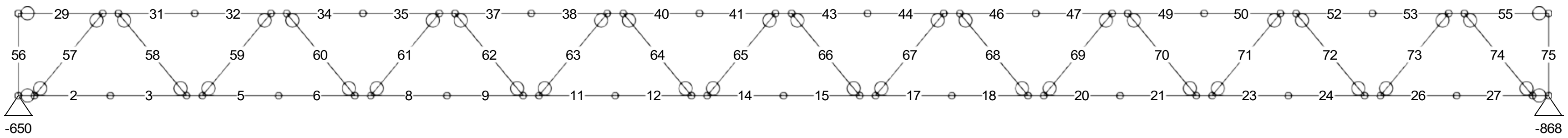
Node Diagram for Truss RJ2
Engineering Status = 52%

LC21 (1.2G + 1.6P)   Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Member Diagram for Truss RJ2
Engineering Status = 52%

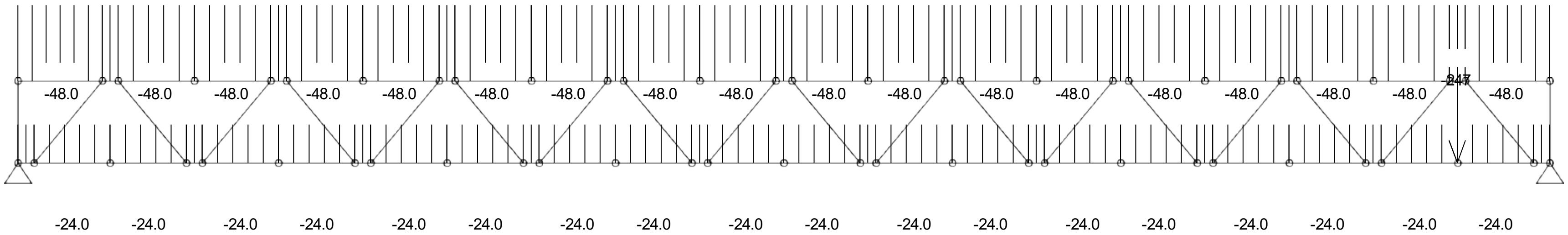
~~LC21 (1.2G + 1.6P) Maximum for this Load Case = 38%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD®

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



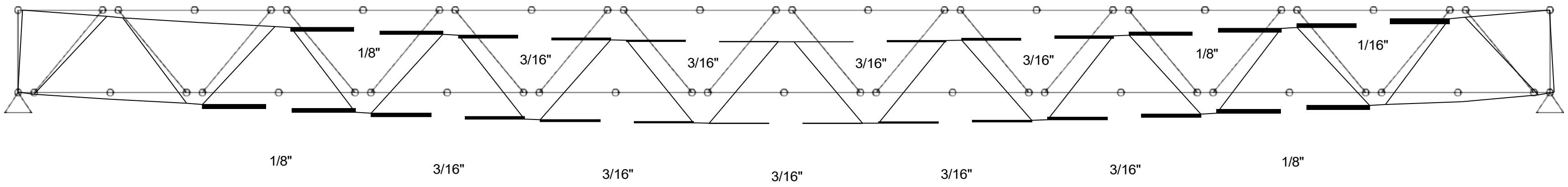
Load Diagram for Truss RJ2
Engineering Status = 52%

LC21 (1.2G + 1.6P) Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



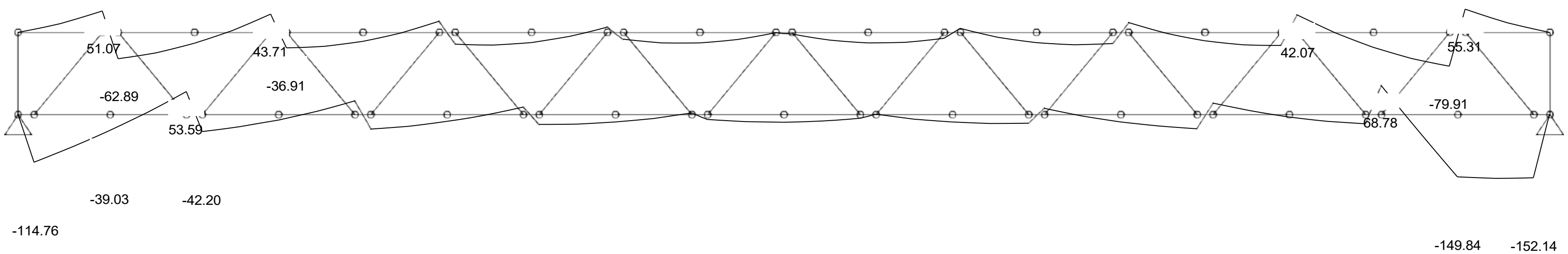
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC21 (1.2G + 1.6P)   Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

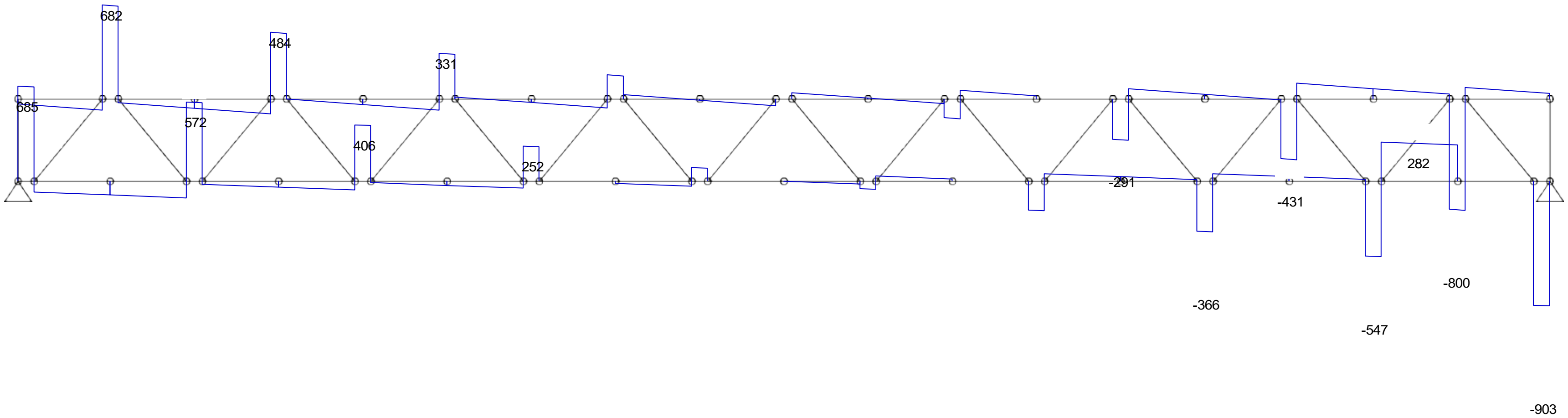
Engineering Status = 52%

LC21 (1.2G + 1.6P) Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

Engineering Status = 52%

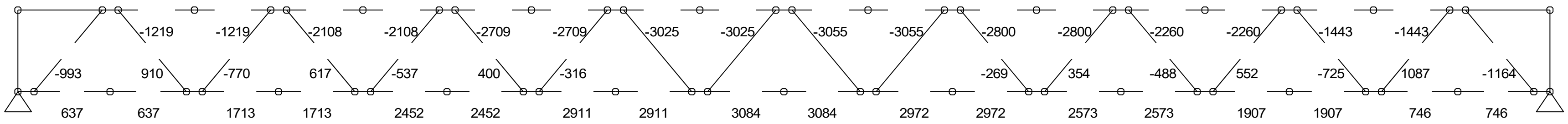
LC21 (1.2G + 1.6P)   Maximum for this Load Case = 38%

Company: Bauhu | Dwg Name: 242201_Exuma, Bahamas roof truss MS Sheet 147 of 1 | Project: Exuma, Bahamas | Job Number:242201

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



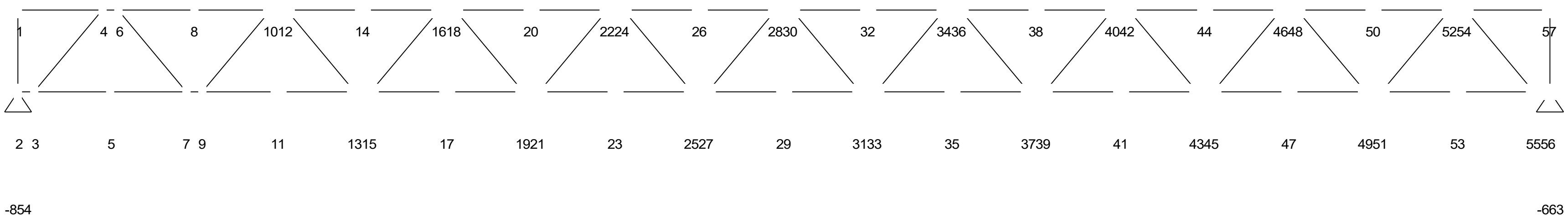
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC21 (1.2G + 1.6P) Maximum for this Load Case = 38%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Node Diagram for Truss RJ2
Engineering Status = 52%

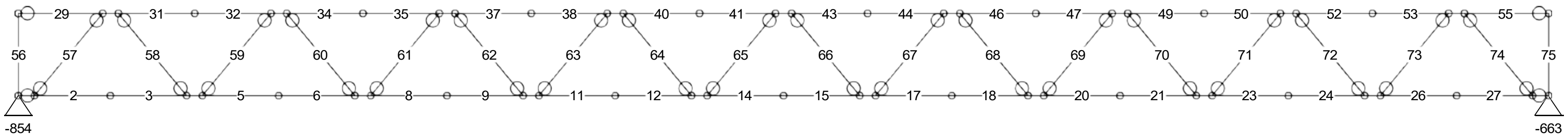
LC22 (1.2G + 1.6P)   Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



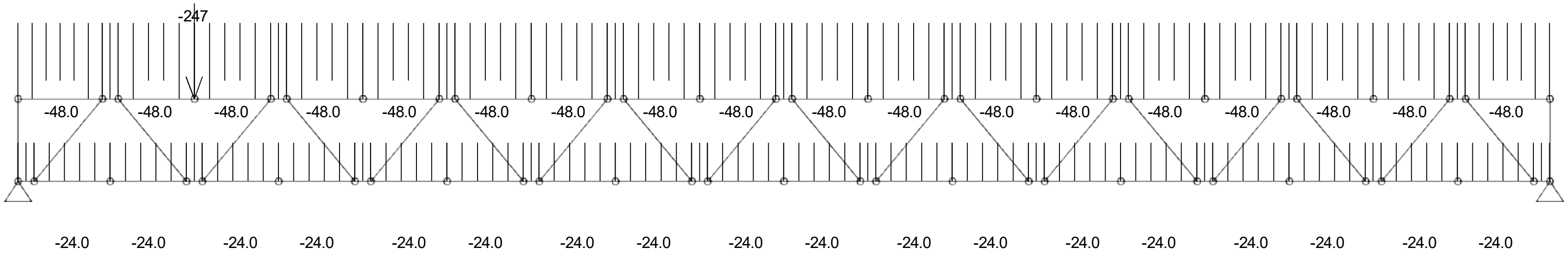
Member Diagram for Truss RJ2
Engineering Status = 52%

LC22 (1.2G + 1.6P) Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



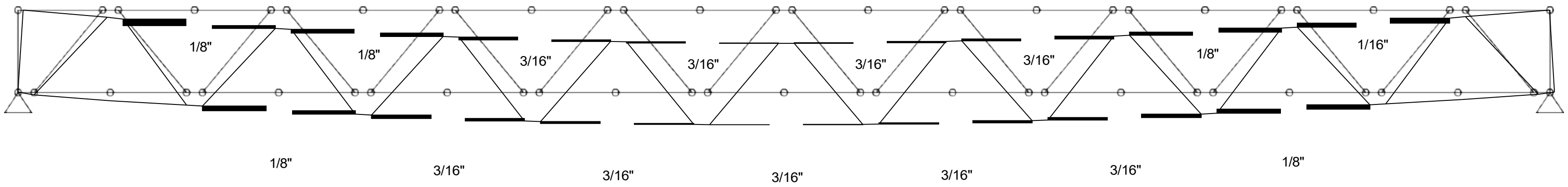
Load Diagram for Truss RJ2
Engineering Status = 52%

LC22 (1.2G + 1.6P) Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



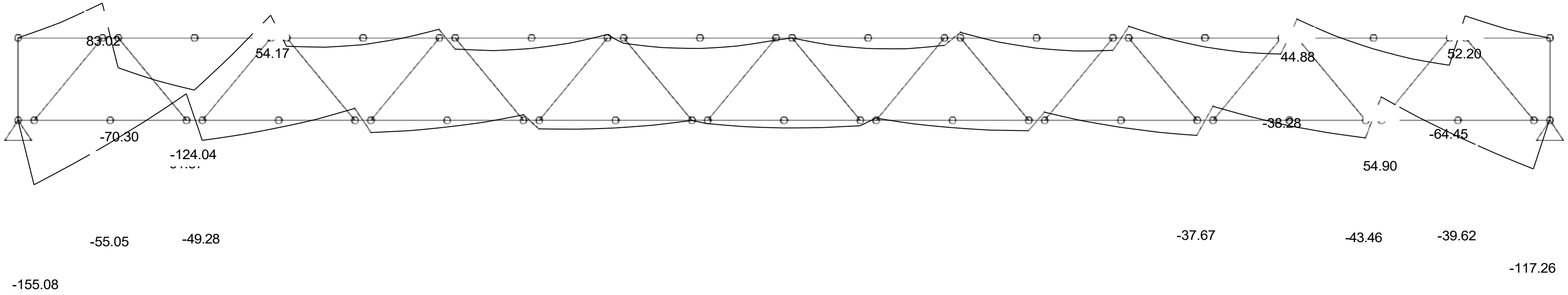
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC22 (1.2G + 1.6P)   Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

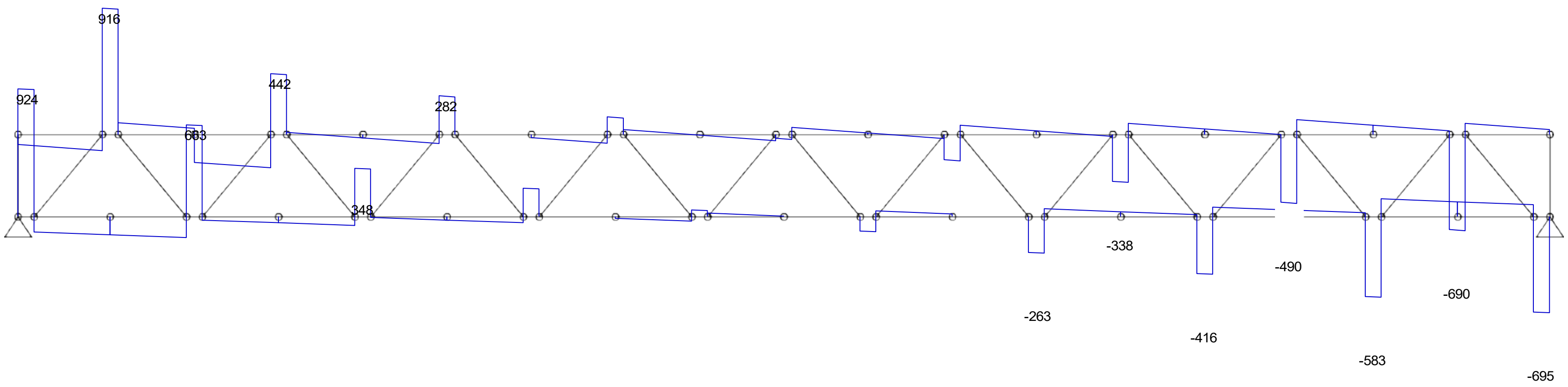
Engineering Status = 52%

LC22 (1.2G + 1.6P) Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



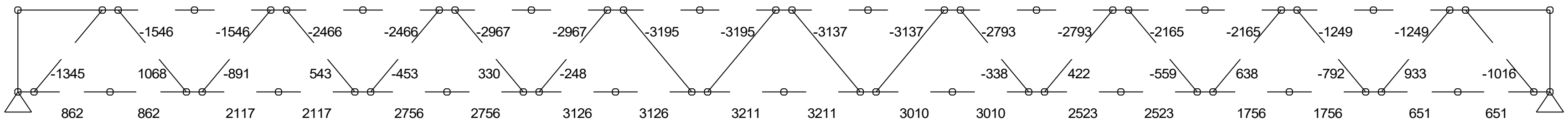
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC22 (1.2G + 1.6P)   Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



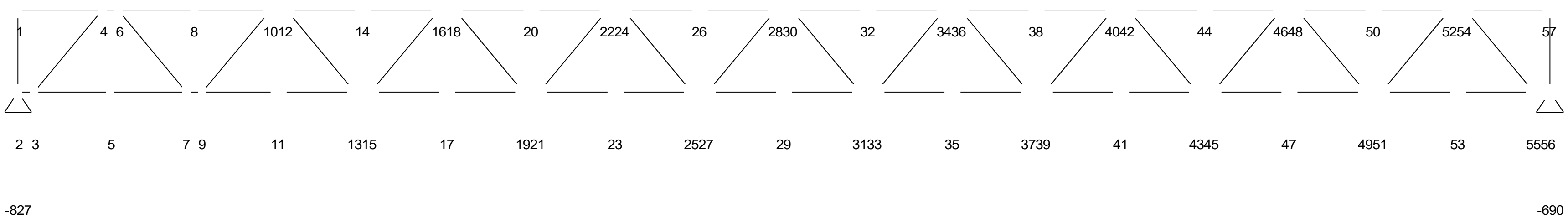
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC22 (1.2G + 1.6P) Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



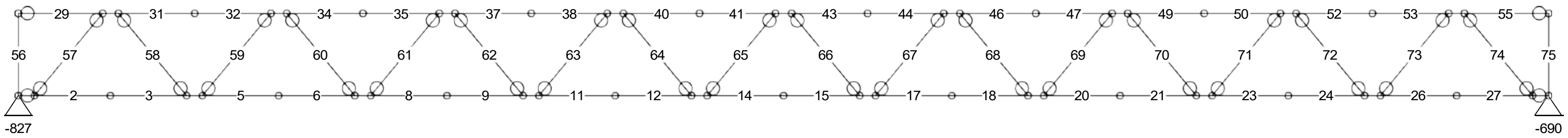
Node Diagram for Truss RJ2
Engineering Status = 52%

LC23 (1.2G + 1.6P)   Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Member Diagram for Truss RJ2
Engineering Status = 52%

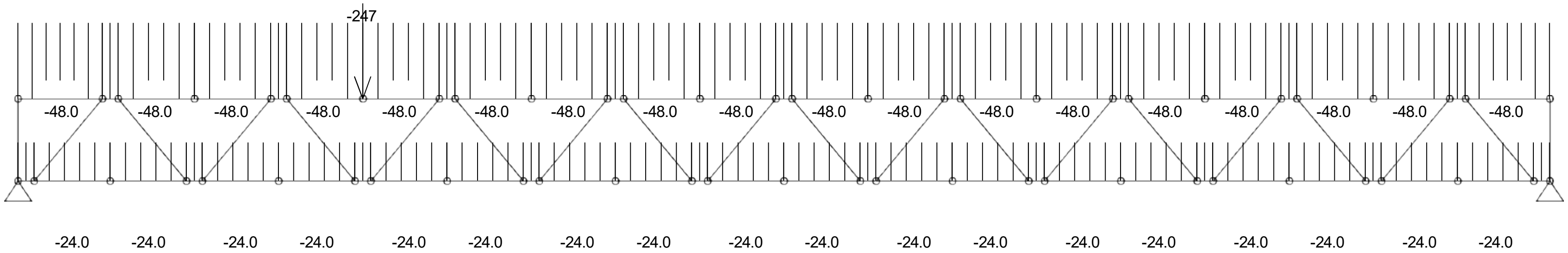
LC23 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



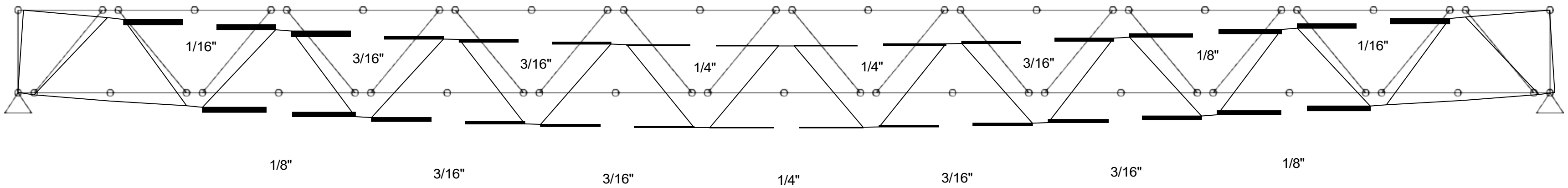
Load Diagram for Truss RJ2
Engineering Status = 52%

LC23 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Deflection Diagram for Truss RJ2
Engineering Status = 52%

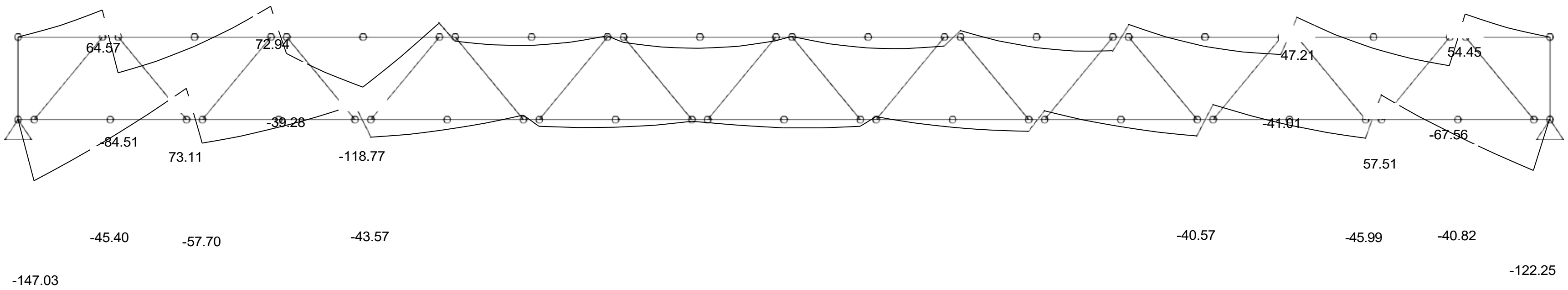
LC23 (1.2G + 1.6P)   Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

Engineering Status = 52%

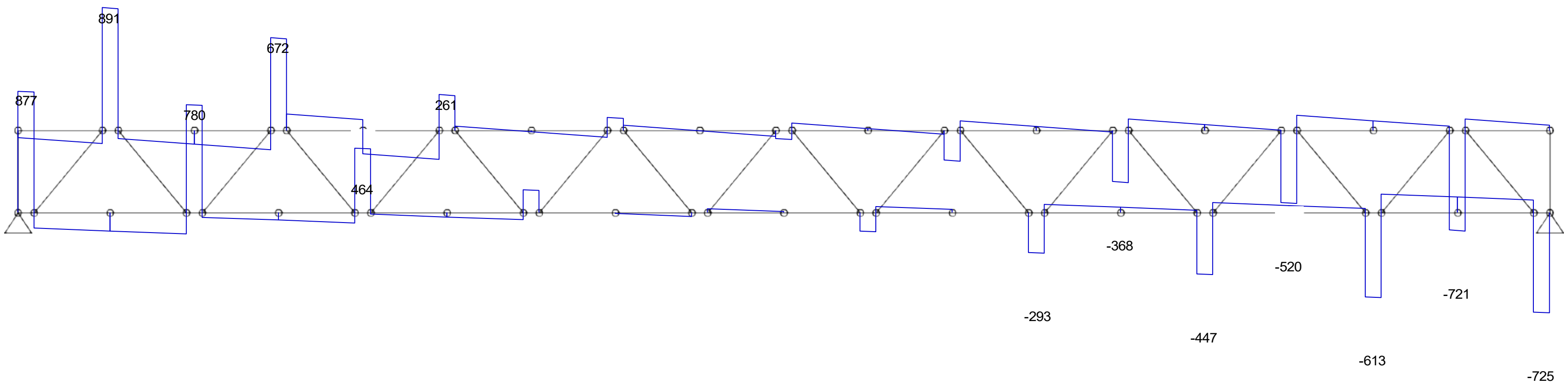
LC23 (1.2G + 1.6P)   Maximum for this Load Case = 42%

Company: Bauhu   Dwg Name: 242201_Exuma, Bahamas roof truss MS   Sheet 160 of 1   Project: Exuma, Bahamas   Job Number:242201

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

Engineering Status = 52%

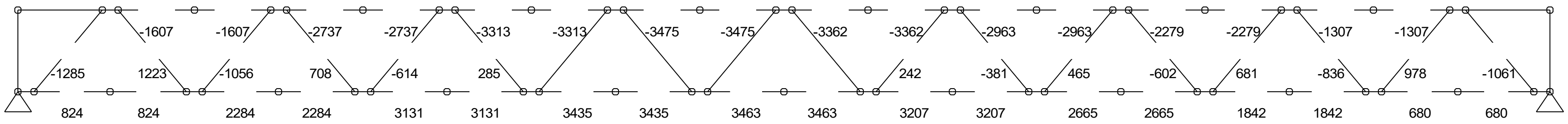
LC23 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name: FRAMECAD_FT_m
Roof Type: SHEET
Wind Speed: 200
Terrain Factor Kz 1.00
Importance Factor I 1.00
Topography Factor Kzt 1.00
Truss Spacing: 2'-0"
Top Chord Restraints: 3'-11 1/4"
Bottom Chord Restraints: 3'-11 1/4"
Design Code: AISI S100-20 LRFD

FRAMECAD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load: 20.0psf
Top Chord Dead Load: 20.0psf
Bottom Chord Live Load: 0.0psf
Bottom Chord Dead Load: 10.0psf
Bottom Chord Services Load: 0.0psf
Top Chord Ground Snow Load: 0.0psf
Suspended Ceilings
Loading Code: IBC 2018 LRFD



Axial Diagram for Truss RJ2
Engineering Status = 52%

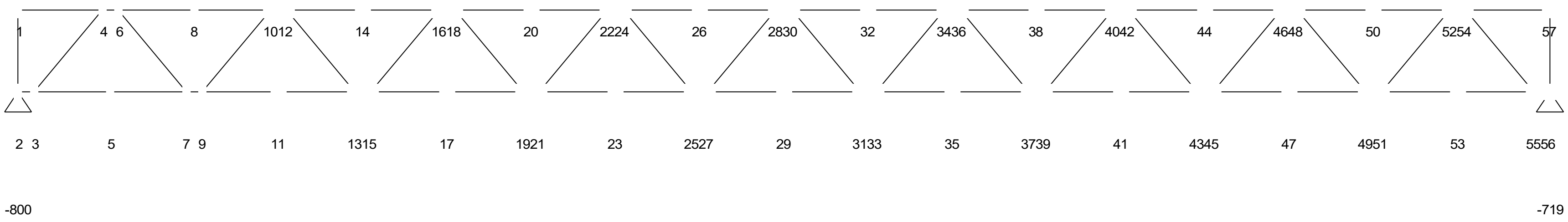
LC23 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



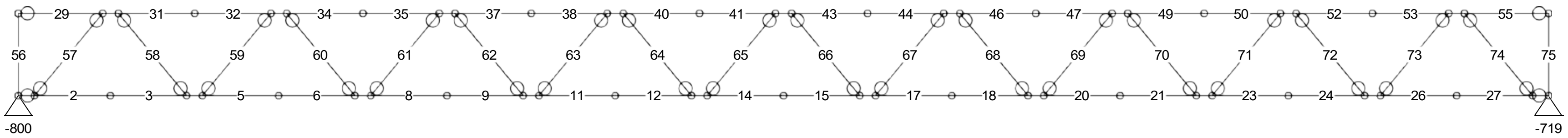
Node Diagram for Truss RJ2
Engineering Status = 52%

LC24 (1.2G + 1.6P)   Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



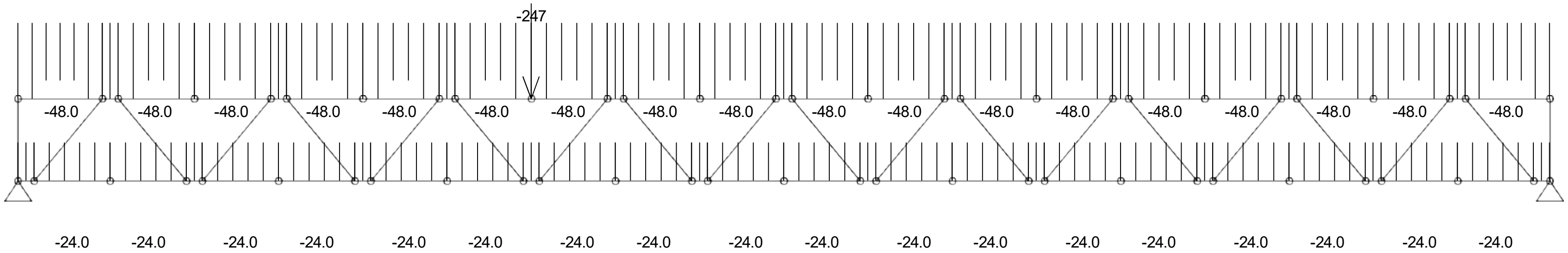
Member Diagram for Truss RJ2
Engineering Status = 52%

LC24 (1.2G + 1.6P) Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Load Diagram for Truss RJ2
Engineering Status = 52%

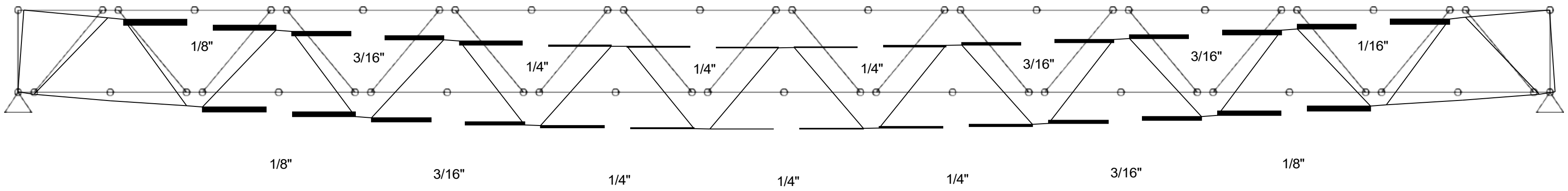
LC24 (1.2G + 1.6P) Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



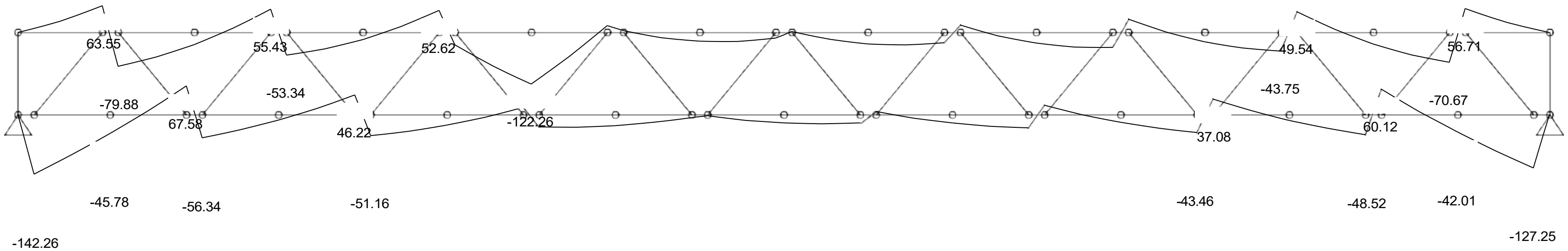
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC24 (1.2G + 1.6P)   Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



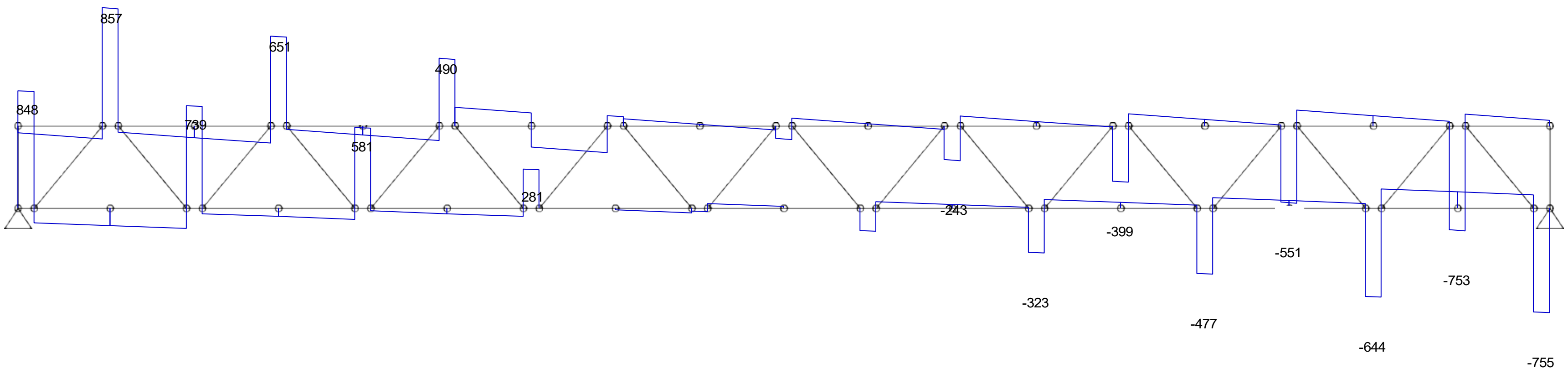
Bending Diagram for Truss RJ2
Engineering Status = 52%

LC24 (1.2G + 1.6P)   Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2
Engineering Status = 52%

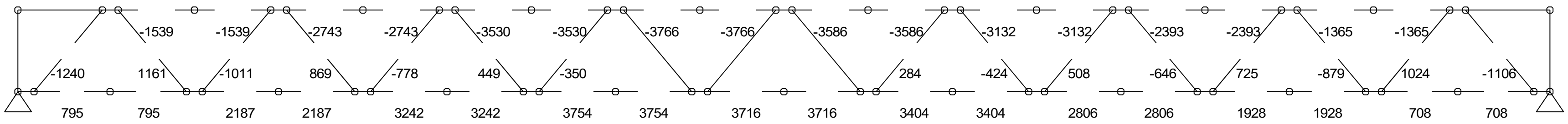
LC24 (1.2G + 1.6P)   Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

FRAMECAD



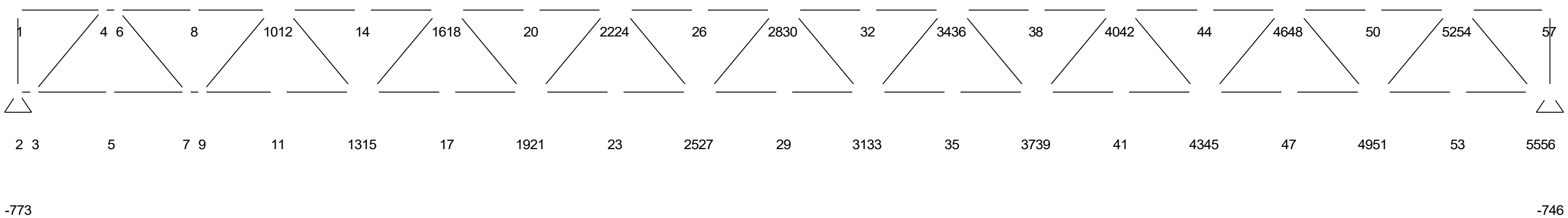
Axial Diagram for Truss RJ2
Engineering Status = 52%

~~LC24 (1.2G + 1.6P) Maximum for this Load Case = 48%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



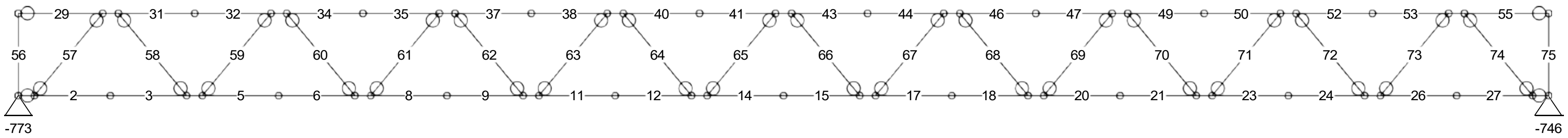
Node Diagram for Truss RJ2
Engineering Status = 52%

LC25 (1.2G + 1.6P)   Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



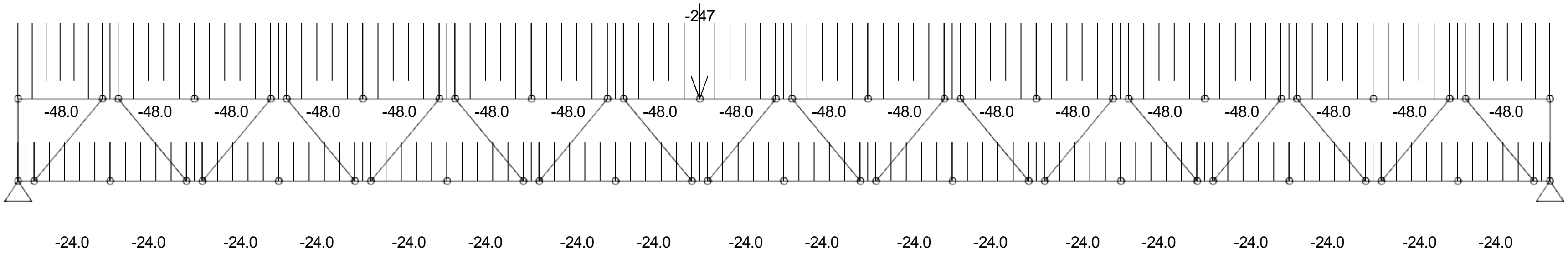
Member Diagram for Truss RJ2
Engineering Status = 52%

~~LC25 (1.2G + 1.6P) Maximum for this Load Case = 51%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



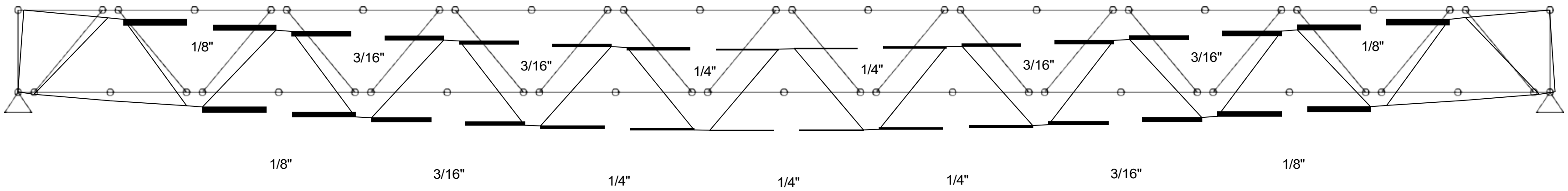
Load Diagram for Truss RJ2
Engineering Status = 52%

LC25 (1.2G + 1.6P)   Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



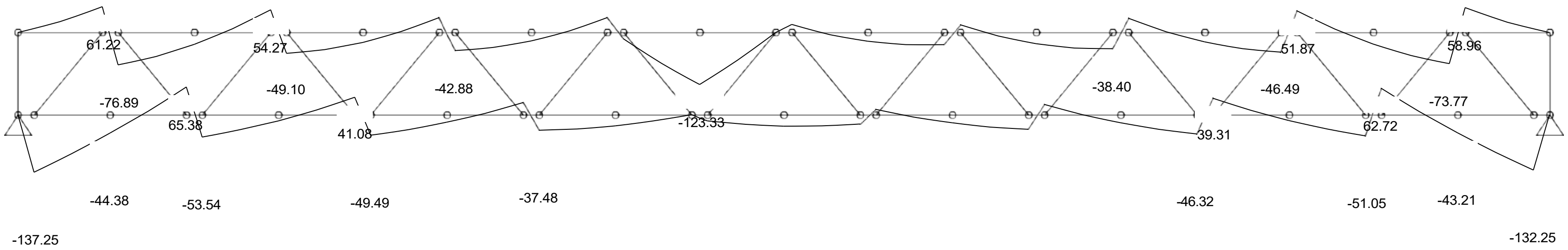
~~Deflection Diagram for Truss RJ2~~
Engineering Status = 52%

LC25 (1.2G + 1.6P)   Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



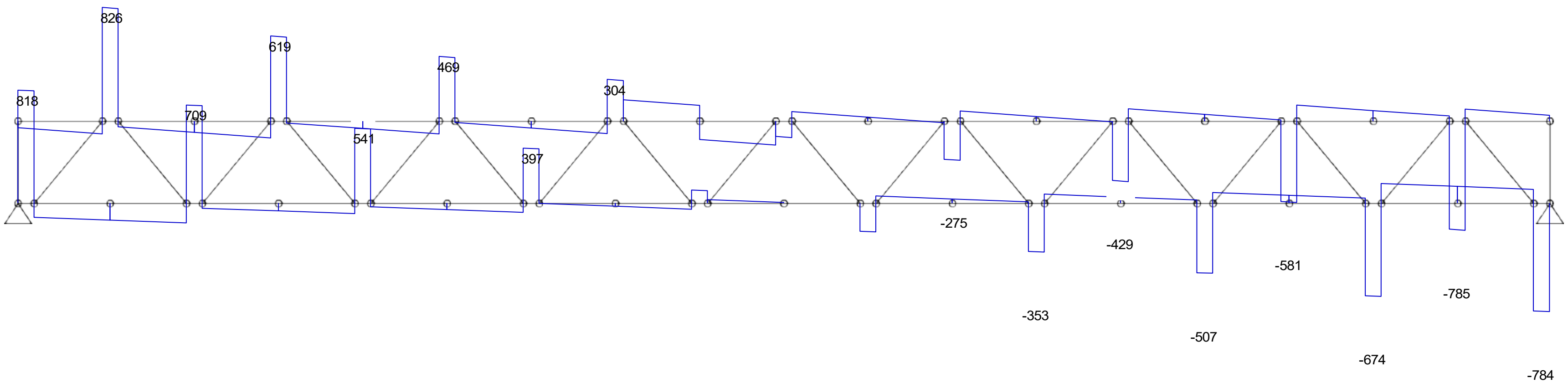
Bending Diagram for Truss RJ2
Engineering Status = 52%

LC25 (1.2G + 1.6P)   Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

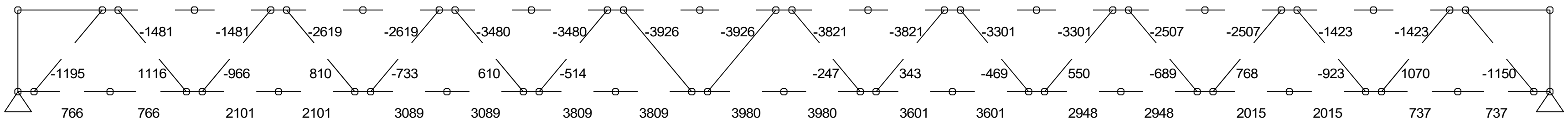
Engineering Status = 52%

LC25 (1.2G + 1.6P)   Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Axial Diagram for Truss RJ2
Engineering Status = 52%

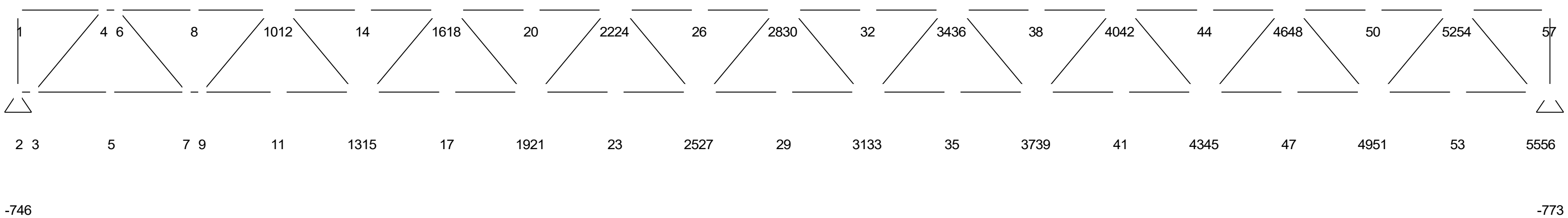
~~LC25 (1.2G + 1.6P) Maximum for this Load Case = 51%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



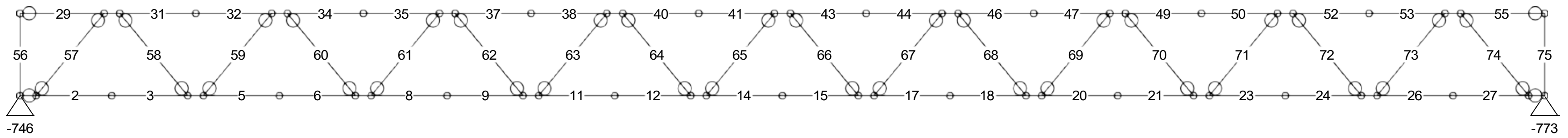
Node Diagram for Truss RJ2
Engineering Status = 52%

LC26 (1.2G + 1.6P)   Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



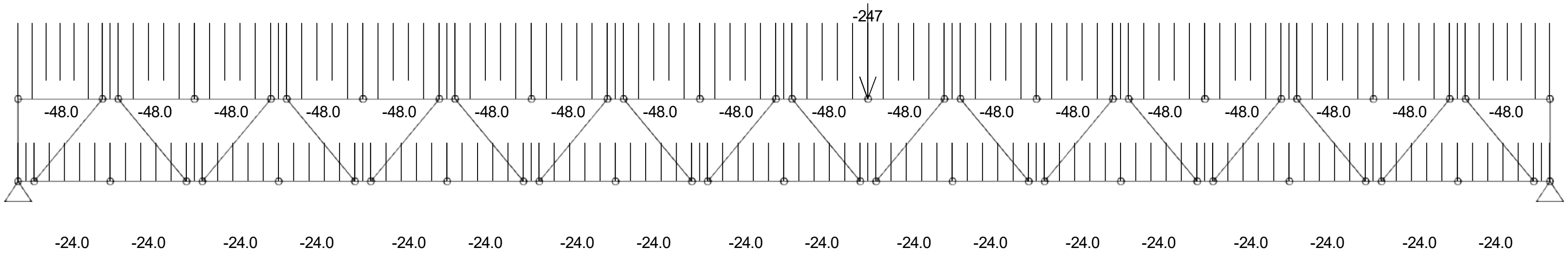
Member Diagram for Truss RJ2
Engineering Status = 52%

~~LC26 (1.2G + 1.6P) Maximum for this Load Case = 51%~~

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



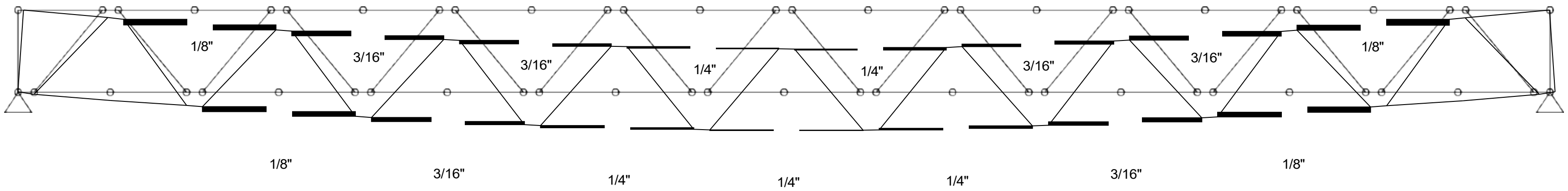
Load Diagram for Truss RJ2
Engineering Status = 52%

LC26 (1.2G + 1.6P) Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Deflection Diagram for Truss RJ2

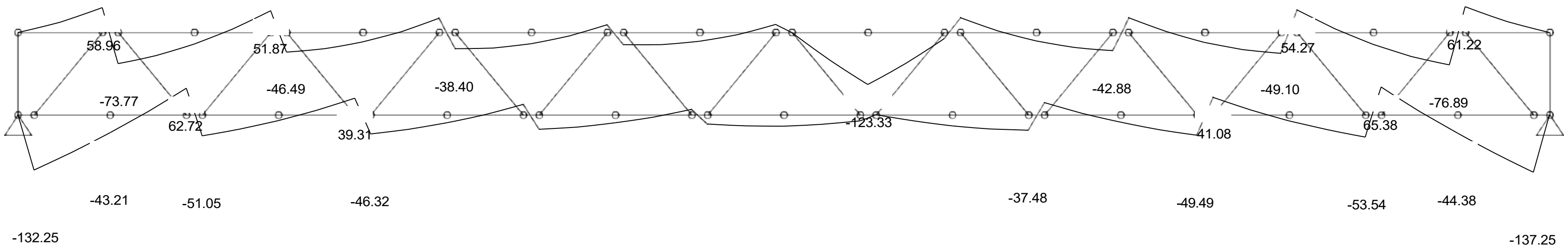
Engineering Status = 52%

LC26 (1.2G + 1.6P)   Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



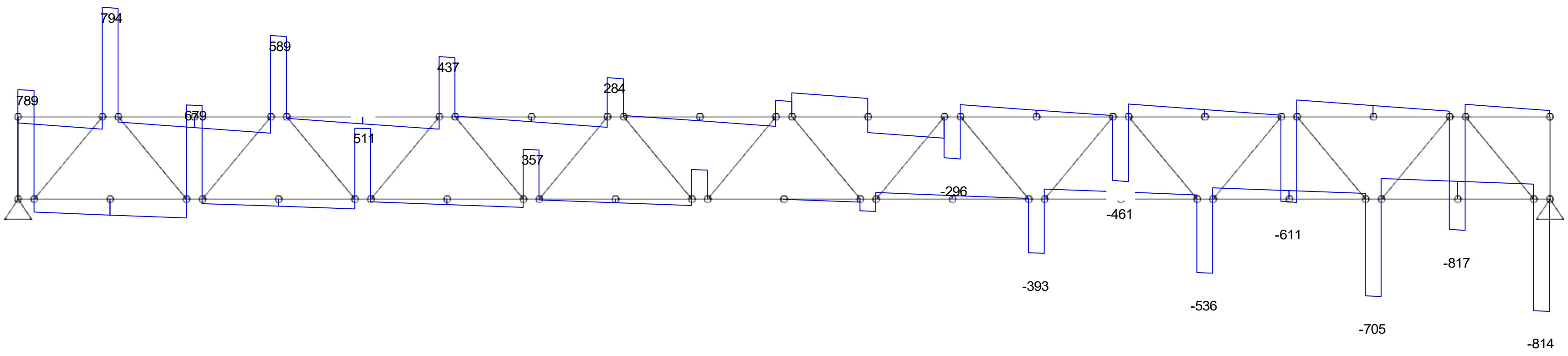
Bending Diagram for Truss RJ2
Engineering Status = 52%

LC26 (1.2G + 1.6P)   Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



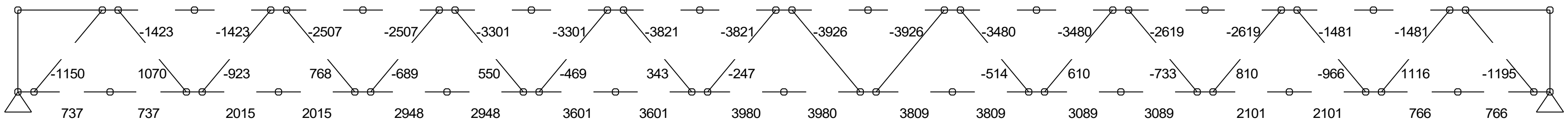
Shear Diagram for Truss RJ2
Engineering Status = 52%

LC26 (1.2G + 1.6P) Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



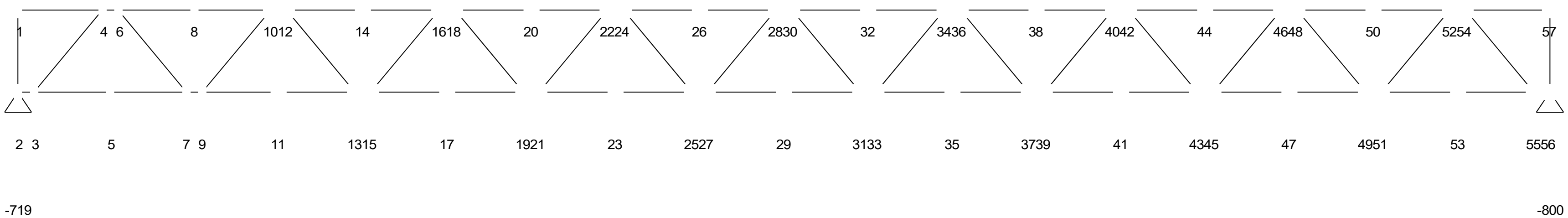
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC26 (1.2G + 1.6P) Maximum for this Load Case = 51%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



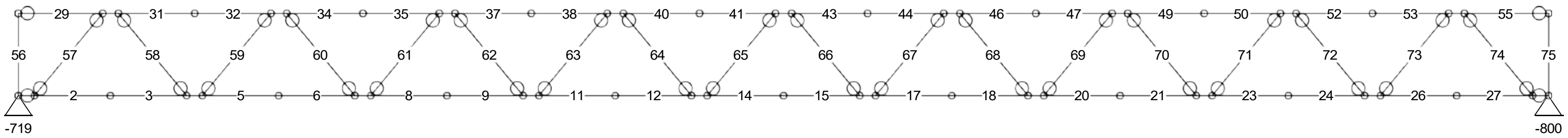
Node Diagram for Truss RJ2
Engineering Status = 52%

LC27 (1.2G + 1.6P) Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



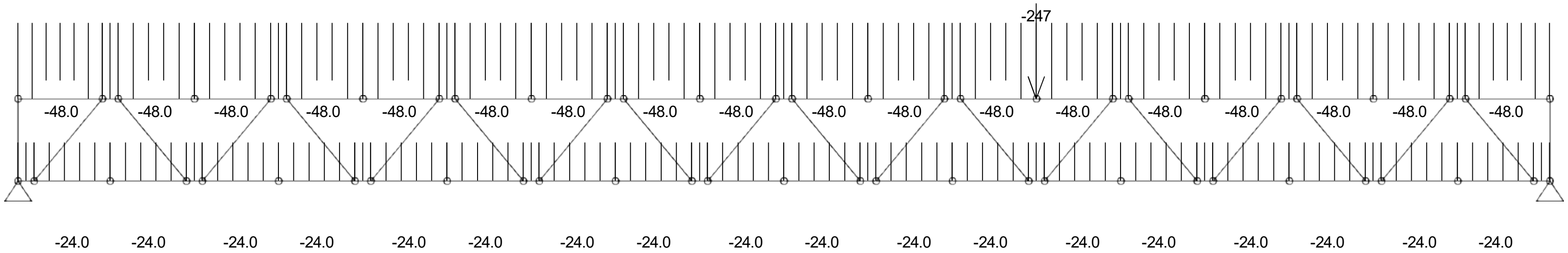
Member Diagram for Truss RJ2
Engineering Status = 52%

LC27 (1.2G + 1.6P) Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



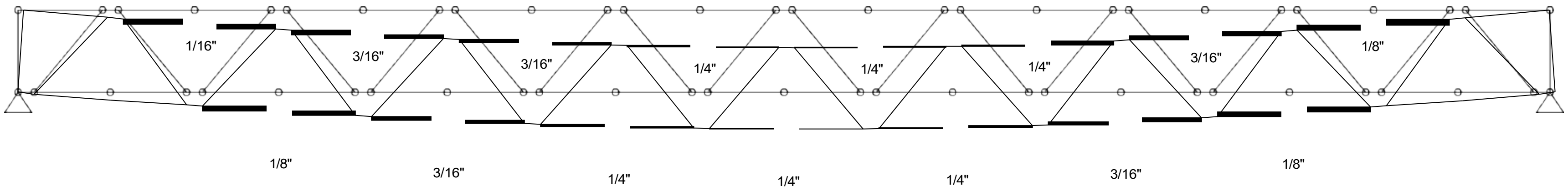
Load Diagram for Truss RJ2
Engineering Status = 52%

LC27 (1.2G + 1.6P) Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD

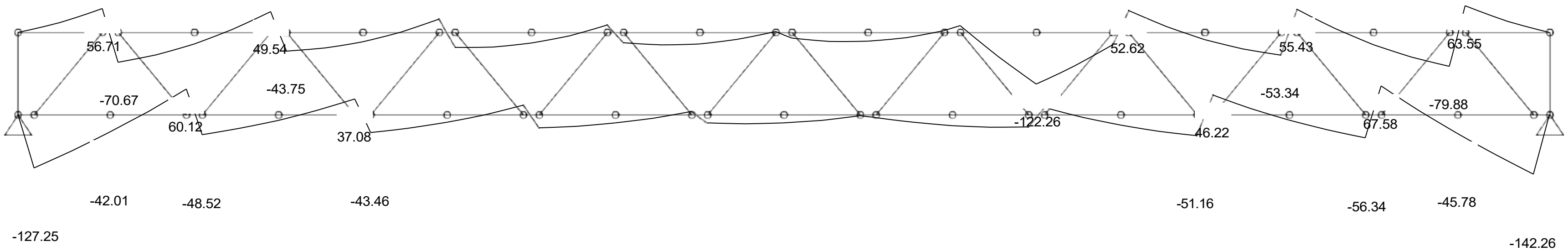


Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC27 (1.2G + 1.6P)   Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m	Top Chord Live Load:	20.0psf
Roof Type:	SHEET	Top Chord Dead Load:	20.0psf
Wind Speed:	200	Bottom Chord Live Load:	0.0psf
Terrain Factor Kz	1.00	Bottom Chord Dead Load:	10.0psf
Importance Factor I	1.00	Bottom Chord Services Load:	0.0psf
Topography Factor Kzt	1.00	Top Chord Ground Snow Load:	0.0psf
Truss Spacing:	2'-0"	Suspended Ceilings	
Top Chord Restraints:	3'-11 1/4"	Loading Code:	IBC 2018 LRFD
Bottom Chord Restraints:	3'-11 1/4"		
Design Code:	AISI S100-20 LRFD		

Only values over 36 lb-f are shown
Bending values shown are in lb-f



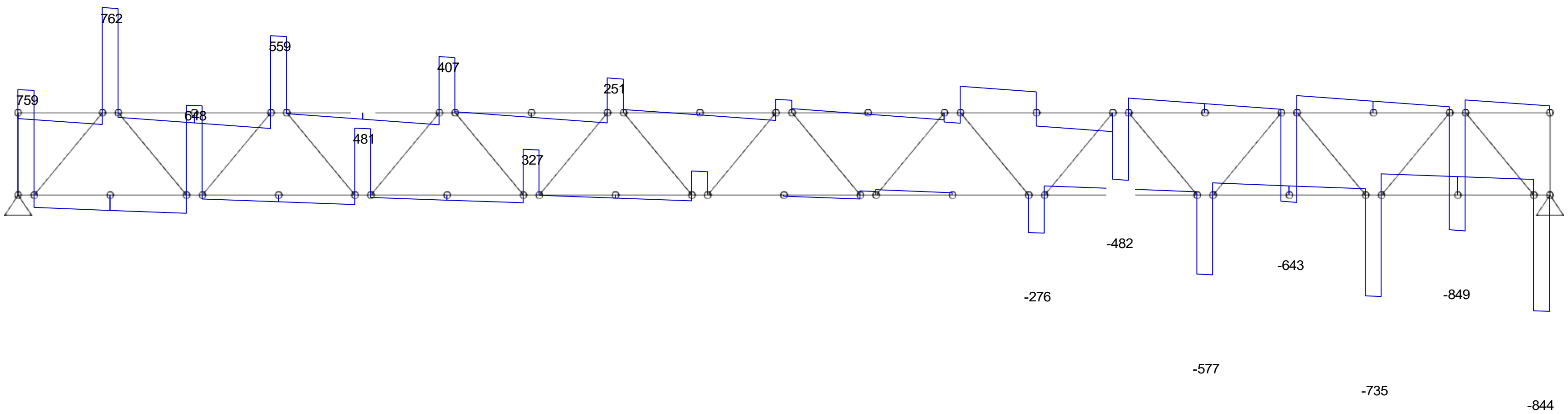
Bending Diagram for Truss RJ2
Engineering Status = 52%

LC27 (1.2G + 1.6P)   Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

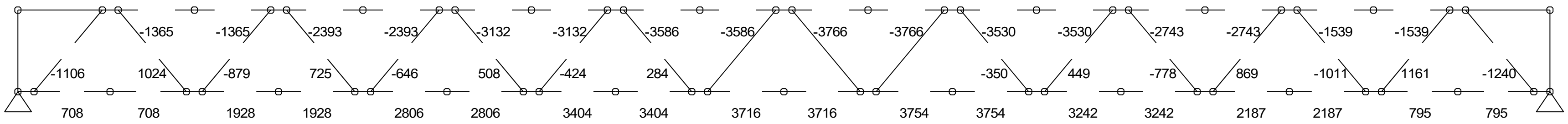
Engineering Status = 52%

LC27 (1.2G + 1.6P) Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



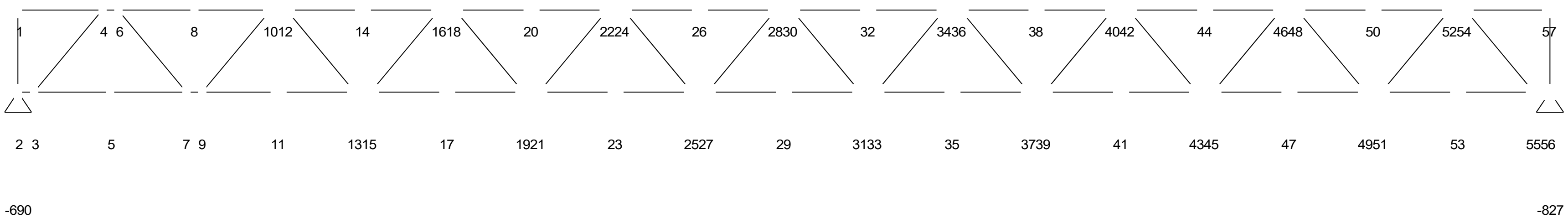
Axial Diagram for Truss RJ2
Engineering Status = 52%

LC27 (1.2G + 1.6P) Maximum for this Load Case = 48%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



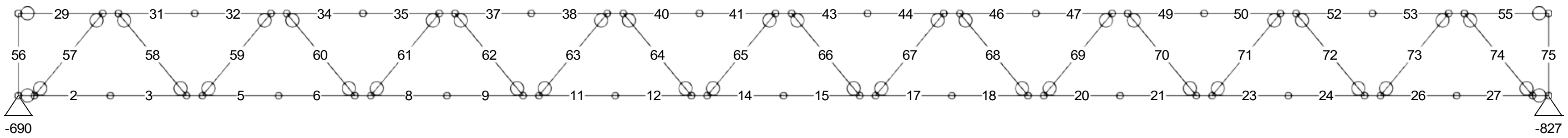
Node Diagram for Truss RJ2
Engineering Status = 52%

LC28 (1.2G + 1.6P)   Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Member Diagram for Truss RJ2
Engineering Status = 52%

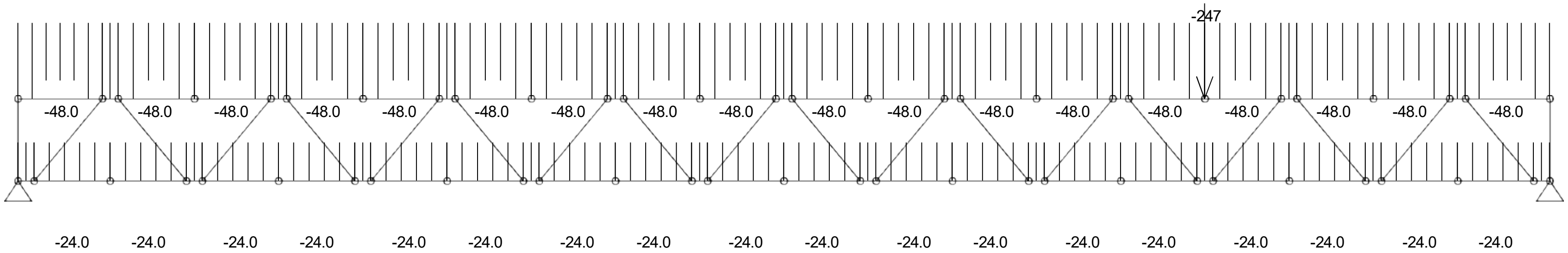
LC28 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



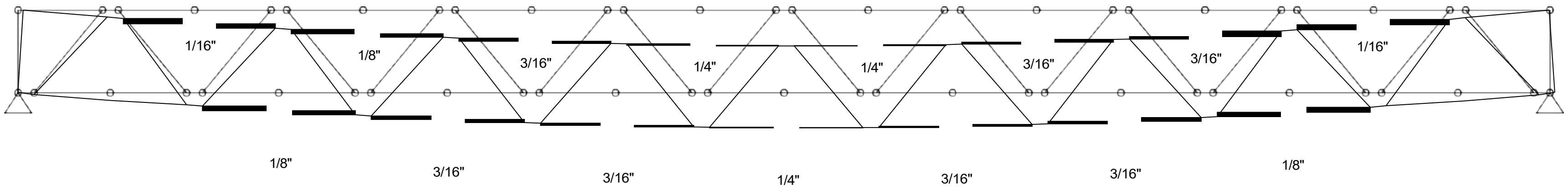
Load Diagram for Truss RJ2
Engineering Status = 52%

LC28 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Deflection Diagram for Truss RJ2
Engineering Status = 52%

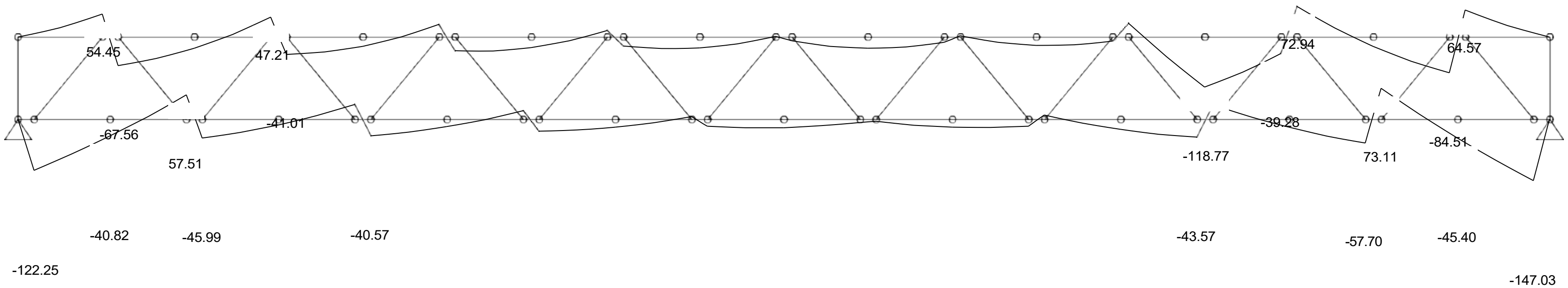
LC28 (1.2G + 1.6P)   Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Bending Diagram for Truss RJ2

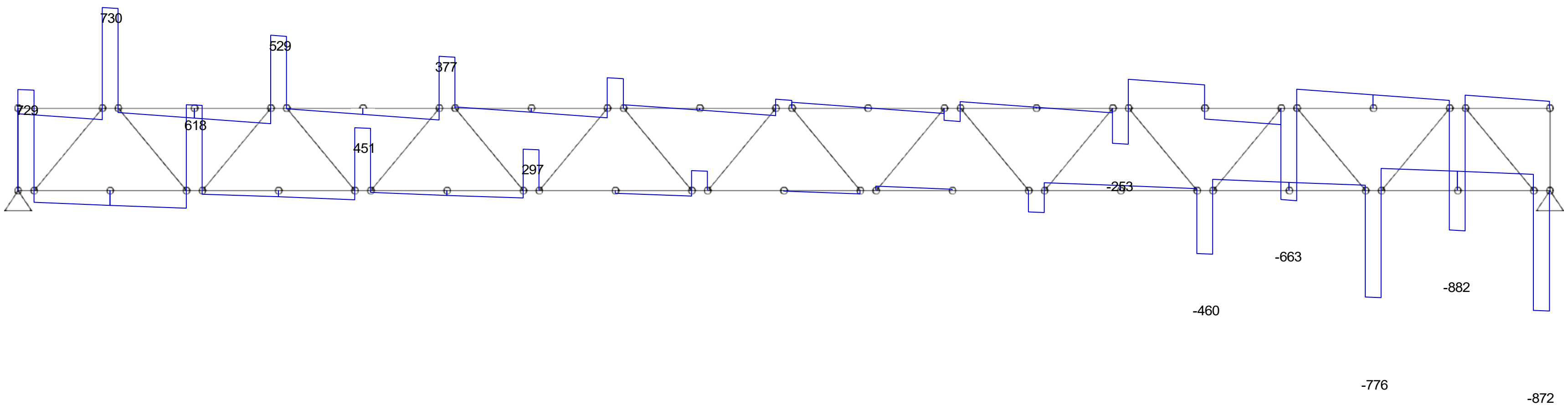
Engineering Status = 52%

LC28 (1.2G + 1.6P)   Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2

Engineering Status = 52%

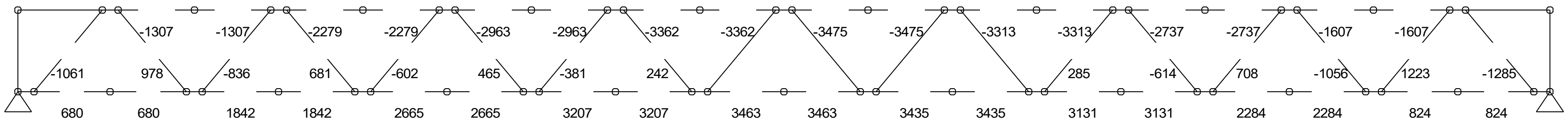
LC28 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Axial Diagram for Truss RJ2
Engineering Status = 52%

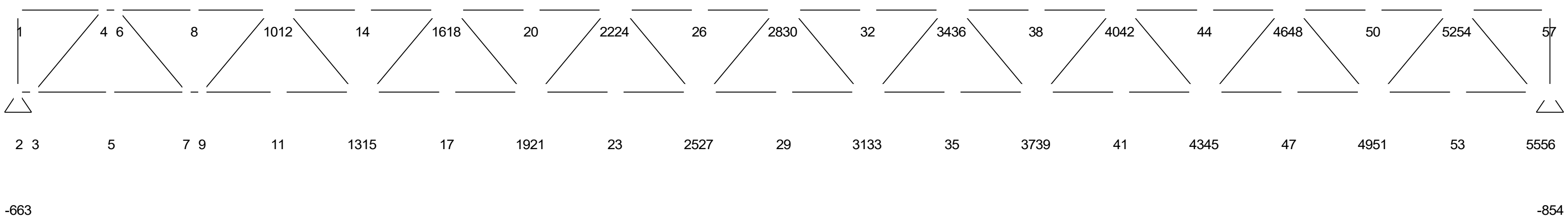
LC28 (1.2G + 1.6P) Maximum for this Load Case = 42%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



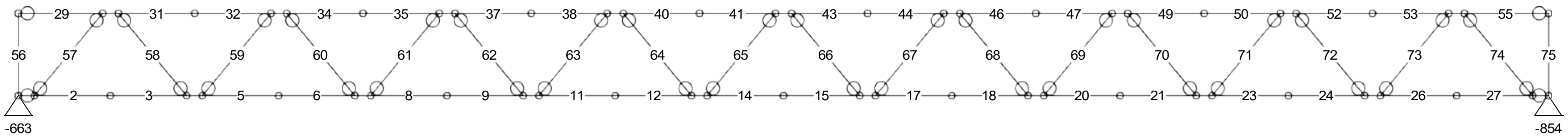
Node Diagram for Truss RJ2
Engineering Status = 52%

LC29 (1.2G + 1.6P) Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

Reactions shown are for this load case only
All force values are shown in lb
A negative value is load downwards
A positive value is load upwards

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



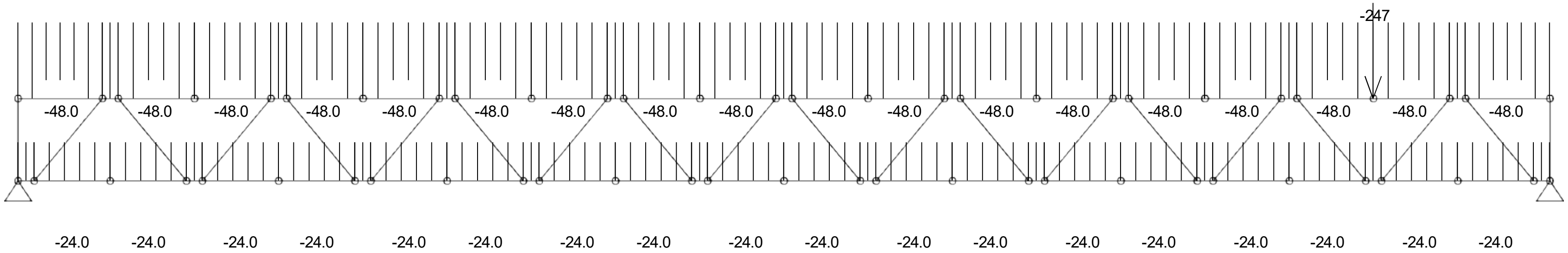
Member Diagram for Truss RJ2
Engineering Status = 52%

LC29 (1.2G + 1.6P) Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

A negative value represents a force downwards
A positive value represents a force upwards
All distributed load values are shown in plf
All point load values are shown in lb
Values left to right are not shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



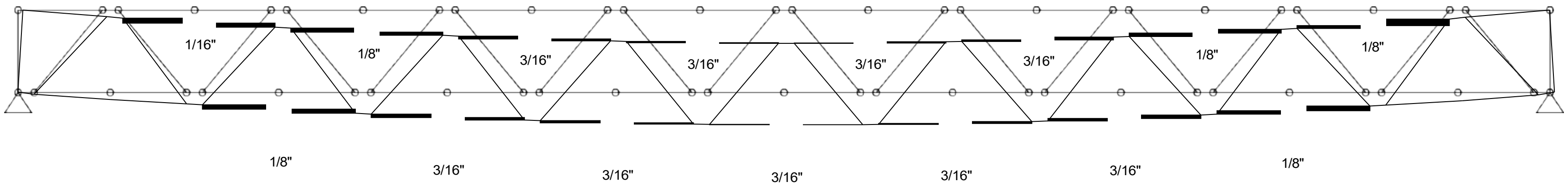
Load Diagram for Truss RJ2
Engineering Status = 52%

LC29 (1.2G + 1.6P) Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All values shown are in inches
Only values over 1/12 inches are shown
Deflection Scaled 20x

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



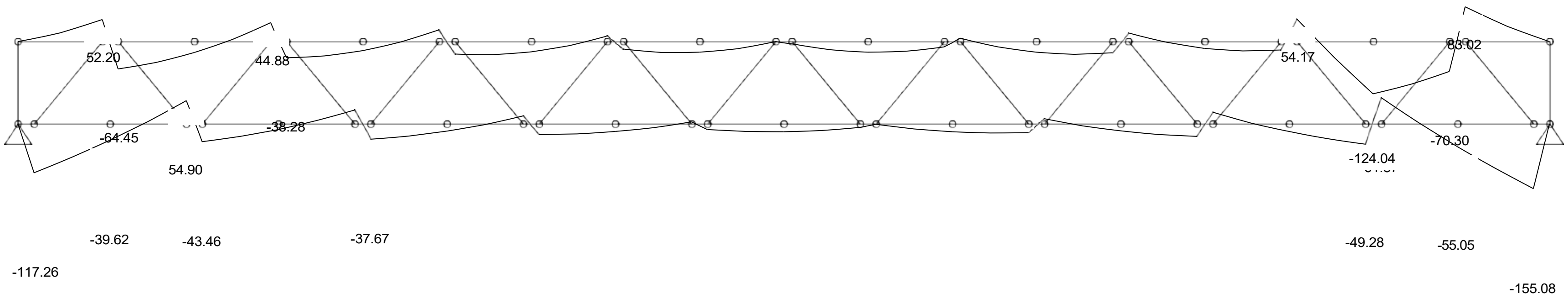
Deflection Diagram for Truss RJ2
Engineering Status = 52%

LC29 (1.2G + 1.6P)   Maximum for this Load Case = 40%

System Name: FRAMECAD_FT_m
Roof Type: SHEET
Wind Speed: 200
Terrain Factor Kz 1.00
Importance Factor I 1.00
Topography Factor Kzt 1.00
Truss Spacing: 2'-0"
Top Chord Restraints: 3'-11 1/4"
Bottom Chord Restraints: 3'-11 1/4"
Design Code: AISI S100-20 LRFD

Only values over 36 lb-f are shown
Bending values shown are in lb-f

Top Chord Live Load: 20.0psf
Top Chord Dead Load: 20.0psf
Bottom Chord Live Load: 0.0psf
Bottom Chord Dead Load: 10.0psf
Bottom Chord Services Load: 0.0psf
Top Chord Ground Snow Load: 0.0psf
Suspended Ceilings
Loading Code: IBC 2018 LRFD



Bending Diagram for Truss RJ2

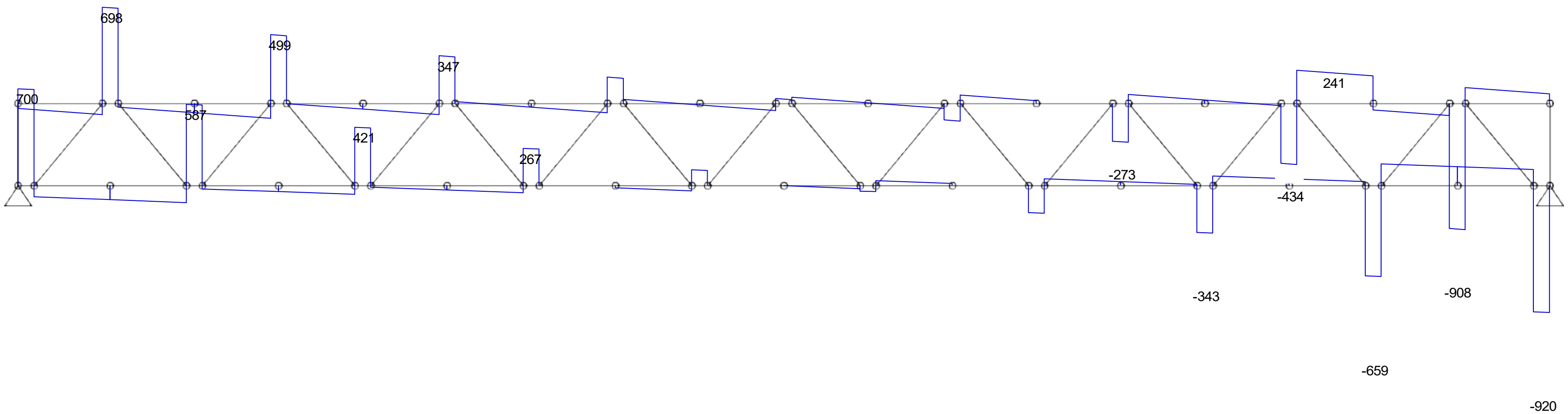
Engineering Status = 52%

LC29 (1.2G + 1.6P)   Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

All force values are shown in lb
Only values over 200 lb are shown

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Shear Diagram for Truss RJ2
Engineering Status = 52%

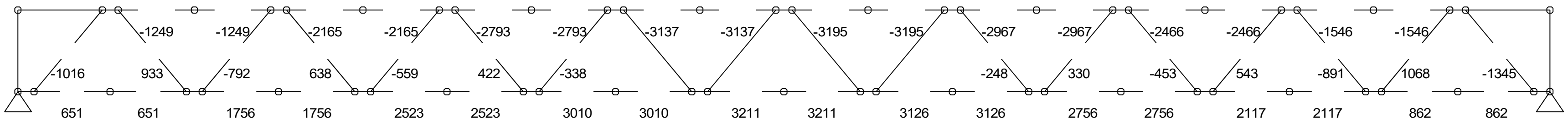
LC29 (1.2G + 1.6P)   Maximum for this Load Case = 40%

System Name:	FRAMECAD_FT_m
Roof Type:	SHEET
Wind Speed:	200
Terrain Factor Kz	1.00
Importance Factor I	1.00
Topography Factor Kzt	1.00
Truss Spacing:	2'-0"
Top Chord Restraints:	3'-11 1/4"
Bottom Chord Restraints:	3'-11 1/4"
Design Code:	AISI S100-20 LRFD

FRAMECAD

Only values over 200 lb are shown
A negative value represents compression
A positive value represents tension
Axial values shown are in lb

Top Chord Live Load:	20.0psf
Top Chord Dead Load:	20.0psf
Bottom Chord Live Load:	0.0psf
Bottom Chord Dead Load:	10.0psf
Bottom Chord Services Load:	0.0psf
Top Chord Ground Snow Load:	0.0psf
Suspended Ceilings	
Loading Code:	IBC 2018 LRFD



Axial Diagram for Truss RJ2
Engineering Status = 52%

LC29 (1.2G + 1.6P) Maximum for this Load Case = 40%

Appendix- 7A

ClarkWestern Building Systems

CW Tech Support: (888) 437-3244
clarkwestern.com

2007 North American Specification LRFD
DATE: 31-01-2024

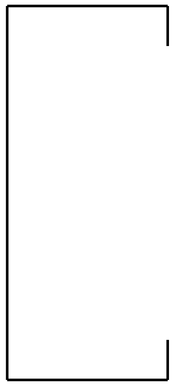
SECTION DESIGNATION: Single

Section Dimensions:

Web Height =	5.905 in
Top Flange =	1.969 in
Bottom Flange =	1.969 in
Stiffening Lip =	0.590 in
Inside Corner Radius =	0.0590 in
Punchout Dia. =	1.340 in
Design Thickness =	0.0590 in

Steel Properties:

Fy =	65.270 ksi
Fu =	71.100 ksi
Fya =	65.270 ksi



COMBINED AXIAL AND BENDING LOADS

INPUT PARAMETERS

Overall Wall Height = 10.21
Lateral Load = 124.62 psf
Load Factor for Lateral Load = 1.00
Lateral Load multiplied by 0.70 for deflection calculations
Studs Considered Fully Braced for Bending

K-phi (flexure) for Distortional Buckling = 0.00 lb*in/in
K-phi (axial) for Distortional Buckling = 0.00 lb*in/in

MAXIMUM FACTORED AXIAL LOADS (lb)

		SPACING		Maximum
BRACING	12 in	16 in	24 in	KL/r
NONE	2172	1698	770	171
MID Pt	5936	4583	2032	86
THIRD Pt	7548	5798	2551	57
SHEATH 2 SIDES	N/A	N/A	N/A	53
DEFLECTION	L/558	L/419	L/279	

Note: Axial loads for sheathing braced design are based on the North American Standard for Cold-Formed Steel Framing - Wall Stud Design, 2007 Edition with 1/2 inch gypsum sheathing and No. 6 fasteners max 12 inches on center

Appendix- 7B

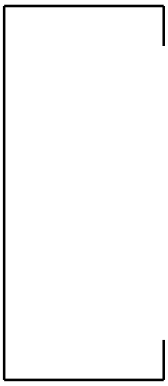
ClarkWestern Building Systems
CW Tech Support: (888) 437-3244
clarkwestern.com

2007 North American Specification LRFD
DATE: 31-01-2024

SECTION DESIGNATION: Single

Section Dimensions:

Web Height =	5.905 in
Top Flange =	1.969 in
Bottom Flange =	1.969 in
Stiffening Lip =	0.429 in
Inside Corner Radius =	0.0590 in
Punchout Dia. =	1.340 in
Design Thickness =	0.0472 in



Steel Properties:

Fy =	65.270 ksi
Fu =	71.100 ksi
Fya =	65.270 ksi

COMBINED AXIAL AND BENDING LOADS

INPUT PARAMETERS

Overall Wall Height = 10.21
Lateral Load = 5.0 psf
Load Factor for Lateral Load = 1.00
Lateral Load multiplied by 0.70 for deflection calculations
Studs Considered Fully Braced for Bending

K-phi (flexure) for Distortional Buckling = 0.00 lb*in/in
K-phi (axial) for Distortional Buckling = 0.00 lb*in/in

MAXIMUM FACTORED AXIAL LOADS (lb)

		SPACING		Maximum
BRACING	12 in	16 in	24 in	KL/r
NONE	2404	2384	2342	178
MID Pt	6413	6352	6232	89
THIRD Pt	8031	7952	7796	59
SHEATH 2 SIDES	N/A	N/A	N/A	53
DEFLECTION	L/10623	L/7967	L/5311	

Note: Axial loads for sheathing braced design are based on the North American Standard for Cold-Formed Steel Framing - Wall Stud Design, 2007 Edition with 1/2 inch gypsum sheathing and No. 6 fasteners max 12 inches on center

Appendix- 8

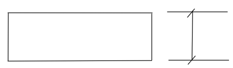
Project: Exuma, Bahamas	242201

Lateral Calculation

17-04-24

PART I: Wind Analysis

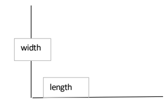
Inputs



3.51 m
11.51 ft

Shear wall tributary height

1.75 m
5.76 ft



Wind Load Data:

E	210000.000 N/mm^2	4385941.197 kip/ft^2

Wind Load Factor = 1.00 | 4.980 kN/m^2 | 0.104 kip/ft^2

Roof To Roof Floor	Tributary width (m)	Tributary width (ft)	Tributary height (m)	Area (m2)	Wind Pressure (kN/m^2)	V (kN)	Factored V(kN)	Remark	Factored V (kips)	Force Distribution factor HRS	Force Distribution factor Brace	Actual Force HRS (kips)	Actual Force Brace (kips)	Actual Force Brace (kN)
Along Length														
Along Grid A	1.7	5.5	1.75	2.94	4.98	14.64	14.6	HRS Frame	3.3	1.00	0.00	3.29	0.00	0.00
Along Grid Bet A&B	3.2	10.5	1.75	5.61	4.98	27.96	28.0	Cross Flat Strap	6.3	0.00	1.00	0.00	6.29	27.96
Along Grid B adjacent	3.8	12.3	1.75	6.59	4.98	32.81	32.8	Cross Flat Strap+HRS Frame	7.4	0.20	0.80	1.48	5.90	26.25
Along Grid Bet B&C	3.2	10.5	1.75	5.60	4.98	27.89	27.9	Cross Flat Strap	6.3	0.00	1.00	0.00	6.27	27.89
Along Grid C	1.0	3.2	1.75	1.69	4.98	8.40	8.4	HRS Frame	1.9	1.00	0.00	1.89	0.00	0.00
Along Width														
Along Grid 1	2.02	6.6	1.75	3.54	4.98	17.65	17.6	HRS Frame	4.0	1.00	0.00	3.97	0.00	0.00
Along Grid 2	4.5	14.7	1.75	7.84	4.98	39.06	39.1	Cross Flat Strap+HRS Frame	8.8	0.30	0.70	2.63	6.15	27.34
Along Grid 3	3.5	11.3	1.75	6.05	4.98	30.14	30.1	HRS Frame	6.8	1.00	0.00	6.78	0.00	0.00

Project: Exuma,	242201

Bracing Calculation

Bracing Capacity						
Strap Capacities						
Strap	Width (mm)		Thickness (mm	Tension capacity(kN)	Area (mm2)	Grade
165x1.2	150		1.2	81.0	180	450
Strap	Width (in)		Thickness (in)	Tension capacity(kip)	Area (in2)	Grade
165x1.2	5.905515		0.04724412	18.2096	0.279000859	65.2671

150x1.2

Ht	WALL	
	3.14 m	

Roof To Roof Floor	Panel Length (m)	Panel Length (ft)	Wind Force (KN)	Wind Force (Kip)	Governing Force (KN)	Shear panel No.	Tension Strap in panel.	Force in Each Panel (KN)	Single Bracing Force(KN)	Single Bracing Capacity (KN)	Single Bracing Capacity (Kip)	Straps	Total Bracing Capacity (KN)	Strap No.	Bracing Wall Length (m)	Type of Bracing	Provided Bracing Angle	Check	Remark
Along Length																			
Along Grid A																			HRS Frame
Along Grid Bet A&B	2.30	7.5	27.960	6.29	**27.960**	2	2	14.0	23.6	48	10.8	150x1.2	96	2	2.3	X- Bracing	53.8	OK	Cross Flat Strap
Along Grid B adjacent	2.00	6.6	26.247	5.90	**26.250**	2	2	13.1	24.4	44	9.8	150x1.2	87	2	2.0	X- Bracing	57.5	OK	Cross Flat Strap+HRS Frame
Along Grid Bet B&C	2.30	7.5	27.885	6.27	**27.890**	2	2	13.9	23.6	48	10.8	150x1.2	96	2	2.3	X- Bracing	53.8	OK	Cross Flat Strap
Along Grid C																			HRS Frame
Along Width																			
Along Grid 1																			HRS Frame
Along Grid 2	1.80	5.9	27.339	6.15	**27.340**	3	3	9.1	18.3	40	9.1	150x1.2	121	3	1.8	X- Bracing	60.2	OK	Cross Flat Strap+HRS Frame
Along Grid 3																			HRS Frame

Project: Exuma,	242201

Hold-Down Calculation

Ht	WALL	0	0
	3.14 m	0.00 m	0.00 m

Roof To Roof Floor	Panel Length (m)	Panel Length (ft)	Wind Force (kN)	Wind Force (Kip)	Governing Force (kN)	Shear panel No.	Strap No.	Force in Each Panel (kN))	Angle of inclination	Uplift Force (kN)	Uplift Force (kip)
Along Length											
Along Grid A											
Along Grid Bet A&B	2.30	7.5	28.0	6.3	28.0	2	2	14.0	54	19.09	4.29
Along Grid B adjacent	2.00	6.6	26.2	5.9	26.3	2	2	13.1	58	20.61	4.63
Along Grid Bet B&C	2.30	7.5	27.9	6.3	27.9	2	2	13.9	54	19.04	4.28
Along Grid C											
Along Width											
Along Grid 1											
Along Grid 2	1.80	5.9	27.3	6.1	27.3	3	3	9.1	60	15.90	3.57
Along Grid 3											

Roof To Roof Floor	Provide Holddown	Uplift Capacity of Holddown (kN)	Uplift Capacity of Holddown (lb)	Remark	Provide Bolt/Anchor/Screw	Factored Boundary Stud force (kN)	Factored Boundary Stud force (lb)	Boundary Stud	Boundary Stud Capacity (lb)	Boundary Stud Check	Shear force (kN)	Shear force (kip)	Uplift Force (kip)
Along Length													
Along Grid A													
Along Grid Bet A&B	HTT4	21.2	4,770.0	OK	M16 Anchor	19.09	4,290.64	Stud 150x1.2	6,232.0	OK	6.99	1.57	4.29
Along Grid B adjacent	HTT4	21.2	4,770.0	OK	M16 Anchor	20.61	4,632.47	Stud 150x1.2	6,232.0	OK	6.56	1.48	4.63
Along Grid Bet B&C	HTT4	21.2	4,770.0	OK	M16 Anchor	19.04	4,279.90	Stud 150x1.2	6,232.0	OK	6.97	1.57	4.28
Along Grid C													
Along Width													
Along Grid 1													
Along Grid 2	HTT4	21.2	4,770.0	OK	M16 Anchor	15.90	3,573.95	Stud 150x1.2	6,232.0	OK	4.56	1.02	3.57
Along Grid 3													

Connection Design:

Holddown to connection

Maximum reaction	KN	lb	Screw Capacity		
Force in each panel	13.98	3142.83	Shear	3.145 kN	707.000 lb

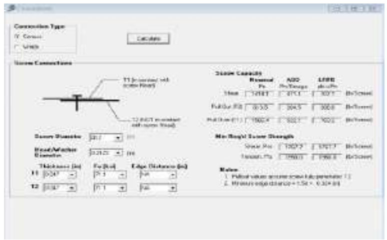
Holdown to Boundary stud connection:

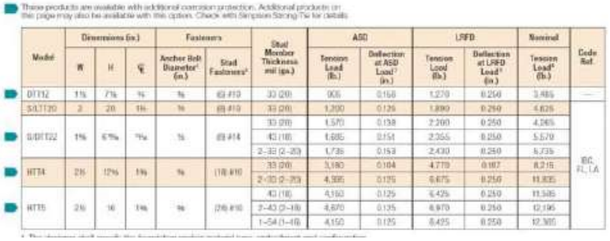
Provide (18) 10g screw 4.8mm gauge to connect hold-down with boundary stud.

Bracing to wall connection:

Using 12g screw 5.5 dia		
No of screw	4.445	

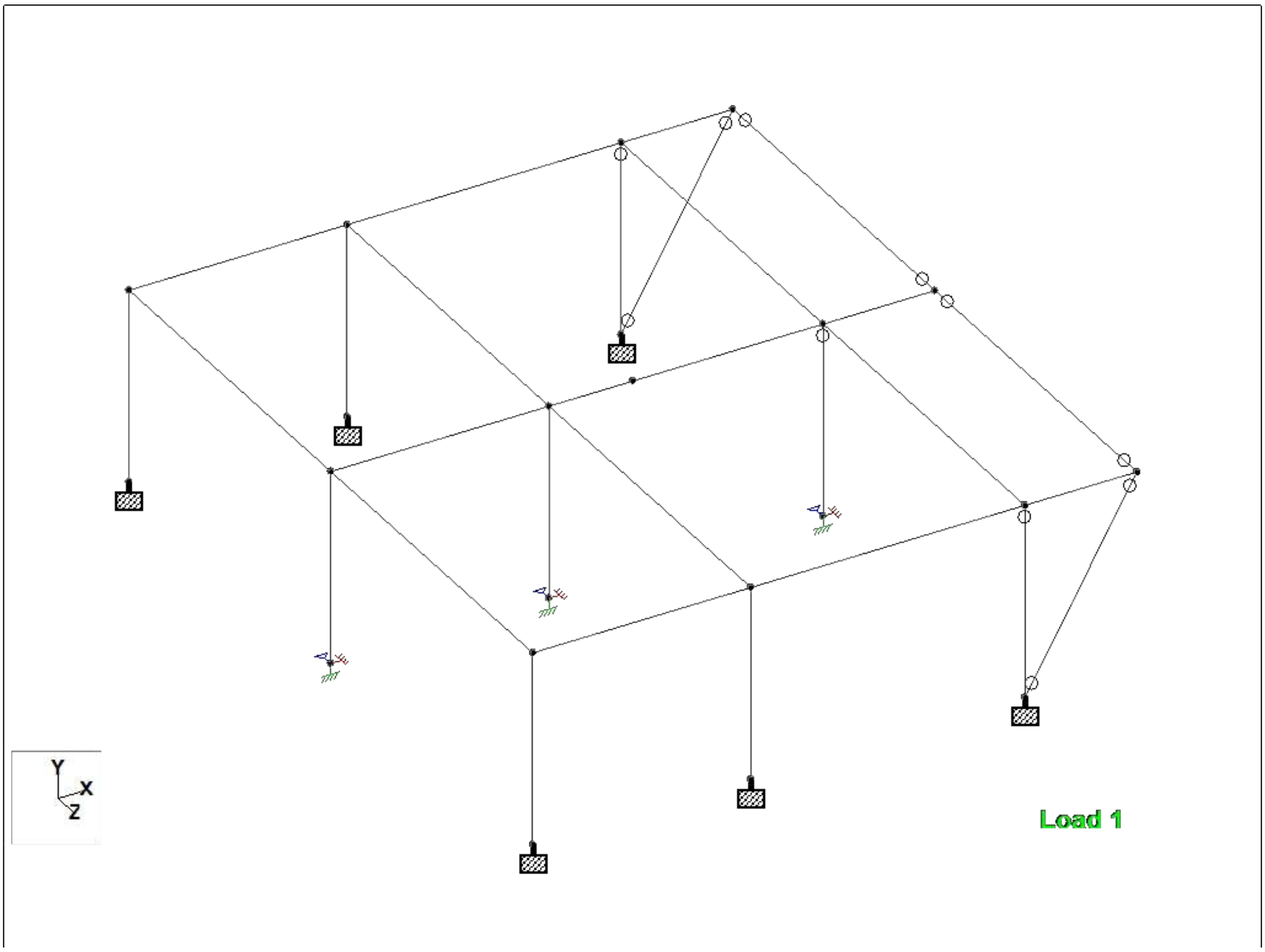
Provide (6) 12g screw 5.5 mm gauge to connect Bracing with boundary stud.





Appendix- 8A

Bentley Software licensed to International Design & Engineering Connected User: Ganesh Maske	Job No. 242201	Sheet No. 1	Rev
Job Title	Part	Ref	
Client Example	By SK	Date 2/6/2024	Chd MS
File Exuma, Bahamas_060224.STD	Date Time 12-Feb-2024 13:59		



Whole Structure

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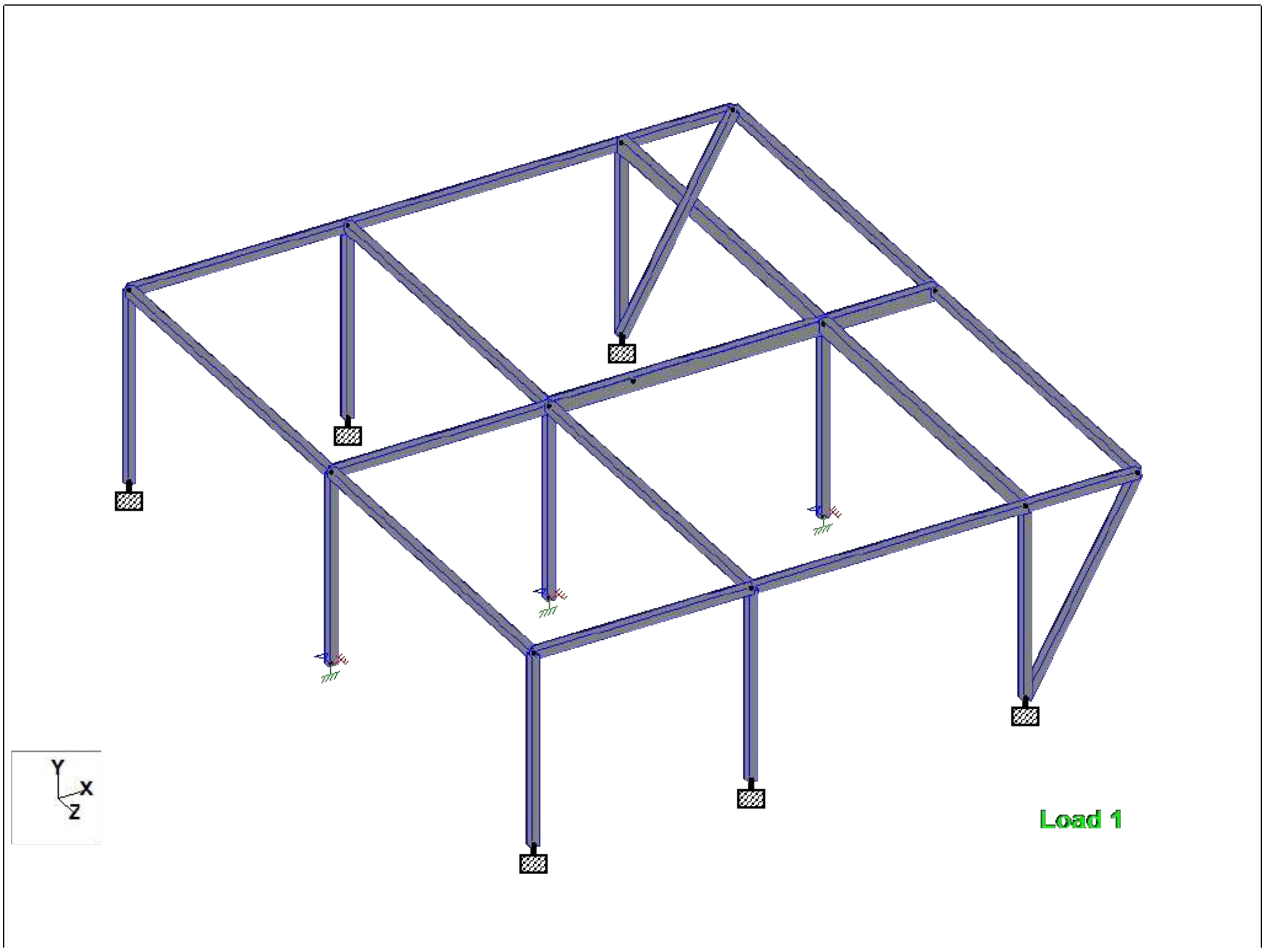
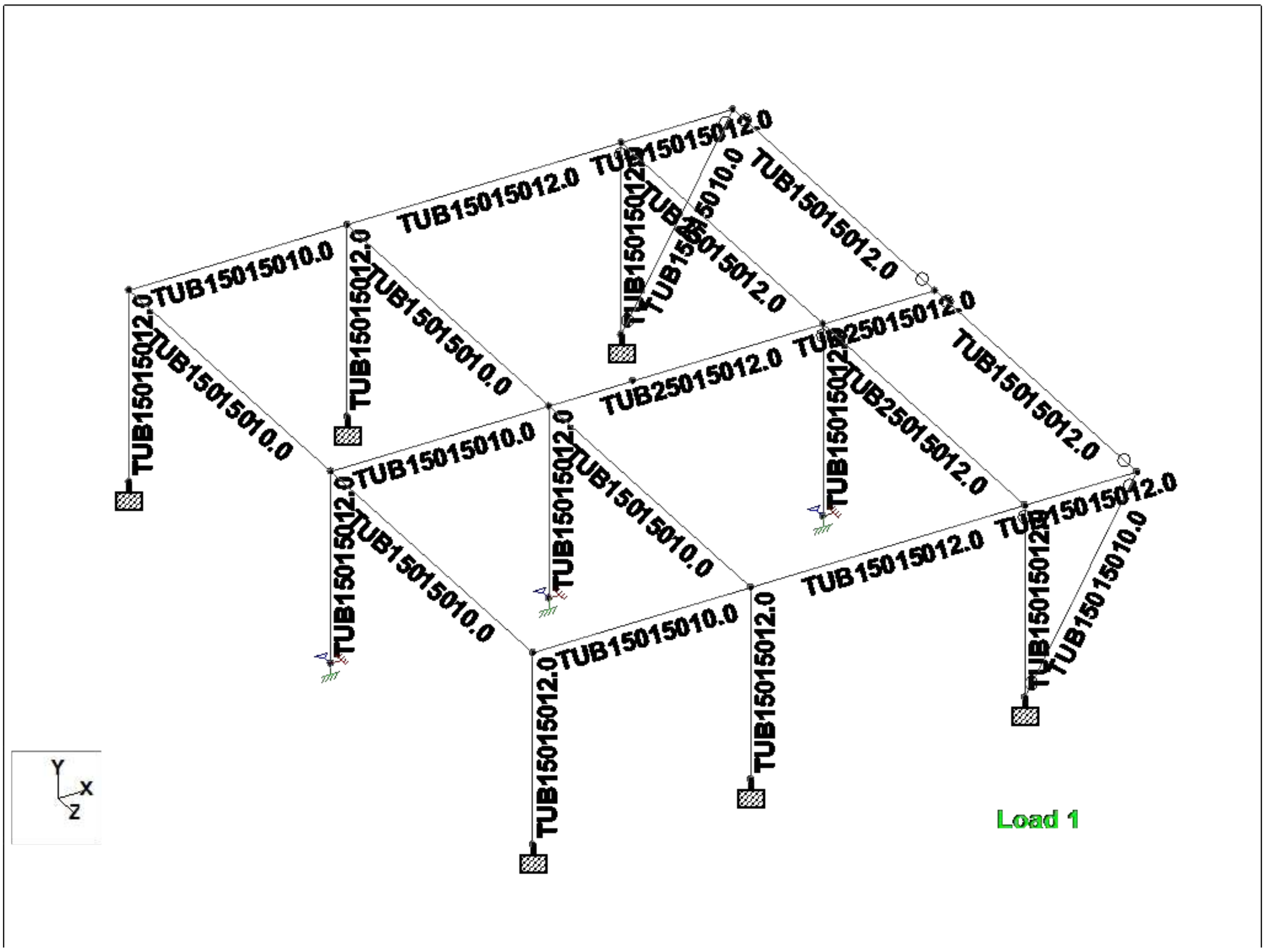
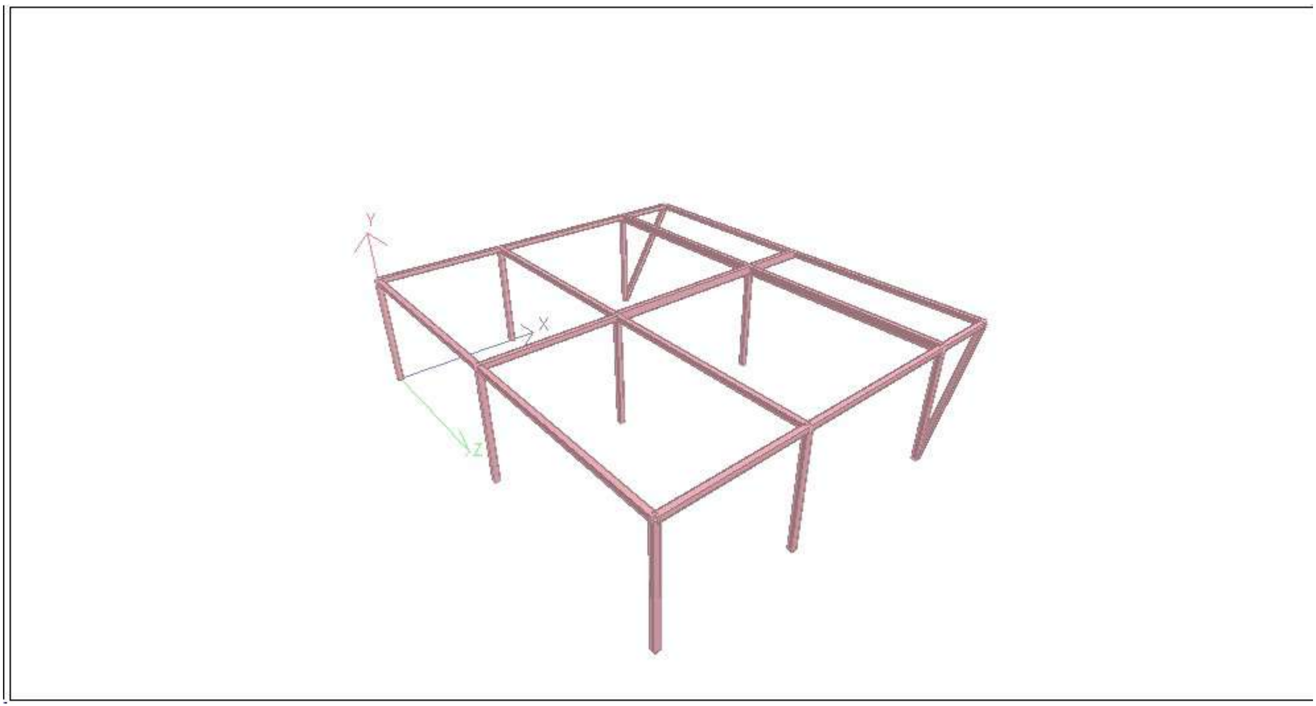
Job Title	Part	Ref	
Client Example	By SK	Date 2/6/2024	Chd MS
File Exuma, Bahamas_060224.STD	Date Time 12-Feb-2024 13:59		



Section Outline



Section Used



3D Model

Sections

Prop	Name	Area (in2)	I_{yy} (in4)	I_{zz} (in4)	J (in4)	Material	Source
1	TUB15015010.0	8.510	42.597	42.597	68.039	STEEL	Legacy
2	TUB15015012.0	10.029	48.603	48.603	78.610	STEEL	Legacy
3	TUB25015012.0	13.749	76.112	171.876	170.290	STEEL	Legacy

Materials

Mat	Name	E (kip/in2)	ν	Density (kip/in3)	α (/°F)
1	STEEL	29,000.000	0.300	0.000	0.000

Supports

Node	X (kip/ft)	Y (kip/ft)	Z (kip/ft)	rX (kip-ft/deg)	rY (kip-ft/deg)	rZ (kip-ft/deg)
1	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
2	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
3	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
4	Fixed	Fixed	Fixed	Fixed	Fixed	-
5	Fixed	Fixed	Fixed	Fixed	Fixed	-
6	Fixed	Fixed	Fixed	Fixed	Fixed	-
7	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
8	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
9	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed

Releases

Beam	Node	x	y	z	rx	ry	rz
3	12	Fixed	Fixed	Fixed	Fixed	Pin	Pin
6	15	Fixed	Fixed	Fixed	Fixed	Pin	Pin
9	18	Fixed	Fixed	Fixed	Fixed	Pin	Pin
25	19	Fixed	Fixed	Fixed	Fixed	Pin	Pin
25	20	Fixed	Fixed	Fixed	Fixed	Pin	Pin
26	20	Fixed	Fixed	Fixed	Fixed	Pin	Pin
26	21	Fixed	Fixed	Fixed	Fixed	Pin	Pin

Beam	Node	x	y	z	rx	ry	rz
29	9	Fixed	Fixed	Fixed	Fixed	Pin	Pin
29	21	Fixed	Fixed	Fixed	Fixed	Pin	Pin
30	3	Fixed	Fixed	Fixed	Fixed	Pin	Pin
30	19	Fixed	Fixed	Fixed	Fixed	Pin	Pin

Basic Load Cases

Reference Load Cases

Number	Name	Type
R1	MASS	Mass

Primary Load Cases

Number	Name	Type
1	DL	Dead
2	LL	Live
3	ROOF LL	Roof Live
4	WL+X	Wind
6	WL-X	Wind
8	WL+Z	Wind
10	WL-Z	Wind

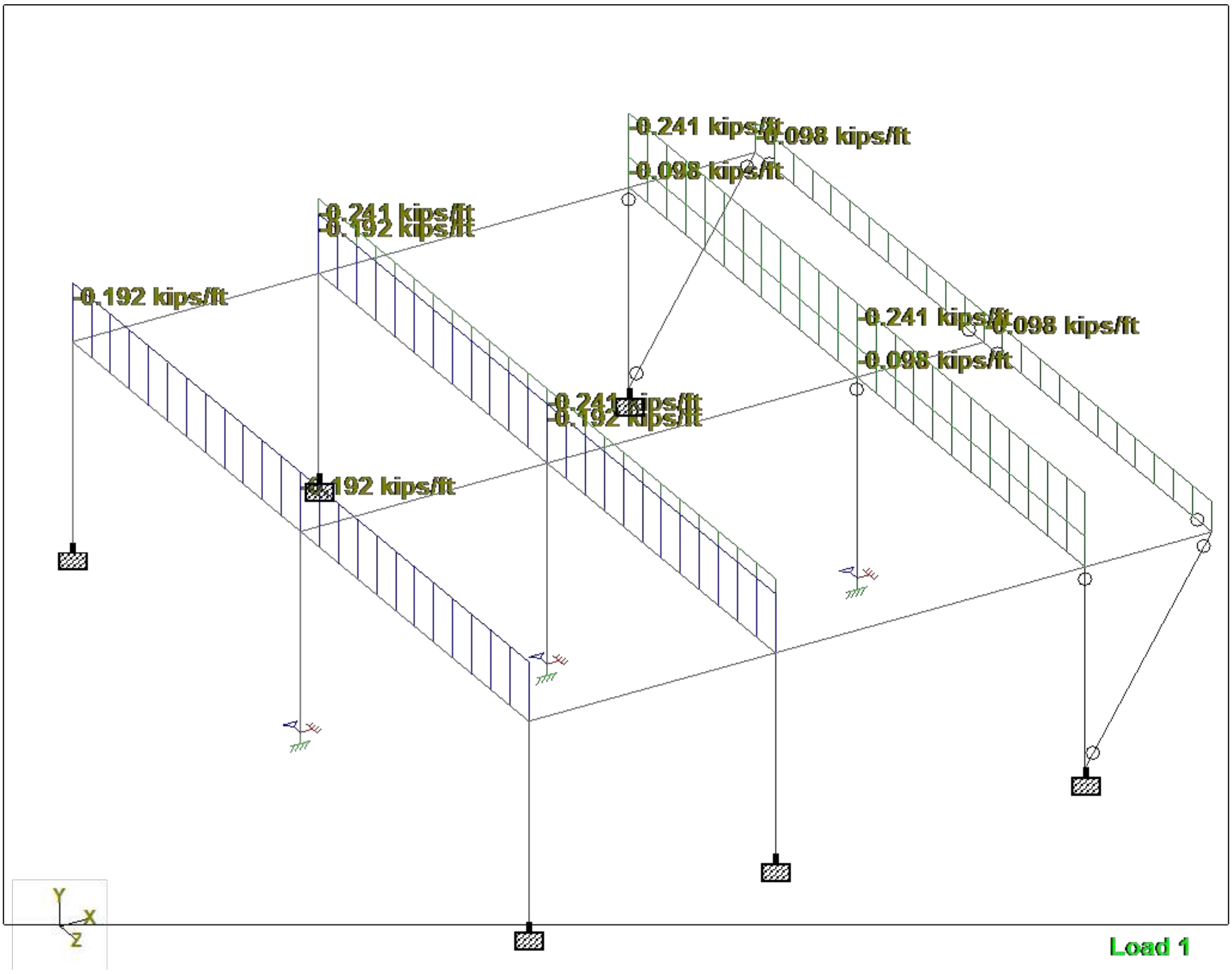
Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
101	COMB - 1.4 DEAD	1	DL	1.400
102	COMB - 1.2 DEAD + 1.6 LIVE + 0.5 ROOF LIVE	1	DL	1.200
		2	LL	1.600
		3	ROOF LL	0.500
103	COMB - 1.2 DEAD + 1 LIVE + 1.6 ROOF LIVE	1	DL	1.200
		2	LL	1.000
		3	ROOF LL	1.600

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
104	COMB - 1.2 DEAD + 1.6 ROOF LIVE + 0.5 WIND (1)	1	DL	1.200
		3	ROOF LL	1.600
		4	WL+X	0.500
106	COMB - 1.2 DEAD + 1.6 ROOF LIVE + 0.5 WIND (3)	1	DL	1.200
		3	ROOF LL	1.600
		6	WL-X	0.500
108	COMB - 1.2 DEAD + 1.6 ROOF LIVE + 0.5 WIND (5)	1	DL	1.200
		3	ROOF LL	1.600
		8	WL+Z	0.500
110	COMB - 1.2 DEAD + 1.6 ROOF LIVE + 0.5 WIND (7)	1	DL	1.200
		3	ROOF LL	1.600
		10	WL-Z	0.500
112	COMB - 1.2 DEAD + 1 LIVE + 0.5 ROOF LIVE + 1 WIND (1)	1	DL	1.200
		2	LL	1.000
		3	ROOF LL	0.500
		4	WL+X	1.000
114	COMB - 1.2 DEAD + 1 LIVE + 0.5 ROOF LIVE + 1 WIND (3)	1	DL	1.200
		2	LL	1.000

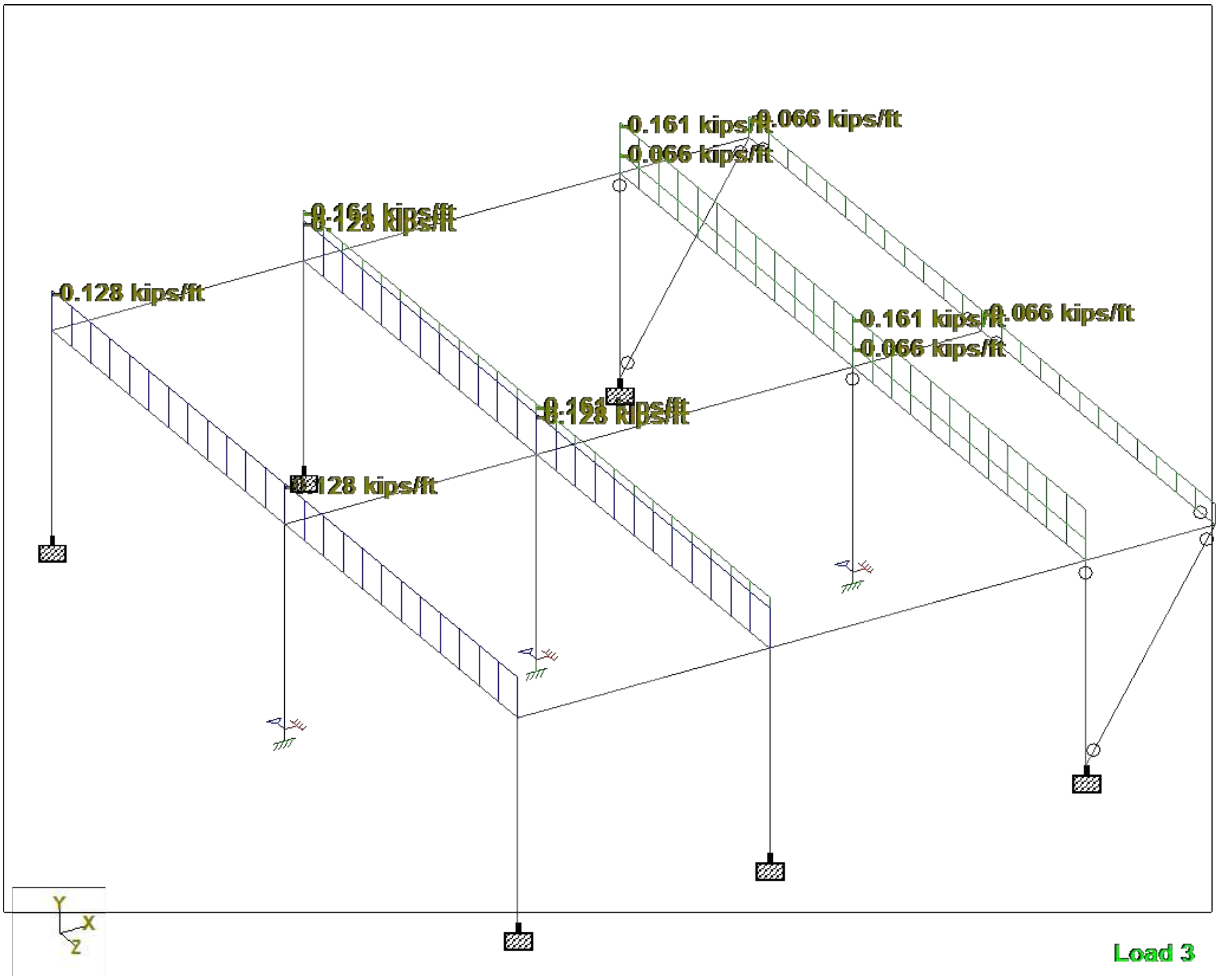
Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
114	COMB - 1.2 DEAD + 1 LIVE + 0.5 ROOF LIVE + 1 WIND (3)	3	ROOF LL	0.500
		6	WL-X	1.000
116	COMB - 1.2 DEAD + 1 LIVE + 0.5 ROOF LIVE + 1 WIND (5)	1	DL	1.200
		2	LL	1.000
		3	ROOF LL	0.500
		8	WL+Z	1.000
118	COMB - 1.2 DEAD + 1 LIVE + 0.5 ROOF LIVE + 1 WIND (7)	1	DL	1.200
		2	LL	1.000
		3	ROOF LL	0.500
		10	WL-Z	1.000
120	COMB - 1.2 DEAD + 1 LIVE	1	DL	1.200
		2	LL	1.000
121	COMB - 0.9 DEAD + 1 WIND (1)	1	DL	0.900
		4	WL+X	1.000
123	COMB - 0.9 DEAD + 1 WIND (3)	1	DL	0.900
		6	WL-X	1.000
125	COMB - 0.9 DEAD + 1 WIND (5)	1	DL	0.900
		8	WL+Z	1.000
127	COMB - 0.9 DEAD + 1 WIND (7)	1	DL	0.900
		10	WL-Z	1.000
129	COMB - 1 LIVE + 1 ROOF LIVE	2	LL	1.000
		3	ROOF LL	1.000

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
130	COMB - 0.9 DEAD	1	DL	0.900
131	COMB - 1 DEAD + 1 LIVE + 1 ROOF LIVE	1	DL	1.000
		2	LL	1.000
		3	ROOF LL	1.000

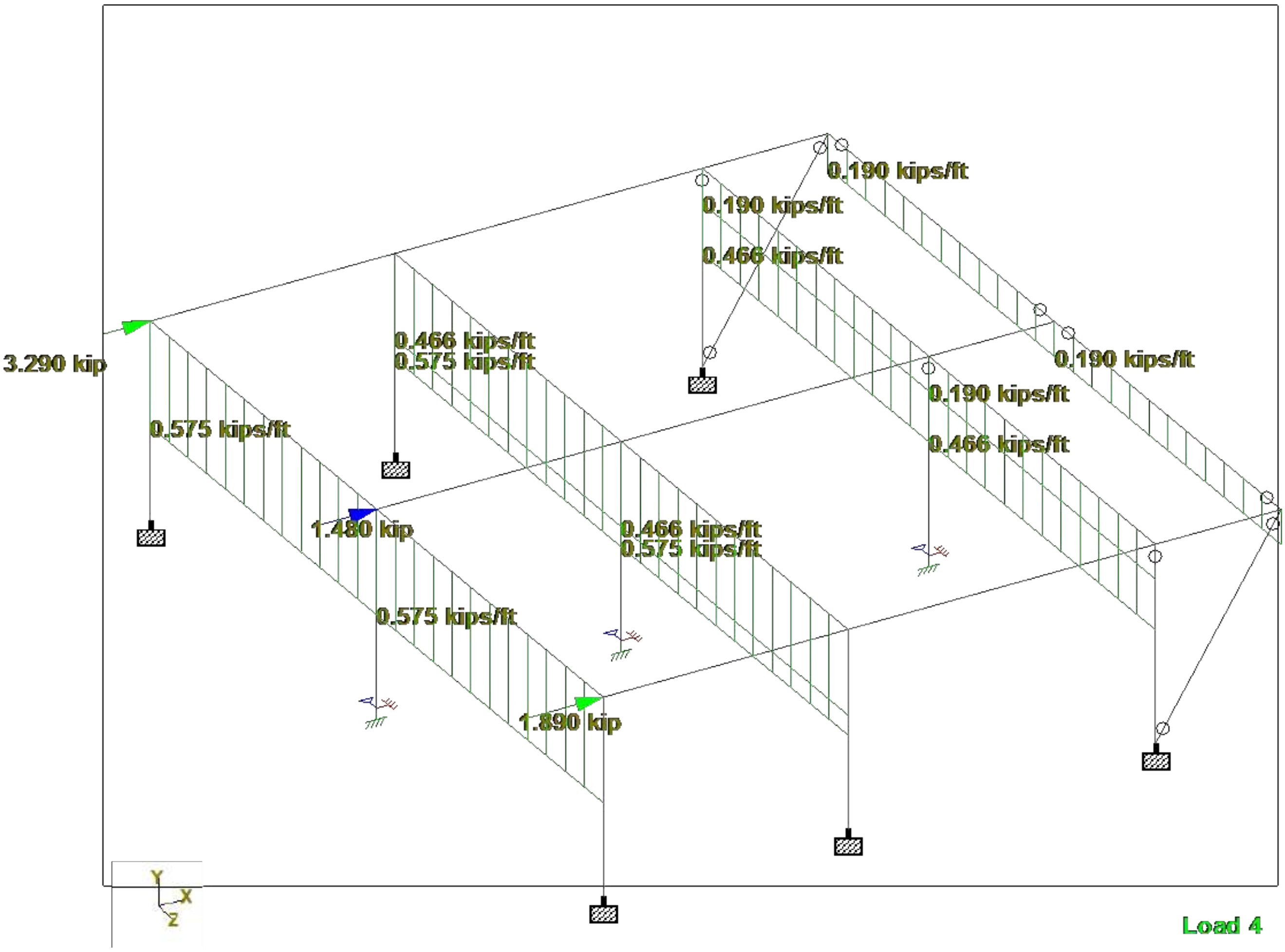


Dead Load

Bentley Software licensed to International Design & Engineering Connected User: Ganesh Maske	Job No. 242201	Sheet No. 9	Rev
Job Title	Part		Ref
Client Example	By SK	Date 2/6/2024	Chd MS
File Exuma, Bahamas_060224.STD	Date Time 12-Feb-2024 13:59		



Live Load



0.190 kips/ft
0.190 kips/ft
0.466 kips/ft
3.290 kip
0.466 kips/ft
0.575 kips/ft
0.190 kips/ft
0.190 kips/ft
0.575 kips/ft
0.466 kips/ft
1.480 kip
0.466 kips/ft
0.575 kips/ft
0.575 kips/ft
1.890 kip
Y
X
Z
Load 4

WL+X

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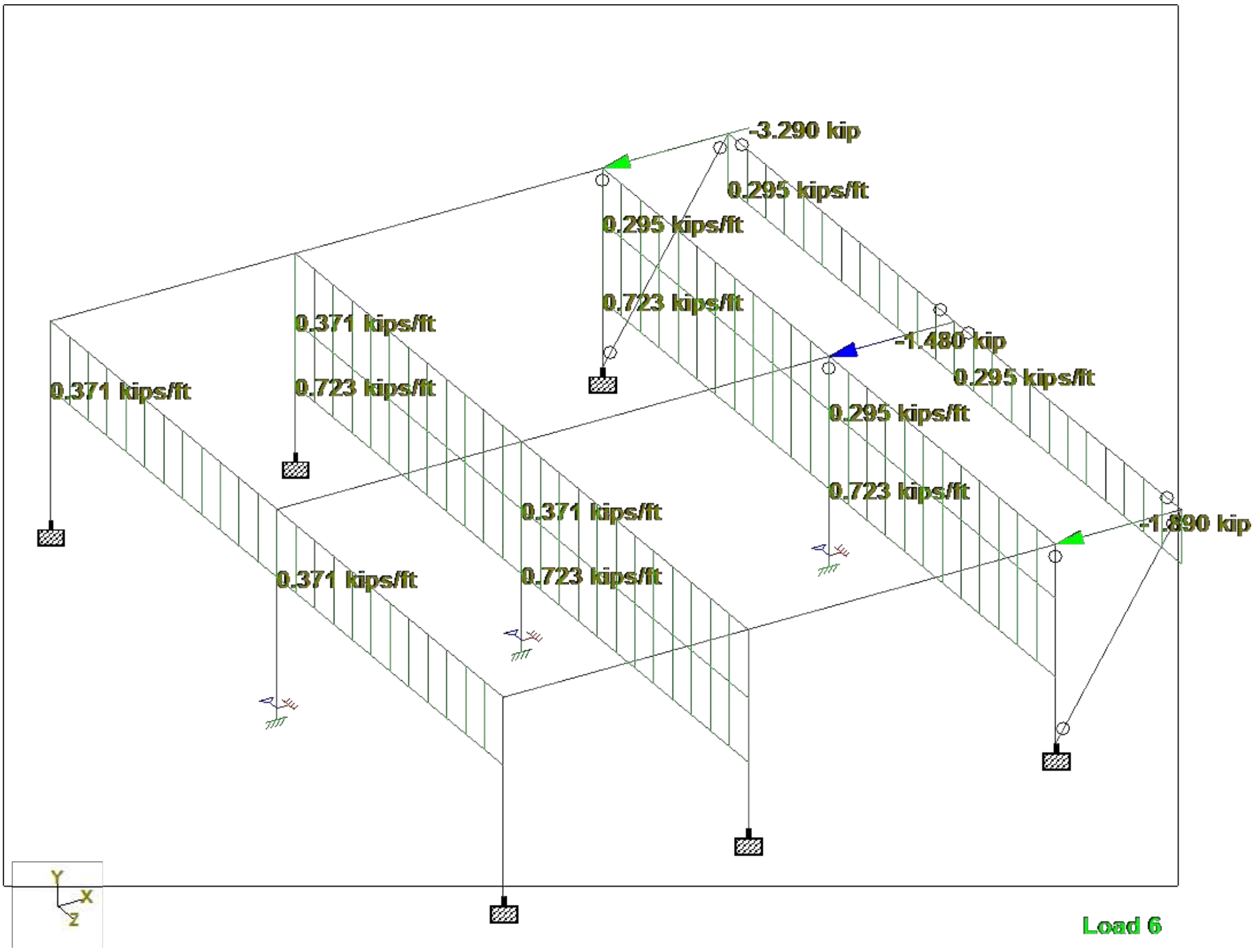
Job Title	Part	Ref	
Client Example	By SK	Date 2/6/2024	Chd MS
File Exuma, Bahamas_060224.STD	Date Time 12-Feb-2024 13:59		



WL-X

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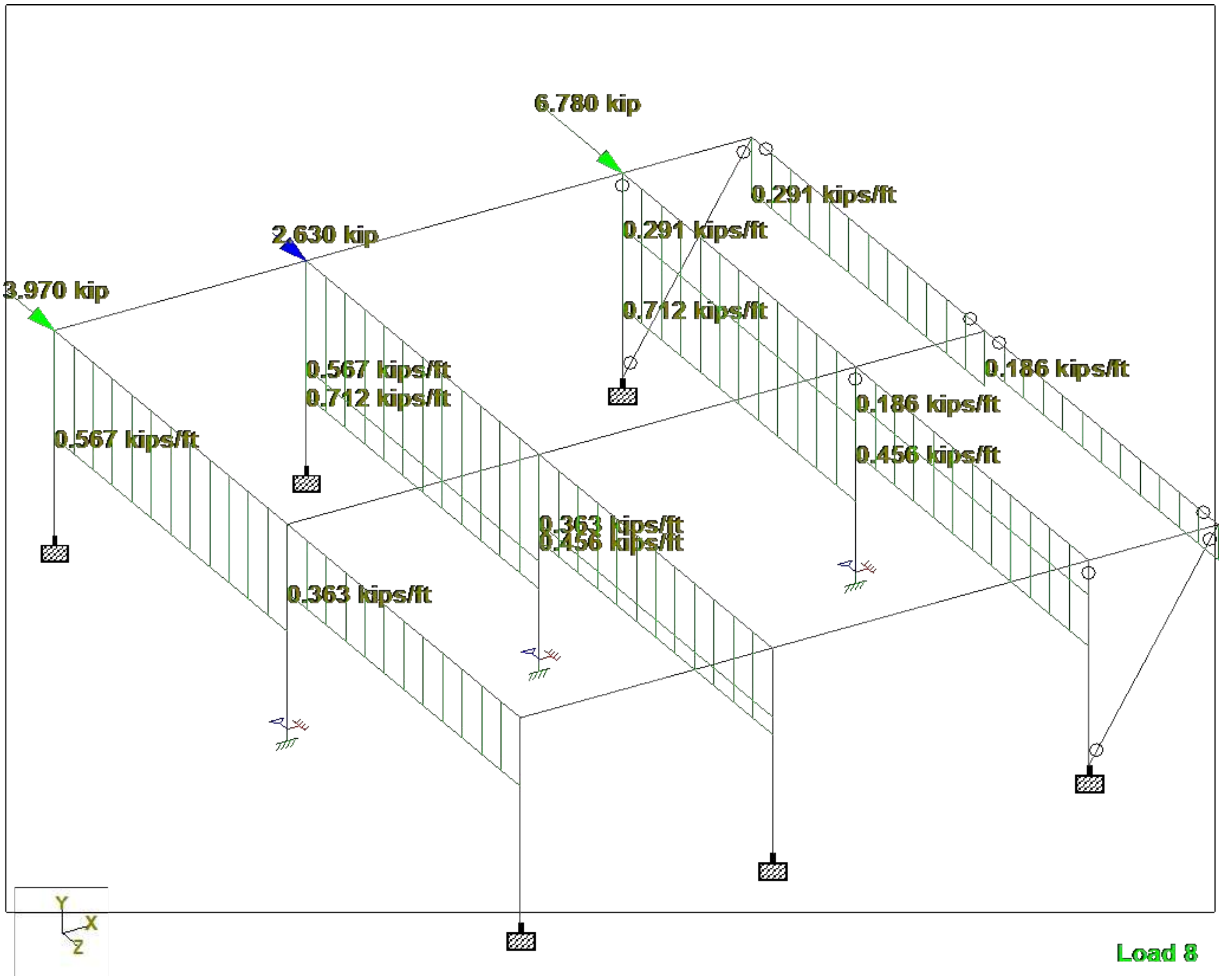
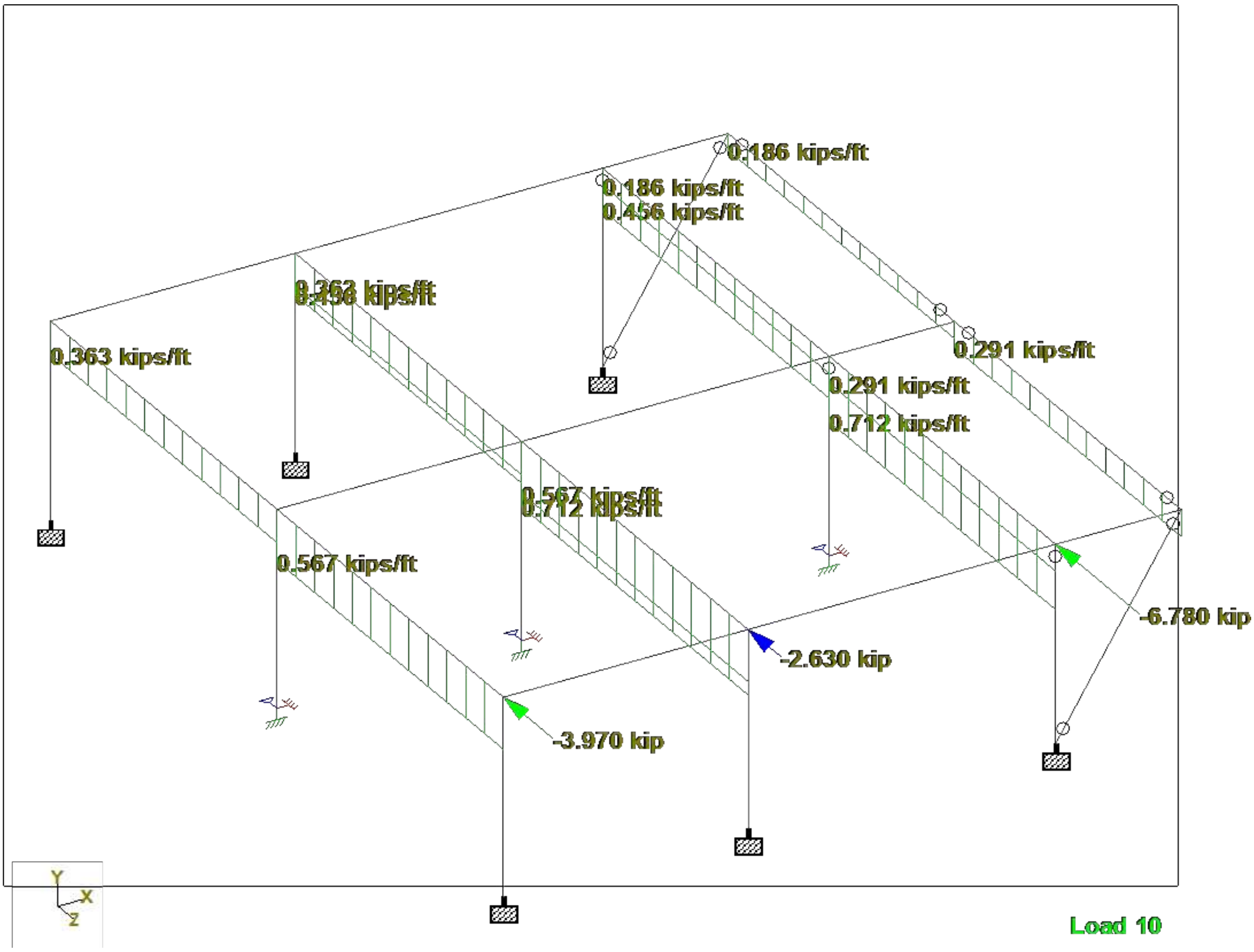
Job Title	Part		Ref
Client Example	By SK	Date 2/6/2024	Chd MS
File Exuma, Bahamas_060224.STD	Date Time 12-Feb-2024 13:59		



WL+Z



WL-Z

Node Displacement Summary

Type	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	10	112	0.266	0.001	-0.018	0.267	-0.002	0.000	-0.002
Min X	10	6	-0.385	0.001	0.018	0.385	-0.003	0.000	0.002
Max Y	20	6	-0.348	0.546	-0.045	0.649	0.000	0.000	0.008
Min Y	20	103	0.134	-0.475	0.000	0.494	0.000	0.000	-0.007
Max Z	19	8	-0.008	0.008	0.499	0.499	-0.005	0.000	0.000
Min Z	19	118	-0.107	0.064	-0.499	0.514	0.000	0.000	0.001
Max rX	17	8	-0.208	0.003	0.388	0.441	0.007	0.000	0.001
Min rX	11	10	-0.208	0.003	-0.388	0.441	-0.007	0.000	0.001
Max rY	10	118	-0.107	0.000	-0.326	0.343	-0.002	0.000	0.001

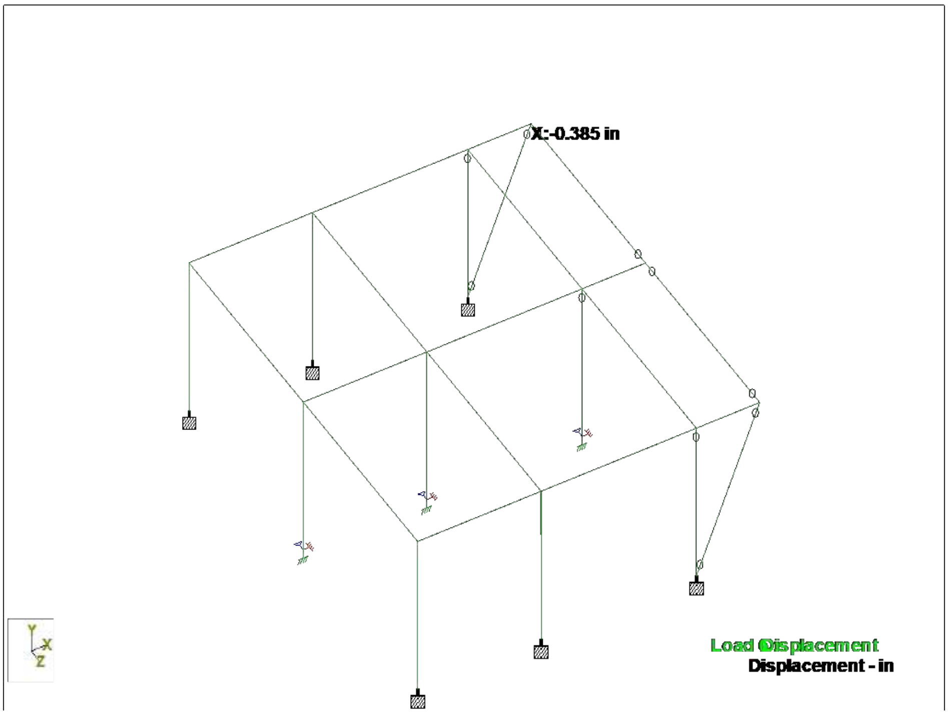
Job No.	Sheet No.	Rev
242201	14	

Job Title	Part	Ref

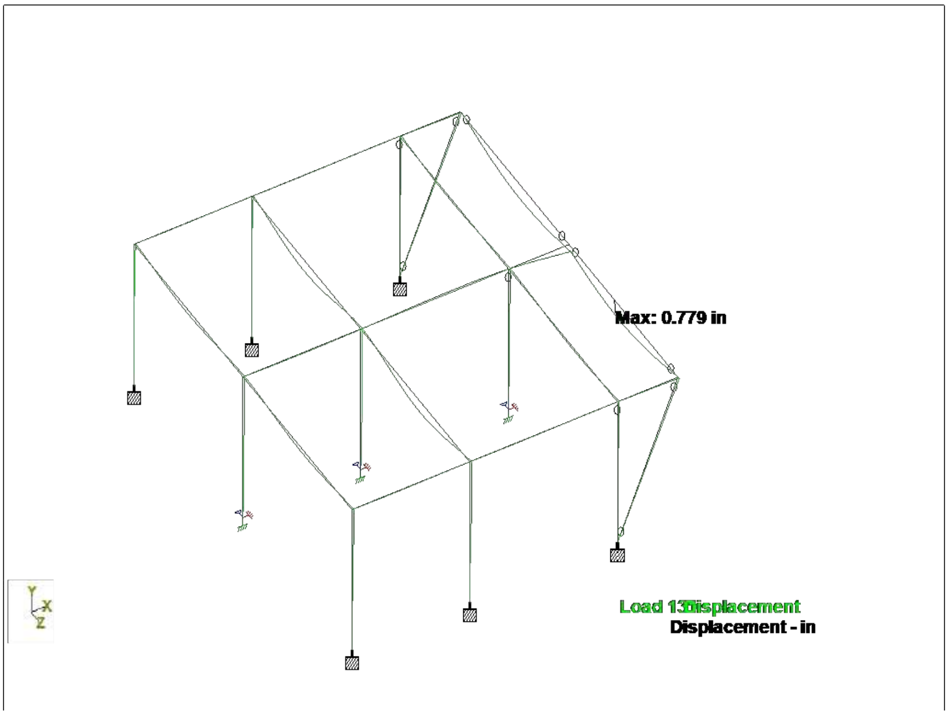
Client	By	Date	Chd
Example	SK	2/6/2024	MS

File	Date Time
Exuma, Bahamas_060224.STD	12-Feb-2024 13:59

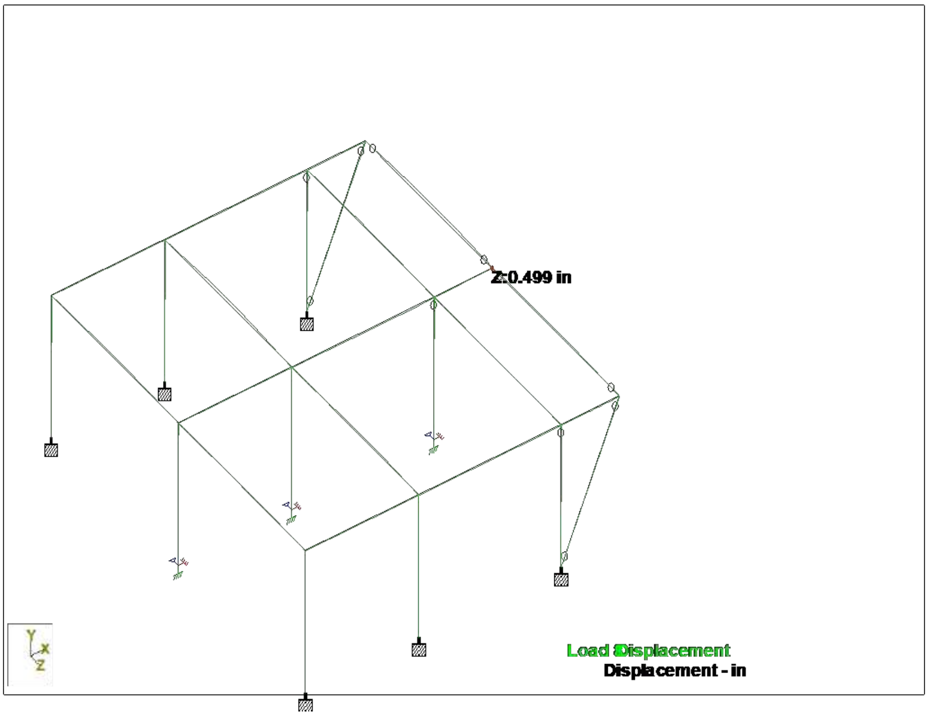
Type	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Min rY	10	8	-0.008	0.003	0.326	0.326	-0.002	0.000	0.000
Max rZ	20	6	-0.348	0.546	-0.045	0.649	0.000	0.000	0.008
Min rZ	20	103	0.134	-0.475	0.000	0.494	0.000	0.000	-0.007
Max Rst	20	8	-0.108	0.427	0.499	0.666	0.002	0.000	0.006



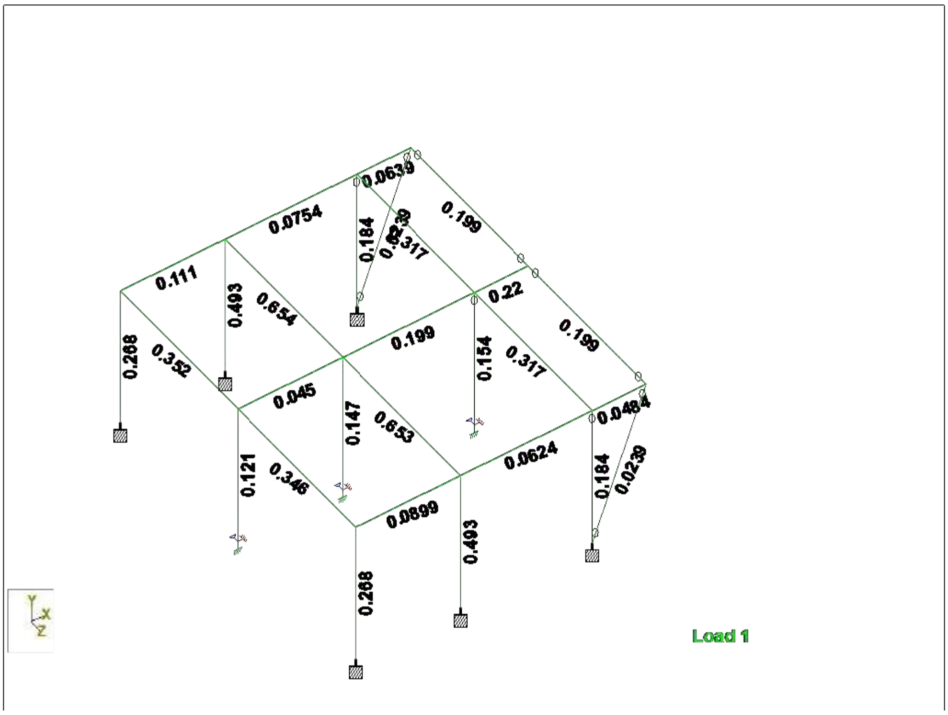
Maximum Lateral Sway In X-Direction



Maximum Vertical Deflection



Maximum Lateral Sway In Z-Direction



Utilization Ratio

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
1	1	-0.302	2.404	0.778	2.795	0.000	2.084
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	-0.135	1.121	0.456	1.636	0.000	0.895
	4	-0.785	-5.856	-1.910	-6.604	0.084	5.100
	6	1.782	-2.436	-1.458	-5.491	-0.083	-11.718
	8	0.014	-5.895	-3.432	-16.833	0.229	-0.163
	10	0.965	-2.254	0.121	4.945	-0.229	-6.342
	101	-0.423	3.366	1.090	3.913	0.000	2.917
	102	-0.430	3.446	1.162	4.172	0.000	2.948
	103	-0.578	4.679	1.663	5.972	0.000	3.933
	104	-0.971	1.751	0.708	2.669	0.042	6.483
	106	0.312	3.461	0.934	3.226	-0.042	-1.926
	108	-0.571	1.732	-0.053	-2.445	0.114	3.851
	110	-0.096	3.552	1.723	8.444	-0.114	0.761
	112	-1.216	-2.410	-0.748	-2.432	0.083	8.048
	114	1.351	1.010	-0.296	-1.319	-0.083	-8.770

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
1	116	-0.416	-2.449	-2.270	-12.661	0.229	2.786
	118	0.535	1.191	1.283	9.117	-0.229	-3.394
	120	-0.363	2.885	0.934	3.354	0.000	2.501
	121	-1.057	-3.692	-1.209	-4.088	0.084	6.975
	123	1.509	-0.272	-0.758	-2.975	-0.083	-9.842
	125	-0.258	-3.731	-2.731	-14.317	0.229	1.713
	127	0.693	-0.091	0.822	7.461	-0.229	-4.467
	129	-0.135	1.121	0.456	1.636	0.000	0.895
	130	-0.272	2.164	0.701	2.516	0.000	1.875
	131	-0.437	3.525	1.234	4.431	0.000	2.979
2	1	-0.424	5.303	1.436	5.157	0.000	2.520
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	-0.184	2.759	0.893	3.207	0.000	1.074
	4	-0.959	-9.656	-3.244	-11.709	0.084	5.722
	6	2.323	-10.743	-3.360	-12.007	-0.083	-13.661
	8	0.075	-12.862	-5.817	-26.263	0.229	-0.380
	10	1.266	-7.188	-0.674	2.952	-0.229	-7.423
	101	-0.593	7.424	2.011	7.220	0.000	3.528
	102	-0.601	7.743	2.170	7.792	0.000	3.561
	103	-0.804	10.777	3.152	11.320	0.000	4.742
	104	-1.283	5.949	1.531	5.466	0.042	7.603
	106	0.358	5.406	1.472	5.317	-0.042	-2.089
	108	-0.766	4.346	0.244	-1.811	0.114	4.552
	110	-0.171	7.183	2.815	12.797	-0.114	1.030
	112	-1.559	-1.914	-1.074	-3.917	0.083	9.283
	114	1.722	-3.000	-1.190	-4.214	-0.083	-10.100
	116	-0.526	-5.119	-3.647	-18.470	0.229	3.181
	118	0.665	0.554	1.496	10.745	-0.229	-3.862
	120	-0.509	6.363	1.723	6.189	0.000	3.024
	121	-1.340	-4.884	-1.951	-7.067	0.084	7.990
	123	1.941	-5.971	-2.068	-7.365	-0.083	-11.393
	125	-0.307	-8.090	-4.525	-21.621	0.229	1.888
	127	0.885	-2.416	0.619	7.594	-0.229	-5.155
	129	-0.184	2.759	0.893	3.207	0.000	1.074

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
2	130	-0.381	4.773	1.293	4.642	0.000	2.268
	131	-0.608	8.062	2.329	8.365	0.000	3.593
3	1	0.859	5.690	0.000	-0.206	-0.341	1.491
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	0.384	2.468	0.000	-0.121	-0.200	0.614
	4	-2.018	-6.709	-0.063	-0.299	0.716	3.406
	6	-0.817	-11.544	0.063	1.195	0.766	-7.949
	8	-2.085	-11.292	-0.888	-8.967	1.353	-0.167
	10	-0.702	-6.648	0.888	9.847	0.104	-4.297
	101	1.202	7.966	0.000	-0.288	-0.477	2.087
	102	1.222	8.062	0.000	-0.308	-0.509	2.096
	103	1.644	10.777	0.000	-0.441	-0.730	2.772
	104	0.635	7.423	-0.031	-0.590	-0.372	4.475
	106	1.235	5.006	0.031	0.157	-0.346	-1.202
	108	0.601	5.132	-0.444	-4.924	-0.053	2.688
	110	1.293	7.453	0.444	4.483	-0.678	0.623
	112	-0.796	1.353	-0.063	-0.607	0.207	5.502
	114	0.405	-3.481	0.063	0.887	0.257	-5.852
	116	-0.863	-3.230	-0.888	-9.274	0.843	1.929
	118	0.520	1.414	0.888	9.539	-0.405	-2.201
	120	1.030	6.828	0.000	-0.247	-0.409	1.789
	121	-1.246	-1.588	-0.063	-0.485	0.409	4.748
	123	-0.045	-6.423	0.063	1.009	0.460	-6.607
	125	-1.312	-6.171	-0.888	-9.152	1.046	1.175
	127	0.071	-1.527	0.888	9.661	-0.202	-2.956
	129	0.384	2.468	0.000	-0.121	-0.200	0.614
	130	0.773	5.121	0.000	-0.185	-0.307	1.342
	131	1.242	8.158	0.000	-0.327	-0.541	2.105
4	1	-0.021	5.463	0.000	0.000	0.000	0.000
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	-0.012	2.873	0.000	0.000	0.000	0.000
	4	-0.172	-13.173	0.102	0.621	0.084	0.000
	6	0.259	-8.074	-0.102	-0.621	-0.083	0.000
	8	0.042	-10.442	-1.119	-8.528	0.229	0.000

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
4	10	0.042	-10.442	1.119	8.528	-0.229	0.000
	101	-0.029	7.648	0.000	0.000	0.000	0.000
	102	-0.031	7.992	0.000	0.000	0.000	0.000
	103	-0.043	11.153	0.000	0.000	0.000	0.000
	104	-0.130	4.567	0.051	0.310	0.042	0.000
	106	0.086	7.116	-0.051	-0.311	-0.042	0.000
	108	-0.022	5.932	-0.560	-4.264	0.114	0.000
	110	-0.022	5.932	0.559	4.264	-0.114	0.000
	112	-0.203	-5.180	0.102	0.621	0.083	0.000
	114	0.228	-0.082	-0.102	-0.621	-0.083	0.000
	116	0.012	-2.449	-1.119	-8.528	0.229	0.000
	118	0.012	-2.449	1.119	8.528	-0.229	0.000
	120	-0.025	6.556	0.000	0.000	0.000	0.000
	121	-0.191	-8.256	0.102	0.621	0.084	0.000
	123	0.240	-3.157	-0.102	-0.621	-0.083	0.000
	125	0.024	-5.525	-1.119	-8.528	0.229	0.000
	127	0.024	-5.525	1.119	8.528	-0.229	0.000
	129	-0.012	2.873	0.000	0.000	0.000	0.000
	130	-0.019	4.917	0.000	0.000	0.000	0.000
	131	-0.032	8.336	0.000	0.000	0.000	0.000
5	1	-0.244	9.970	0.000	0.000	0.000	0.000
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	-0.117	5.782	0.000	0.000	0.000	0.000
	4	-0.012	-21.207	-0.026	-0.154	0.084	0.000
	6	0.877	-21.548	0.026	0.154	-0.083	0.000
	8	0.425	-21.012	-0.900	-8.605	0.229	0.000
	10	0.425	-21.012	0.900	8.605	-0.229	0.000
	101	-0.342	13.958	0.000	0.000	0.000	0.000
	102	-0.352	14.855	0.000	0.000	0.000	0.000
	103	-0.480	21.215	0.000	0.000	0.000	0.000
	104	-0.486	10.612	-0.013	-0.077	0.042	0.000
	106	-0.042	10.441	0.013	0.077	-0.042	0.000
	108	-0.268	10.709	-0.450	-4.303	0.114	0.000
	110	-0.268	10.709	0.450	4.303	-0.114	0.000

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
5	112	-0.364	-6.352	-0.026	-0.154	0.083	0.000
	114	0.526	-6.693	0.026	0.154	-0.083	0.000
	116	0.074	-6.157	-0.900	-8.605	0.229	0.000
	118	0.074	-6.157	0.900	8.605	-0.229	0.000
	120	-0.293	11.964	0.000	0.000	0.000	0.000
	121	-0.232	-12.234	-0.026	-0.154	0.084	0.000
	123	0.657	-12.576	0.026	0.154	-0.083	0.000
	125	0.205	-12.039	-0.900	-8.605	0.229	0.000
	127	0.205	-12.039	0.900	8.605	-0.229	0.000
	129	-0.117	5.782	0.000	0.000	0.000	0.000
	130	-0.220	8.973	0.000	0.000	0.000	0.000
	131	-0.361	15.752	0.000	0.000	0.000	0.000
6	1	0.000	14.818	0.000	0.000	0.000	0.000
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	0.000	7.710	0.000	0.000	0.000	0.000
	4	0.000	-22.150	-0.063	-0.681	0.084	0.000
	6	0.000	-34.865	0.063	0.681	-0.083	0.000
	8	0.000	-28.020	-0.888	-9.646	0.229	0.000
	10	0.000	-28.020	0.888	9.646	-0.229	0.000
	101	0.000	20.746	0.000	0.000	0.000	0.000
	102	0.000	21.637	0.000	0.000	0.000	0.000
	103	0.000	30.119	0.000	0.000	0.000	0.000
	104	0.000	19.044	-0.031	-0.341	0.042	0.000
	106	0.000	12.686	0.031	0.341	-0.042	0.000
	108	0.000	16.109	-0.444	-4.823	0.114	0.000
	110	0.000	16.109	0.444	4.823	-0.114	0.000
	112	0.000	-0.513	-0.063	-0.681	0.083	0.000
	114	0.000	-13.228	0.063	0.681	-0.083	0.000
	116	0.000	-6.382	-0.888	-9.646	0.229	0.000
	118	0.000	-6.382	0.888	9.646	-0.229	0.000
	120	0.000	17.782	0.000	0.000	0.000	0.000
	121	0.000	-8.813	-0.063	-0.681	0.084	0.000
	123	0.000	-21.528	0.063	0.681	-0.083	0.000
	125	0.000	-14.683	-0.888	-9.646	0.229	0.000

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
6	127	0.000	-14.683	0.888	9.646	-0.229	0.000
	129	0.000	7.710	0.000	0.000	0.000	0.000
	130	0.000	13.337	0.000	0.000	0.000	0.000
	131	0.000	22.529	0.000	0.000	0.000	0.000
7	1	-0.302	2.404	-0.778	-2.795	0.000	2.084
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	-0.135	1.121	-0.456	-1.636	0.000	0.895
	4	-0.439	-5.703	2.069	7.683	0.084	2.846
	6	1.435	-2.588	1.300	4.412	-0.083	-9.464
	8	0.965	-2.254	-0.121	-4.944	0.229	-6.342
	10	0.014	-5.895	3.432	16.833	-0.229	-0.163
	101	-0.423	3.366	-1.090	-3.914	0.000	2.918
	102	-0.430	3.446	-1.162	-4.172	0.000	2.948
	103	-0.578	4.679	-1.663	-5.972	0.000	3.933
	104	-0.798	1.828	-0.629	-2.130	0.042	5.356
	106	0.139	3.385	-1.013	-3.766	-0.042	-0.799
	108	-0.096	3.552	-1.723	-8.444	0.114	0.762
	110	-0.571	1.732	0.053	2.445	-0.114	3.851
	112	-0.869	-2.257	0.907	3.511	0.083	5.794
	114	1.005	0.857	0.138	0.240	-0.083	-6.516
	116	0.535	1.191	-1.283	-9.117	0.229	-3.394
	118	-0.416	-2.449	2.270	12.661	-0.229	2.785
	120	-0.363	2.885	-0.934	-3.355	0.000	2.501
	121	-0.711	-3.539	1.368	5.167	0.084	4.721
	123	1.163	-0.424	0.599	1.896	-0.083	-7.589
	125	0.693	-0.091	-0.822	-7.460	0.229	-4.467
	127	-0.258	-3.731	2.731	14.317	-0.229	1.713
	129	-0.135	1.121	-0.456	-1.636	0.000	0.895
	130	-0.272	2.164	-0.701	-2.516	0.000	1.876
	131	-0.437	3.525	-1.234	-4.431	0.000	2.979
8	1	-0.424	5.303	-1.436	-5.157	0.000	2.520
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	-0.184	2.759	-0.893	-3.207	0.000	1.074
	4	-0.524	-9.728	3.197	11.422	0.084	3.151

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
8	6	1.888	-10.672	3.407	12.294	-0.083	-11.091
	8	1.266	-7.188	0.674	-2.952	0.229	-7.423
	10	0.075	-12.862	5.817	26.263	-0.229	-0.381
	101	-0.593	7.424	-2.011	-7.220	0.000	3.528
	102	-0.601	7.743	-2.170	-7.792	0.000	3.561
	103	-0.804	10.777	-3.152	-11.320	0.000	4.742
	104	-1.066	5.913	-1.554	-5.609	0.042	6.318
	106	0.140	5.442	-1.449	-5.173	-0.042	-0.803
	108	-0.171	7.183	-2.815	-12.797	0.114	1.031
	110	-0.766	4.346	-0.244	1.811	-0.114	4.552
	112	-1.125	-1.985	1.028	3.629	0.083	6.712
	114	1.287	-2.929	1.237	4.501	-0.083	-7.530
	116	0.665	0.554	-1.496	-10.745	0.229	-3.862
	118	-0.526	-5.119	3.647	18.470	-0.229	3.180
	120	-0.509	6.363	-1.723	-6.189	0.000	3.024
	121	-0.905	-4.955	1.905	6.780	0.084	5.419
	123	1.507	-5.899	2.114	7.652	-0.083	-8.823
	125	0.885	-2.416	-0.619	-7.594	0.229	-5.155
	127	-0.307	-8.090	4.525	21.621	-0.229	1.887
	129	-0.184	2.759	-0.893	-3.207	0.000	1.074
	130	-0.381	4.773	-1.293	-4.642	0.000	2.268
	131	-0.608	8.062	-2.329	-8.365	0.000	3.594
9	1	0.859	5.690	0.000	0.206	0.341	1.491
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	0.384	2.468	0.000	0.121	0.200	0.614
	4	-1.751	-6.826	-0.063	-1.016	-0.470	1.898
	6	-1.086	-11.426	0.063	0.120	-1.013	-6.441
	8	-0.702	-6.648	-0.888	-9.847	-0.104	-4.298
	10	-2.085	-11.292	0.888	8.966	-1.353	-0.168
	101	1.202	7.966	0.000	0.288	0.477	2.088
	102	1.222	8.062	0.000	0.308	0.509	2.097
	103	1.644	10.777	0.000	0.441	0.730	2.772
	104	0.769	7.364	-0.031	-0.067	0.495	3.721
	106	1.101	5.064	0.031	0.501	0.223	-0.448

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
9	108	1.293	7.453	-0.444	-4.482	0.678	0.624
	110	0.602	5.132	0.444	4.924	0.053	2.689
	112	-0.528	1.236	-0.063	-0.708	0.040	3.994
	114	0.137	-3.364	0.063	0.428	-0.504	-4.344
	116	0.520	1.414	-0.888	-9.539	0.405	-2.201
	118	-0.863	-3.230	0.888	9.274	-0.844	1.929
	120	1.031	6.828	0.000	0.247	0.409	1.789
	121	-0.978	-1.705	-0.063	-0.830	-0.163	3.240
	123	-0.313	-6.305	0.063	0.305	-0.706	-5.098
	125	0.071	-1.527	-0.888	-9.662	0.203	-2.956
	127	-1.313	-6.171	0.888	9.152	-1.046	1.174
	129	0.384	2.468	0.000	0.121	0.200	0.614
	130	0.773	5.121	0.000	0.185	0.307	1.342
	131	1.242	8.158	0.000	0.327	0.541	2.106

Reaction Summary

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
Max FX	2	6	2.323	-10.743	-3.360	-12.007	-0.083	-13.661
Min FX	9	10	-2.085	-11.292	0.888	8.966	-1.353	-0.168
Max FY	6	103	0.000	30.119	0.000	0.000	0.000	0.000
Min FY	6	6	0.000	-34.865	0.063	0.681	-0.083	0.000
Max FZ	8	10	0.075	-12.862	5.817	26.263	-0.229	-0.381
Min FZ	2	8	0.075	-12.862	-5.817	-26.263	0.229	-0.380
Max MX	8	10	0.075	-12.862	5.817	26.263	-0.229	-0.381
Min MX	2	8	0.075	-12.862	-5.817	-26.263	0.229	-0.380
Max MY	3	8	-2.085	-11.292	-0.888	-8.967	1.353	-0.167
Min MY	9	10	-2.085	-11.292	0.888	8.966	-1.353	-0.168
Max MZ	2	112	-1.559	-1.914	-1.074	-3.917	0.083	9.283
Min MZ	2	6	2.323	-10.743	-3.360	-12.007	-0.083	-13.661

Beam Displacement Summary

Type	Beam	L/C	d (ft)	X (in)	Y (in)	Z (in)	Resultant (in)
Max X	30	131	12.688	0.102	-0.064	0.000	0.120
Min X	30	127	12.688	-0.143	0.087	-0.499	0.526
Max Y	30	131	0.000	0.000	0.000	0.000	0.000
Min Y	30	131	12.688	0.102	-0.064	0.000	0.120
Max Z	30	129	0.000	0.000	0.000	0.000	0.000
Min Z	30	127	12.688	-0.143	0.087	-0.499	0.526
Max Rst	25	6	10.253	-0.366	1.218	-0.045	1.273

Utilization Ratio

Beam	Parameter	Analysis Property	Design Property	Status	Actual Ratio	Allowable Ratio	Normalized Ratio (Actual/Allowable)	Code	Clause	L/C
1	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.268	1.000	0.268	360-16 L	Eq.H1-1b	8
2	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.493	1.000	0.493	360-16 L	Eq.H1-1b	8
3	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.184	1.000	0.184	360-16 L	Eq.H1-1b	10
4	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.121	1.000	0.121	360-16 L	Eq.H1-1b	10
5	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.147	1.000	0.147	360-16 L	Eq.H1-1b	6
6	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.154	1.000	0.154	360-16 L	Eq.H1-1b	8
7	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.268	1.000	0.268	360-16 L	Eq.H1-1b	10
8	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.493	1.000	0.493	360-16 L	Eq.H1-1b	10
9	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.184	1.000	0.184	360-16 L	Eq.H1-1b	8
10	Parameter 1	TUB15015010.0	TUB15015010.0	PASS	0.111	1.000	0.111	360-16 L	Cl.F7.1	6
11	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.075	1.000	0.075	360-16 L	Cl.F7.1	6

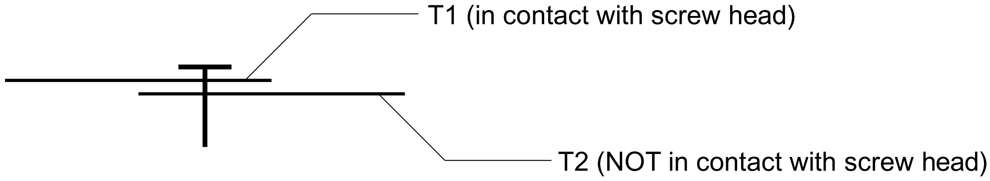
Beam	Parameter	Analysis Property	Design Property	Status	Actual Ratio	Allowable Ratio	Normalized Ratio (Actual/Allowable)	Code	Clause	L/C
14	Parameter 1	TUB15015010.0	TUB15015010.0	PASS	0.090	1.000	0.090	360-16 L	Cl.F7.1	6
15	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.062	1.000	0.062	360-16 L	Cl.F7.1	6
16	Parameter 1	TUB15015010.0	TUB15015010.0	PASS	0.352	1.000	0.352	360-16 L	Cl.F7.4	4
17	Parameter 1	TUB15015010.0	TUB15015010.0	PASS	0.346	1.000	0.346	360-16 L	Cl.F7.4	4
18	Parameter 1	TUB15015010.0	TUB15015010.0	PASS	0.654	1.000	0.654	360-16 L	Cl.F7.4	6
19	Parameter 1	TUB15015010.0	TUB15015010.0	PASS	0.653	1.000	0.653	360-16 L	Cl.F7.4	6
20	Parameter 1	TUB25015012.0	TUB25015012.0	PASS	0.317	1.000	0.317	360-16 L	Cl.F7.4	6
21	Parameter 1	TUB25015012.0	TUB25015012.0	PASS	0.317	1.000	0.317	360-16 L	Cl.F7.4	6
22	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.064	1.000	0.064	360-16 L	Cl.F7.1	112
23	Parameter 1	TUB25015012.0	TUB25015012.0	PASS	0.220	1.000	0.220	360-16 L	Cl.F7.1	6
24	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.048	1.000	0.048	360-16 L	Cl.F7.1	112
25	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.199	1.000	0.199	360-16 L	Cl.F7.4	6
26	Parameter 1	TUB15015012.0	TUB15015012.0	PASS	0.199	1.000	0.199	360-16 L	Cl.F7.4	6
27	Parameter 1	TUB15015010.0	TUB15015010.0	PASS	0.045	1.000	0.045	360-16 L	Cl.F7.1	6
28	Parameter 1	TUB25015012.0	TUB25015012.0	PASS	0.199	1.000	0.199	360-16 L	Cl.F7.4	6
29	Parameter 1	TUB15015010.0	TUB15015010.0	PASS	0.024	1.000	0.024	360-16 L	Cl.H3.1	10
30	Parameter 1	TUB15015010.0	TUB15015010.0	PASS	0.024	1.000	0.024	360-16 L	Cl.H3.1	8

Appendix- 9

ClarkWestern Building Systems
CW Tech Support: (888) 437-3244
clarkwestern.com

2007 North American Specification LRFD
DATE: 05-02-2024

Exuma, Bahamas 242201



Screw Connection Input Parameters

T1 = 0.0590 in   Fu(1) = 71.1 ksi   Edge Dist = NA
T2 = 0.0590 in   Fu(2) = 71.1 ksi   Edge Dist = NA
Screw Diameter = #12 (0.216 in)
Screw Head Diameter = 0.3125

Results

	Nominal Pn (lb)	**ASD Pn/Omega (lb)**	**LRFD phi x Pn (lb)**	**Min Req'd Screw Strength, Pss (lb)**
Shear	1989.0	663.0	994.5	2486.2
Pullout (T2)	770.2	256.7	385.1	2457.9
Pullver (T1)	1966.4	655.5	983.2	2457.9

Notes:
1. Pullout values assume screw fully penetrates T2
2. Minimum edge distance = 1.5d = 0.324 (in)

Appendix- 10

Project:	**Exuma,**
Project no:	**Bahamas**
Author:	**242201**

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	SK/MS
Description	L Shape Base Plate for SHS150x150X10
Date	09-Feb-24
Code	AISC/ACI

Material

Steel	A36, Unknown, A572 Gr.50, A913 Gr.50
Concrete	4000 psi, 4500 psi, 5000 psi

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	L Shape Base Plate for SHS150x150
Analysis	Stress, strain/ simplified loading
Design code	AISC - LRFD (2016)

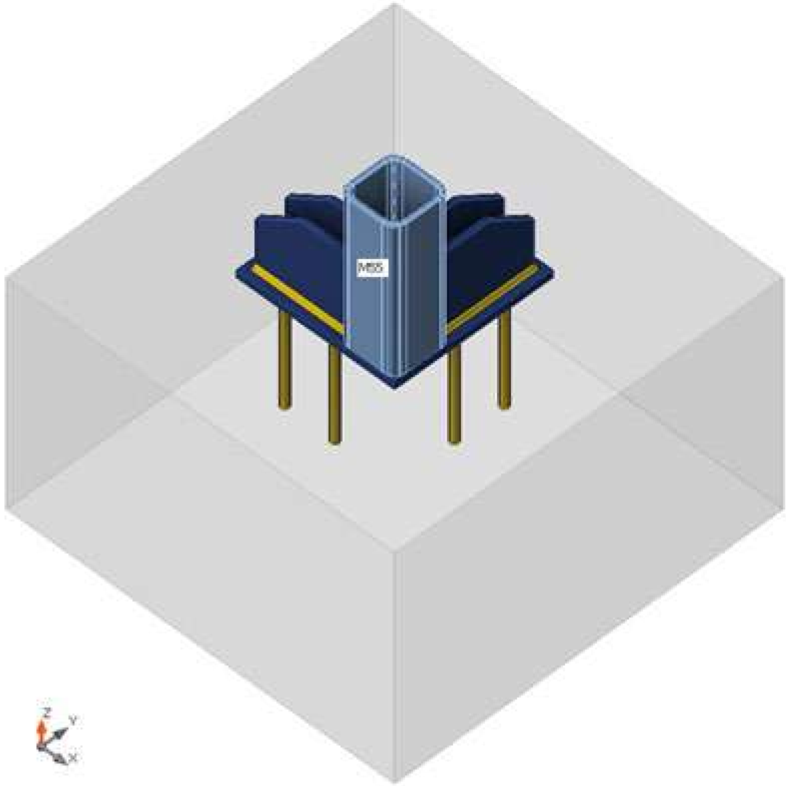
Members

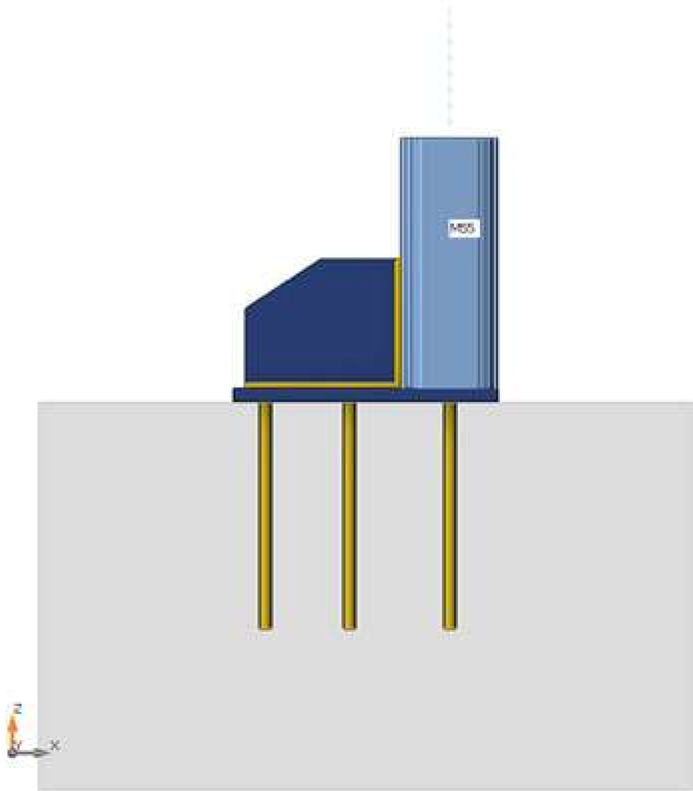
Geometry

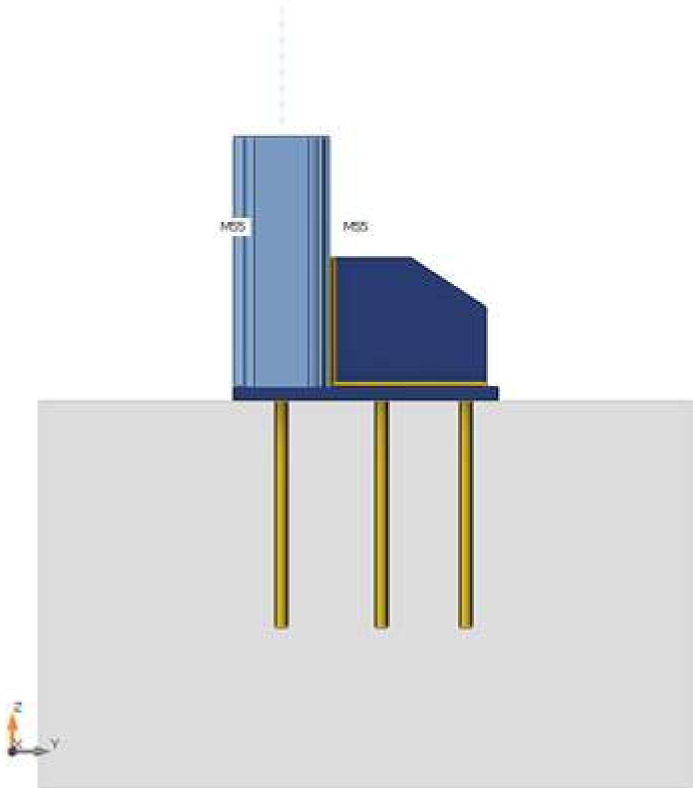
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
M55	20 - SHS150/150/12.5	0.0	90.0	0.0	3/4	0"	-1/8

Supports and forces

Name	Support	Forces in	X [in]
M55 / end		Node	0"



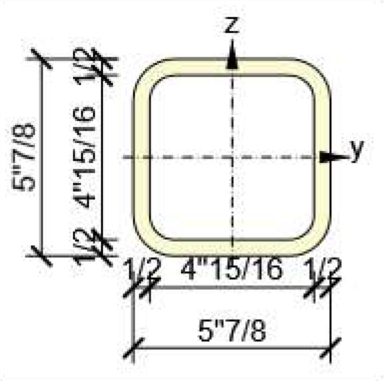




Cross-sections

Name	Material
20 - SHS150/150/12.5	A913 Gr.50

Cross-sections

Name	Material	Drawing
20 - SHS150/150/12.5	A913 Gr.50	

Anchors

Name	Bolt assembly	Diameter [in]	f_u [ksi]	Gross area [in^2]
3/4 A325	3/4 A325	3/4	120.0	0.4418

Load effects (Equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE8	M55 / End	8.09	-4.53	-0.31	0.23	1.89	-21.62
LE10	M55 / End	5.97	-1.94	-2.07	0.08	7.37	-11.39

Foundation block

Item	Value	Unit
CB 1		
Dimensions	3'-3"3/4 x 3'-3"3/4	in
Depth	1'-11"5/8	in
Anchor	3/4 A325	
Anchoring length	1'-1"3/4	in
Shear force transfer	Anchors	

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Loc. deformation	0.1 < 3%	OK
Anchors	70.3 < 100%	OK
Welds	75.4 < 100%	OK
Concrete block	12.3 < 100%	OK
Buckling	Not calculated	

Plates

Name	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
M55	1/2	LE8	38.5	0.0	0.0	OK
Plate 6	13/16	LE8	20.4	0.0	0.0	OK
RIB1	9/16	LE8	21.0	0.0	0.0	OK
RIB2	9/16	LE8	20.3	0.0	0.0	OK
RIB3	9/16	LE10	8.7	0.0	0.0	OK
RIB4	9/16	LE8	6.5	0.0	0.0	OK

Design data

Material	F_y [ksi]	ε_{lim} [%]
A913 Gr.50	50.0	5.0

Symbol explanation

t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain

Detailed result for M55

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for Plate 6

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for RIB1

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for RIB2

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for RIB3

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for RIB4

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

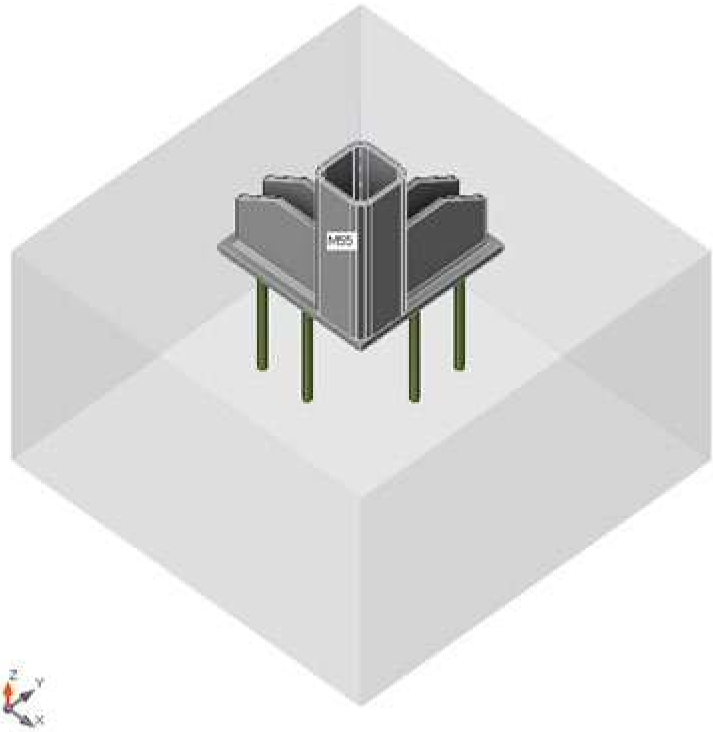
ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Loc. deformation

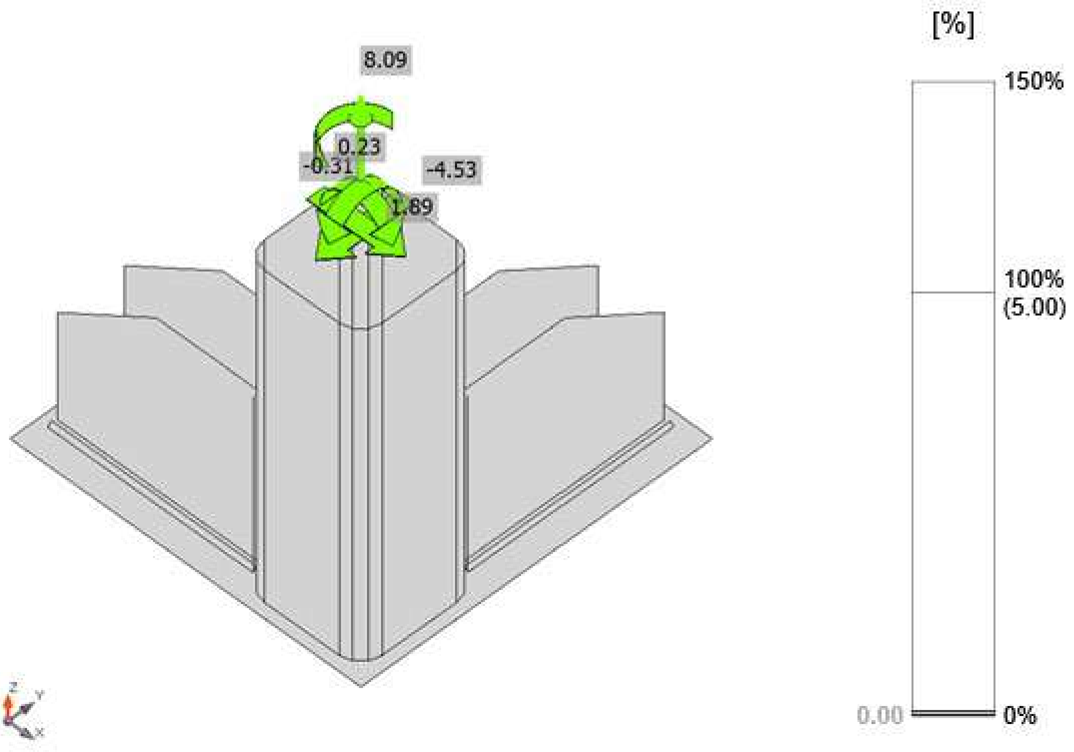
Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
M55	5"7/8	LE8		3/16	0.1	OK

Symbol explanation

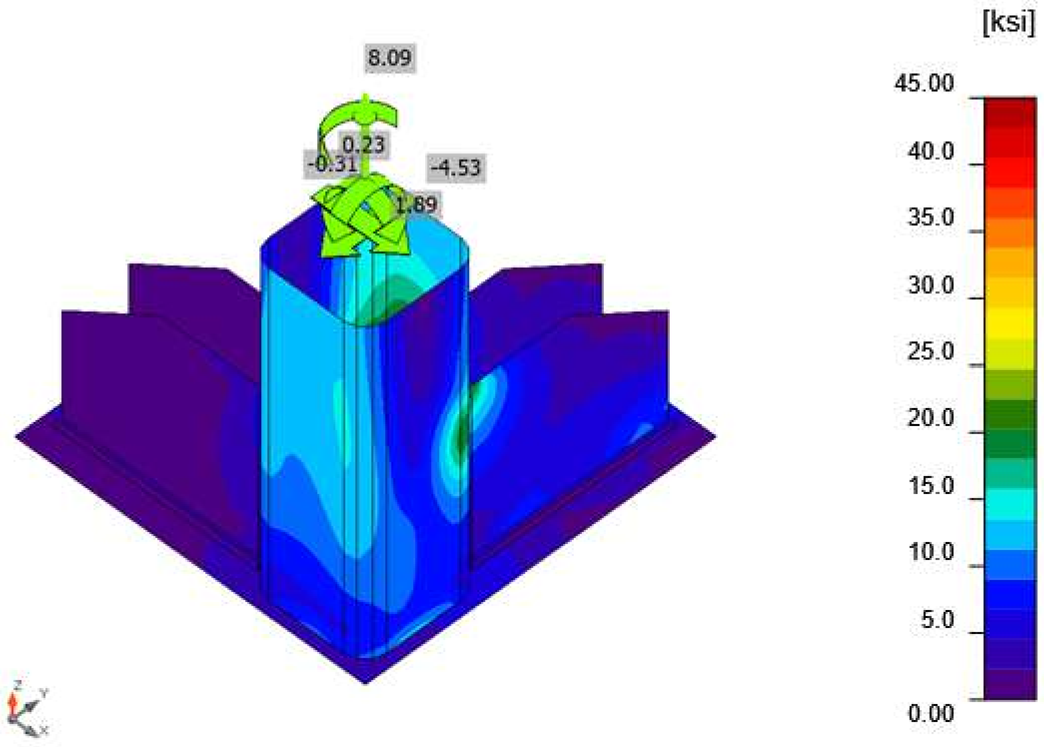
d_0	Cross-section size
δ	Local cross-section deformation
δ_{lim}	Allowed deformation



Overall check, LE8

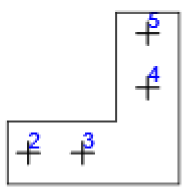


Strain check, LE8



Equivalent stress, LE8

Anchors

Shape	Item	Loads	N_f [kip]	V [kip]	ϕN_{cbg} [kip]	ϕV_{cbg} [kip]	ϕV_{cp} [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	A2	LE8	4.22	0.28	41.60	17.11	109.48	70.3	27.3	67.1	OK	OK
	A3	LE8	1.26	0.86	41.60	17.11	109.48	70.3	27.3	67.1	OK	OK
	A4	LE8	9.21	1.68	41.60	32.65	109.48	70.3	10.7	58.0	OK	OK
	A5	LE8	14.57	1.79	41.60	32.65	109.48	70.3	11.5	58.3	OK	OK

Design data

Grade	ϕN_{sa} [kip]	ϕV_{sa} [kip]
3/4 A325 - 1	28.06	15.63

Symbol explanation

N_f	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
ϕN_{cbg}	Concrete breakout strength in tension – ACI 318-14 – 17.4.2
ϕV_{cbg}	Concrete breakout strength in shear – ACI 318-14 – 17.5.2
ϕV_{cp}	Concrete pryout strength in shear – ACI 318-14 – 17.5.3
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
ϕN_{sa}	Steel strength of anchor in tension - ACI 318-14 – 17.4.1
ϕV_{sa}	Steel strength of anchor in shear - ACI 318-14 – 17.5.1

Detailed result for A2

Following checks of anchors loaded in tension are not provided and should be checked using information in relevant Technical Product Specification (based on the 5 percent fractile of tests performed and evaluated according to ACI 355.2):

- *Pull-out failure of fastener (for post-installed mechanical anchors) - ACI 318-19 – 17.6.3*
- *Bond strength of adhesive anchor (for post-installed bonded anchors) - ACI 318-19 – 17.6.5*
- *Concrete splitting failure during installation should be evaluated by ACI 355.2 requirements*

Concrete blow-out failure is provided only for anchors with washer plates.

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} =$ 28.06 kip ≥ $N_f =$ 4.22 kip

Where:

ϕ = 0.70 – resistance factor

$A_{se,N}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \text{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
  - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)

The check is performed for group of anchors that form common tension breakout cone: A2, A3, A4, A5

$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot N_b =$ 41.60 kip ≥ $N_{fg} =$ 29.26 kip

Where:

N_{fg} = 29.26 kip – sum of tension forces of anchors with common concrete breakout cone area

ϕ = 0.70 – resistance factor

A_{Nc} = 1436.0097 in^2 – concrete breakout cone area for group of anchors

A_{Nc0} = 871.8767 in^2 – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N}$ = 0.98 – modification factor for edge distance:

- $\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$, where:
    - $c_{a,min} =$ 1'-1"3/4 in – minimum distance from the anchor to the edge
    - $h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$ 9"13/16 in – depth of embedment, where:
        - $h_{emb} =$ 1'-1"3/4 in – anchor length
        - $c_{a,max} =$ 1'-2"3/4 in – maximum distance from the anchor to one of the three closest edges
        - $s =$ 5"1/8 in – maximum spacing between anchors

$\Psi_{ec,N}$ = 0.71 – modification factor for eccentrically loaded group of anchors

- $\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$, where:
    - $\Psi_{ecx,N} = \frac{1}{1+\frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$ 0.86 – modification factor that depends on eccentricity in x-direction
    - $e_{x,N} =$ 2"7/16 in – tension load eccentricity in x-direction
    - $\Psi_{ecy,N} = \frac{1}{1+\frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$ 0.82 – modification factor that depends on eccentricity in y-direction
    - $e_{y,N} =$ 3"3/16 in – tension load eccentricity in y-direction
    - $h_{ef} =$ 9"13/16 in – depth of embedment

$\Psi_{c,N}$ = 1.00 – modification factor for concrete conditions

N_b = 52.18 kip – basic concrete breakout strength of a single anchor in tension:

- $N_b = k_c \cdot \lambda_a \cdot \sqrt{f'_c} \cdot h_{ef}^{1.5}$, where:
    - $k_c =$ 24.0 – coefficient for cast-in anchors
    - $\lambda_a =$ 1.00 – modification factor for lightweight concrete
    - $f'_c =$ 5.0 ksi – concrete compressive strength
    - $h_{ef} =$ 9"13/16 in – depth of embedment

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 15.63 kip ≥ $V =$ 0.28 kip

Where:

$\phi = 0.65$ – resistance factor

$A_{se,V} = 0.3340\ \text{in}^2$ – tensile stress area

$f_{uta} = 120.0$ ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \text{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
  - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete shear breakout check (ACI 318-14 – 17.5.2)

The check is performed for group of anchors that form common shear breakout cone: A2, A3

$\phi V_{cbg} = \phi \cdot \frac{A_{Vc}}{A_{Vc0}} \cdot \Psi_{ec,V} \cdot \Psi_{ed,V} \cdot \Psi_{c,V} \cdot \Psi_{h,V} \cdot \Psi_{\alpha,V} \cdot V_{br} =$ 17.11 kip ≥ $V_g =$ 4.68 kip

Where:

V_g = 4.68 kip – sum of shear forces of anchors on common base plate

ϕ = 0.65 – resistance factor

A_{Vc} = 881.4674 in^2 – projected concrete failure area of an anchor or group of anchors

A_{Vc0} = 980.8613 in^2 – projected concrete failure area of one anchor when not limited by corner influences, spacing or member thickness

$\Psi_{ec,V}$ = 0.93 – modification factor for anchor groups loaded eccentrically in shear:

- $\Psi_{ec,V} = \frac{1}{1+\frac{2 \cdot e'_V}{3 \cdot c_a}}$, where:
  - e'_V = 1"5/8 in – shear load eccentricity
  - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load

$\Psi_{ed,V}$ = 0.89 – modification factor for edge effect:

- $\Psi_{ed,V} = 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_a} \leq 1$, where:
  - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
  - c_{a2} = 1'-1"3/4 in – edge distance in direction perpendicular to the load

$\Psi_{c,V}$ = 1.00 – modification factor for concrete conditions

$\Psi_{h,V}$ = 1.00 – modification factor for anchors located in a shallow concrete member:

- $\Psi_{h,V} = \frac{}{\sqrt{\frac{1.5 \cdot c_{a1}}{h_a}}} \geq 1$, where:
  - h_a = 1'-11"5/8 in – thickness of member in which an anchor is anchored measured parallel to anchor axis

$\Psi_{\alpha,V}$ = 1.00 – modification factor for anchors loaded at an angle with the concrete edge

- $\Psi_{\alpha,V} = \sqrt{\frac{1}{(\cos \alpha_V)^2 + (0.5 \cdot \sin \alpha_V)^2}}$, where:
  - α_V = 3.4 ° – angle between direction of shear force and direction perpendicular to concrete edge

V_b = 35.47 kip – basic concrete breakout strength of a single anchor in shear:

- $V_b = \min(7.0 \cdot (\frac{l_e}{d})^{0.2} \cdot \lambda_a \cdot \sqrt{d_a} \cdot \sqrt{f_c'} \cdot c_a^{1.5}, 9.0 \cdot \lambda_a \cdot \sqrt{f_c'} \cdot c_a^{1.5})$, where:
    - l_e = 6" in – effective length
    - d_a = 3/4 in – anchor diameter
    - λ_a = 1.00 – modification factor for lightweight concrete
    - f_c' = 5.0 ksi – concrete compressive strength
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
        - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} =$ 109.48 kip ≥ $V_g =$ 4.53 kip

Where:

ϕ = 0.65 – resistance factor

k_{cp} = 2.00 – concrete pry-out factor

N_{cp} = 84.22 kip – concrete cone tension break-out resistance in case all anchors are in tension

V_g = 4.53 kip – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

$U_{tt}^{5/3} + U_{ts}^{5/3} =$ 0.67 ≤ 1.0

Where:

U_{tt} = 0.70 – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

U_{ts} = 0.27 – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Detailed result for A3

Following checks of anchors loaded in tension are not provided and should be checked using information in relevant Technical Product Specification (based on the 5 percent fractile of tests performed and evaluated according to ACI 355.2):

- *Pull-out failure of fastener (for post-installed mechanical anchors) - ACI 318-19 – 17.6.3*
- *Bond strength of adhesive anchor (for post-installed bonded anchors) - ACI 318-19 – 17.6.5*
- *Concrete splitting failure during installation should be evaluated by ACI 355.2 requirements*

Concrete blow-out failure is provided only for anchors with washer plates.

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} =$ 28.06 kip ≥ $N_f =$ 1.26 kip

Where:

ϕ = 0.70 – resistance factor

$A_{se,N}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\text{ ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
    - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
    - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)

The check is performed for group of anchors that form common tension breakout cone: A2, A3, A4, A5

$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot N_b =$ 41.60 kip ≥ $N_{fg} =$ 29.26 kip

Where:

N_{fg} = 29.26 kip – sum of tension forces of anchors with common concrete breakout cone area

ϕ = 0.70 – resistance factor

A_{Nc} = 1436.0097 in^2 – concrete breakout cone area for group of anchors

A_{Nc0} = 871.8767 in^2 – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N}$ = 0.98 – modification factor for edge distance:

- $\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$, where:
  - $c_{a,min} =$ 1'-1"3/4 in – minimum distance from the anchor to the edge
  - $h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$ 9"13/16 in – depth of embedment, where:
    - $h_{emb} =$ 1'-1"3/4 in – anchor length
    - $c_{a,max} =$ 1'-2"3/4 in – maximum distance from the anchor to one of the three closest edges
    - $s =$ 5"1/8 in – maximum spacing between anchors

$\Psi_{ec,N}$ = 0.71 – modification factor for eccentrically loaded group of anchors

- $\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$, where:
  - $\Psi_{ecx,N} = \frac{1}{1+\frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$ 0.86 – modification factor that depends on eccentricity in x-direction
  - $e_{x,N} =$ 2"7/16 in – tension load eccentricity in x-direction
  - $\Psi_{ecy,N} = \frac{1}{1+\frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$ 0.82 – modification factor that depends on eccentricity in y-direction
  - $e_{y,N} =$ 3"3/16 in – tension load eccentricity in y-direction
  - $h_{ef} =$ 9"13/16 in – depth of embedment

$\Psi_{c,N}$ = 1.00 – modification factor for concrete conditions

N_b = 52.18 kip – basic concrete breakout strength of a single anchor in tension:

- $N_b = k_c \cdot \lambda_a \cdot \sqrt{f_c'} \cdot h_{ef}^{1.5}$, where:
  - $k_c =$ 24.0 – coefficient for cast-in anchors
  - $\lambda_a =$ 1.00 – modification factor for lightweight concrete
  - $f_c' =$ 5.0 ksi – concrete compressive strength
  - $h_{ef} =$ 9"13/16 in – depth of embedment

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 15.63 kip $\geq$ $V =$ 0.86 kip

Where:

ϕ = 0.65 – resistance factor

$A_{se,V}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \text{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
    - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
    - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete shear breakout check (ACI 318-14 – 17.5.2)

The check is performed for group of anchors that form common shear breakout cone: A2, A3

$\phi V_{cbg} = \phi \cdot \frac{A_{Vc}}{A_{Vc0}} \cdot \Psi_{ec,V} \cdot \Psi_{ed,V} \cdot \Psi_{c,V} \cdot \Psi_{h,V} \cdot \Psi_{\alpha,V} \cdot V_{br} =$ 17.11 kip ≥ $V_g =$ 4.68 kip

Where:

V_g = 4.68 kip – sum of shear forces of anchors on common base plate

ϕ = 0.65 – resistance factor

A_{Vc} = 881.4674 in^2 – projected concrete failure area of an anchor or group of anchors

A_{Vc0} = 980.8613 in^2 – projected concrete failure area of one anchor when not limited by corner influences, spacing or member thickness

$\Psi_{ec,V}$ = 0.93 – modification factor for anchor groups loaded eccentrically in shear:

- $\Psi_{ec,V} = \frac{1}{1+\frac{2 \cdot e'_V}{3 \cdot c_a}}$, where:
  - e'_V = 1"5/8 in – shear load eccentricity
  - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load

$\Psi_{ed,V}$ = 0.89 – modification factor for edge effect:

- $\Psi_{ed,V} = 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_a} \leq 1$, where:
  - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
  - c_{a2} = 1'-1"3/4 in – edge distance in direction perpendicular to the load

$\Psi_{c,V}$ = 1.00 – modification factor for concrete conditions

$\Psi_{h,V}$ = 1.00 – modification factor for anchors located in a shallow concrete member:

- $\Psi_{h,V} = \frac{}{\sqrt{\frac{1.5 \cdot c_{a1}}{h_a}}} \geq 1$, where:
  - h_a = 1'-11"5/8 in – thickness of member in which an anchor is anchored measured parallel to anchor axis

$\Psi_{\alpha,V}$ = 1.00 – modification factor for anchors loaded at an angle with the concrete edge

- $\Psi_{\alpha,V} = \sqrt{\frac{1}{(\cos \alpha_V)^2+(0.5 \cdot \sin \alpha_V)^2}}$, where:
  - α_V = 3.4 ° – angle between direction of shear force and direction perpendicular to concrete edge

V_b = 35.47 kip – basic concrete breakout strength of a single anchor in shear:

- $V_b = \min(7.0 \cdot (\frac{l_e}{d})^{0.2} \cdot \lambda_a \cdot \sqrt{d_a} \cdot \sqrt{f'_c} \cdot c_a^{1.5}, 9.0 \cdot \lambda_a \cdot \sqrt{f'_c} \cdot c_a^{1.5})$, where:
    - l_e = 6" in – effective length
    - d_a = 3/4 in – anchor diameter
    - λ_a = 1.00 – modification factor for lightweight concrete
    - f'_c = 5.0 ksi – concrete compressive strength
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
        - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} =$ 109.48 kip ≥ $V_g =$ 4.53 kip

Where:

ϕ = 0.65 – resistance factor

k_{cp} = 2.00 – concrete pry-out factor

N_{cp} = 84.22 kip – concrete cone tension break-out resistance in case all anchors are in tension

V_g = 4.53 kip – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

$U_{tt}{}^{5/3} + U_{ts}{}^{5/3} =$ 0.67 ≤ 1.0

Where:

U_{tt} = 0.70 – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

U_{ts} = 0.27 – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Detailed result for A4

Following checks of anchors loaded in tension are not provided and should be checked using information in relevant Technical Product Specification (based on the 5 percent fractile of tests performed and evaluated according to ACI 355.2):

- *Pull-out failure of fastener (for post-installed mechanical anchors) - ACI 318-19 – 17.6.3*
- *Bond strength of adhesive anchor (for post-installed bonded anchors) - ACI 318-19 – 17.6.5*
- *Concrete splitting failure during installation should be evaluated by ACI 355.2 requirements*

Concrete blow-out failure is provided only for anchors with washer plates.

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} =$ 28.06 kip ≥ $N_f =$ 9.21 kip

Where:

ϕ = 0.70 – resistance factor

$A_{se,N}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125 \text{ ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - f_{ya} = 92.0 ksi – specified yield strength of anchor steel
  - f_u = 120.0 ksi – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)

The check is performed for group of anchors that form common tension breakout cone: A2, A3, A4, A5

$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot N_b =$ 41.60 kip ≥ $N_{fg} =$ 29.26 kip

Where:

N_{fg} = 29.26 kip – sum of tension forces of anchors with common concrete breakout cone area

ϕ = 0.70 – resistance factor

A_{Nc} = 1436.0097 in^2 – concrete breakout cone area for group of anchors

A_{Nc0} = 871.8767 in^2 – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N}$ = 0.98 – modification factor for edge distance:

- $\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$, where:
  - $c_{a,min} =$ 1'-1"3/4 in – minimum distance from the anchor to the edge
  - $h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$ 9"13/16 in – depth of embedment, where:
    - $h_{emb} =$ 1'-1"3/4 in – anchor length
    - $c_{a,max} =$ 1'-2"3/4 in – maximum distance from the anchor to one of the three closest edges
    - $s =$ 5"1/8 in – maximum spacing between anchors

$\Psi_{ec,N}$ = 0.71 – modification factor for eccentrically loaded group of anchors

- $\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$, where:
  - $\Psi_{ecx,N} = \frac{1}{1+\frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$ 0.86 – modification factor that depends on eccentricity in x-direction
  - $e_{x,N} =$ 2"7/16 in – tension load eccentricity in x-direction
  - $\Psi_{ecy,N} = \frac{1}{1+\frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$ 0.82 – modification factor that depends on eccentricity in y-direction
  - $e_{y,N} =$ 3"3/16 in – tension load eccentricity in y-direction
  - $h_{ef} =$ 9"13/16 in – depth of embedment

$\Psi_{c,N}$ = 1.00 – modification factor for concrete conditions

N_b = 52.18 kip – basic concrete breakout strength of a single anchor in tension:

- $N_b = k_c \cdot \lambda_a \cdot \sqrt{f'_c} \cdot h_{ef}^{1.5}$, where:
  - $k_c =$ 24.0 – coefficient for cast-in anchors
  - $\lambda_a =$ 1.00 – modification factor for lightweight concrete
  - $f'_c =$ 5.0 ksi – concrete compressive strength
  - $h_{ef} =$ 9"13/16 in – depth of embedment

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 15.63 kip ≥ $V =$ 1.68 kip

Where:

$\phi = 0.65$ – resistance factor

$A_{se,V} = 0.3340 \text{ in}^2$ – tensile stress area

$f_{uta} = 120.0$ ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125 \text{ ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
  - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete shear breakout check (ACI 318-14 – 17.5.2)

The check is performed for group of anchors that form common shear breakout cone: A4, A5

$\phi V_{cbg} = \phi \cdot \frac{A_{Vc}}{A_{Vc0}} \cdot \Psi_{ec,V} \cdot \Psi_{ed,V} \cdot \Psi_{c,V} \cdot \Psi_{h,V} \cdot \Psi_{\alpha,V} \cdot V_{br} =$ 32.65 kip ≥ $V_g =$ 2.15 kip

Where:

V_g = 2.15 kip – sum of shear forces of anchors on common base plate

ϕ = 0.65 – resistance factor

A_{Vc} = 881.4674 in^2 – projected concrete failure area of an anchor or group of anchors

A_{Vc0} = 980.8613 in^2 – projected concrete failure area of one anchor when not limited by corner influences, spacing or member thickness

$\Psi_{ec,V}$ = 0.90 – modification factor for anchor groups loaded eccentrically in shear:

- $\Psi_{ec,V} = \frac{1}{1+\frac{2 \cdot e_V'}{3 \cdot c_a}}$, where:
  - e_V' = 2"9/16 in – shear load eccentricity
  - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load

$\Psi_{ed,V}$ = 0.89 – modification factor for edge effect:

- $\Psi_{ed,V} = 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_a} \leq 1$, where:
  - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
  - c_{a2} = 1'-1"3/4 in – edge distance in direction perpendicular to the load

$\Psi_{c,V}$ = 1.00 – modification factor for concrete conditions

$\Psi_{h,V}$ = 1.00 – modification factor for anchors located in a shallow concrete member:

- $\Psi_{h,V} = \sqrt{\frac{1.5 \cdot c_{a1}}{h_a}} \geq 1$, where:
  - h_a = 1'-11"5/8 in – thickness of member in which an anchor is anchored measured parallel to anchor axis

$\Psi_{\alpha,V}$ = 1.98 – modification factor for anchors loaded at an angle with the concrete edge

- $\Psi_{\alpha,V} = \sqrt{\frac{1}{(\cos \alpha_V)^2+(0.5 \cdot \sin \alpha_V)^2}}$, where:
  - α_V = 85.6 ° – angle between direction of shear force and direction perpendicular to concrete edge

V_b = 35.47 kip – basic concrete breakout strength of a single anchor in shear:

- $V_b = \min(7.0 \cdot (\frac{l_e}{d})^{0.2} \cdot \lambda_a \cdot \sqrt{d_a} \cdot \sqrt{f'_c} \cdot c_a^{1.5}, 9.0 \cdot \lambda_a \cdot \sqrt{f'_c} \cdot c_a^{1.5})$, where:
    - l_e = 6" in – effective length
    - d_a = 3/4 in – anchor diameter
    - λ_a = 1.00 – modification factor for lightweight concrete
    - f'_c = 5.0 ksi – concrete compressive strength
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
        - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} =$ 109.48 kip ≥ $V_g =$ 4.53 kip

Where:

ϕ = 0.65 – resistance factor

k_{cp} = 2.00 – concrete pry-out factor

N_{cp} = 84.22 kip – concrete cone tension break-out resistance in case all anchors are in tension

V_g = 4.53 kip – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

$U_{tt}{}^{5/3} + U_{ts}{}^{5/3} =$ 0.58 ≤ 1.0

Where:

U_{tt} = 0.70 – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

U_{ts} = 0.11 – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Detailed result for A5

Following checks of anchors loaded in tension are not provided and should be checked using information in relevant Technical Product Specification (based on the 5 percent fractile of tests performed and evaluated according to ACI 355.2):

- *Pull-out failure of fastener (for post-installed mechanical anchors) - ACI 318-19 – 17.6.3*
- *Bond strength of adhesive anchor (for post-installed bonded anchors) - ACI 318-19 – 17.6.5*
- *Concrete splitting failure during installation should be evaluated by ACI 355.2 requirements*

Concrete blow-out failure is provided only for anchors with washer plates.

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} =$ 28.06 kip ≥ $N_f =$ 14.57 kip

Where:

ϕ = 0.70 – resistance factor

$A_{se,N}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125 \text{ ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - f_{ya} = 92.0 ksi – specified yield strength of anchor steel
  - f_u = 120.0 ksi – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)

The check is performed for group of anchors that form common tension breakout cone: A2, A3, A4, A5

$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot N_b =$ 41.60 kip ≥ $N_{fg} =$ 29.26 kip

Where:

N_{fg} = 29.26 kip – sum of tension forces of anchors with common concrete breakout cone area

ϕ = 0.70 – resistance factor

A_{Nc} = 1436.0097 in^2 – concrete breakout cone area for group of anchors

A_{Nc0} = 871.8767 in^2 – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N}$ = 0.98 – modification factor for edge distance:

- $\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$, where:
  - $c_{a,min} =$ 1'-1"3/4 in – minimum distance from the anchor to the edge
  - $h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$ 9"13/16 in – depth of embedment, where:
    - $h_{emb} =$ 1'-1"3/4 in – anchor length
    - $c_{a,max} =$ 1'-2"3/4 in – maximum distance from the anchor to one of the three closest edges
    - $s =$ 5"1/8 in – maximum spacing between anchors

$\Psi_{ec,N}$ = 0.71 – modification factor for eccentrically loaded group of anchors

- $\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$, where:
  - $\Psi_{ecx,N} = \frac{1}{1 + \frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$ 0.86 – modification factor that depends on eccentricity in x-direction
  - $e_{x,N} =$ 2"7/16 in – tension load eccentricity in x-direction
  - $\Psi_{ecy,N} = \frac{1}{1 + \frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$ 0.82 – modification factor that depends on eccentricity in y-direction
  - $e_{y,N} =$ 3"3/16 in – tension load eccentricity in y-direction
  - $h_{ef} =$ 9"13/16 in – depth of embedment

$\Psi_{c,N}$ = 1.00 – modification factor for concrete conditions

N_b = 52.18 kip – basic concrete breakout strength of a single anchor in tension:

- $N_b = k_c \cdot \lambda_a \cdot \sqrt{f'_c} \cdot h_{ef}^{1.5}$, where:
  - $k_c =$ 24.0 – coefficient for cast-in anchors
  - $\lambda_a =$ 1.00 – modification factor for lightweight concrete
  - $f'_c =$ 5.0 ksi – concrete compressive strength
  - $h_{ef} =$ 9"13/16 in – depth of embedment

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 15.63 kip $\geq$ $V =$ 1.79 kip

Where:

$\phi = 0.65$ – resistance factor

$A_{se,V} = 0.3340\ \text{in}^2$ – tensile stress area

$f_{uta} = 120.0$ ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \text{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
    - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
    - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete shear breakout check (ACI 318-14 – 17.5.2)

The check is performed for group of anchors that form common shear breakout cone: A4, A5

$\phi V_{cbg} = \phi \cdot \frac{A_{Vc}}{A_{Vc0}} \cdot \Psi_{ec,V} \cdot \Psi_{ed,V} \cdot \Psi_{c,V} \cdot \Psi_{h,V} \cdot \Psi_{\alpha,V} \cdot V_{br} =$ 32.65 kip $\geq$ $V_g =$ 2.15 kip

Where:

V_g = 2.15 kip – sum of shear forces of anchors on common base plate

ϕ = 0.65 – resistance factor

A_{Vc} = 881.4674 in^2 – projected concrete failure area of an anchor or group of anchors

A_{Vc0} = 980.8613 in^2 – projected concrete failure area of one anchor when not limited by corner influences, spacing or member thickness

$\Psi_{ec,V}$ = 0.90 – modification factor for anchor groups loaded eccentrically in shear:

- $\Psi_{ec,V} = \frac{1}{1+\frac{2 \cdot e'_V}{3 \cdot c_a}}$, where:
  - e'_V = 2"9/16 in – shear load eccentricity
  - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load

$\Psi_{ed,V}$ = 0.89 – modification factor for edge effect:

- $\Psi_{ed,V} = 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_a} \leq 1$, where:
  - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
  - c_{a2} = 1'-1"3/4 in – edge distance in direction perpendicular to the load

$\Psi_{c,V}$ = 1.00 – modification factor for concrete conditions

$\Psi_{h,V}$ = 1.00 – modification factor for anchors located in a shallow concrete member:

- $\Psi_{h,V} = \sqrt{\frac{1.5 \cdot c_{a1}}{h_a}} \geq 1$, where:
  - h_a = 1'-11"5/8 in – thickness of member in which an anchor is anchored measured parallel to anchor axis

$\Psi_{\alpha,V}$ = 1.98 – modification factor for anchors loaded at an angle with the concrete edge

- $\Psi_{\alpha,V} = \sqrt{\frac{1}{(\cos \alpha_V)^2 + (0.5 \cdot \sin \alpha_V)^2}}$, where:
  - α_V = 85.6 ° – angle between direction of shear force and direction perpendicular to concrete edge

V_b = 35.47 kip – basic concrete breakout strength of a single anchor in shear:

- $V_b = \min(7.0 \cdot (\frac{l_e}{d})^{0.2} \cdot \lambda_a \cdot \sqrt{d_a} \cdot \sqrt{f_c'} \cdot c_a^{1.5}, 9.0 \cdot \lambda_a \cdot \sqrt{f_c'} \cdot c_a^{1.5})$, where:
  - l_e = 6" in – effective length
  - d_a = 3/4 in – anchor diameter
  - λ_a = 1.00 – modification factor for lightweight concrete
  - f_c' = 5.0 ksi – concrete compressive strength
  - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load
    - c_{a1} = 1'-2"3/4 in – edge distance in direction of the load

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} =$ 109.48 kip ≥ $V_g =$ 4.53 kip

Where:

ϕ = 0.65 – resistance factor

k_{cp} = 2.00 – concrete pry-out factor

N_{cp} = 84.22 kip – concrete cone tension break-out resistance in case all anchors are in tension

V_g = 4.53 kip – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

${U_{tt}}^{5/3} + {U_{ts}}^{5/3} =$ 0.58 ≤ 1.0

Where:

U_{tt} = 0.70 – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

U_{ts} = 0.11 – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
Plate 6	M55-w 1	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
Plate 6	M55-arc 1	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-arc 2	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-arc 3	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-w 2	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
Plate 6	M55-arc 4	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-arc 5	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-arc 6	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-w 3	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
Plate 6	M55-arc 7	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-arc 8	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-arc 9	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-w 4	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
Plate 6	M55-arc 10	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-arc 11	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	M55-arc 12	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
Plate 6	RIB1	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	9"3/8	9/16	LE8	1.12	5.09	22.1	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	9"3/8	9/16	LE8	0.78	6.66	11.7	OK	OK
M55-w 2	RIB1	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE8	2.71	3.92	69.0	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE8	2.99	3.98	75.1	OK	OK
Plate 6	RIB2	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	9"3/8	9/16	LE8	0.79	5.90	13.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	9"3/8	9/16	LE8	1.35	5.38	25.0	OK	OK
M55-w 2	RIB2	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	7"13/16	3/8	LE8	2.70	3.58	75.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	7"13/16	3/8	LE8	2.62	3.49	75.1	OK	OK
Plate 6	RIB3	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	9"7/16	9/16	LE10	0.35	6.08	5.7	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	9"3/8	9/16	LE10	0.50	4.74	10.6	OK	OK
M55-w 3	RIB3	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	7"13/16	3/8	LE10	1.26	3.45	36.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	7"13/16	3/8	LE10	1.12	3.52	31.7	OK	OK
Plate 6	RIB4	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	9"3/8	9/16	LE10	0.45	6.19	7.2	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	9"7/16	9/16	LE8	0.45	4.66	9.7	OK	OK
M55-w 3	RIB4	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE10	0.70	4.21	16.6	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE8	0.64	3.18	20.0	OK	OK

Design data

Material	F_{exx} [ksi]
E70xx	0.0
E70xx	70.0

Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
◢	Fillet weld
F_{exx}	Ultimate strength as rated by electrode classification number

Detailed result for Plate 6 / M55-w 1 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 1 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 3 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-w 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 5 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 6 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-w 3 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 7 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 8 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 9 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-w 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 10 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 11 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / M55-arc 12 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 6 / RIB1 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 5.09 kip ≥ $F_n =$ 1.12 kip

Where:

F_{nw} = 46.2 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 19.9° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1470 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Plate 6 / RIB1 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 6.66 kip ≥ $F_n =$ 0.78 kip

Where:

F_{nw} = 60.4 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 66.4° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1470 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for M55-w 2 / RIB1 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.92 kip ≥ $F_n =$ 2.71 kip

Where:

F_{nw} = 53.3 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 41.4° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0981 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for M55-w 2 / RIB1 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.98 kip ≥ $F_n =$ 2.99 kip

Where:

F_{nw} = 54.1 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 43.7° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0981 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Plate 6 / RIB2 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 5.90 kip ≥ $F_n =$ 0.79 kip

Where:

F_{nw} = 60.2 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 65.5° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1306 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Plate 6 / RIB2 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 5.38 kip ≥ $F_n =$ 1.35 kip

Where:

F_{nw} = 54.9 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 46.3° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1306 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for M55-w 2 / RIB2 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.58 kip ≥ $F_n =$ 2.70 kip

Where:

F_{nw} = 54.7 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 45.7° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0872 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for M55-w 2 / RIB2 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.49 kip ≥ $F_n =$ 2.62 kip

Where:

F_{nw} = 53.3 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 41.5° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0872 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Plate 6 / RIB3 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 6.08 kip ≥ $F_n =$ 0.35 kip

Where:

F_{nw} = 61.8 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 74.1° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1311 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Plate 6 / RIB3 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 4.74 kip ≥ $F_n =$ 0.50 kip

Where:

F_{nw} = 48.3 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 26.8° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1306 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for M55-w 3 / RIB3 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.45 kip ≥ $F_n =$ 1.26 kip

Where:

F_{nw} = 52.8 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 39.8° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0872 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for M55-w 3 / RIB3 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.52 kip ≥ $F_n =$ 1.12 kip

Where:

F_{nw} = 53.7 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 42.7° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0872 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Plate 6 / RIB4 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 6.19 kip ≥ $F_n =$ 0.45 kip

Where:

F_{nw} = 56.2 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 50.3° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1470 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Plate 6 / RIB4 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 4.66 kip ≥ $F_n =$ 0.45 kip

Where:

F_{nw} = 42.1 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 1.7° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1475 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for M55-w 3 / RIB4 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 4.21 kip ≥ $F_n =$ 0.70 kip

Where:

F_{nw} = 57.2 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 53.6° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0981 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for M55-w 3 / RIB4 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.18 kip ≥ $F_n =$ 0.64 kip

Where:

F_{nw} = 43.2 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 8.5° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0981 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Concrete block

Item	Loads	A_1 [in^2]	A_2 [in^2]	σ [ksi]	$\Phi f_{p,max}$ [ksi]	Ut [%]	Status
CB 1	LE8	31.7069	917.7310	0.7	5.5	12.3	OK

Symbol explanation

A_1	Loaded area
A_2	Supporting area
σ	Average stress in concrete
$\phi f_{p,max}$	Concrete bearing resistance
Ut	Utilization

Detailed result for CB 1

Concrete block compressive resistance check (AISC 360-16 – J8)

$\phi_c f_{p,max} =$ 5.5 ksi ≥ $\sigma =$ 0.7 ksi

Where:

$f_{p,max}$ = 8.5 ksi – concrete block design bearing strength:

- $f_{p,max} = 0.85 \cdot f'_c \cdot \sqrt{\frac{A_2}{A_1}} \leq 1.7 \cdot f'_c$, where:
  - $f'_c =$ 5.0 ksi – concrete compressive strength
  - $A_1 =$ 31.7069 in^2 – base plate area in contact with concrete surface
  - $A_2 =$ 917.7310 in^2 – concrete supporting surface

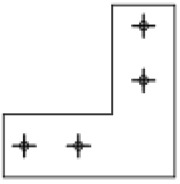
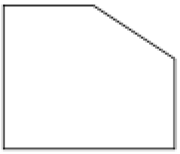
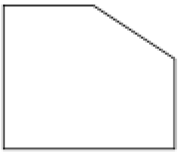
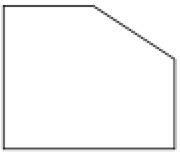
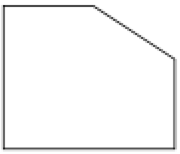
ϕ_c = 0.65 – resistance factor for concrete

Buckling

Buckling analysis was not calculated.

Bill of material

Manufacturing operations

Name	Plates [in]	Shape	Nr.	Welds Leg size [in]	Length [in]	Bolts	Nr.
Plate 6	P13/16x1'-4"1/8-1'-4"1/8 (A913 Gr.50)		1			3/4 A325	4
RIB1	P9/16x9"7/16-7"7/8 (A913 Gr.50)		1	Double fillet: 3/8	1'-5"5/16		
RIB2	P9/16x9"7/16-7"7/8 (A913 Gr.50)		1	Double fillet: 5/16	1'-5"5/16		
RIB3	P9/16x9"7/16-7"7/8 (A913 Gr.50)		1	Double fillet: 5/16	1'-5"5/16		
RIB4	P9/16x9"7/16-7"7/8 (A913 Gr.50)		1	Double fillet: 3/8	1'-5"5/16		

Welds

Type	Material	Throat thickness [in]	Leg size [in]	Length [in]
Butt	E70xx	-	-	1'-7"7/8
Double fillet	E70xx	1/4	3/8	2'-10"5/8
Double fillet	E70xx	1/4	5/16	2'-10"5/8

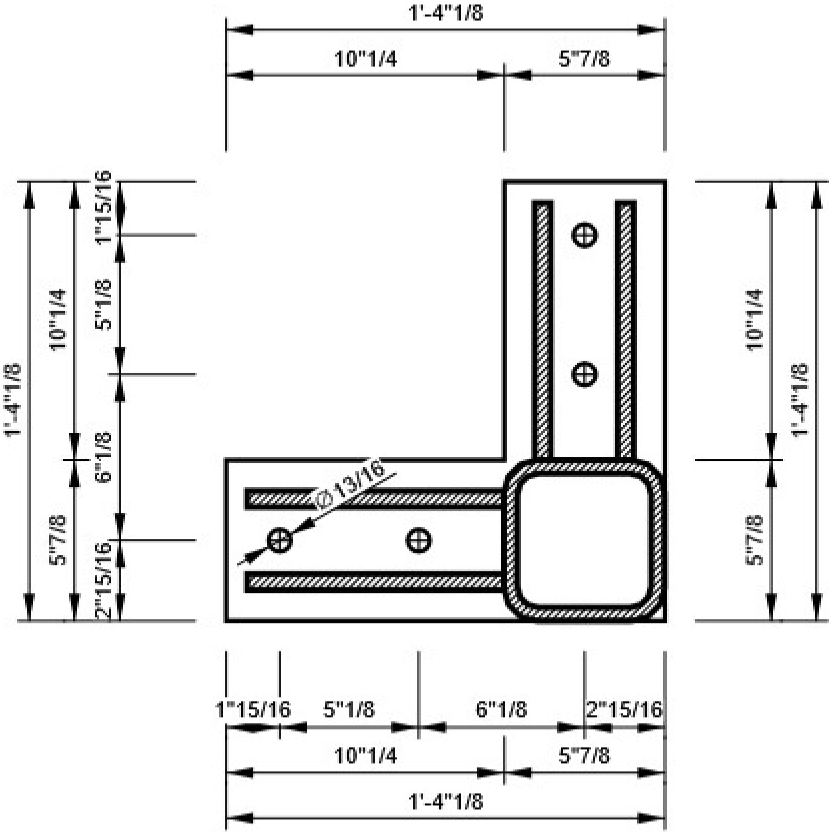
Anchors

Name	Length [in]	Drill length [in]	Count
3/4 A325	1'-2"9/16	1'-1"3/4	4

Drawing

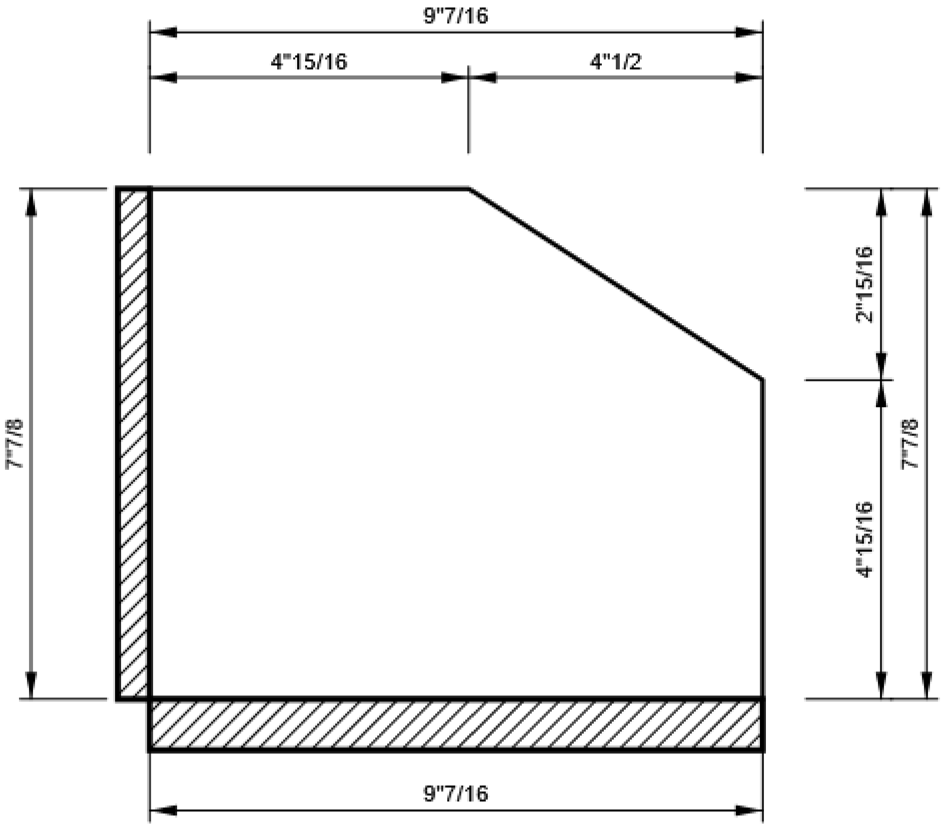
Plate 6

P13/16x1'-4"1/8-1'-4"1/8 (A913 Gr.50)



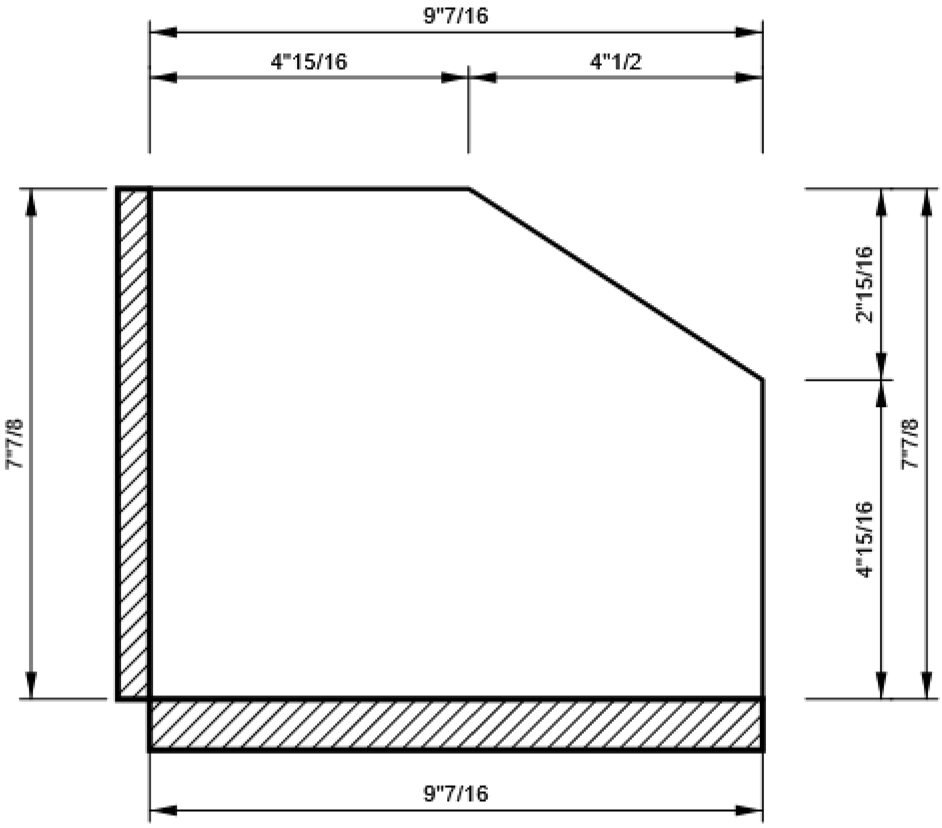
RIB1

P9/16x7"7/8-9"7/16 (A913 Gr.50)



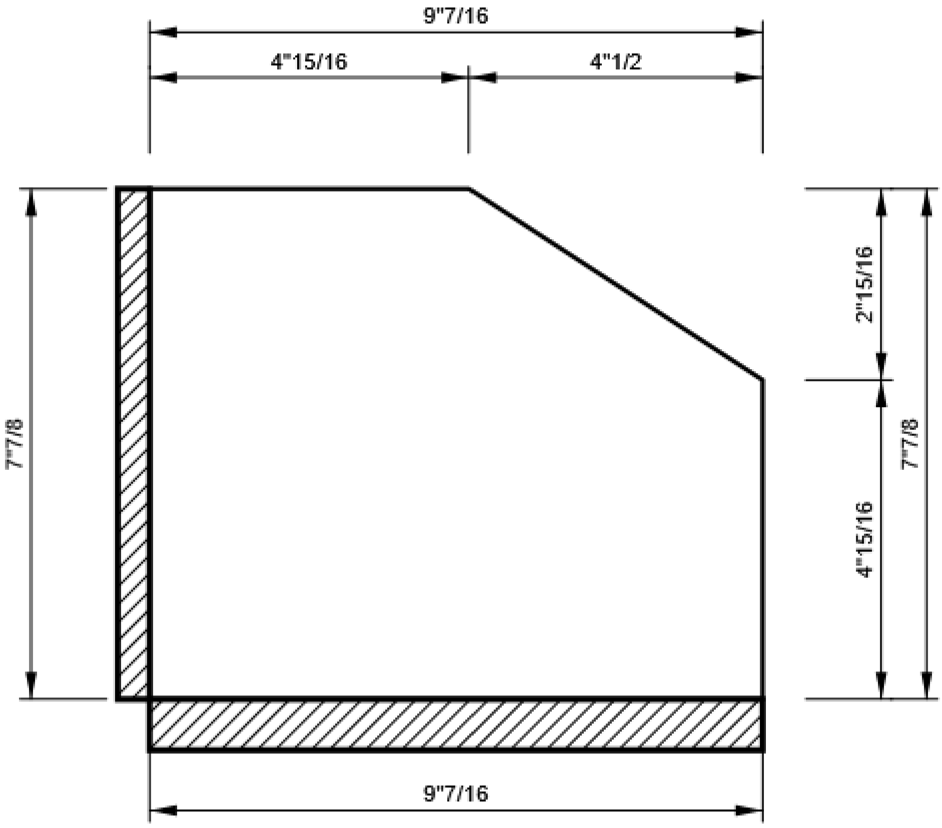
RIB2

P9/16x7"7/8-9"7/16 (A913 Gr.50)



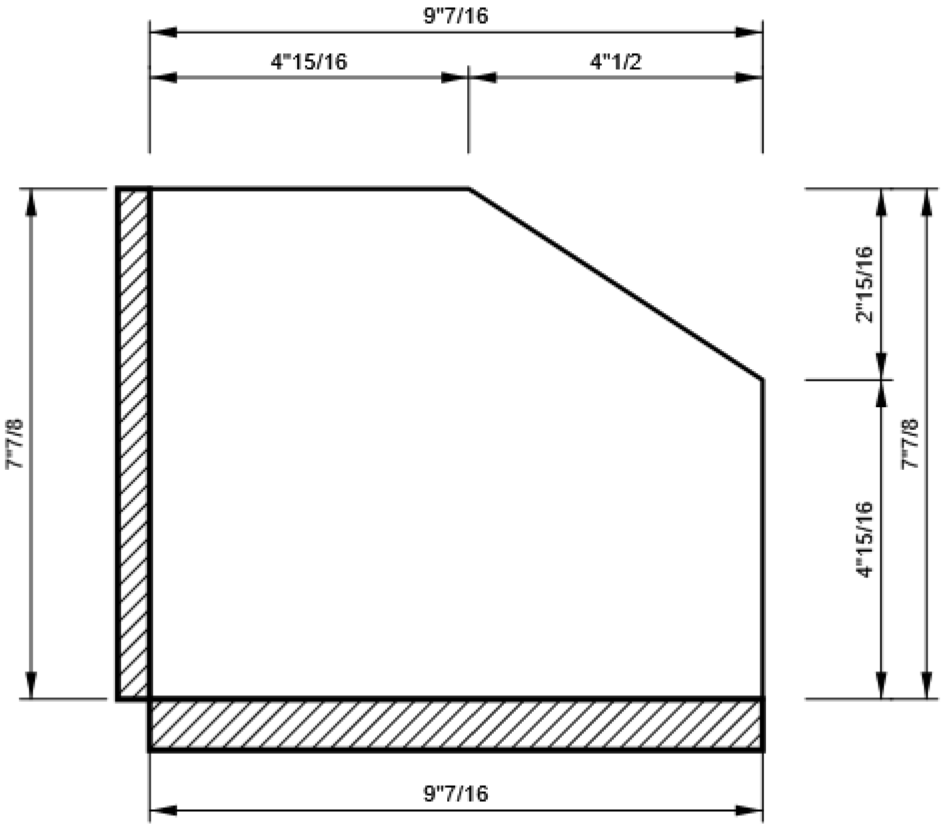
RIB3

P9/16x7"7/8-9"7/16 (A913 Gr.50)



RIB4

P9/16x7"7/8-9"7/16 (A913 Gr.50)



Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Appendix- 11

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	SK/MS
Description	Straight Base Plate
Date	09-Feb-24
Code	AISC/ACI

Material

Steel	A913 Gr.50
Concrete	5000 psi

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Straight Base Plate
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

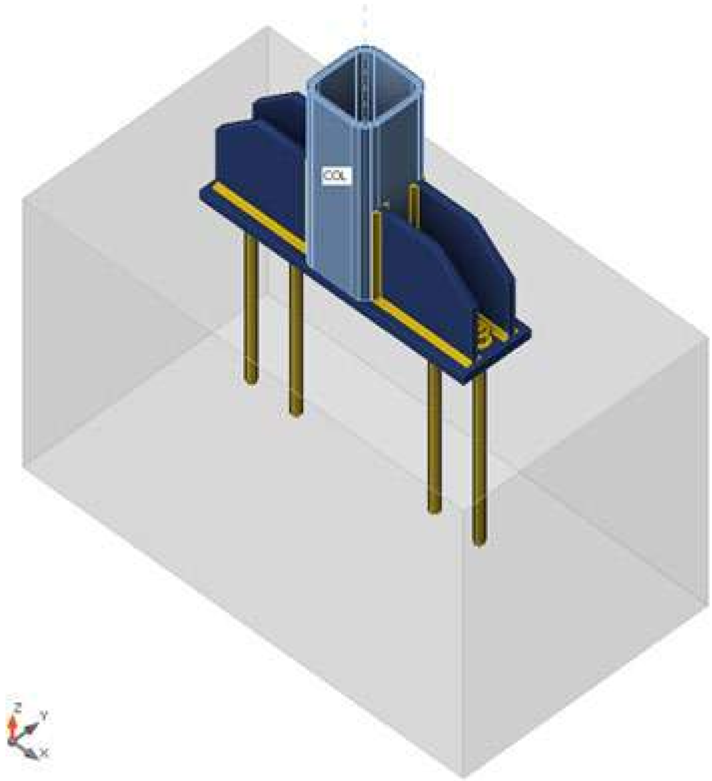
Members

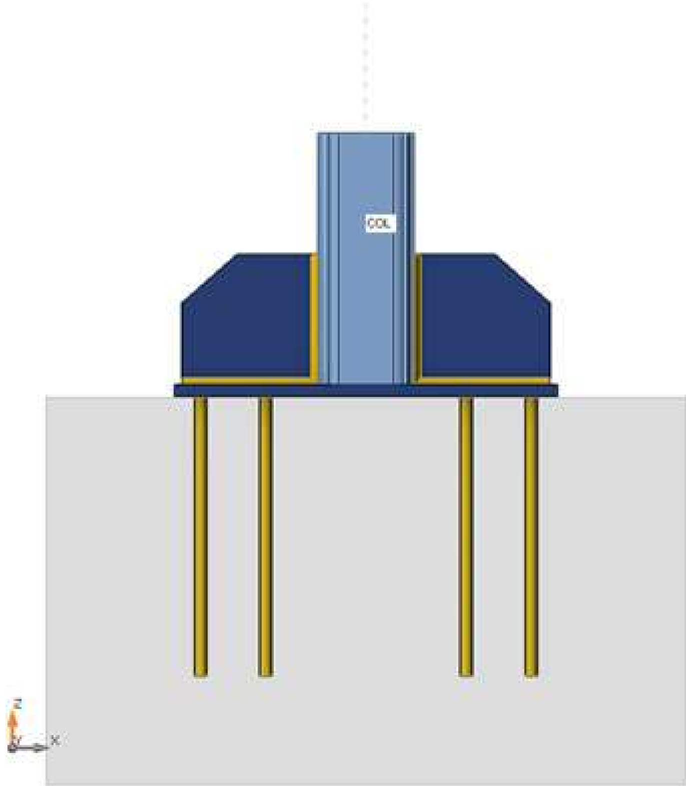
Geometry

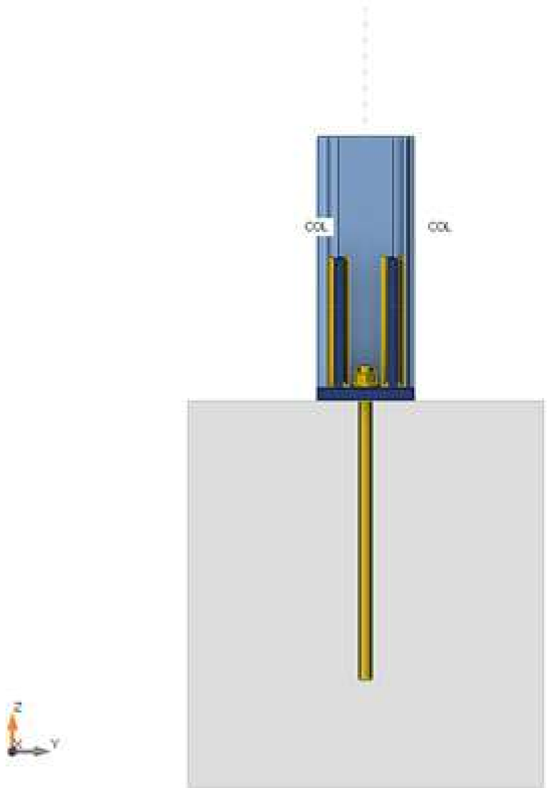
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
COL	3 - SHS150/150/12.5	0.0	90.0	0.0	0"	0"	0"

Supports and forces

Name	Support	Forces in	X [in]
COL / end		Node	0"



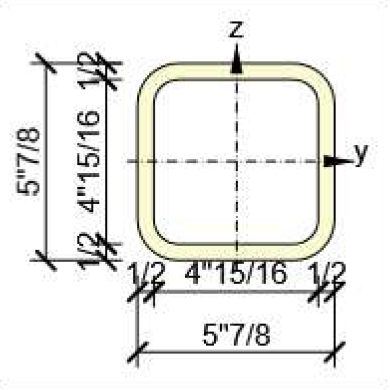




Cross-sections

Name	Material
3 - SHS150/150/12.5	A913 Gr.50

Cross-sections

Name	Material	Drawing
3 - SHS150/150/12.5	A913 Gr.50	

Anchors

Name	Bolt assembly	Diameter [in]	f_u [ksi]	Gross area [in^2]
3/4 A325	3/4 A325	3/4	120.0	0.4418

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	COL / End	21.55	0.00	-0.06	0.08	0.69	0.00
LE2	COL / End	14.84	0.00	-0.88	0.23	9.60	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.06	0.00	21.55	0.00	0.69	0.08
LE2	0.88	0.00	14.84	0.00	9.60	0.23

Foundation block

Item	Value	Unit
CB 1		
Dimensions	1'-9"5/8 x 3'-3"	in
Depth	1'-11"5/8	in
Anchor	3/4 A325	
Anchoring length	1'-4"15/16	in
Shear force transfer	Anchors	

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Loc. deformation	0.0 < 3%	OK
Anchors	56.3 < 100%	OK
Welds	44.1 < 100%	OK
Concrete block	Not calculated	
Buckling	Not calculated	

Plates

Name	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
COL	1/2	LE2	19.4	0.0	0.0	OK
BP1	13/16	LE2	12.1	0.0	0.0	OK
RIB1a	9/16	LE1	7.9	0.0	0.0	OK
RIB1b	9/16	LE1	7.8	0.0	0.0	OK
RIB2a	9/16	LE2	14.0	0.0	0.0	OK
RIB2b	9/16	LE2	13.8	0.0	0.0	OK

Design data

Material	F_y [ksi]	ε_{lim} [%]
A913 Gr.50	50.0	5.0

Symbol explanation

t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain

Detailed result for COL

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for BP1

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for RIB1a

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for RIB1b

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for RIB2a

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Detailed result for RIB2b

Design values used in the analysis

$\phi F_y =$ 45.0 ksi

Where:

F_y = 50.0 ksi – characteristic yield strength

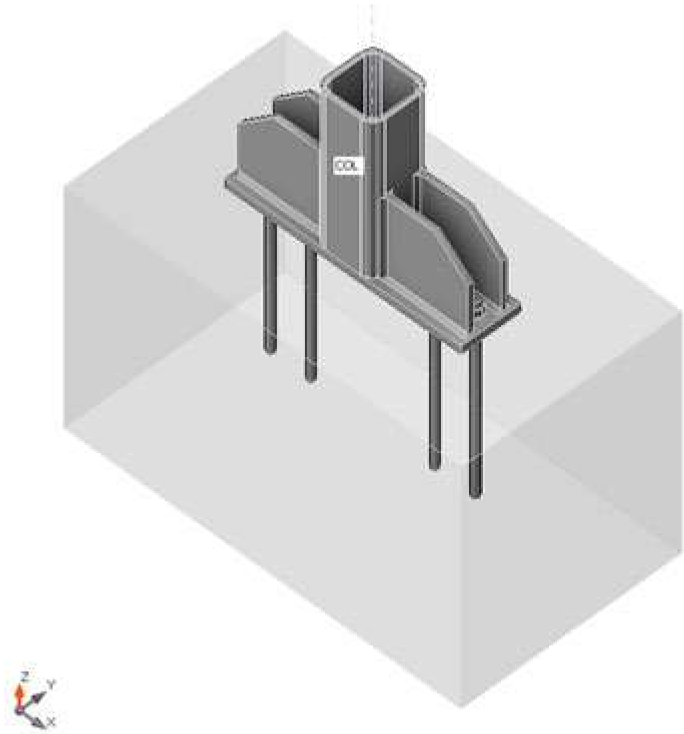
ϕ = 0.90 – resistance factor for steel material AISC 360-16 – B3.1

Loc. deformation

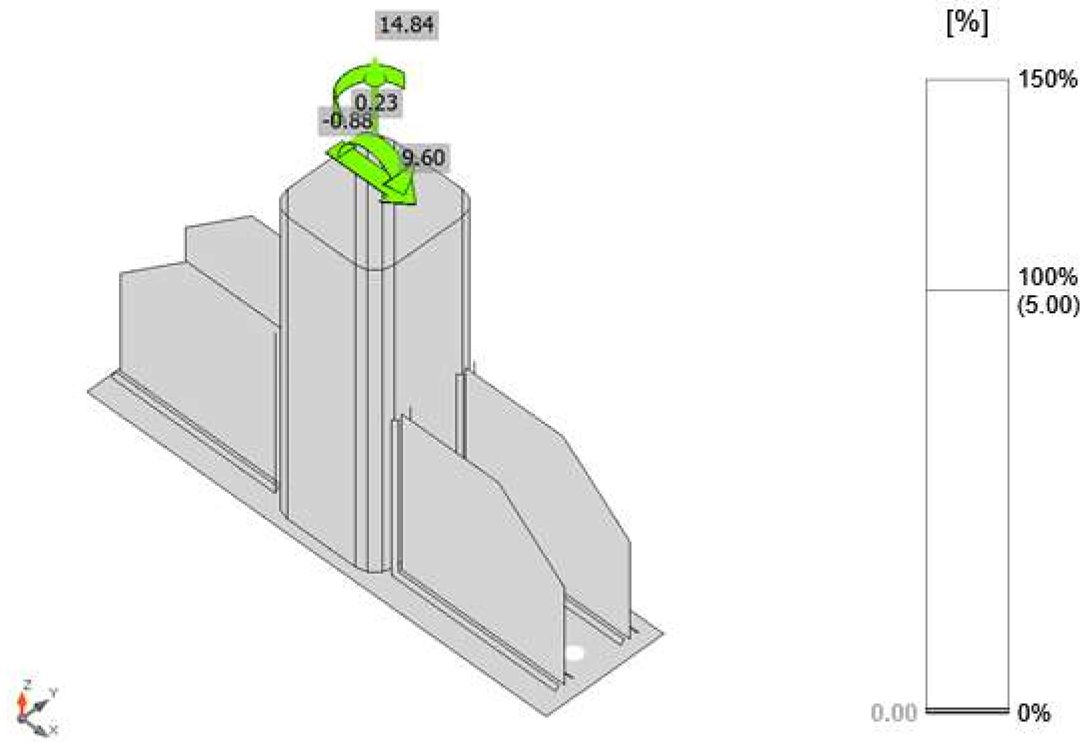
Name	d_0 [in]	Loads	δ [in]	$δ_{lim}$ [in]	$δ/d_0$ [%]	Check status
COL	5"7/8	LE1		3/16	0.0	OK

Symbol explanation

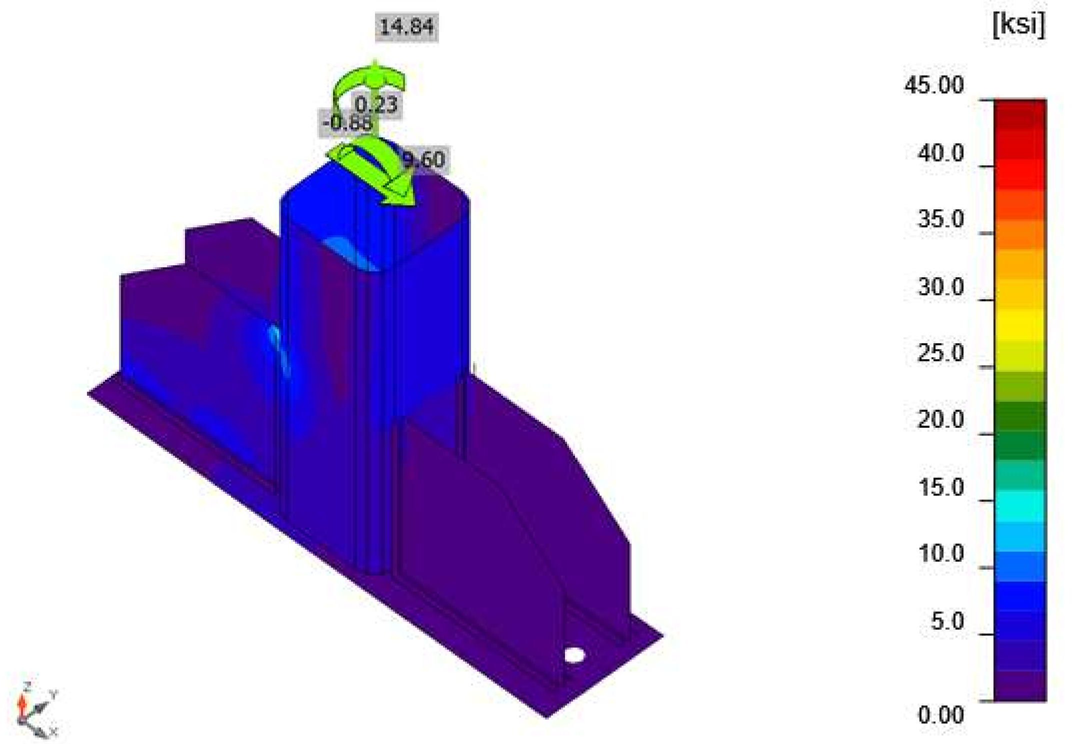
d_0	Cross-section size
δ	Local cross-section deformation
$δ_{lim}$	Allowed deformation



Overall check, LE2

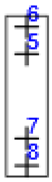


Strain check, LE2



Equivalent stress, LE2

Anchors

Shape	Item	Loads	N_f [kip]	V [kip]	ϕN_{cbg} [kip]	ϕV_{cbg} [kip]	ϕV_{cp} [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	A5	LE1	6.76	0.17	38.29	-	73.61	56.3	1.1	38.4	OK	OK
	A6	LE1	4.51	0.19	38.29	-	73.61	56.3	1.2	38.4	OK	OK
	A7	LE1	6.35	0.14	38.29	-	73.61	56.3	0.9	38.4	OK	OK
	A8	LE1	3.94	0.16	38.29	8.45	73.61	56.3	1.0	38.4	OK	OK

Design data

Grade	ϕN_{sa} [kip]	ϕV_{sa} [kip]
3/4 A325 - 1	28.06	15.63

Symbol explanation

N_f	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
ϕN_{cbg}	Concrete breakout strength in tension – ACI 318-14 – 17.4.2
ϕV_{cbg}	Concrete breakout strength in shear – ACI 318-14 – 17.5.2
ϕV_{cp}	Concrete pryout strength in shear – ACI 318-14 – 17.5.3
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
ϕN_{sa}	Steel strength of anchor in tension - ACI 318-14 – 17.4.1
ϕV_{sa}	Steel strength of anchor in shear - ACI 318-14 – 17.5.1

Detailed result for A5

Following checks of anchors loaded in tension are not provided and should be checked using information in relevant Technical Product Specification (based on the 5 percent fractile of tests performed and evaluated according to ACI 355.2):

- *Pull-out failure of fastener (for post-installed mechanical anchors) - ACI 318-19 – 17.6.3*
- *Bond strength of adhesive anchor (for post-installed bonded anchors) - ACI 318-19 – 17.6.5*
- *Concrete splitting failure during installation should be evaluated by ACI 355.2 requirements*

Concrete blow-out failure is provided only for anchors with washer plates.

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} =$ 28.06 kip $\geq$ $N_f =$ 6.76 kip

Where:

$\phi = 0.70$ – resistance factor

$A_{se,N} = 0.3340$ in^2 – tensile stress area

$f_{uta} = 120.0$ ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125 \text{ ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
    - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
    - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)

The check is performed for group of anchors that form common tension breakout cone: A5, A6, A7, A8

$$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot N_b = \quad 38.29 \text{ kip} \geq N_{fg} = \quad 21.55 \text{ kip}$$

Where:

N_{fg} = 21.55 kip – sum of tension forces of anchors with common concrete breakout cone area

ϕ = 0.70 – resistance factor

A_{Nc} = 842.4422 in^2 – concrete breakout cone area for group of anchors

A_{Nc0} = 468.8714 in^2 – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N}$ = 0.96 – modification factor for edge distance:

- $\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$, where:
  - $c_{a,min} =$ 9"7/16 in – minimum distance from the anchor to the edge
  - $h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$ 7"3/16 in – depth of embedment, where:
    - $h_{emb} =$ 1'-4"15/16 in – anchor length
    - $c_{a,max} =$ 10"13/16 in – maximum distance from the anchor to one of the three closest edges
    - $s =$ 3"15/16 in – maximum spacing between anchors

$\Psi_{ec,N}$ = 0.97 – modification factor for eccentrically loaded group of anchors

- $\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$, where:
  - $\Psi_{ecx,N} = \frac{1}{1+\frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$ 1.00 – modification factor that depends on eccentricity in x-direction
  - $e_{x,N} =$ 0" in – tension load eccentricity in x-direction
  - $\Psi_{ecy,N} = \frac{1}{1+\frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$ 0.97 – modification factor that depends on eccentricity in y-direction
  - $e_{y,N} =$ 3/8 in – tension load eccentricity in y-direction
  - $h_{ef} =$ 7"3/16 in – depth of embedment

$\Psi_{c,N}$ = 1.00 – modification factor for concrete conditions

N_b = 32.77 kip – basic concrete breakout strength of a single anchor in tension:

- $N_b = k_c \cdot \lambda_a \cdot \sqrt{f_c'} \cdot h_{ef}^{1.5}$, where:
  - $k_c =$ 24.0 – coefficient for cast-in anchors
  - $\lambda_a =$ 1.00 – modification factor for lightweight concrete
  - $f_c' =$ 5.0 ksi – concrete compressive strength
  - $h_{ef} =$ 7"3/16 in – depth of embedment

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 15.63 kip ≥ $V =$ 0.17 kip

Where:

ϕ = 0.65 – resistance factor

$A_{se,V}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \text{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - f_{ya} = 92.0 ksi – specified yield strength of anchor steel
  - f_u = 120.0 ksi – specified ultimate strength of anchor steel

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} =$ 73.61 kip ≥ $V_g =$ 0.07 kip

Where:

ϕ = 0.65 – resistance factor

k_{cp} = 2.00 – concrete pry-out factor

N_{cp} = 56.62 kip – concrete cone tension break-out resistance in case all anchors are in tension

V_g = 0.07 kip – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

$U_{tt}^{5/3} + U_{ts}^{5/3} =$ 0.38 ≤ 1.0

Where:

U_{tt} = 0.56 – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

U_{ts} = 0.01 – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Detailed result for A6

Following checks of anchors loaded in tension are not provided and should be checked using information in relevant Technical Product Specification (based on the 5 percent fractile of tests performed and evaluated according to ACI 355.2):

- *Pull-out failure of fastener (for post-installed mechanical anchors) - ACI 318-19 – 17.6.3*
- *Bond strength of adhesive anchor (for post-installed bonded anchors) - ACI 318-19 – 17.6.5*
- *Concrete splitting failure during installation should be evaluated by ACI 355.2 requirements*

Concrete blow-out failure is provided only for anchors with washer plates.

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} =$ 28.06 kip ≥ $N_f =$ 4.51 kip

Where:

$\phi = 0.70$ – resistance factor

$A_{se,N} = 0.3340$ in^2 – tensile stress area

$f_{uta} = 120.0$ ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \text{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
  - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)

The check is performed for group of anchors that form common tension breakout cone: A5, A6, A7, A8

$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot N_b =$ 38.29 kip ≥ $N_{fg} =$ 21.55 kip

Where:

N_{fg} = 21.55 kip – sum of tension forces of anchors with common concrete breakout cone area

ϕ = 0.70 – resistance factor

A_{Nc} = 842.4422 in^2 – concrete breakout cone area for group of anchors

A_{Nc0} = 468.8714 in^2 – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N}$ = 0.96 – modification factor for edge distance:

- $\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$, where:
    - $c_{a,min} =$ 9"7/16 in – minimum distance from the anchor to the edge
    - $h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$ 7"3/16 in – depth of embedment, where:
        - $h_{emb} =$ 1'-4"15/16 in – anchor length
        - $c_{a,max} =$ 10"13/16 in – maximum distance from the anchor to one of the three closest edges
        - $s =$ 3"15/16 in – maximum spacing between anchors

$\Psi_{ec,N}$ = 0.97 – modification factor for eccentrically loaded group of anchors

- $\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$, where:
    - $\Psi_{ecx,N} = \frac{1}{1+\frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$ 1.00 – modification factor that depends on eccentricity in x-direction
    - $e_{x,N} =$ 0" in – tension load eccentricity in x-direction
    - $\Psi_{ecy,N} = \frac{1}{1+\frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$ 0.97 – modification factor that depends on eccentricity in y-direction
    - $e_{y,N} =$ 3/8 in – tension load eccentricity in y-direction
    - $h_{ef} =$ 7"3/16 in – depth of embedment

$\Psi_{c,N}$ = 1.00 – modification factor for concrete conditions

N_b = 32.77 kip – basic concrete breakout strength of a single anchor in tension:

- $N_b = k_c \cdot \lambda_a \cdot \sqrt{f'_c} \cdot h_{ef}^{1.5}$, where:
    - $k_c =$ 24.0 – coefficient for cast-in anchors
    - $\lambda_a =$ 1.00 – modification factor for lightweight concrete
    - $f'_c =$ 5.0 ksi – concrete compressive strength
    - $h_{ef} =$ 7"3/16 in – depth of embedment

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 15.63 kip ≥ $V =$ 0.19 kip

Where:

ϕ = 0.65 – resistance factor

$A_{se,V}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\text{ ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
  - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} =$ 73.61 kip ≥ $V_g =$ 0.07 kip

Where:

ϕ = 0.65 – resistance factor

k_{cp} = 2.00 – concrete pry-out factor

N_{cp} = 56.62 kip – concrete cone tension break-out resistance in case all anchors are in tension

V_g = 0.07 kip – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

$U_{tt}^{5/3} + U_{ts}^{5/3} =$ 0.38 ≤ 1.0

Where:

U_{tt} = 0.56 – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

U_{ts} = 0.01 – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Detailed result for A7

Following checks of anchors loaded in tension are not provided and should be checked using information in relevant Technical Product Specification (based on the 5 percent fractile of tests performed and evaluated according to ACI 355.2):

- *Pull-out failure of fastener (for post-installed mechanical anchors) - ACI 318-19 – 17.6.3*
- *Bond strength of adhesive anchor (for post-installed bonded anchors) - ACI 318-19 – 17.6.5*
- *Concrete splitting failure during installation should be evaluated by ACI 355.2 requirements*

Concrete blow-out failure is provided only for anchors with washer plates.

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} =$ 28.06 kip ≥ $N_f =$ 6.35 kip

Where:

$\phi = 0.70$ – resistance factor

$A_{se,N} = 0.3340\ \text{in}^2$ – tensile stress area

$f_{uta} = 120.0$ ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \text{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
    - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
    - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)

The check is performed for group of anchors that form common tension breakout cone: A5, A6, A7, A8

$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot N_b =$ 38.29 kip ≥ $N_{fg} =$ 21.55 kip

Where:

N_{fg} = 21.55 kip – sum of tension forces of anchors with common concrete breakout cone area

ϕ = 0.70 – resistance factor

A_{Nc} = 842.4422 in^2 – concrete breakout cone area for group of anchors

A_{Nc0} = 468.8714 in^2 – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N}$ = 0.96 – modification factor for edge distance:

- $\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$, where:
  - $c_{a,min} =$ 9"7/16 in – minimum distance from the anchor to the edge
  - $h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$ 7"3/16 in – depth of embedment, where:
    - $h_{emb} =$ 1'-4"15/16 in – anchor length
    - $c_{a,max} =$ 10"13/16 in – maximum distance from the anchor to one of the three closest edges
    - $s =$ 3"15/16 in – maximum spacing between anchors

$\Psi_{ec,N}$ = 0.97 – modification factor for eccentrically loaded group of anchors

- $\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$, where:
  - $\Psi_{ecx,N} = \frac{1}{1 + \frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$ 1.00 – modification factor that depends on eccentricity in x-direction
  - $e_{x,N} =$ 0" in – tension load eccentricity in x-direction
  - $\Psi_{ecy,N} = \frac{1}{1 + \frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$ 0.97 – modification factor that depends on eccentricity in y-direction
  - $e_{y,N} =$ 3/8 in – tension load eccentricity in y-direction
  - $h_{ef} =$ 7"3/16 in – depth of embedment

$\Psi_{c,N}$ = 1.00 – modification factor for concrete conditions

N_b = 32.77 kip – basic concrete breakout strength of a single anchor in tension:

- $N_b = k_c \cdot \lambda_a \cdot \sqrt{f'_c} \cdot h_{ef}^{1.5}$, where:
  - $k_c =$ 24.0 – coefficient for cast-in anchors
  - $\lambda_a =$ 1.00 – modification factor for lightweight concrete
  - $f'_c =$ 5.0 ksi – concrete compressive strength
  - $h_{ef} =$ 7"3/16 in – depth of embedment

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 15.63 kip ≥ $V =$ 0.14 kip

Where:

ϕ = 0.65 – resistance factor

$A_{se,V}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \text{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
  - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} =$ 73.61 kip ≥ $V_g =$ 0.07 kip

Where:

ϕ = 0.65 – resistance factor

k_{cp} = 2.00 – concrete pry-out factor

N_{cp} = 56.62 kip – concrete cone tension break-out resistance in case all anchors are in tension

V_g = 0.07 kip – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

$U_{tt}^{5/3} + U_{ts}^{5/3} =$ 0.38 ≤ 1.0

Where:

U_{tt} = 0.56 – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

U_{ts} = 0.01 – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Detailed result for A8

Following checks of anchors loaded in tension are not provided and should be checked using information in relevant Technical Product Specification (based on the 5 percent fractile of tests performed and evaluated according to ACI 355.2):

- *Pull-out failure of fastener (for post-installed mechanical anchors) - ACI 318-19 – 17.6.3*
- *Bond strength of adhesive anchor (for post-installed bonded anchors) - ACI 318-19 – 17.6.5*
- *Concrete splitting failure during installation should be evaluated by ACI 355.2 requirements*

Concrete blow-out failure is provided only for anchors with washer plates.

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot f_{uta} =$ 28.06 kip ≥ $N_f =$ 3.94 kip

Where:

ϕ = 0.70 – resistance factor

$A_{se,N}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \text{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
    - $f_{ya} =$ 92.0 ksi – specified yield strength of anchor steel
    - $f_u =$ 120.0 ksi – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)

The check is performed for group of anchors that form common tension breakout cone: A5, A6, A7, A8

$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot N_b =$ 38.29 kip ≥ $N_{fg} =$ 21.55 kip

Where:

N_{fg} = 21.55 kip – sum of tension forces of anchors with common concrete breakout cone area

ϕ = 0.70 – resistance factor

A_{Nc} = 842.4422 in^2 – concrete breakout cone area for group of anchors

A_{Nc0} = 468.8714 in^2 – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N}$ = 0.96 – modification factor for edge distance:

- $\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$, where:
  - $c_{a,min} =$ 9"7/16 in – minimum distance from the anchor to the edge
  - $h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$ 7"3/16 in – depth of embedment, where:
    - $h_{emb} =$ 1'-4"15/16 in – anchor length
    - $c_{a,max} =$ 10"13/16 in – maximum distance from the anchor to one of the three closest edges
    - $s =$ 3"15/16 in – maximum spacing between anchors

$\Psi_{ec,N}$ = 0.97 – modification factor for eccentrically loaded group of anchors

- $\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$, where:
  - $\Psi_{ecx,N} = \frac{1}{1+\frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$ 1.00 – modification factor that depends on eccentricity in x-direction
  - $e_{x,N} =$ 0" in – tension load eccentricity in x-direction
  - $\Psi_{ecy,N} = \frac{1}{1+\frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$ 0.97 – modification factor that depends on eccentricity in y-direction
  - $e_{y,N} =$ 3/8 in – tension load eccentricity in y-direction
  - $h_{ef} =$ 7"3/16 in – depth of embedment

$\Psi_{c,N}$ = 1.00 – modification factor for concrete conditions

N_b = 32.77 kip – basic concrete breakout strength of a single anchor in tension:

- $N_b = k_c \cdot \lambda_a \cdot \sqrt{f_c'} \cdot h_{ef}^{1.5}$, where:
  - $k_c =$ 24.0 – coefficient for cast-in anchors
  - $\lambda_a =$ 1.00 – modification factor for lightweight concrete
  - $f_c' =$ 5.0 ksi – concrete compressive strength
  - $h_{ef} =$ 7"3/16 in – depth of embedment

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 15.63 kip ≥ $V =$ 0.16 kip

Where:

ϕ = 0.65 – resistance factor

$A_{se,V}$ = 0.3340 in^2 – tensile stress area

f_{uta} = 120.0 ksi – specified tensile strength of anchor steel:

- $f_{uta} = \min(125\ \mathrm{ksi}, 1.9 \cdot f_{ya}, f_u)$, where:
  - f_{ya} = 92.0 ksi – specified yield strength of anchor steel
  - f_u = 120.0 ksi – specified ultimate strength of anchor steel

Concrete shear breakout check (ACI 318-14 – 17.5.2)

The check is performed for group of anchors that form common shear breakout cone: A8

$\phi V_{cbg} = \phi \cdot \frac{A_{Vc}}{A_{Vc0}} \cdot \Psi_{ec,V} \cdot \Psi_{ed,V} \cdot \Psi_{c,V} \cdot \Psi_{h,V} \cdot \Psi_{\alpha,V} \cdot V_{br} =$ 8.45 kip ≥ $V_g =$ 0.06 kip

Where:

V_g = 0.06 kip – sum of shear forces of anchors on common base plate

ϕ = 0.65 – resistance factor

A_{Vc} = 306.8989 in^2 – projected concrete failure area of an anchor or group of anchors

A_{Vc0} = 401.7564 in^2 – projected concrete failure area of one anchor when not limited by corner influences, spacing or member thickness

$\Psi_{ec,V}$ = 1.00 – modification factor for anchor groups loaded eccentrically in shear:

- $\Psi_{ec,V} = \frac{1}{1+\frac{2 \cdot e'_V}{3 \cdot c_a}}$, where:
  - e'_V = 0" in – shear load eccentricity
  - c_{a1} = 9"7/16 in – edge distance in direction of the load
    - c_{a1} = 9"7/16 in – edge distance in direction of the load

$\Psi_{ed,V}$ = 0.93 – modification factor for edge effect:

- $\Psi_{ed,V} = 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_a} \leq 1$, where:
  - c_{a1} = 9"7/16 in – edge distance in direction of the load
    - c_{a1} = 9"7/16 in – edge distance in direction of the load
  - c_{a2} = 10"13/16 in – edge distance in direction perpendicular to the load

$\Psi_{c,V}$ = 1.00 – modification factor for concrete conditions

$\Psi_{h,V}$ = 1.00 – modification factor for anchors located in a shallow concrete member:

- $\Psi_{h,V} = \sqrt{\frac{1.5 \cdot c_{a1}}{h_a}} \geq 1$, where:
  - h_a = 1'-11"5/8 in – thickness of member in which an anchor is anchored measured parallel to anchor axis

$\Psi_{\alpha,V}$ = 1.01 – modification factor for anchors loaded at an angle with the concrete edge

- $\Psi_{\alpha,V} = \sqrt{\frac{1}{(\cos \alpha_V)^2 + (0.5 \cdot \sin \alpha_V)^2}}$, where:
  - α_V = 8.4 ° – angle between direction of shear force and direction perpendicular to concrete edge

V_b = 18.16 kip – basic concrete breakout strength of a single anchor in shear:

- $V_b = \min(7.0 \cdot (\frac{l_e}{d})^{0.2} \cdot \lambda_a \cdot \sqrt{d_a} \cdot \sqrt{f'_c} \cdot c_a^{1.5}, 9.0 \cdot \lambda_a \cdot \sqrt{f'_c} \cdot c_a^{1.5})$, where:
  - l_e = 6" in – effective length
  - d_a = 3/4 in – anchor diameter
  - λ_a = 1.00 – modification factor for lightweight concrete
  - f'_c = 5.0 ksi – concrete compressive strength
  - c_{a1} = 9"7/16 in – edge distance in direction of the load
    - c_{a1} = 9"7/16 in – edge distance in direction of the load

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot N_{cp} =$ 73.61 kip ≥ $V_g =$ 0.07 kip

Where:

ϕ = 0.65 – resistance factor

k_{cp} = 2.00 – concrete pry-out factor

N_{cp} = 56.62 kip – concrete cone tension break-out resistance in case all anchors are in tension

V_g = 0.07 kip – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

$U_{tt}^{5/3} + U_{ts}^{5/3} =$ 0.38 ≤ 1.0

Where:

U_{tt} = 0.56 – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

U_{ts} = 0.01 – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
BP1	COL-w 1	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
BP1	COL-arc 1	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-arc 2	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-arc 3	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-w 2	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
BP1	COL-arc 4	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-arc 5	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-arc 6	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-w 3	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
BP1	COL-arc 7	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-arc 8	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-arc 9	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-w 4	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
BP1	COL-arc 10	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-arc 11	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	COL-arc 12	E70xx	-	-	1/2	-	-	-	-	-	OK	OK
BP1	RIB1a	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	8"3/16	7/16	LE1	0.48	4.13	11.6	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	8"1/4	7/16	LE2	0.16	4.55	3.5	OK	OK
COL-w 1	RIB1a	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE1	0.61	3.86	15.9	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE1	0.84	3.84	22.0	OK	OK
BP1	RIB1b	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	8"1/4	7/16	LE1	0.13	3.88	3.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	8"3/16	7/16	LE1	0.41	3.46	11.7	OK	OK
COL-w 1	RIB1b	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE1	0.84	3.84	21.8	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE1	0.61	3.86	15.9	OK	OK
BP1	RIB2a	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	8"3/16	7/16	LE2	0.62	3.94	15.9	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	8"3/16	7/16	LE2	0.25	3.75	6.7	OK	OK
COL-w 3	RIB2a	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE2	1.29	3.71	34.7	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE2	1.64	3.72	44.1	OK	OK
BP1	RIB2b	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	8"3/16	7/16	LE2	0.23	3.99	5.8	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	8"3/16	7/16	LE2	0.51	3.40	14.9	OK	OK
COL-w 3	RIB2b	E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE2	1.62	3.72	43.6	OK	OK
		E70xx	◢ 1/4 ◣	◢ 3/8 ◣	7"13/16	3/8	LE2	1.28	3.71	34.6	OK	OK

Design data

Material	F_{exx} [ksi]
E70xx	0.0
E70xx	70.0

Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
◢	Fillet weld
F_{exx}	Ultimate strength as rated by electrode classification number

Detailed result for BP1 / COL-w 1 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 1 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 3 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-w 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 5 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 6 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-w 3 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 7 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 8 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 9 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-w 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 10 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 11 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / COL-arc 12 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for BP1 / RIB1a - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 4.13 kip ≥ $F_n =$ 0.48 kip

Where:

F_{nw} = 53.6 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 42.4° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1026 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for BP1 / RIB1a - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 4.55 kip ≥ $F_n =$ 0.16 kip

Where:

F_{nw} = 58.8 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 59.5° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1031 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for COL-w 1 / RIB1a - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.86 kip ≥ $F_n =$ 0.61 kip

Where:

F_{nw} = 55.0 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 46.6° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0935 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for COL-w 1 / RIB1a - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.84 kip ≥ $F_n =$ 0.84 kip

Where:

F_{nw} = 54.8 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 46.0° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0935 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for BP1 / RIB1b - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.88 kip ≥ $F_n =$ 0.13 kip

Where:

F_{nw} = 50.2 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 32.4° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1031 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for BP1 / RIB1b - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.46 kip ≥ $F_n =$ 0.41 kip

Where:

F_{nw} = 45.0 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 16.0° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1026 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for COL-w 1 / RIB1b - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.84 kip ≥ $F_n =$ 0.84 kip

Where:

F_{nw} = 54.8 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 46.0° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0935 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for COL-w 1 / RIB1b - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.86 kip ≥ $F_n =$ 0.61 kip

Where:

F_{nw} = 55.0 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 46.6° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0935 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for BP1 / RIB2a - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.94 kip ≥ $F_n =$ 0.62 kip

Where:

F_{nw} = 51.1 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 35.1° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1026 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for BP1 / RIB2a - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.75 kip ≥ $F_n =$ 0.25 kip

Where:

F_{nw} = 48.7 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 27.8° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1026 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for COL-w 3 / RIB2a - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.71 kip ≥ $F_n =$ 1.29 kip

Where:

F_{nw} = 53.0 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 40.5° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0935 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for COL-w 3 / RIB2a - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.72 kip ≥ $F_n =$ 1.64 kip

Where:

F_{nw} = 53.1 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 40.9° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0935 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for BP1 / RIB2b - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.99 kip ≥ $F_n =$ 0.23 kip

Where:

F_{nw} = 51.9 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 37.3° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1026 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for BP1 / RIB2b - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.40 kip ≥ $F_n =$ 0.51 kip

Where:

F_{nw} = 44.2 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 13.0° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.1026 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for COL-w 3 / RIB2b - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.72 kip ≥ $F_n =$ 1.62 kip

Where:

F_{nw} = 53.1 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 40.9° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0935 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for COL-w 3 / RIB2b - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.71 kip ≥ $F_n =$ 1.28 kip

Where:

F_{nw} = 53.0 ksi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70.0 ksi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 40.5° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0935 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Concrete block

Item	Loads	A_1 [in^2]	A_2 [in^2]	σ [ksi]	$\phi f_{p,max}$ [ksi]	Ut [%]	Status
CB 1	LE1	0.0000	0.0000	-	-	-	OK

Symbol explanation

A_1	Loaded area
A_2	Supporting area
σ	Average stress in concrete
$\phi f_{p,max}$	Concrete bearing resistance
Ut	Utilization

Detailed result for CB 1

Concrete block compressive resistance check (AISC 360-16 – J8)

$\phi_c f_{p,max} =$ - ksi < $\sigma =$ - ksi

Where:

$f_{p,max}$ = - ksi – concrete block design bearing strength:

- $f_{p,max} = 0.85 \cdot f'_c \cdot \sqrt{\frac{A_2}{A_1}} \leq 1.7 \cdot f'_c$, where:
  - $f'_c =$ 5.0 ksi – concrete compressive strength
  - $A_1 =$ 0.0000 in^2 – base plate area in contact with concrete surface
  - $A_2 =$ 0.0000 in^2 – concrete supporting surface

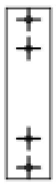
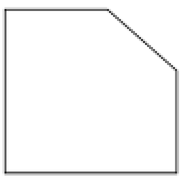
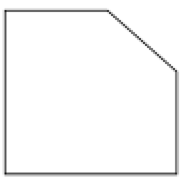
ϕ_c = 0.65 – resistance factor for concrete

Buckling

Buckling analysis was not calculated.

Bill of material

Manufacturing operations

Name	Plates [in]	Shape	Nr.	Welds Leg size [in]	Length [in]	Bolts	Nr.
BP1	P13/16x5"7/8-1'-11"1/4 (A913 Gr.50)		1	Butt: 11/16	1'-7"7/8	3/4 A325	4
RIB1	P9/16x8"1/4-7"7/8 (A913 Gr.50)		2	Double fillet: 3/8	2'-8"5/16		
RIB2	P9/16x8"1/4-7"7/8 (A913 Gr.50)		2	Double fillet: 3/8	2'-8"5/16		

Welds

Type	Material	Throat thickness [in]	Leg size [in]	Length [in]
Butt	E70xx	-	-	1'-7"7/8
Double fillet	E70xx	1/4	3/8	5'-4"9/16

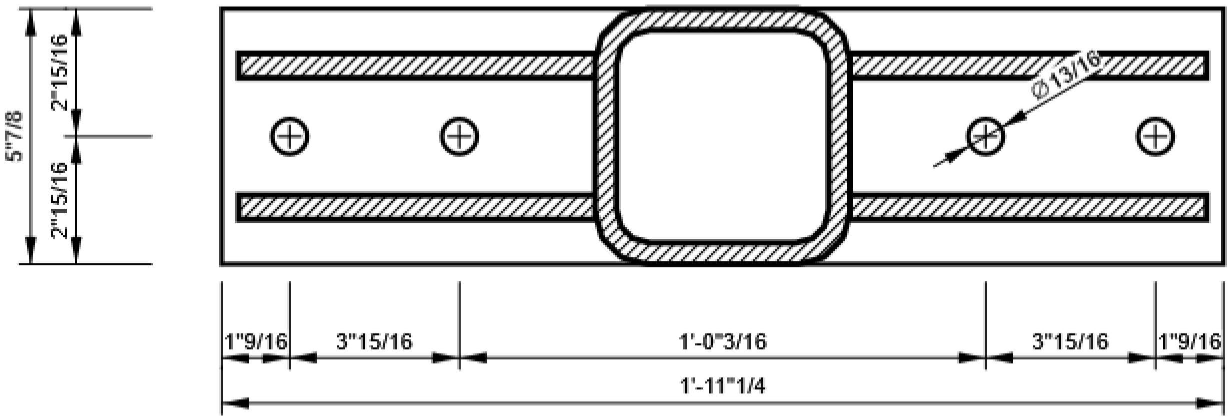
Anchors

Name	Length [in]	Drill length [in]	Count
3/4 A325	1'-5"3/4	1'-4"15/16	4

Drawing

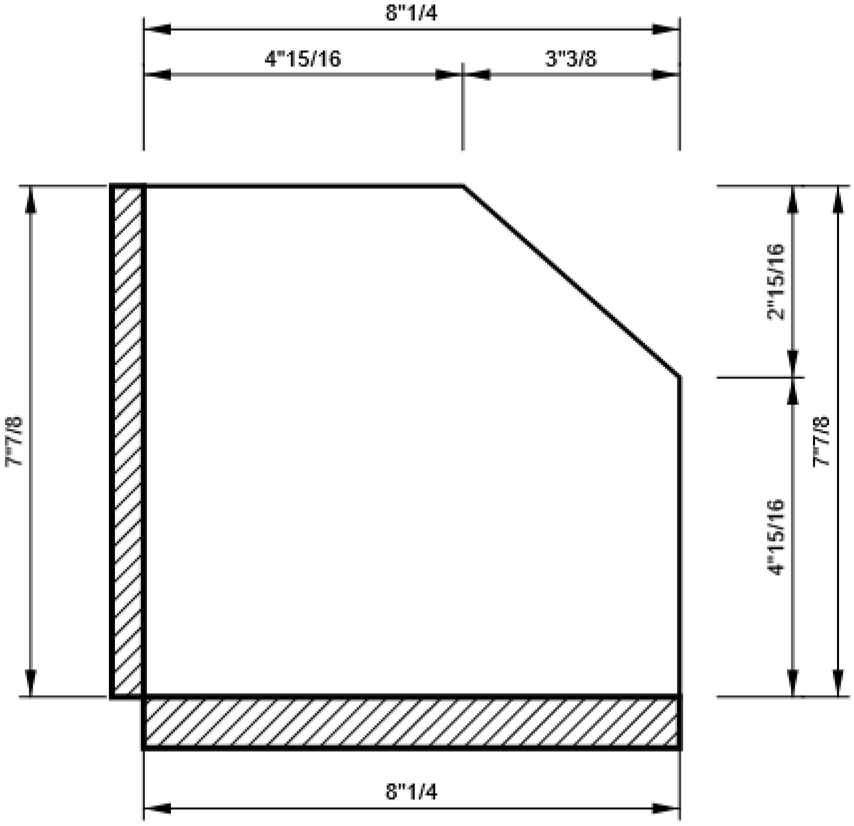
BP1

P13/16x1'-11"1/4-5"7/8 (A913 Gr.50)



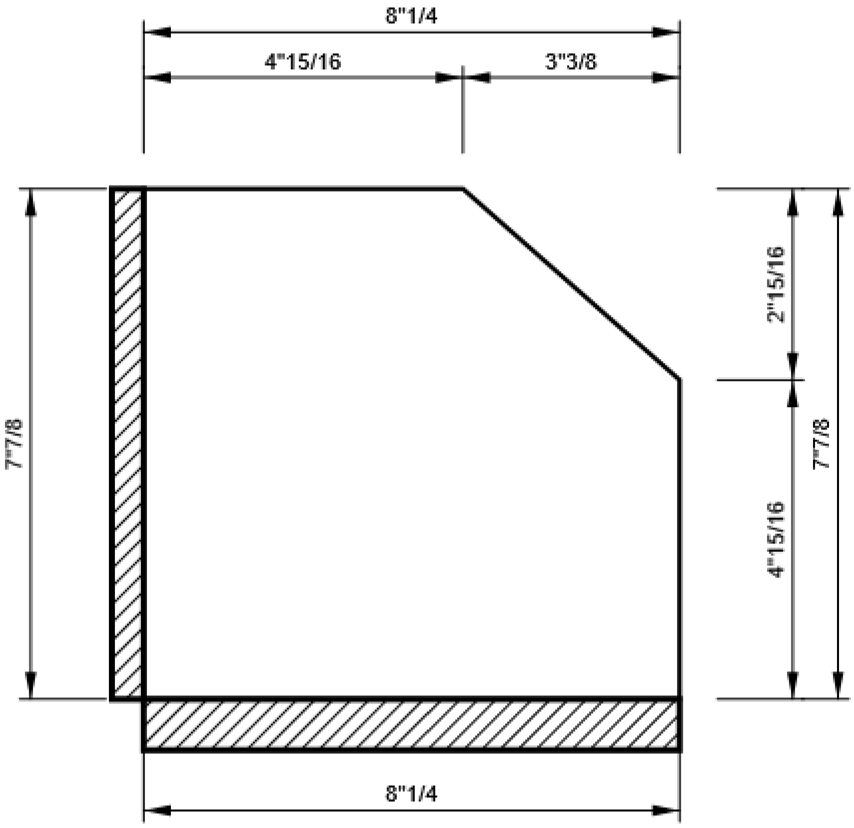
RIB1

P9/16x7"7/8-8"1/4 (A913 Gr.50)



RIB2

P9/16x7"7/8-8"1/4 (A913 Gr.50)



Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Appendix- 12

Anchor Designer™ Software
Version 3.1.2303.1

Company:		Date:	4/22/2024
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1. Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: A615 Grade 60 Rebar
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 15.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 16.75
c_{ac} (inch): 25.94
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 48.00
State: Cracked
Compressive strength, f'_c (psi): 5000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 10.25 x 6.00 x 0.75

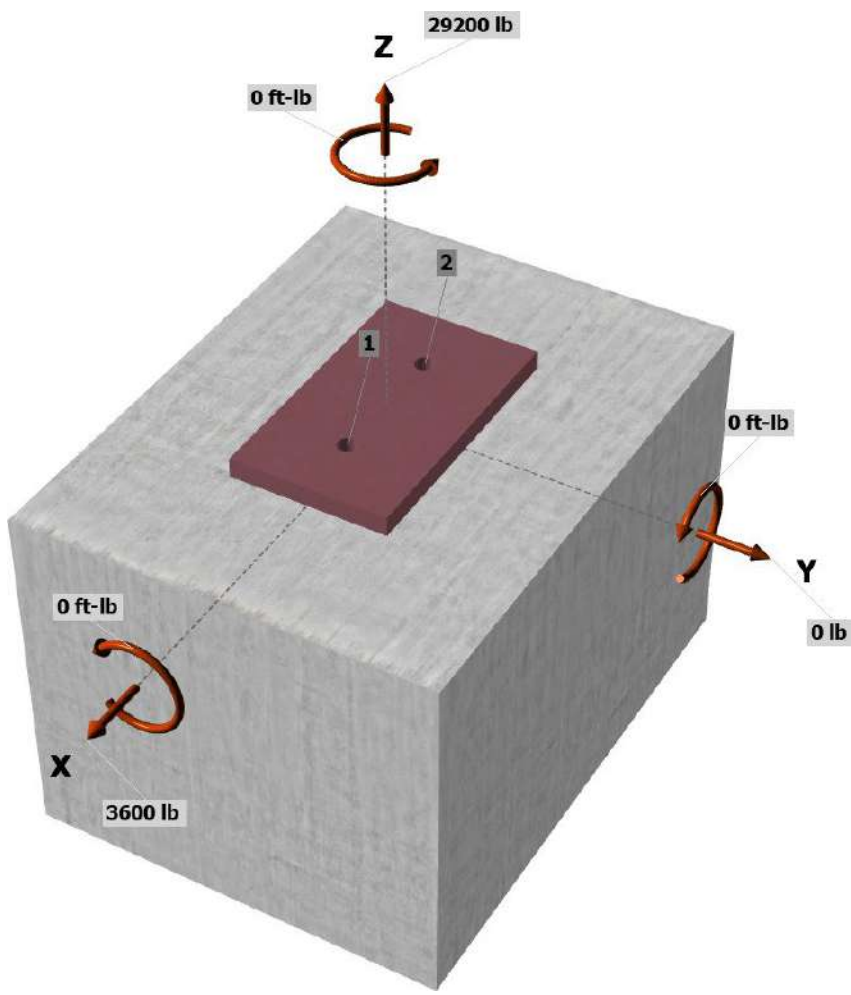
Recommended Anchor
Anchor Name: SET-3G™ - SET-3G w/ #6 A615 Gr. 60 Rebar
Code Report: ICC-ES ESR-4057

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

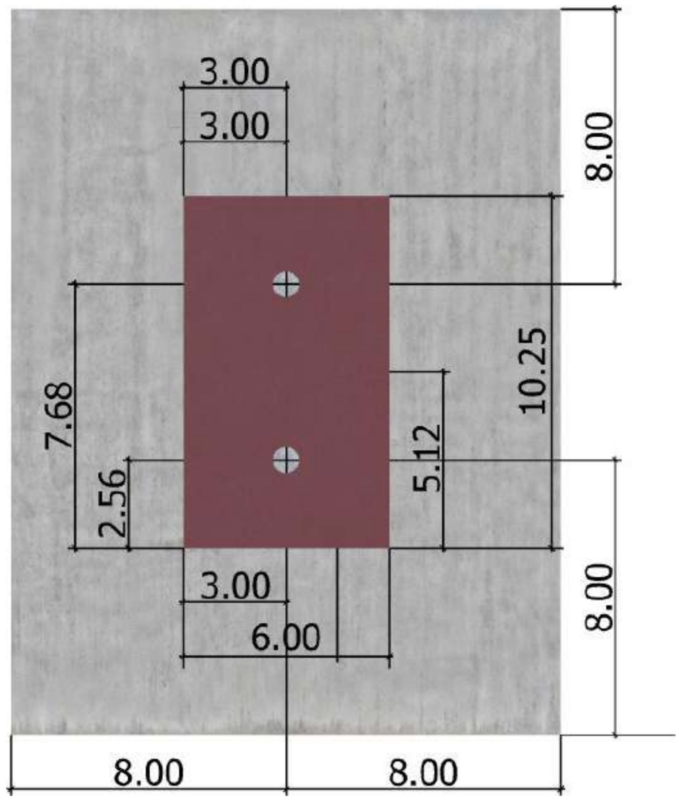
Strength level loads:

N_{ua} [lb]: 29200
V_{uax} [lb]: 3600
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>

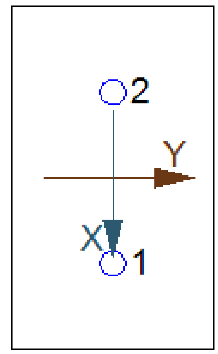


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	14600.0	1800.0	0.0	1800.0
2	14600.0	1800.0	0.0	1800.0
Sum	29200.0	3600.0	0.0	3600.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
39600	0.75	29700

6. Adhesive Strength of Anchor in Tension (Sec. 17.6.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} (f'_c / 2{,}500)^n$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	f'_c (psi)	n	$\tau_{k,cr}$ (psi)
1310	1.00	1.00	5000	0.25	1558

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.6.5.2.1)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1558	0.75	15.000	55059

$\phi N_{ag} = \phi (A_{Na} / A_{Na0}) \Psi_{ec,Na} \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1b)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ec,Na}$	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba}(lb)	ϕ	ϕN_{ag} (lb)
337.92	388.23	9.85	8.00	1.000	0.944	1.000	55059	0.65	29394

8. Steel Strength of Anchor in Shear (Sec. 17.7.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
23760	1.0	0.65	15444

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.7.3)

$\phi V_{cpg} = \phi \min|k_{cp}N_{ag} ; k_{cp}N_{cbg}| = \phi \min|k_{cp}(A_{Na} / A_{Na0}) \Psi_{ec,Na} \Psi_{ed,Na} \Psi_{cp,Na}N_{ba} ; k_{cp}(A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N}N_b|$ (Sec. 17.5.1.2 & Eq. 17.7.3.1b)

k_{cp}	A_{Na} (in^2)	A_{Na0} (in^2)	$\Psi_{ed,Na}$	$\Psi_{ec,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (lb)
2.0	337.92	388.23	0.944	1.000	1.000	55059	45222

A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	N_{cb} (lb)	ϕ
337.92	256.00	1.000	1.000	1.000	1.000	14806	19544	0.70

ϕV_{cpg} (lb)
27361

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	14600	29700	0.49	Pass
Adhesive	**29200**	**29394**	**0.99**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	1800	15444	0.12	Pass
Pryout	**3600**	**27361**	**0.13**	**Pass (Governs)**

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.8.1	0.99	0.00	99.3%	1.0	Pass

SET-3G w/ #6 A615 Gr. 60 Rebar with hef = 15.000 inch meets the selected design criteria.

12. Warnings

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.5.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.5.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

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Design:	3.Holddown to concrete	Date:	15/02/2024
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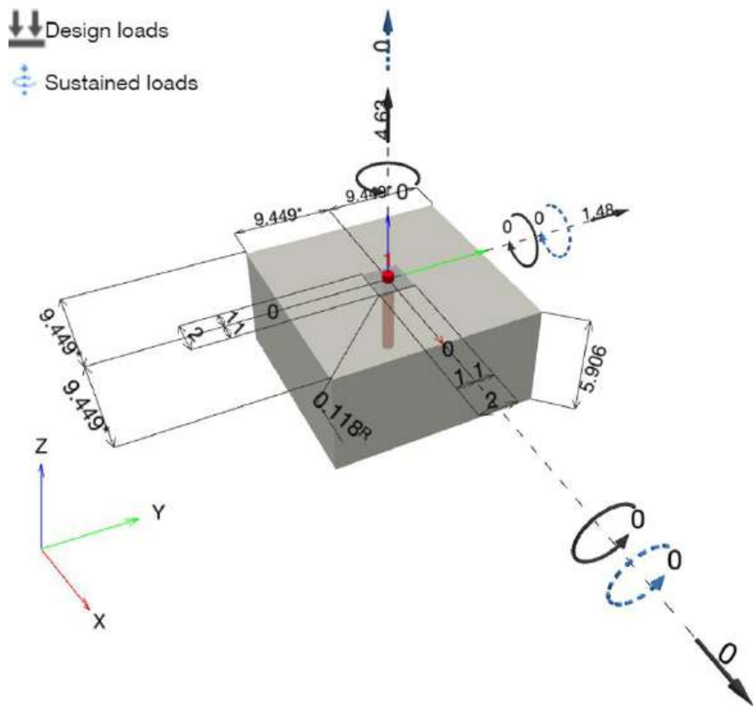
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 + HAS-U 8.8 HDG M16**
Item number:	2223703 HAS-U 8.8 HDG M16x190 (element) / 2022696 HIT-HY 200-A (adhesive)
Insert item # alternative:	2390251 HAS 8.8 HDG M16x190
Effective embedment depth:	$h_{ef,act}$ = 4.331 in. ($h_{ef,limit}$ = - in.)
Material:	8.8
Evaluation Service Report:	ESR-3187
Issued I Valid:	01/05/2021 | 01/03/2022
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.118 in.
Anchor plate^R :	l_x x l_y x t = 2.000 in. x 2.000 in. x 0.118 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, C20/25, f_c' = 2,901 psi; h = 5.906 in., Temp. short/long: 0/0 °C
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

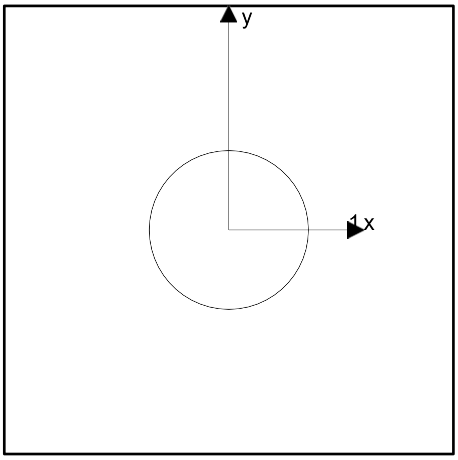
Geometry [in.] & Loading [kip, ft.lb]



1.1 Design results

Case	Description	Forces [kip] / Moments [ft.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 4.630; V_x = 0.000; V_y = 1.480; M_x = 0.000; M_y = 0.000; M_z = 0.000;	no	99

2 Load case/Resulting anchor forces



Anchor reactions [kip]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	4.630	1.480	0.000	1.480

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0.000 [kip]
resulting compression force in (x/y)=(0.000/0.000): 0.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity f N_n [kip]	Utilization $b_N = N_{ua}$/f N_n	Status
Steel Strength*	4.630	18.339	26	OK
Bond Strength**	4.630	6.642	70	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	4.630	5.364	87	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value	refer to ICC-ES ESR-3187
} N_{sa} 2: N_{ua}	ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.24	116,030

Calculations

N_{sa} [kip]
28.214

Results

N_{sa} [kip]	$}_{steel}$	} N_{sa} [kip]	N_{ua} [kip]
28.214	0.650	18.339	4.630

3.2 Bond Strength

$N_a = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na} \psi_{cp,Na} N_{ba}$ — ACI 318-19 Eq. (17.6.5.1a)

$\phi N_a \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2 c_{Na})^2$ — ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10 d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ — ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.5.4.1b)

$\psi_{cp,Na} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.5.5.1b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ — ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,252	0.630	4.331	9.449	1.000	1,192

c_{ac} [in.]	λ_a
9.900	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
8.973	322.06	322.06	1.000

$\psi_{cp,Na}$	N_{ba} [kip]
1.000	10.219

Results

N_a [kip]	ϕ_{bond}	ϕN_a [kip]	N_{ua} [kip]
10.219	0.650	6.642	4.630

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3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N} \, \psi_{c,N} \, \psi_{cp,N} \, N_b$ ACI 318-19 Eq. (17.6.2.1a)

$\phi \, N_{cb} \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 \, h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \, \lambda_a \sqrt{f'_c} \, h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
4.331	9.449	1.000	9.900	17	1.000	2,901

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
168.80	168.80	1.000	1.000	8.252

Results

N_{cb} [kip]	$\phi_{concrete}$	$\phi \, N_{cb}$ [kip]	N_{ua} [kip]
8.252	0.650	5.364	4.630

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Fastening point:			

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1.480	10.184	15	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	1.480	11.552	13	OK
Concrete edge failure in direction y+**	1.480	3.850	39	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3187

ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.24	116,030

Calculations

V_{sa} [kip]
16.973

Results

V_{sa} [kip]	ϕ_{steel}	ϕ V_{sa} [kip]	V_{ua} [kip]
16.973	0.600	10.184	1.480

4.2 Pryout Strength (Concrete Breakout Strength controls)

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$	ACI 318-19 Eq. (17.7.3.1a)
$\phi\ V_{cp} \geq V_{ua}$	ACI 318-19 Table 17.5.2
A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-19 Eq. (17.6.2.1.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.6.1b)
$N_b = k_c\ \lambda_a \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2	4.331	9.449	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
9.900	17	1.000	2,901

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
168.80	168.80	1.000	1.000	8.252

Results

V_{cp} [kip]	$\phi_{concrete}$	$\phi\ V_{cp}$ [kip]	V_{ua} [kip]
16.503	0.700	11.552	1.480

4.3 Concrete edge failure in direction y+

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ ACI 318-19 Eq. (17.7.2.1a)

$\phi\ V_{cb} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5\ c_a^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\sqrt{f'_c}\ c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
6.299	9.449	1.000	5.906	4.331

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.630	2,901	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
111.60	178.56	1.000	1.265	6.956

Results

V_{cb} [kip]	$\phi_{concrete}$	$\phi\ V_{cb}$ [kip]	V_{ua} [kip]
5.499	0.700	3.850	1.480

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V		Utilization $\beta_{N,V}$ [%]	Status
0.863	0.384	5/3	99	OK

$\beta_{NV} = \beta_N + \beta_V <= 1$

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

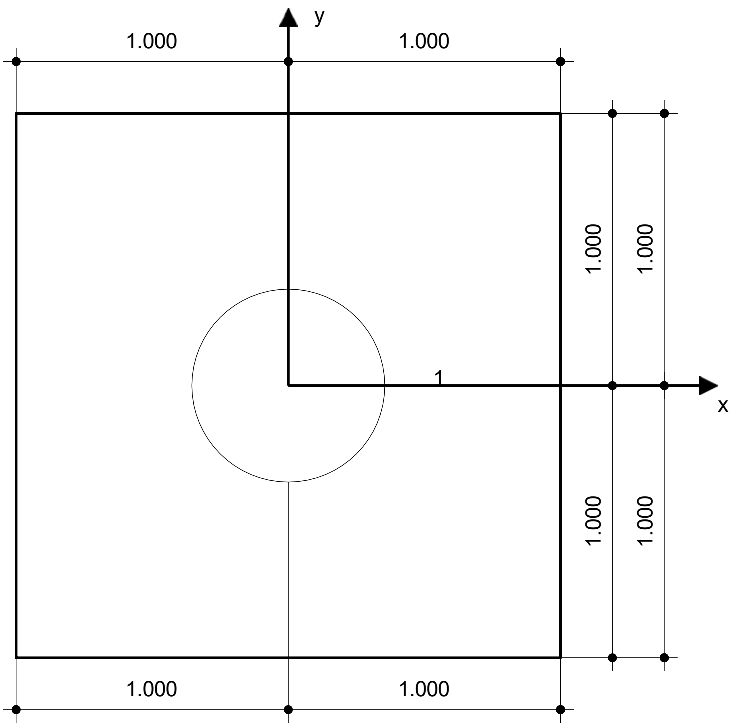
7 Installation data

Profile: no profile	Anchor type and diameter: HIT-HY 200 + HAS-U 8.8 HDG M16
	Item number: 2223703 HAS-U 8.8 HDG M16x190 (element) / 2022696 HIT-HY 200-A (adhesive)
Hole diameter in the fixture: d_f = 0.709 in.	Insert item # alternative: 2390251 HAS 8.8 HDG M16x190
Plate thickness (input): 0.118 in.	Maximum installation torque: 80 Nm
Recommended plate thickness: not calculated	Hole diameter in the base material: 0.709 in.
Drilling method: Hammer drilled	Hole depth in the base material: 4.331 in.
Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required	Minimum thickness of the base material: 5.748 in.

Hilti HAS-U or HAS threaded rod with HIT-HY 200 injection mortar with 4.330709 in embedment h_ef, M16, Hot dip galvanized, Hammer drilled installation per ESR-3187

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}

1	0.000	0.000	9.449	9.449	9.449	9.449

8 Remarks; Your Cooperation Duties

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Project data

Project name	Exuma, Bahamas
Project number	242201
Author	MS/SK
Description	Roof Beam-Column Connection
Date	09-02-2024
Code	AISC/ACI

Material

Steel	A36, A913 Gr.50
Concrete	4000 psi

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Roof Beam-Column Connection
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

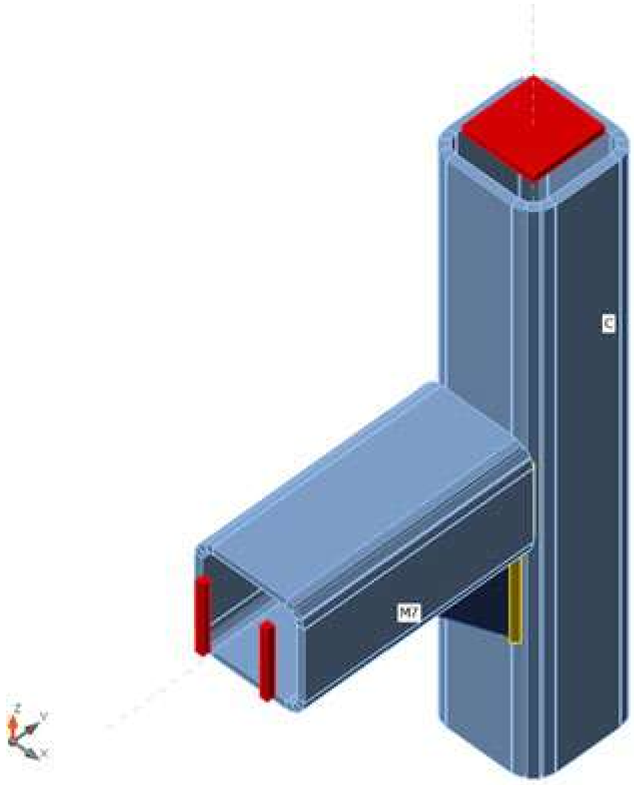
Members

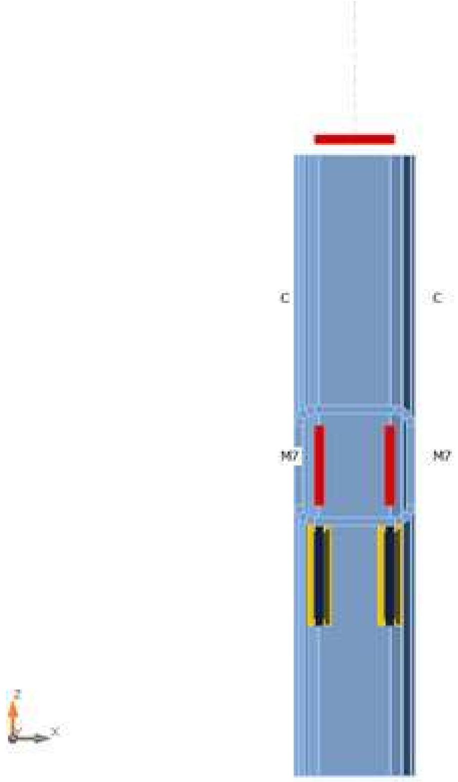
Geometry

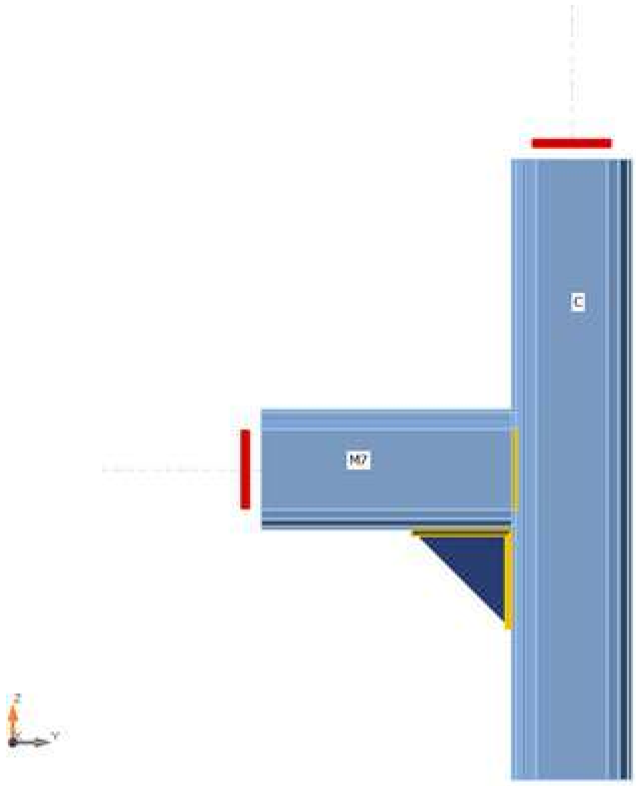
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
C	5 - SHS150/150/12.5	0.0	-90.0	-90.0	0"	0"	0"
M7	6 - SHS150/150/10.0	-90.0	0.0	0.0	2"15/16	0"	0"

Supports and forces

Name	Support	Forces in	X [in]
C / begin	N-Vy-Vz-Mx-My-Mz	Node	0"
C / end		Node	0"
M7 / end	Vy-Mx-Mz	Node	0"



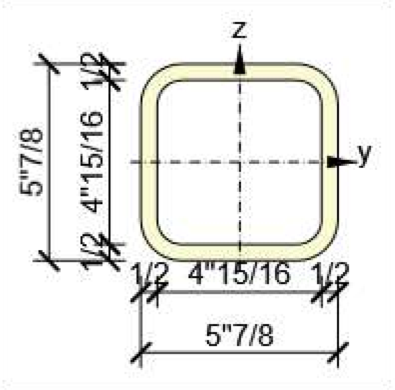
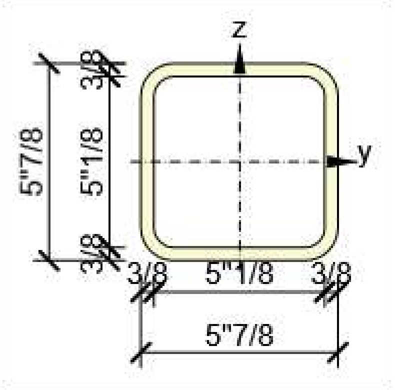




Cross-sections

Name	Material
5 - SHS150/150/12.5	A913 Gr.50
6 - SHS150/150/10.0	A913 Gr.50

Cross-sections

Name	Material	Drawing
5 - SHS150/150/12.5	A913 Gr.50	
6 - SHS150/150/10.0	A913 Gr.50	

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	C / End	-11.200	0.000	0.000	0.00	43.70	0.00
	M7 / End	0.000	0.000	-11.200	0.00	40.90	0.00
	C / Begin	0.000	0.000	0.000	0.00	0.00	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.000	0.000	0.000	-0.04	0.00	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.5 < 5.0%	OK
Loc. deformation	0.3 < 3%	OK
Welds	76.4 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

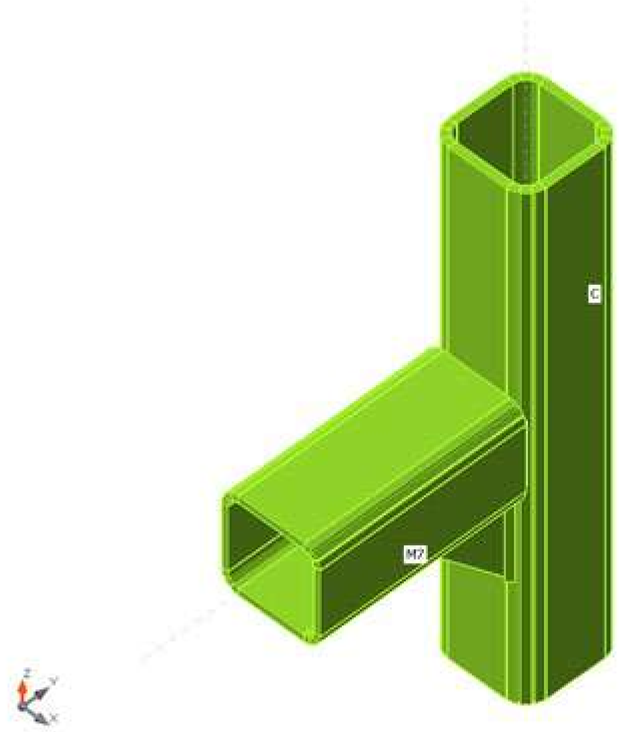
Name	t_p [in]	Loads	σ_{Ed} [psi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [psi]	Status
C	1/2	LE1	45044.9	0.2	0.0	OK
M7	3/8	LE1	45150.0	0.5	0.0	OK
RIB4a	1/2	LE1	45067.6	0.2	0.0	OK
RIB4b	1/2	LE1	45067.7	0.2	0.0	OK

Design data

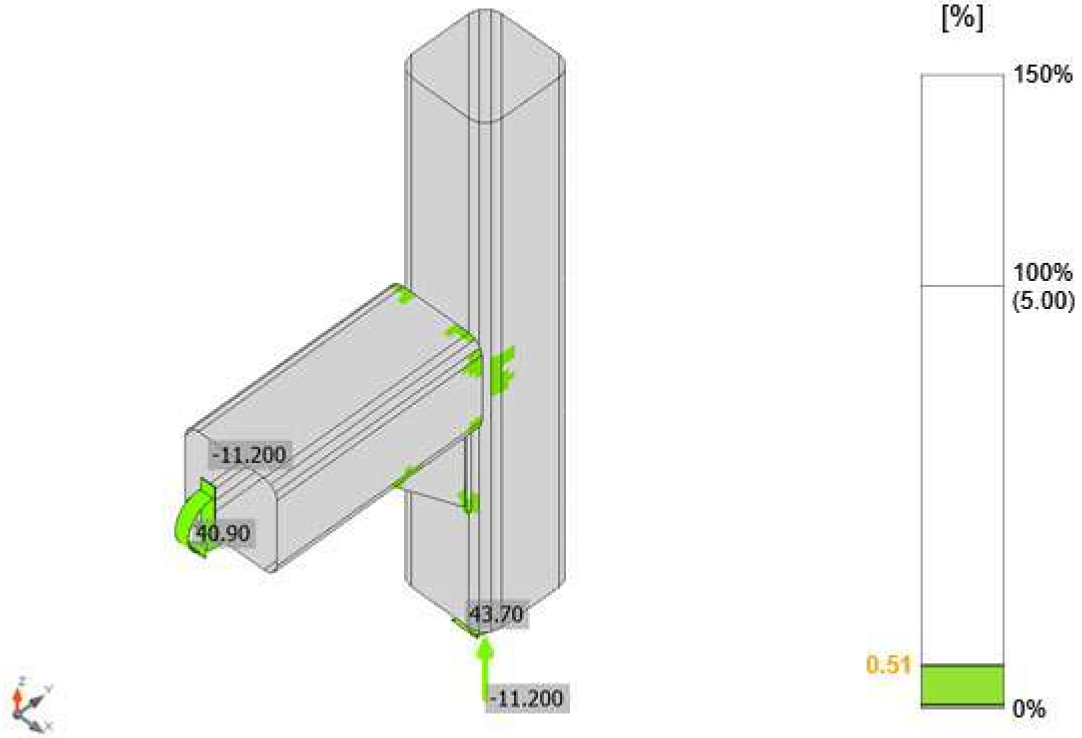
Material	F_y [psi]	ε_{lim} [%]
A913 Gr.50	50000.3	5.0

Loc. deformation

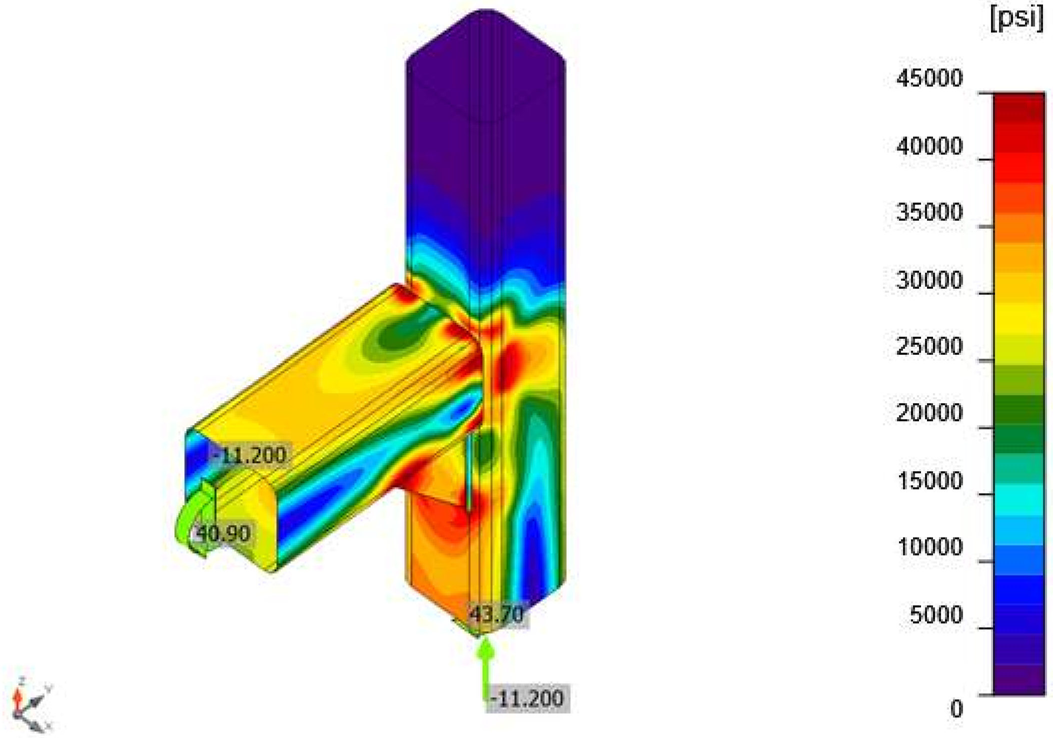
Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
C	5"7/8	LE1		3/16	0.2	OK
M7	5"7/8	LE1		3/16	0.3	OK



Overall check, LE1



Strain check, LE1



Equivalent stress, LE1

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
C-w 1	M7-w 3	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
C-w 1	M7-w 4	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
C-w 1	M7-w 1	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
C-w 1	M7-w 2	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
C-w 1	RIB4a	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	3/8	LE1	2.817	3.742	75.3	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	3/8	LE1	2.817	3.742	75.3	OK	OK
M7-w 1	RIB4a	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	3/8	LE1	2.838	3.742	75.9	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	3/8	LE1	2.860	3.742	76.4	OK	OK
C-w 1	RIB4b	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	3/8	LE1	2.817	3.742	75.3	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	3/8	LE1	2.817	3.742	75.3	OK	OK
M7-w 1	RIB4b	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	3/8	LE1	2.860	3.742	76.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	3/8	LE1	2.838	3.742	75.9	OK	OK

Detailed result for C-w 1 / M7-w 3 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for C-w 1 / M7-w 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for C-w 1 / M7-w 1 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for C-w 1 / M7-w 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for C-w 1 / RIB4a - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.795 kip $\geq$ $F_n =$ 2.817 kip

Where:

$F_{nw} =$ 55934.2 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\vartheta)$, where:
  - F_{EXX} $=$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - $\vartheta =$ 49.5° – angle of loading measured from the weld longitudinal axis

$A_{we} =$ 0.0905 in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 3.742 kip ≥ $F_n =$ 2.817 kip

Where:

$F_{nBM} = 39000.1$ psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
  - $F_u = $ 65000.1 psi – tensile strength of the connected material

$A_{BM} = 0.1279$ in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot \sqrt{2}$, where:
  - $A_{we} = $ 0.0905 in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Detailed result for C-w 1 / RIB4a - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.795 kip ≥ $F_n =$ 2.817 kip

Where:

$F_{nw} = 55938.8$ psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\vartheta)$, where:
  - $F_{EXX} = $ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - $\vartheta = $ 49.5° – angle of loading measured from the weld longitudinal axis

$A_{we} = 0.0905$ in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 3.742 kip ≥ $F_n =$ 2.817 kip

Where:

$F_{nBM} = 39000.1$ psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
  - $F_u = $ 65000.1 psi – tensile strength of the connected material

$A_{BM} = 0.1279$ in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot$
  - $A_{we} =$

, where: $\sqrt{2}$
0.0905 in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Detailed result for M7-w 1 / RIB4a - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.773 kip ≥ $F_n =$ 2.838 kip

Where:

$F_{nw} =$ 55618.7 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\vartheta)$, where:
  -
  - F_{EXX} $=$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength

    $\vartheta =$ 48.5° – angle of loading measured from the weld longitudinal axis

$A_{we} =$ 0.0905 in^2 – effective area of weld critical element

$\phi =$ 0.75 – resistance factor for welded connections

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 3.742 kip ≥ $F_n =$ 2.838 kip

Where:

$F_{nBM} =$ 39000.1 psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
  - F_u $=$ 65000.1 psi – tensile strength of the connected material

$A_{BM} =$ 0.1279 in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot \sqrt{2}$, where:
  - A_{we} $=$ 0.0905 in^2 – effective area of weld critical element

$\phi =$ 0.75 – resistance factor for welded connections

Detailed result for M7-w 1 / RIB4a - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.752 kip ≥ $F_n =$ 2.860 kip

Where:

$F_{nw} =$ 55302.4 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\vartheta)$, where:
  - F_{EXX} $=$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - $\vartheta =$ 47.5° – angle of loading measured from the weld longitudinal axis

$A_{we} = 0.0905$ in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

∘

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 3.742 kip ≥ $F_n =$ 2.860 kip

Where:

$F_{nBM} = 39000.1$ psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
  - $F_u = 65000.1$ psi – tensile strength of the connected material

$A_{BM} = 0.1279$ in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot \sqrt{2}$, where:
  - $A_{we} = 0.0905$ in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Detailed result for C-w 1 / RIB4b - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.795 kip ≥ $F_n =$ 2.817 kip

Where:

$F_{nw} = 55937.0$ psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\vartheta)$, where:
  - $F_{EXX} = 70000.0$ psi – electrode classification number, i.e. minimum specified tensile strength
  - $\vartheta = 49.5°$ – angle of loading measured from the weld longitudinal axis

$A_{we} = 0.0905$ in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 3.742 kip ≥ $F_n =$ 2.817 kip

Where:

$F_{nBM} = 39000.1$ psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
  - $F_u = 65000.1$ psi – tensile strength of the connected material

$A_{BM} = 0.1279$ in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot$
  - $A_{we} =$

, where: $\sqrt{2}$
0.0905 in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Detailed result for C-w 1 / RIB4b - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.795 kip ≥ $F_n =$ 2.817 kip

Where:

$F_{nw} = 55932.4$ psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\vartheta)$, where:
    -
    - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength

    $\vartheta =$ 49.5° – angle of loading measured from the weld longitudinal axis

$A_{we} = 0.0905$ in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 3.742 kip ≥ $F_n =$ 2.817 kip

Where:

$F_{nBM} = 39000.1$ psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
    - $F_u =$ 65000.1 psi – tensile strength of the connected material

$A_{BM} = 0.1279$ in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot \sqrt{2}$, where:
    - $A_{we} =$ 0.0905 in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Detailed result for M7-w 1 / RIB4b - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.752 kip ≥ $F_n =$ 2.860 kip

Where:

$F_{nw} = 55303.5$ psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\vartheta)$, where:
    - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
    - $\vartheta =$ 47.5° – angle of loading measured from the weld longitudinal axis

$A_{we} = 0.0905 \text{ in}^2$ – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

∘

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 3.742 kip ≥ $F_n =$ 2.860 kip

Where:

$F_{nBM} = 39000.1$ psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
  - $F_u =$ 65000.1 psi – tensile strength of the connected material

$A_{BM} = 0.1279$ in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot \sqrt{2}$, where:
  - $A_{we} =$ 0.0905 in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Detailed result for M7-w 1 / RIB4b - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.773 kip ≥ $F_n =$ 2.838 kip

Where:

$F_{nw} = 55619.9$ psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\vartheta)$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - $\vartheta =$ 48.5° – angle of loading measured from the weld longitudinal axis

$A_{we} = 0.0905$ in^2 – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 3.742 kip ≥ $F_n =$ 2.838 kip

Where:

$F_{nBM} = 39000.1$ psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
  - $F_u =$ 65000.1 psi – tensile strength of the connected material

$A_{BM} = 0.1279$ in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot$
  - $A_{we} =$

$\sqrt{2}$
, where: 0.0905 in^2 – effective area of weld critical element

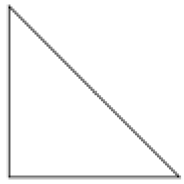
$\phi = 0.75$ – resistance factor for welded connections

Buckling

Buckling analysis was not calculated.

Bill of material

Manufacturing operations

Name	Plates [in]	Shape	Nr.	Welds [in]	Length [in]	Bolts	Nr.
RIB4	P1/2x4"15/16-4"15/16 (A913 Gr.50)		2	Double fillet: a = 1/4	1'-7"11/16		

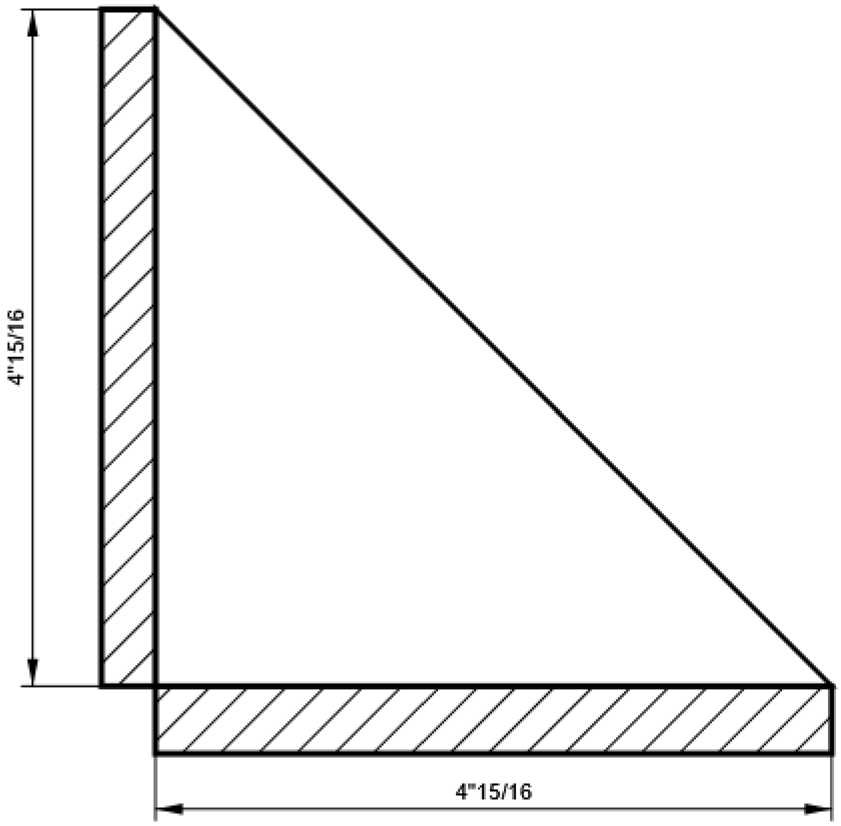
Welds

Type	Material	Throat thickness [in]	Leg size [in]	Length [in]
Butt	E70xx	-	-	1'-3"3/4
Double fillet	E70xx	1/4	5/16	1'-7"11/16

Drawing

RIB4

P1/2x4"15/16-4"15/16 (A913 Gr.50)



Symbol explanation

Symbol	Symbol explanation
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain
◢	Fillet weld
T_h	Throat thickness of weld
L_s	Leg size of weld
L	Length of weld
L_c	Length of critical weld element
F_n	Force in weld critical element
ϕRn_w	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	Yes		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	MS/SK
Description	Roof Beam-Column Connection
Date	09-02-2024
Code	AISC/ACI

Material

Steel	A36, A913 Gr.50
Concrete	4000 psi

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Roof Beam-Column Connection
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

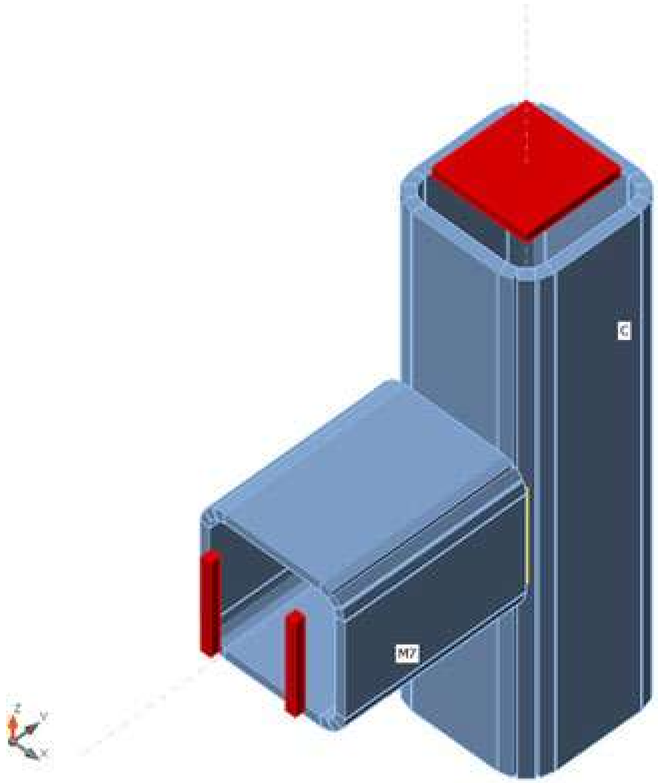
Members

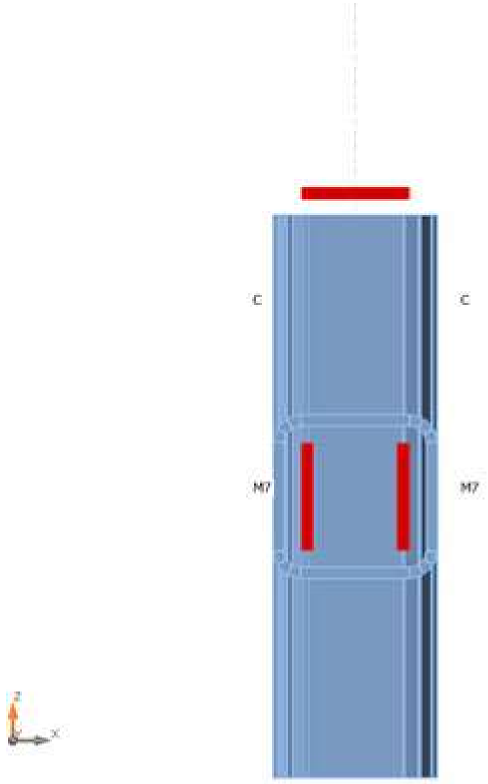
Geometry

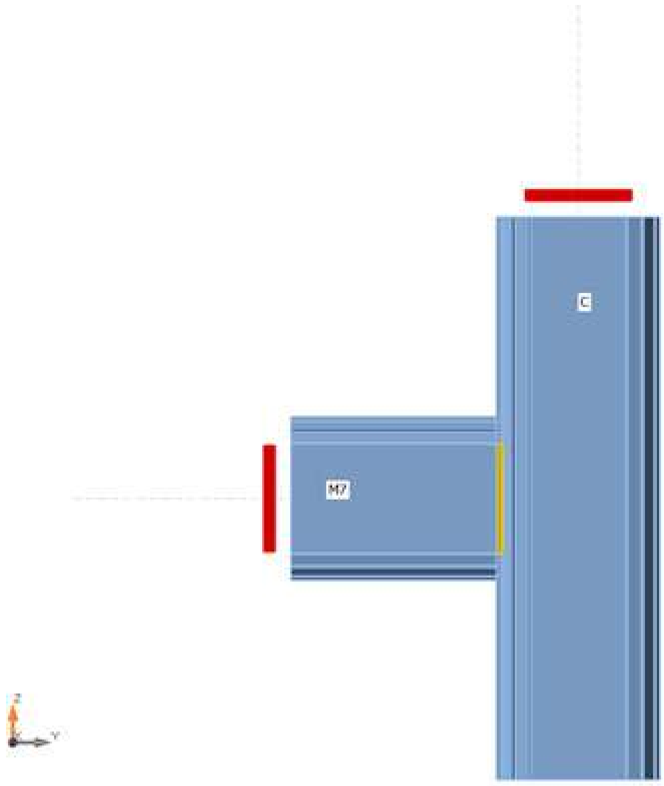
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
C	5 - SHS150/150/12.5	0.0	-90.0	-90.0	0"	0"	0"
M7	6 - SHS150/150/10.0	-90.0	0.0	0.0	2"15/16	0"	0"

Supports and forces

Name	Support	Forces in	X [in]
C / begin	N-Vy-Vz-Mx-My-Mz	Node	0"
C / end		Node	0"
M7 / end	Vy-Mx-Mz	Node	0"



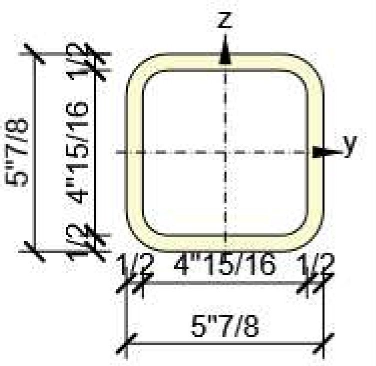
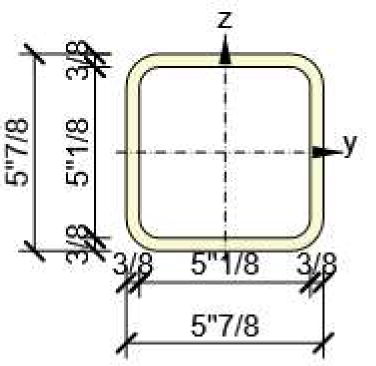




Cross-sections

Name	Material
5 - SHS150/150/12.5	A913 Gr.50
6 - SHS150/150/10.0	A913 Gr.50

Cross-sections

Name	Material	Drawing
5 - SHS150/150/12.5	A913 Gr.50	
6 - SHS150/150/10.0	A913 Gr.50	

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	C / End	-5.300	0.000	0.000	0.00	20.50	0.00
	M7 / End	0.000	0.000	-5.300	0.00	19.20	0.00
	C / Begin	0.000	0.000	0.000	0.00	0.00	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.000	0.000	0.000	0.00	0.00	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.1 < 5.0%	OK
Loc. deformation	0.1 < 3%	OK
Welds	0.0 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

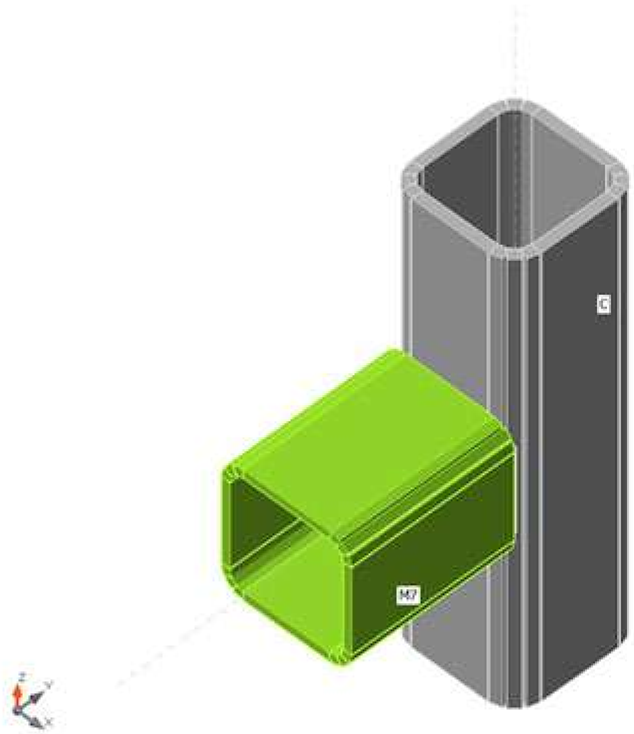
Name	t_p [in]	Loads	σ_{Ed} [psi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [psi]	Status
C	1/2	LE1	40696.1	0.0	0.0	OK
M7	3/8	LE1	38460.6	0.1	0.0	OK

Design data

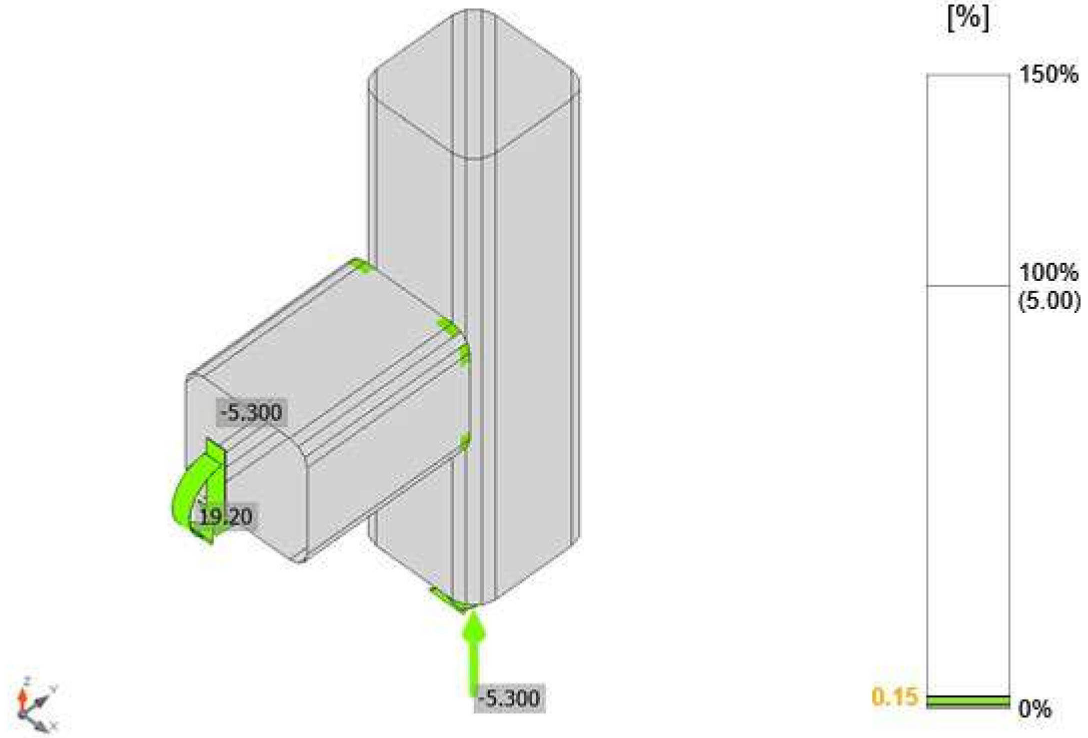
Material	F_y [psi]	ε_{lim} [%]
A913 Gr.50	50000.3	5.0

Loc. deformation

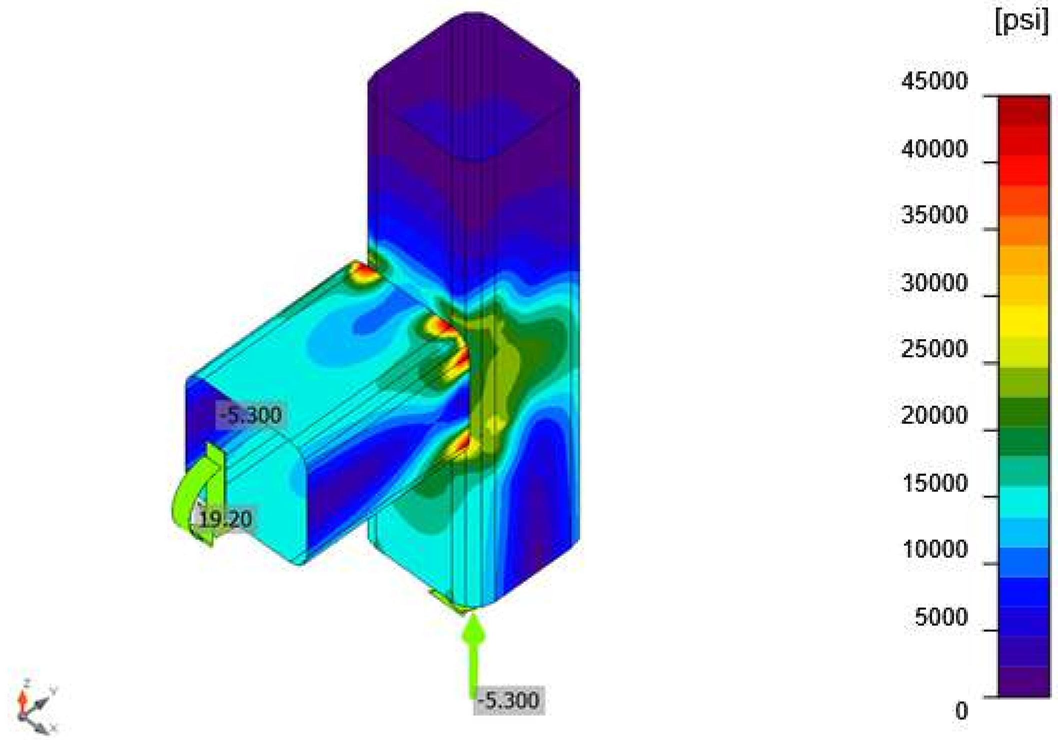
Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
C	5"7/8	LE1		3/16	0.1	OK
M7	5"7/8	LE1		3/16	0.0	OK



Overall check, LE1



Strain check, LE1



Equivalent stress, LE1

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
C-w 1	M7-w 3	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
C-w 1	M7-w 4	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
C-w 1	M7-w 1	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
C-w 1	M7-w 2	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK

Detailed result for C-w 1 / M7-w 3

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Buckling

Buckling analysis was not calculated.

Bill of material

Welds

Type	Material	Throat thickness [in]	Leg size [in]	Length [in]
Butt	E70xx	-	-	1'-3"3/4

Symbol explanation

Symbol	Symbol explanation
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain
T_h	Throat thickness of weld
L_s	Leg size of weld
L	Length of weld
L_c	Length of critical weld element
F_n	Force in weld critical element
ϕRn_w	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	Yes		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	SK/MS
Description	Roof Beam Splice - Vertical
Date	09-02-2024
Code	AISC/ACI

Material

Steel	A36, A913 Gr.50

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Vertical Splice For Roof Beam
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

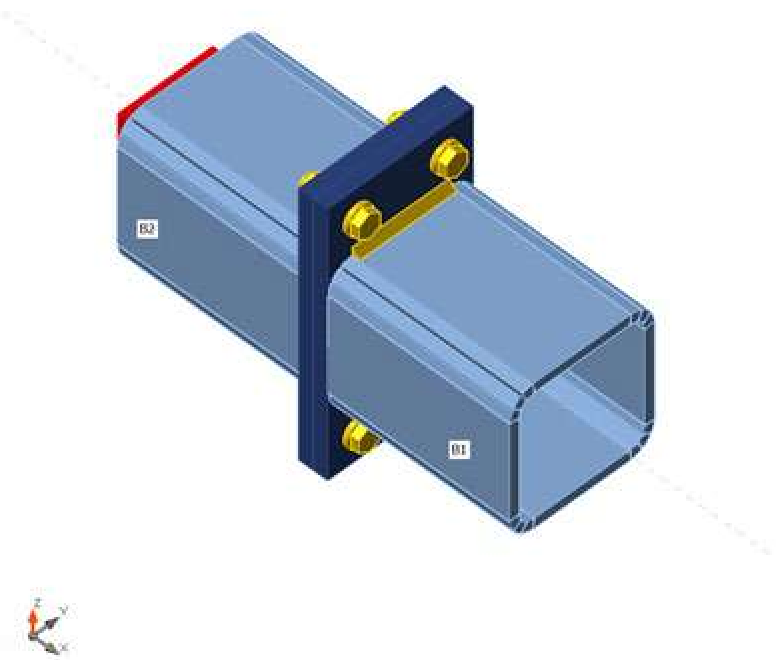
Members

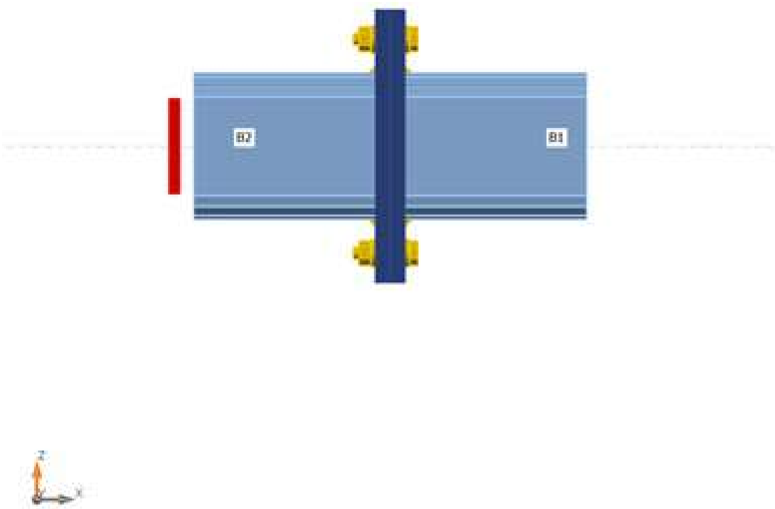
Geometry

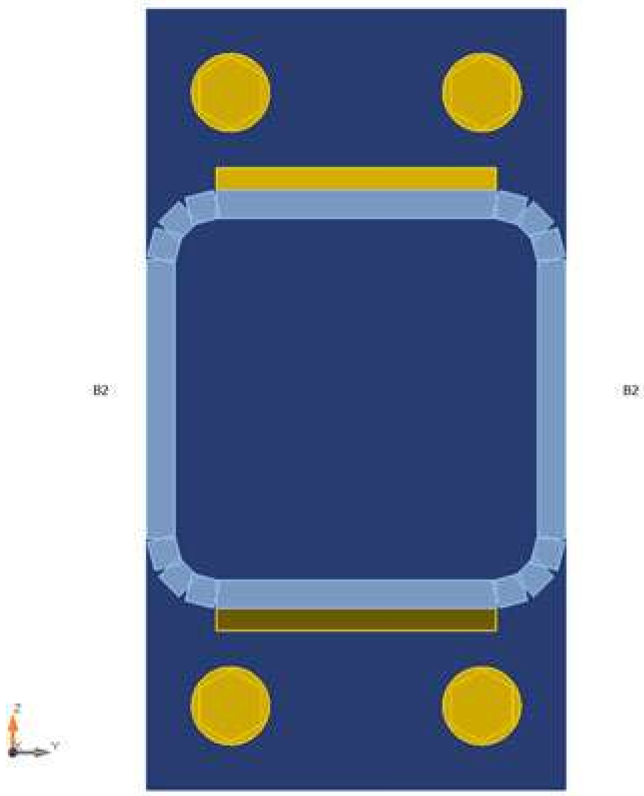
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
B1	2 - SHS150/150/10.0	0.0	0.0	0.0	0"	0"	0"
B2	2 - SHS150/150/10.0	180.0	0.0	0.0	0"	0"	0"

Supports and forces

Name	Support	Forces in	X [in]
B1 / end		Node	0"
B2 / end	N-Vy-Vz-Mx-My-Mz	Node	0"



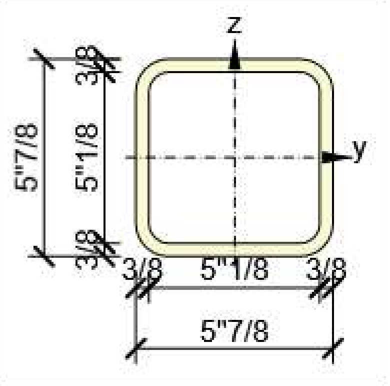




Cross-sections

Name	Material
2 - SHS150/150/10.0	A913 Gr.50

Cross-sections

Name	Material	Drawing
2 - SHS150/150/10.0	A913 Gr.50	

Bolts

Name	Bolt assembly	Diameter [in]	f_u [psi]	Gross area [in^2]
1/2 A325	1/2 A325	1/2	120000.0	0.1964

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	B1 / End	0.000	0.000	-5.230	0.00	19.28	0.00
	B2 / End	0.000	0.000	5.230	0.00	19.28	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.000	0.000	0.000	0.00	0.00	0.00

Check

Summary

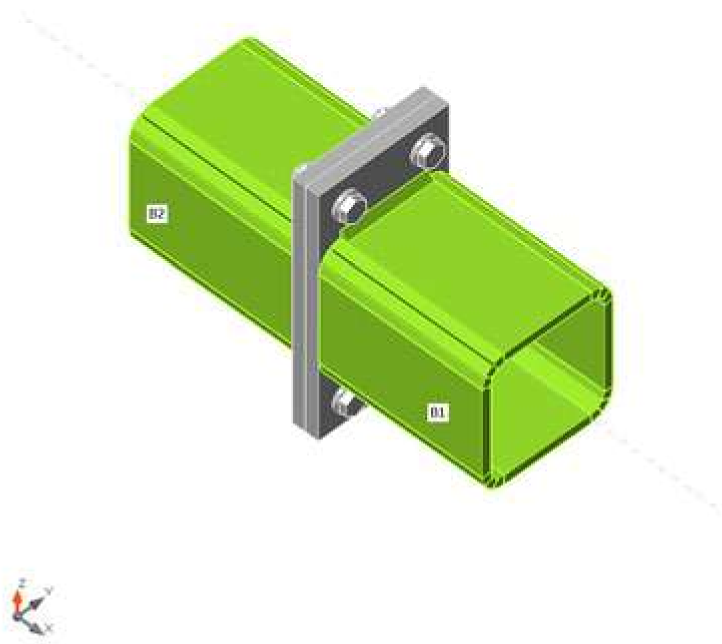
Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Bolts	0.3 < 100%	OK
Welds	80.5 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

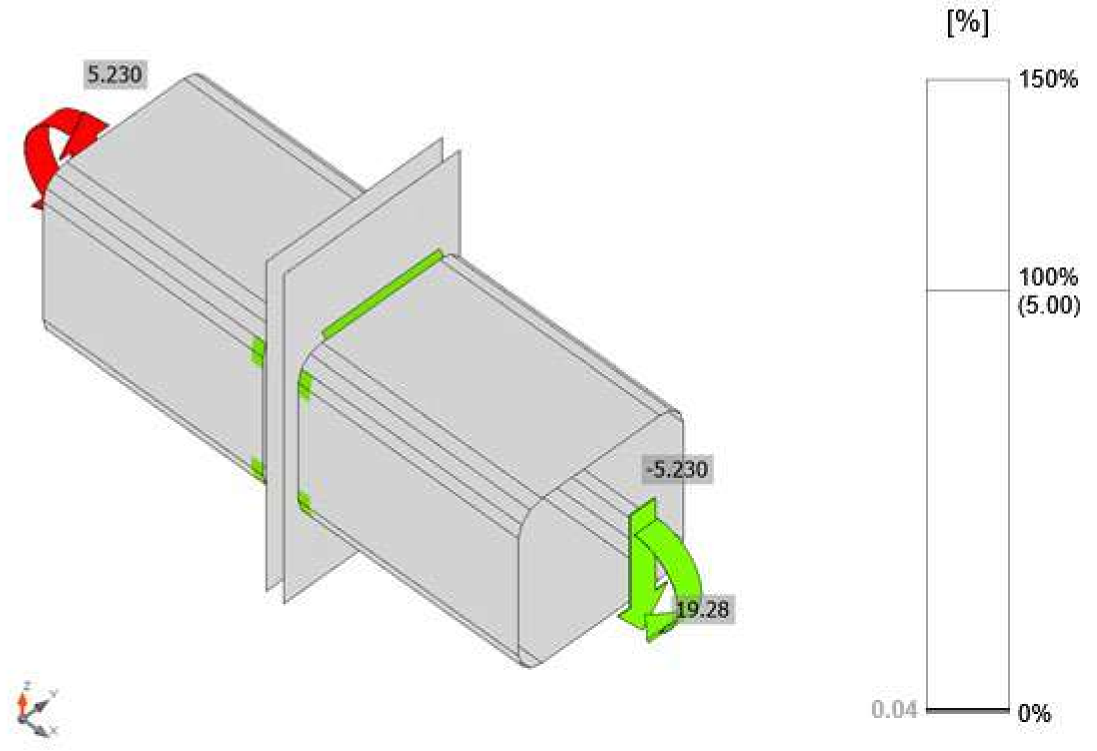
Name	t_p [in]	Loads	σ_{Ed} [psi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [psi]	Status
B1	3/8	LE1	37923.7	0.0	0.0	OK
B2	3/8	LE1	39202.0	0.0	0.0	OK
PP1a	9/16	LE1	4011.3	0.0	70.6	OK
PP1b	9/16	LE1	434.6	0.0	70.6	OK

Design data

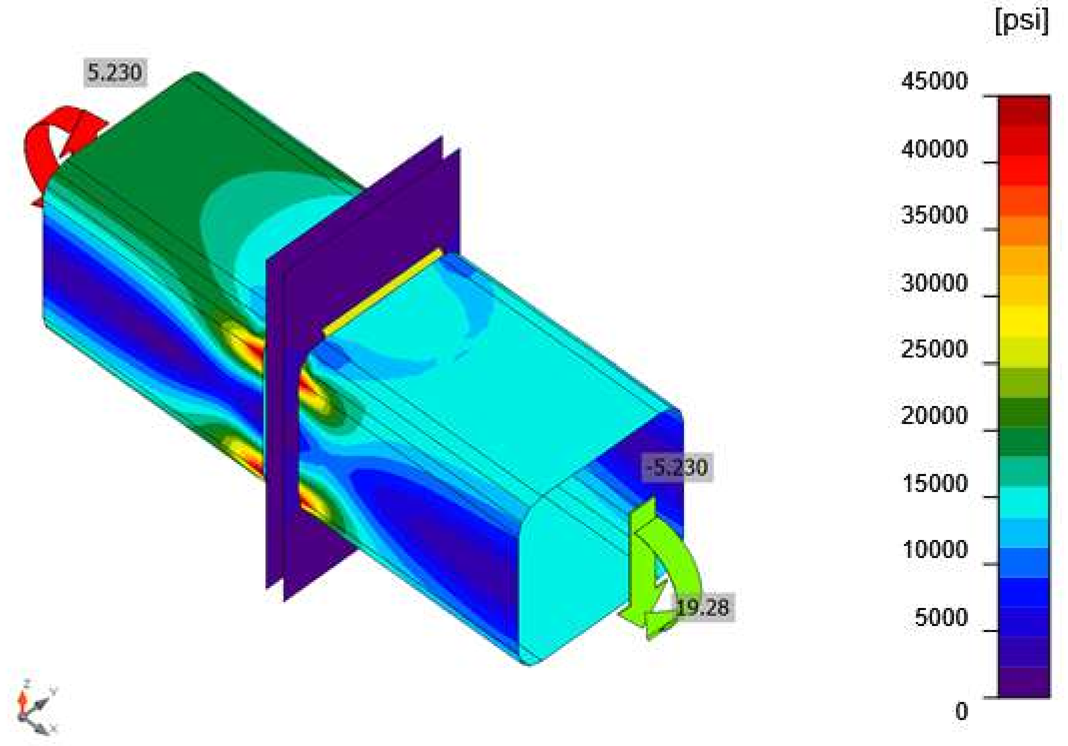
Material	F_y [psi]	ε_{lim} [%]
A913 Gr.50	50000.3	5.0



Overall check, LE1

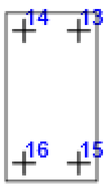


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B13	1/2 A325 - 1	LE1	0.029	0.010	31.085	0.2	0.1	-	OK	OK
	B14	1/2 A325 - 1	LE1	0.034	0.010	31.087	0.3	0.1	-	OK	OK
	B15	1/2 A325 - 1	LE1	0.033	0.010	31.086	0.2	0.1	-	OK	OK
	B16	1/2 A325 - 1	LE1	0.034	0.010	31.088	0.3	0.1	-	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
1/2 A325 - 1	13.243	7.946

Detailed result for B13

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 13.243 kip ≥ $F_t =$ 0.029 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.1964 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 7.946 kip ≥ $V =$ 0.010 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.1964 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 31.085 kip ≥ $V =$ 0.010 kip

Where:

l_c = 7/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 1/2 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B14

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 13.243 kip ≥ $F_t =$ 0.034 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.1964 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 7.946 kip ≥ $V =$ 0.010 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.1964 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 31.087 kip ≥ $V =$ 0.010 kip

Where:

l_c = 7/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 1/2 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B15

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 13.243 kip ≥ $F_t =$ 0.33 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.1964 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 7.946 kip ≥ $V =$ 0.10 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.1964 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 31.086 kip ≥ $V =$ 0.10 kip

Where:

l_c = 7/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 1/2 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B16

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 13.243 kip ≥ $F_t =$ 0.34 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.1964 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 7.946 kip ≥ $V =$ 0.10 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.1964 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 31.088 kip ≥ $V =$ 0.10 kip

Where:

l_c = 7/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 1/2 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
PP1a	B1-w 2	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
PP1a	B1-w 4	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
PP1a	B2-w 2	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
PP1a	B2-w 4	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
PP1a	B1	E70xx	◢ 1/4	◢ 5/16	7"13/16	3/8	LE1	2.175	2.748	79.1	OK	OK
PP1a	B2	E70xx	◢ 1/4	◢ 5/16	7"13/16	3/8	LE1	2.214	2.748	80.5	OK	OK

Detailed result for PP1a / B1-w 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B1-w 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B2-w 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B2-w 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B1 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 2.748 kip ≥ $F_n =$ 2.175 kip

Where:

F_{nw} = 42000.0 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX}$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - Directional strength increase is not used for HSS welds

A_{we} = 0.0872 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for PP1a / B2 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 2.748 kip ≥ $F_n =$ 2.214 kip

Where:

F_{nw} = 42000.0 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX}$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - Directional strength increase is not used for HSS welds

A_{we} = 0.0872 in^2 – effective area of weld critical element

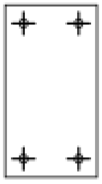
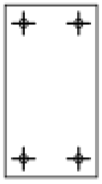
ϕ = 0.75 – resistance factor for welded connections

Buckling

Buckling analysis was not calculated.

Bill of material

Manufacturing operations

Name	Plates [in]	Shape	Nr.	Welds [in]	Length [in]	Bolts	Nr.
PP1	P9/16x5"7/8-11" (A913 Gr.50)		1			1/2 A325	4
	P9/16x5"7/8-11" (A913 Gr.50)		1				

Welds

Type	Material	Throat thickness [in]	Leg size [in]	Length [in]
Fillet	E70xx	1/4	5/16	1'-3"3/4
Butt	E70xx	-	-	1'-3"3/4

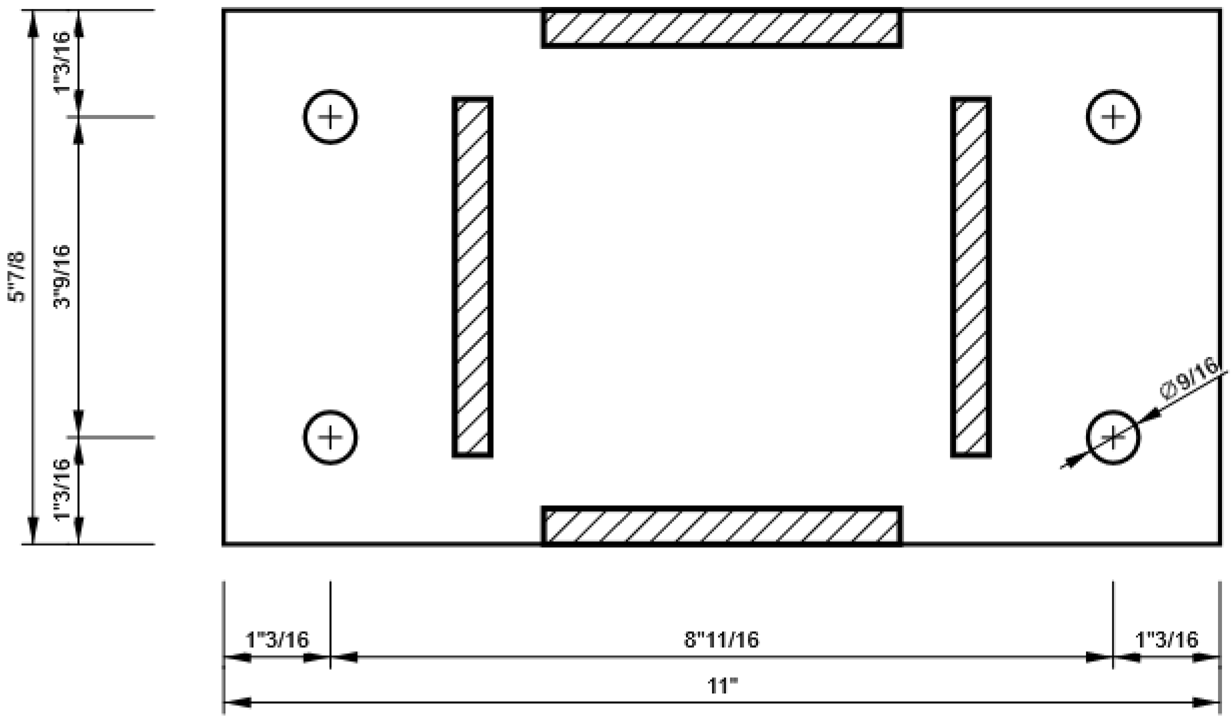
Bolts

Name	Grip length [in]	Count
1/2 A325	1"3/16	4

Drawing

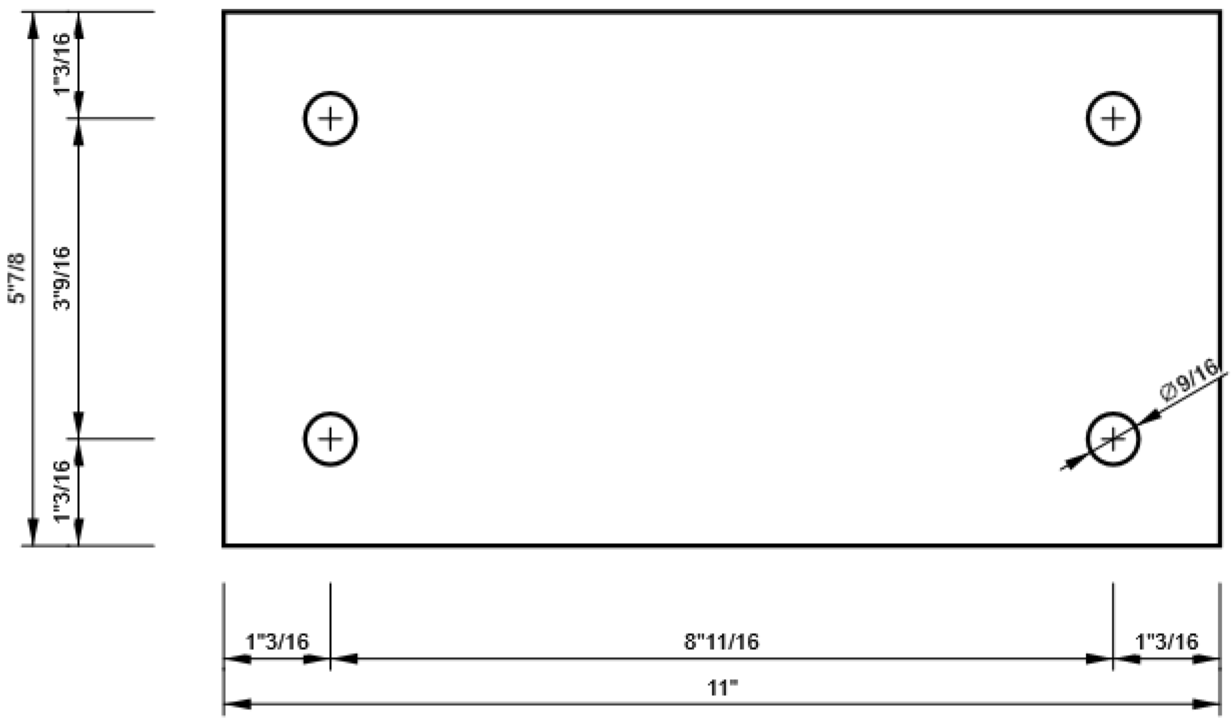
PP1 - PP1a

P9/16x11"-5"7/8 (A913 Gr.50)



PP1 - PP1b

P9/16x11"-5"7/8 (A913 Gr.50)



Symbol explanation

Symbol	Symbol explanation
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain
F_t	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
$\phi Rn_{Bearing}$	Bolt bearing resistance
Ut	Utilization
Ut_s	Utilization in shear
Ut_{ts}	Interaction of tension and shear EN 1993-1-8 – Tab. 3.4
$\phi Rn_{Tension}$	Bolt tension resistance - AISC 360-16 – J3.6
ϕRn_{Shear}	Bolt shear resistance - AISC 360-16 – J3.6
◢	Fillet weld
T_h	Throat thickness of weld
L_s	Leg size of weld
L	Length of weld
L_c	Length of critical weld element
F_n	Force in weld critical element
ϕRn_w	Weld resistance - AISC 360-16 – J2-4

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	No		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Appendix- 17

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	SK/MS
Description	Roof Beam Horizontal Splice
Date	09-02-2024
Code	AISC/ACI

Material

Steel	A36, A913 Gr.50

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Roof Beam Horizontal Splice
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

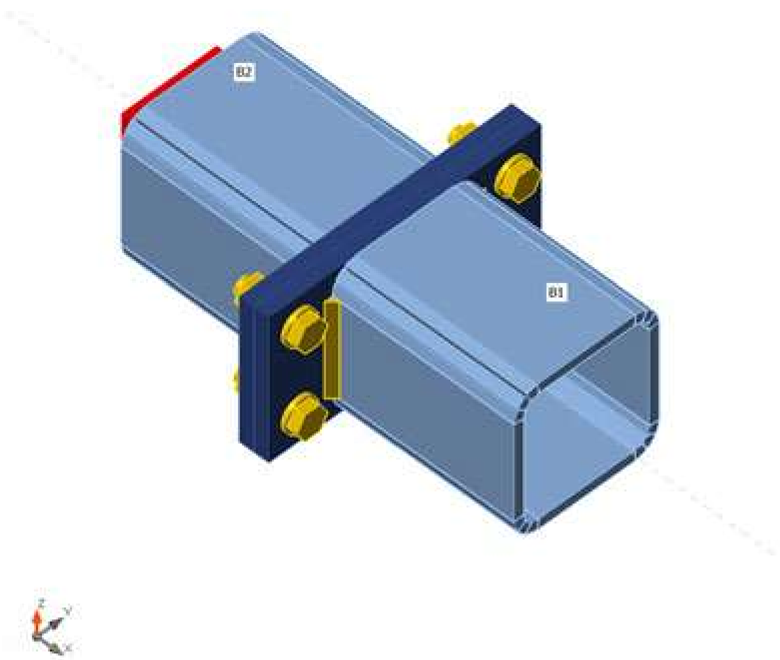
Members

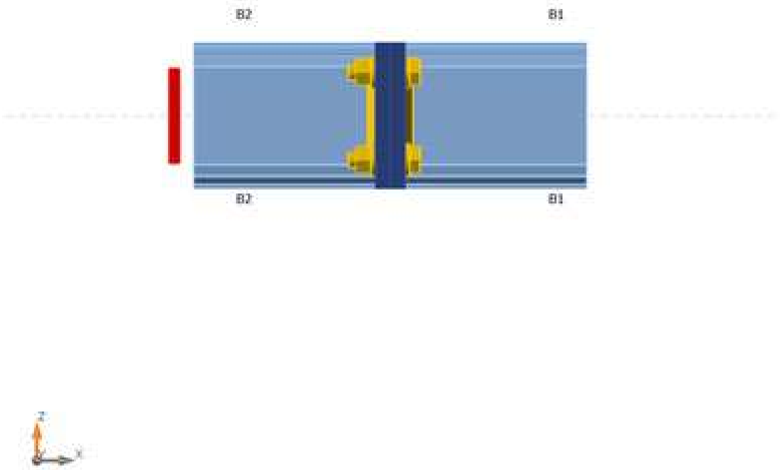
Geometry

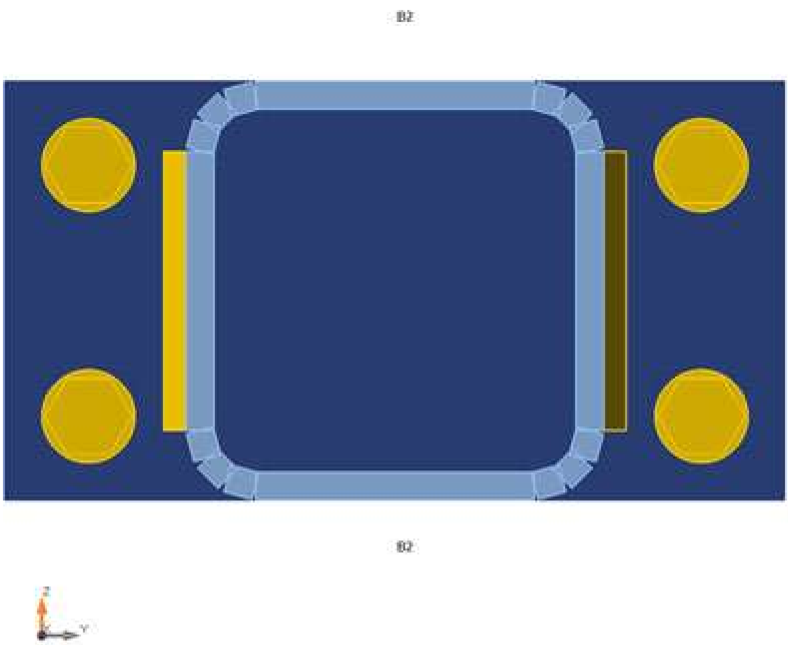
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
B1	2 - SHS150/150/10.0	0.0	0.0	90.0	0"	0"	0"
B2	2 - SHS150/150/10.0	180.0	0.0	90.0	0"	0"	0"

Supports and forces

Name	Support	Forces in	X [in]
B1 / end		Node	0"
B2 / end	N-Vy-Vz-Mx-My-Mz	Node	0"



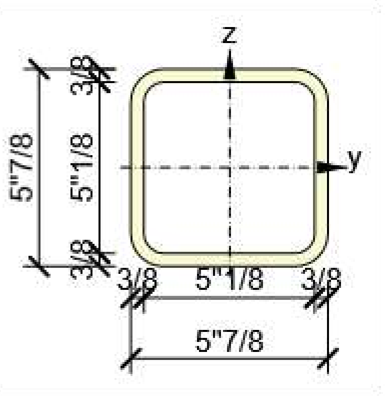




Cross-sections

Name	Material
2 - SHS150/150/10.0	A913 Gr.50

Cross-sections

Name	Material	Drawing
2 - SHS150/150/10.0	A913 Gr.50	

Bolts

Name	Bolt assembly	Diameter [in]	f_u [psi]	Gross area [in^2]
5/8 A325	5/8 A325	5/8	120000.0	0.3068

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	B1 / End	0.000	-11.200	0.000	0.00	0.00	-40.91
	B2 / End	0.000	11.200	0.000	0.00	0.00	-40.91

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.000	0.000	0.000	0.00	0.00	0.00

Check

Summary

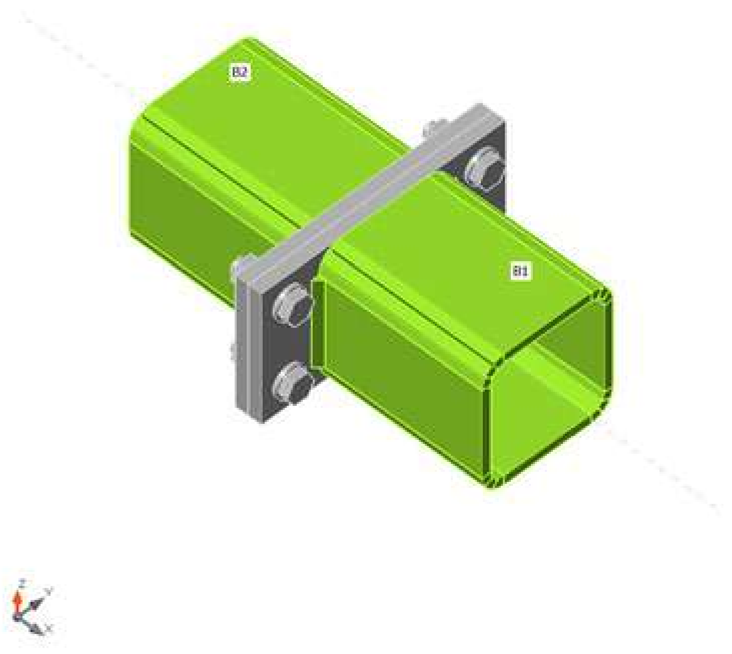
Name	Value	Check status
Analysis	100.0%	OK
Plates	0.3 < 5.0%	OK
Bolts	1.3 < 100%	OK
Welds	80.2 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

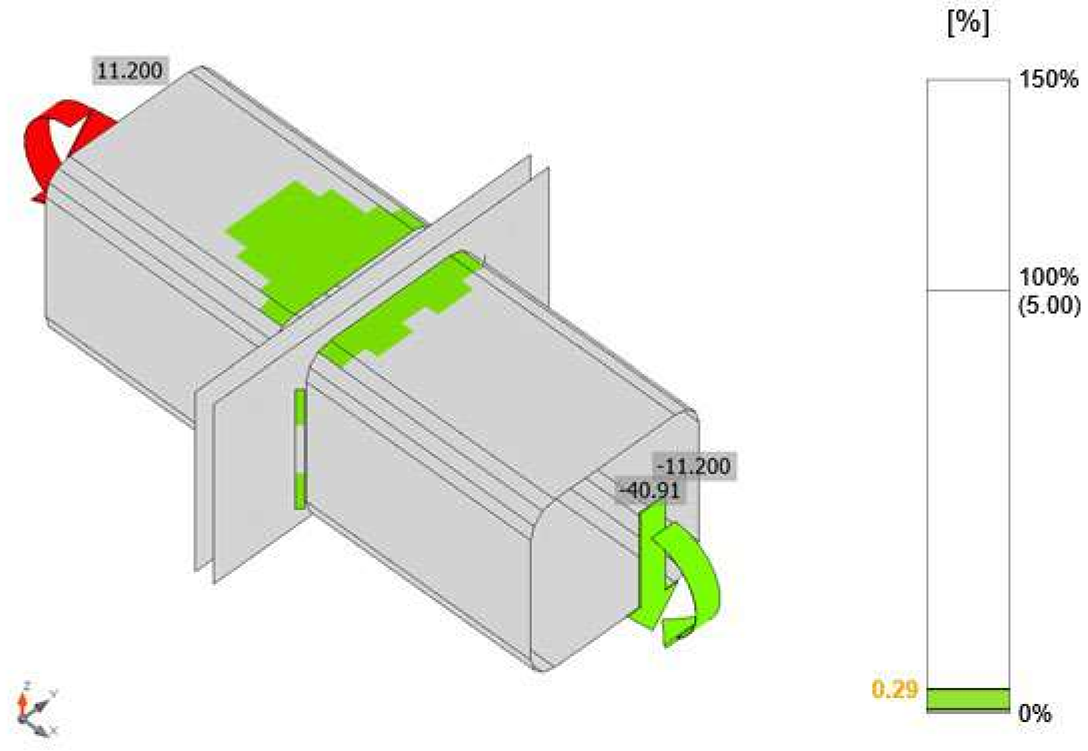
Name	t_p [in]	Loads	σ_{Ed} [psi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [psi]	Status
B1	3/8	LE1	45077.8	0.3	0.0	OK
B2	3/8	LE1	45083.9	0.3	0.0	OK
PP1a	9/16	LE1	4165.0	0.0	500.3	OK
PP1b	9/16	LE1	2461.7	0.0	500.3	OK

Design data

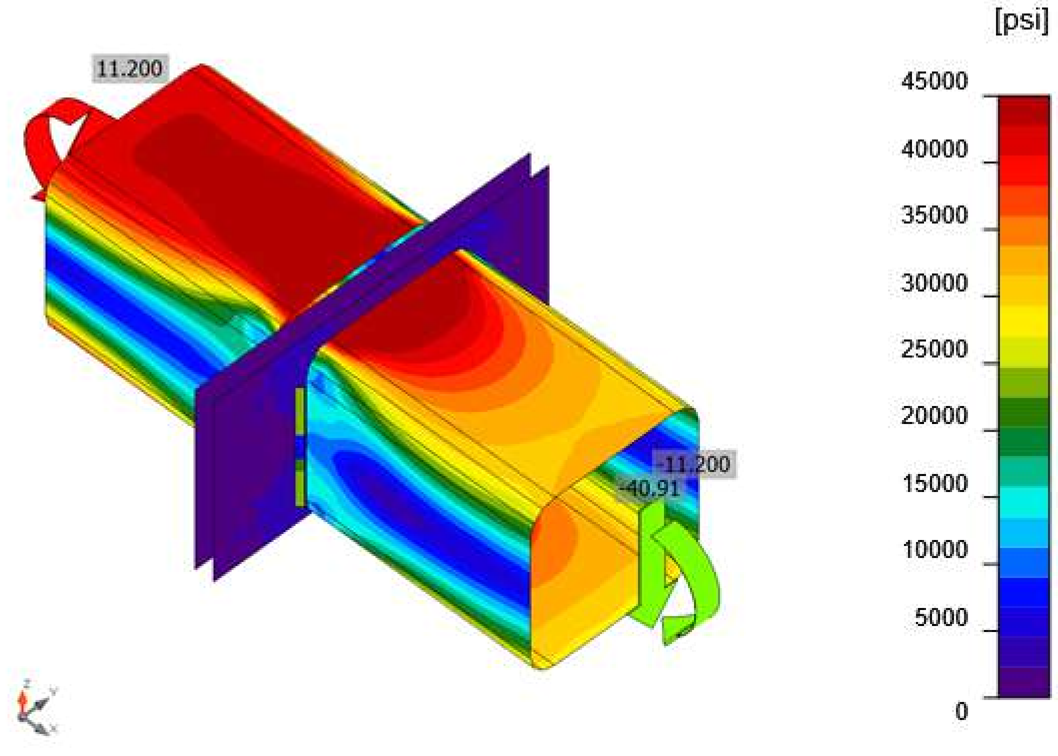
Material	F_y [psi]	ε_{lim} [%]
A913 Gr.50	50000.3	5.0



Overall check, LE1

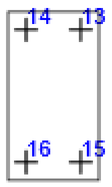


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B13	5/8 A325 - 1	LE1	0.264	0.032	29.553	1.3	0.3	-	OK	OK
	B14	5/8 A325 - 1	LE1	0.042	0.006	37.724	0.2	0.1	-	OK	OK
	B15	5/8 A325 - 1	LE1	0.266	0.032	29.450	1.3	0.3	-	OK	OK
	B16	5/8 A325 - 1	LE1	0.045	0.007	40.297	0.2	0.1	-	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
5/8 A325 - 1	20.691	12.415

Detailed result for B13

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.264 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 12.415 kip ≥ $V =$ 0.032 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 29.553 kip ≥ $V =$ 0.032 kip

Where:

l_c = 7/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B14

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.042 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 12.415 kip ≥ $V =$ 0.06 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 37.724 kip ≥ $V =$ 0.06 kip

Where:

l_c = 1"1/16 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B15

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.266 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 12.415 kip ≥ $V =$ 0.032 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 29.450 kip ≥ $V =$ 0.032 kip

Where:

l_c = 7/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B16

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.045 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 12.415 kip ≥ $V =$ 0.07 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 40.297 kip ≥ $V =$ 0.07 kip

Where:

l_c = 1"3/16 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
PP1a	B1-w 2	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
PP1a	B1-w 4	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
PP1a	B2-w 2	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
PP1a	B2-w 4	E70xx	-	-	3"15/16	-	-	-	-	-	OK	OK
PP1a	B1	E70xx	◢ 1/4	◢ 5/16	7"13/16	3/8	LE1	2.136	2.748	77.7	OK	OK
PP1a	B2	E70xx	◢ 1/4	◢ 5/16	7"13/16	3/8	LE1	2.204	2.748	80.2	OK	OK

Detailed result for PP1a / B1-w 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B1-w 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B2-w 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B2-w 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B1 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 2.748 kip ≥ $F_n =$ 2.136 kip

Where:

F_{nw} = 42000.0 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX}$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - Directional strength increase is not used for HSS welds

A_{we} = 0.0872 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for PP1a / B2 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 2.748 kip ≥ $F_n =$ 2.204 kip

Where:

F_{nw} = 42000.0 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX}$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - Directional strength increase is not used for HSS welds

A_{we} = 0.0872 in^2 – effective area of weld critical element

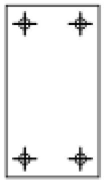
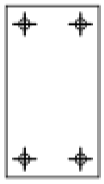
ϕ = 0.75 – resistance factor for welded connections

Buckling

Buckling analysis was not calculated.

Bill of material

Manufacturing operations

Name	Plates [in]	Shape	Nr.	Welds [in]	Length [in]	Bolts	Nr.
PP1	P9/16x5"7/8-11" (A913 Gr.50)		1			5/8 A325	4
	P9/16x5"7/8-11" (A913 Gr.50)		1				

Welds

Type	Material	Throat thickness [in]	Leg size [in]	Length [in]
Fillet	E70xx	1/4	5/16	1'-3"3/4
Butt	E70xx	-	-	1'-3"3/4

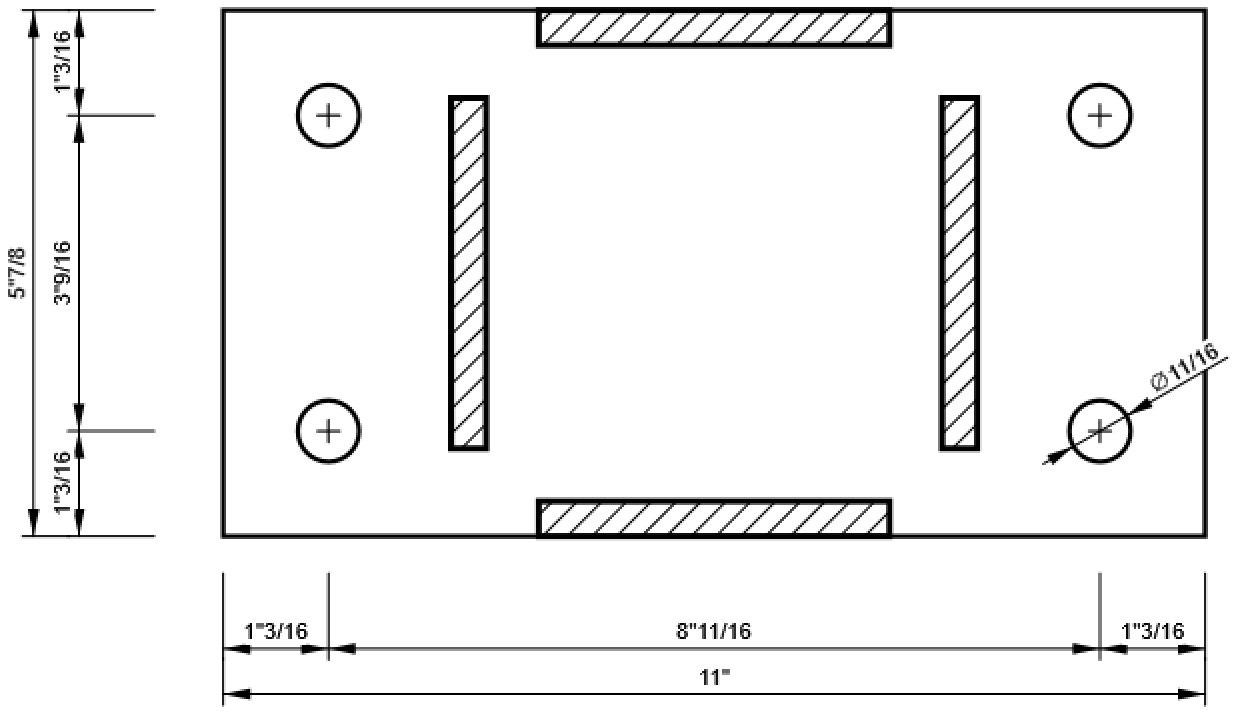
Bolts

Name	Grip length [in]	Count
5/8 A325	1"3/16	4

Drawing

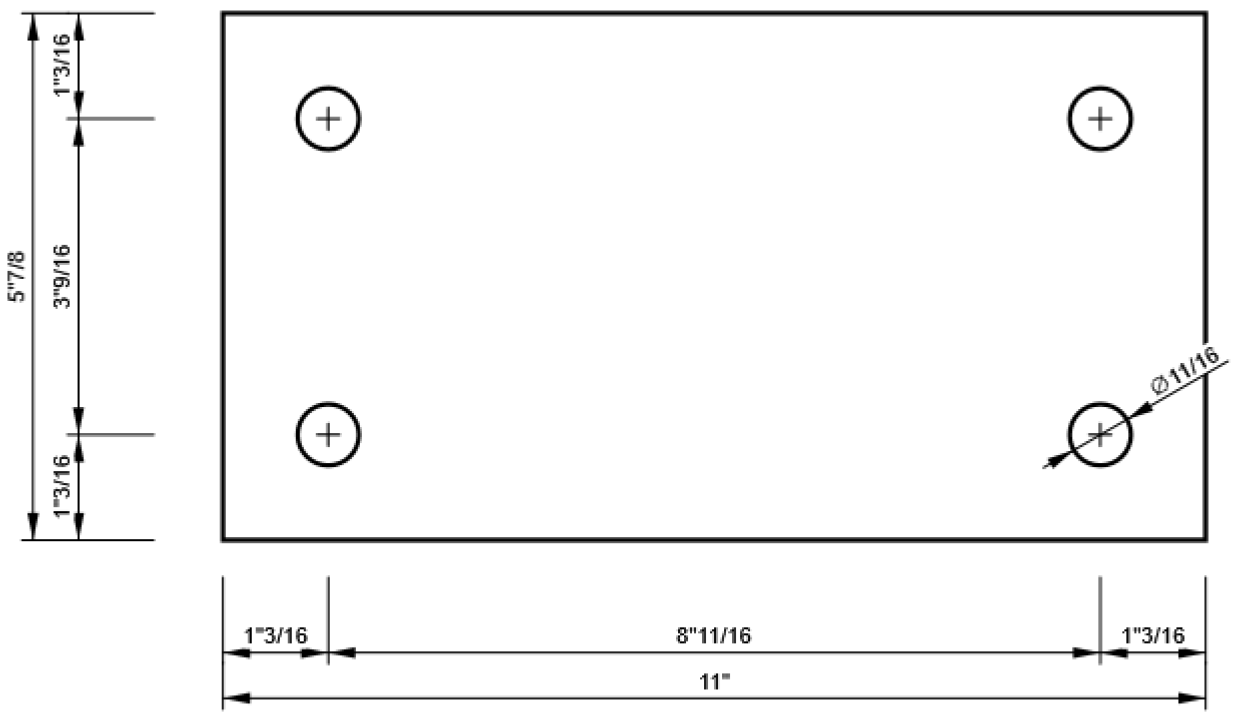
PP1 - PP1a

P9/16x11"-5"7/8 (A913 Gr.50)



PP1 - PP1b

P9/16x11"-5"7/8 (A913 Gr.50)



Symbol explanation

Symbol	Symbol explanation
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain
F_t	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
$\phi Rn_{Bearing}$	Bolt bearing resistance
Ut	Utilization
Ut_s	Utilization in shear
Ut_{ts}	Interaction of tension and shear EN 1993-1-8 – Tab. 3.4
$\phi Rn_{Tension}$	Bolt tension resistance - AISC 360-16 – J3.6
ϕRn_{Shear}	Bolt shear resistance - AISC 360-16 – J3.6
◢	Fillet weld
T_h	Throat thickness of weld
L_s	Leg size of weld
L	Length of weld
L_c	Length of critical weld element
F_n	Force in weld critical element
ϕRn_w	Weld resistance - AISC 360-16 – J2-4

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	No		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Appendix- 18

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	SK/MS
Description	RHS250x150 Beam Horizontal Splice
Date	09-02-2024
Code	AISC/ACI

Material

Steel	A36, A913 Gr.50

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	RHS250x150 Beam Horizontal Splice
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

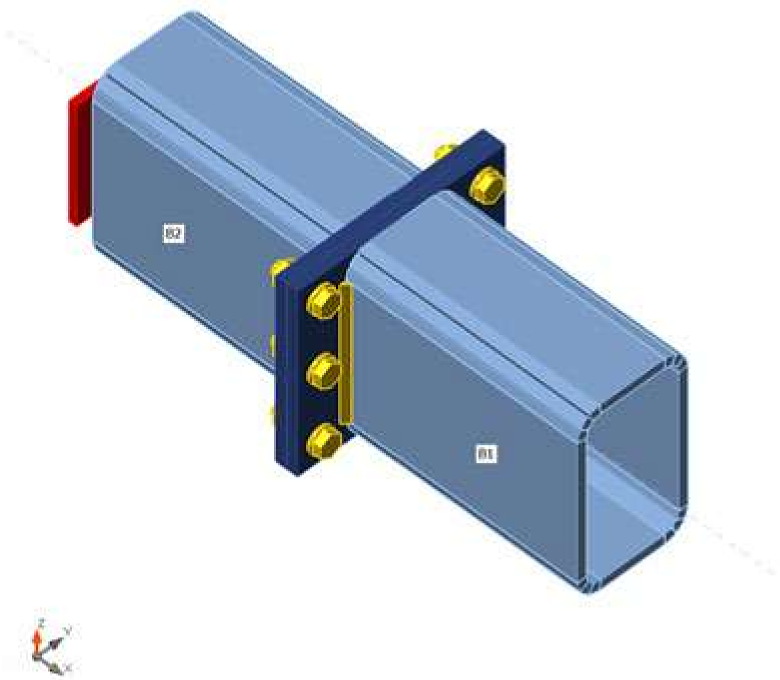
Members

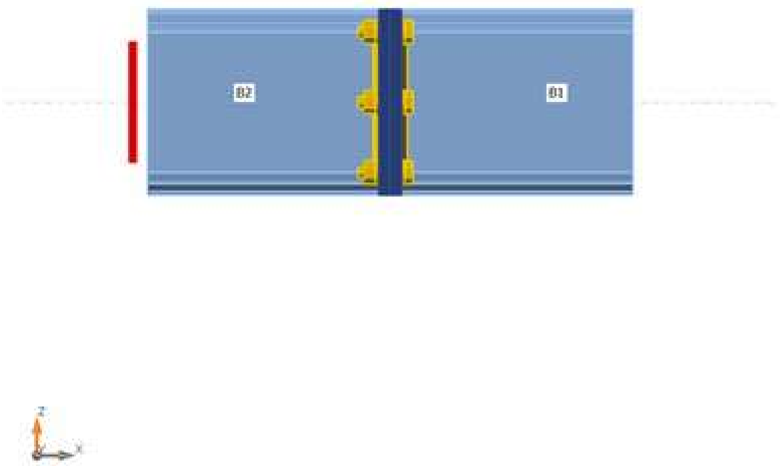
Geometry

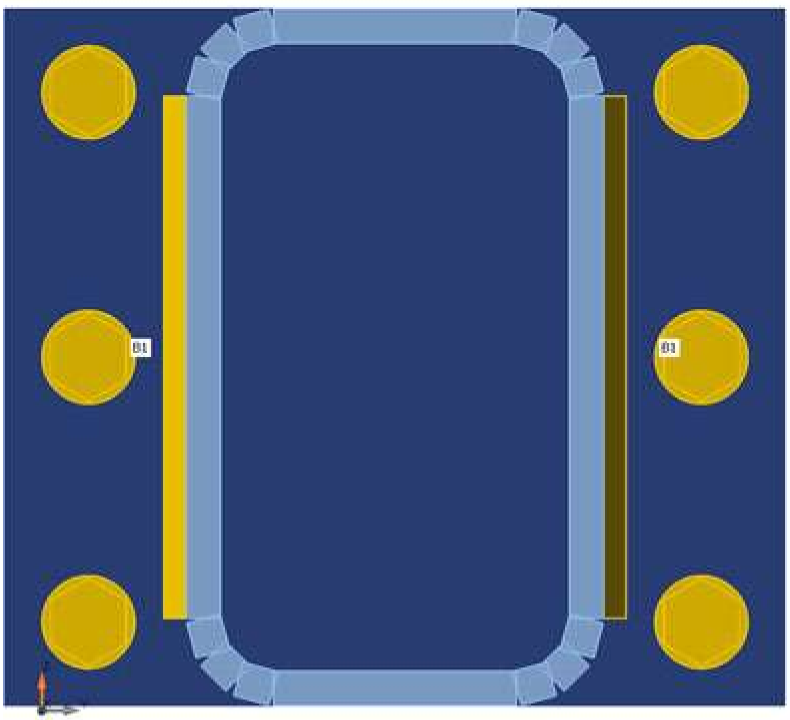
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
B1	3 - RHS250/150/12.5	0.0	0.0	0.0	0"	0"	0"
B2	3 - RHS250/150/12.5	180.0	0.0	0.0	0"	0"	0"

Supports and forces

Name	Support	Forces in	X [in]
B1 / end		Node	0"
B2 / end	N-Vy-Vz-Mx-My-Mz	Node	0"



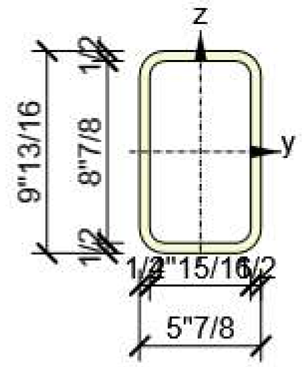




Cross-sections

Name	Material
3 - RHS250/150/12.5	A913 Gr.50

Cross-sections

Name	Material	Drawing
3 - RHS250/150/12.5	A913 Gr.50	

Bolts

Name	Bolt assembly	Diameter [in]	f_u [psi]	Gross area [in^2]
5/8 A325	5/8 A325	5/8	120000.0	0.3068

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	B1 / End	0.000	0.000	-10.590	0.00	43.25	0.00
	B2 / End	0.000	0.000	10.590	0.00	43.25	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.000	0.000	0.000	0.00	0.00	0.00

Check

Summary

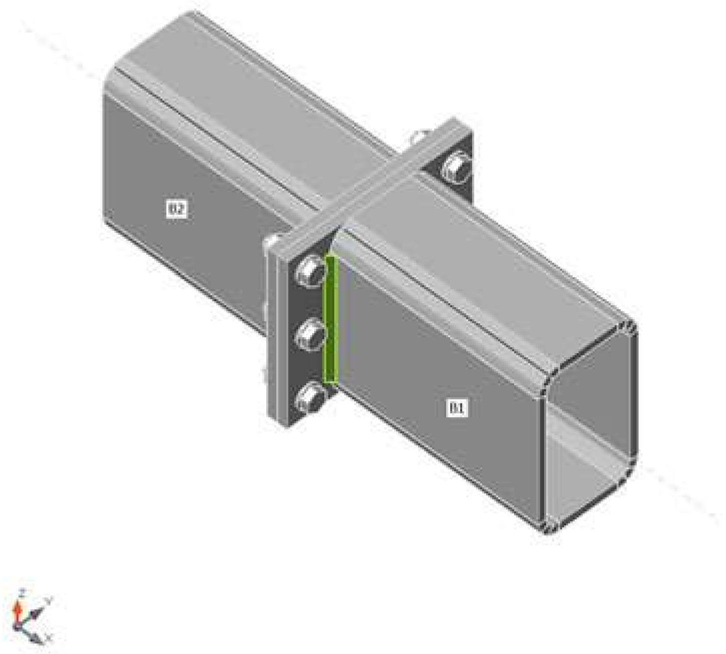
Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Bolts	0.4 < 100%	OK
Welds	76.3 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

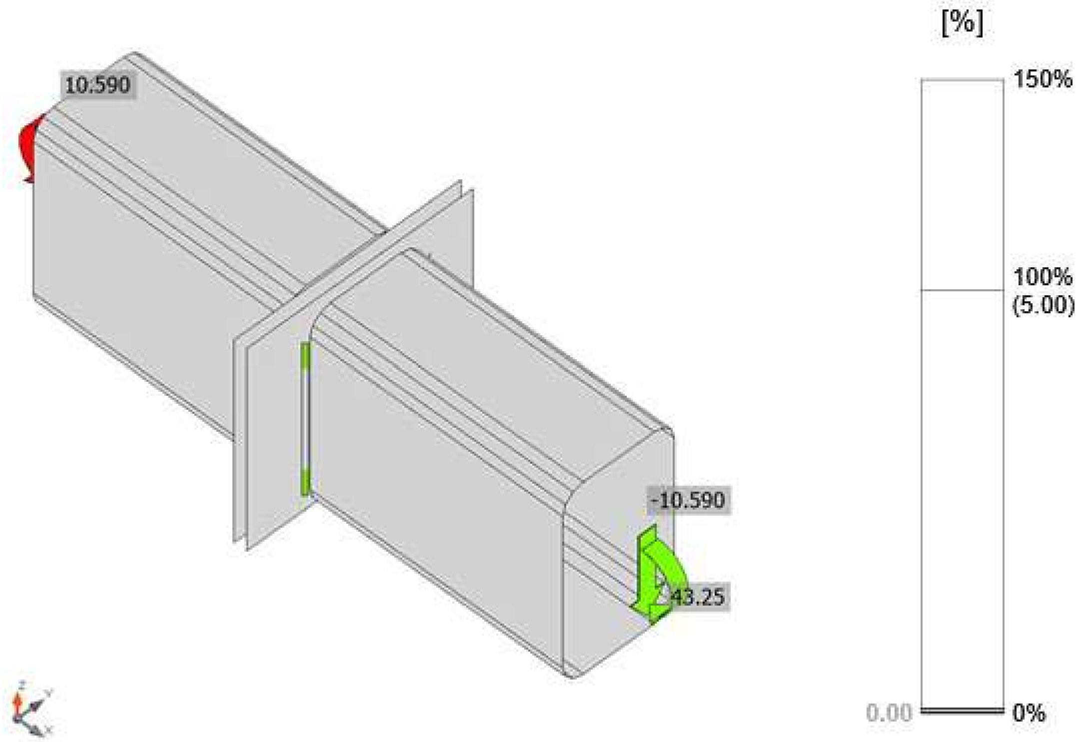
Name	t_p [in]	Loads	σ_{Ed} [psi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [psi]	Status
B1	1/2	LE1	26288.3	0.0	0.0	OK
B2	1/2	LE1	24312.3	0.0	0.0	OK
PP1a	9/16	LE1	2128.1	0.0	129.7	OK
PP1b	9/16	LE1	606.2	0.0	129.7	OK

Design data

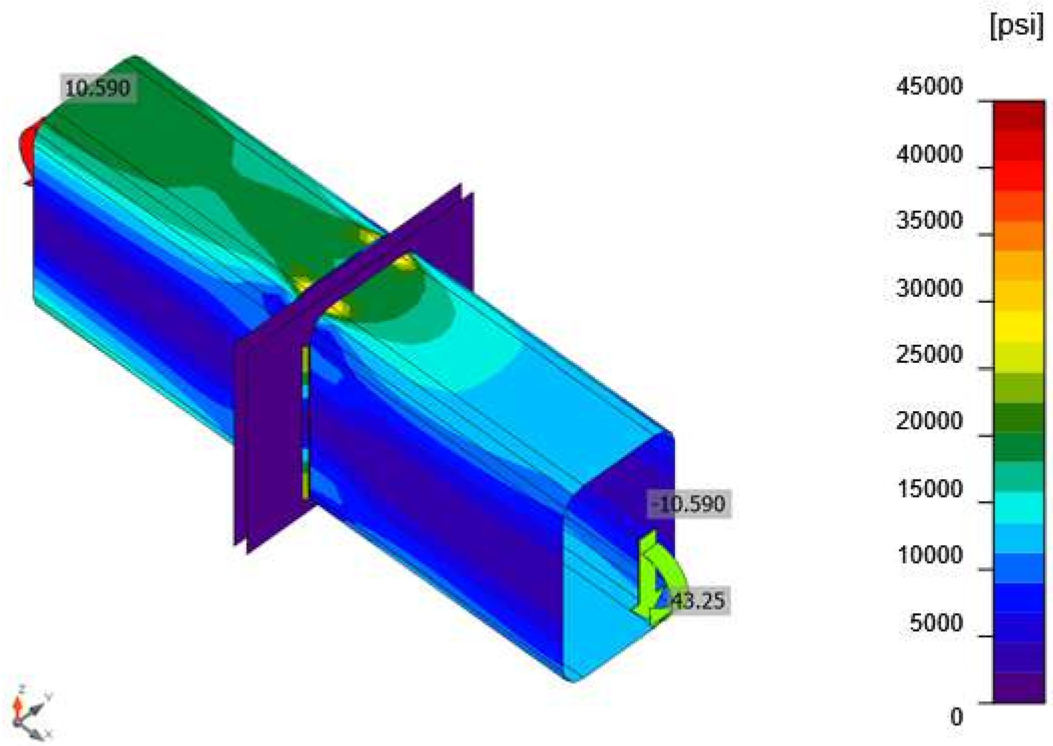
Material	F_y [psi]	ε_{lim} [%]
A913 Gr.50	50000.3	5.0



Overall check, LE1

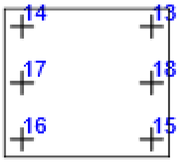


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B13	5/8 A325 - 1	LE1	0.076	0.016	29.632	0.4	0.1	-	OK	OK
	B14	5/8 A325 - 1	LE1	0.079	0.015	29.611	0.4	0.1	-	OK	OK
	B15	5/8 A325 - 1	LE1	0.017	0.011	29.425	0.1	0.1	-	OK	OK
	B16	5/8 A325 - 1	LE1	0.016	0.011	29.112	0.1	0.1	-	OK	OK
	B17	5/8 A325 - 1	LE1	0.036	0.002	42.446	0.2	0.0	-	OK	OK
	B18	5/8 A325 - 1	LE1	0.039	0.002	39.661	0.2	0.0	-	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
5/8 A325 - 1	20.691	12.415

Detailed result for B13

Tension resistance check (AISC 360-16 – J3-1)

$$\phi R_n = \phi \cdot F_{nt} \cdot A_b = \ 20.691 \text{ kip} \geq F_t = \ 0.076 \text{ kip}$$

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$$\phi R_n = \phi \cdot F_{nv} \cdot A_b = \ 12.415 \text{ kip} \geq V = \ 0.016 \text{ kip}$$

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$$

$$\phi R_n = \ 29.632 \text{ kip} \geq V = \ 0.16 \text{ kip}$$

Where:

l_c = 7/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B14

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.079 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 12.415 kip ≥ $V =$ 0.015 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 29.611 kip ≥ $V =$ 0.15 kip

Where:

l_c = 7/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B15

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.17 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 12.415 kip ≥ $V =$ 0.11 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 29.425 kip ≥ $V =$ 0.11 kip

Where:

l_c = 7/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B16

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.16 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 12.415 kip ≥ $V =$ 0.11 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 29.112 kip ≥ $V =$ 0.11 kip

Where:

l_c = 13/16 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B17

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.036 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 12.415 kip ≥ $V =$ 0.002 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 42.446 kip ≥ $V =$ 0.002 kip

Where:

l_c = 1"1/4 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 9/16 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B18

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.039 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 12.415 kip ≥ $V =$ 0.002 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 39.661 kip $\geq$ $V =$ 0.002 kip

Where:

l_c = 1"1/8 in	– clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
t = 9/16 in	– thickness of the plate
d = 5/8 in	– diameter of a bolt
F_u = 65000.1 psi	– tensile strength of the connected material
ϕ = 0.75	– resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
PP1a	B1-w 3	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
PP1a	B1-w 1	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
PP1a	B2-w 3	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
PP1a	B2-w 1	E70xx	-	-	3"7/16	-	-	-	-	-	OK	OK
PP1a	B1	E70xx	◢ 1/4	◢ 5/16	1'-2"11/16	5/8	LE1	3.227	4.268	75.6	OK	OK
PP1a	B2	E70xx	◢ 1/4	◢ 5/16	1'-2"11/16	5/8	LE1	3.257	4.268	76.3	OK	OK

Detailed result for PP1a / B1-w 3 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B1-w 1 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B2-w 3 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B2-w 1 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for PP1a / B1 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 4.268 kip ≥ $F_n =$ 3.227 kip

Where:

F_{nw} = 42000.0 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX}$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - Directional strength increase is not used for HSS welds

A_{we} = 0.1355 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for PP1a / B2 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 4.268 kip ≥ $F_n =$ 3.257 kip

Where:

F_{nw} = 42000.0 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX}$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - Directional strength increase is not used for HSS welds

A_{we} = 0.1355 in^2 – effective area of weld critical element

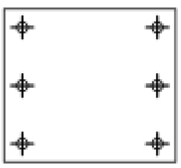
ϕ = 0.75 – resistance factor for welded connections

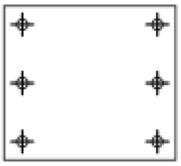
Buckling

Buckling analysis was not calculated.

Bill of material

Manufacturing operations

Name	Plates [in]	Shape	Nr.	Welds [in]	Length [in]	Bolts	Nr.
PP1	P9/16x11"-9"13/16 (A913 Gr.50)		1			5/8 A325	6

	P9/16x11"-9"13/16 (A913 Gr.50)		1				

Welds

Type	Material	Throat thickness [in]	Leg size [in]	Length [in]
Butt	E70xx	-	-	1'-1"3/4
Fillet	E70xx	1/4	5/16	2'-5"1/2

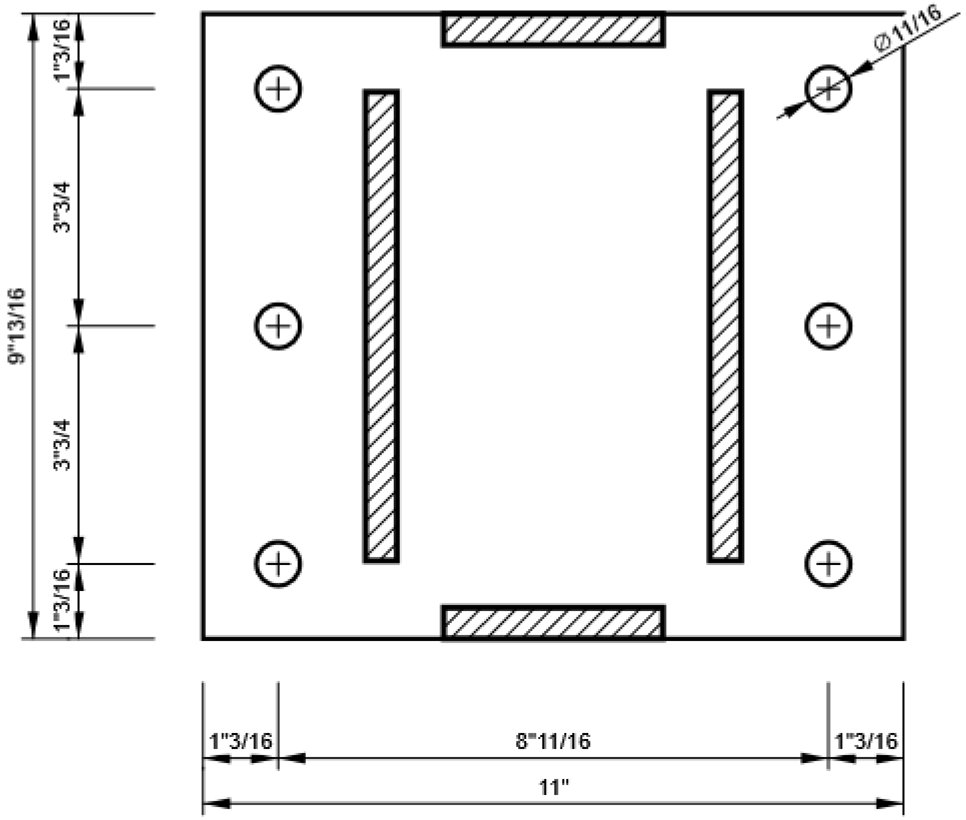
Bolts

Name	Grip length [in]	Count
5/8 A325	1"3/16	6

Drawing

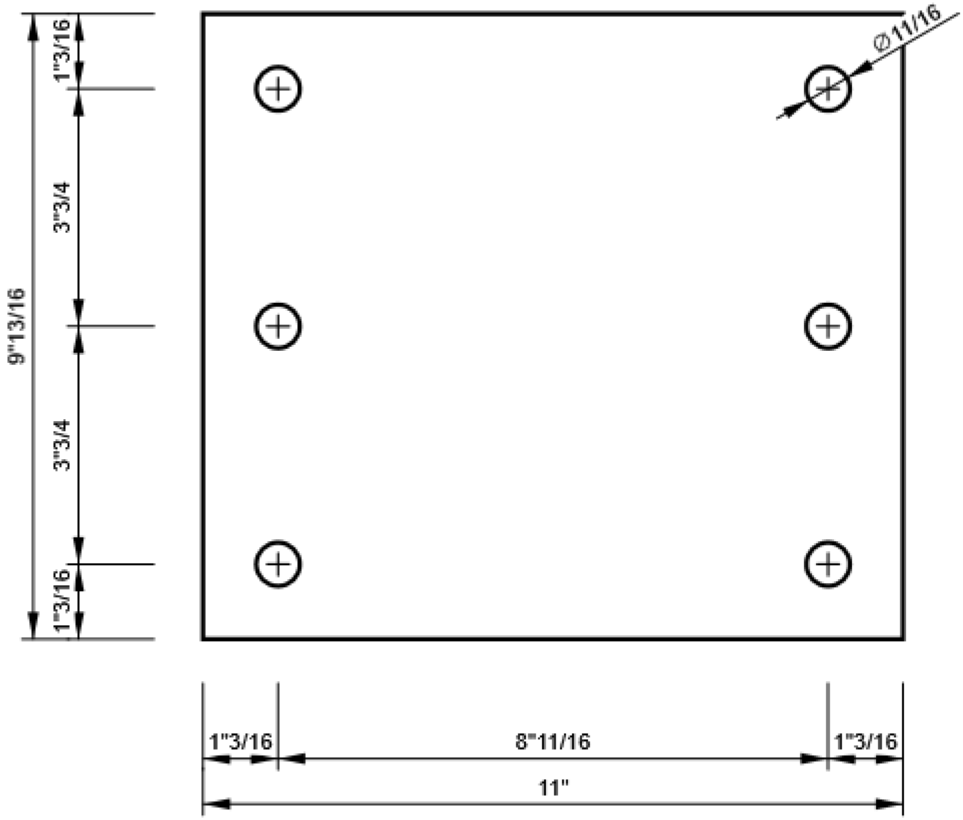
PP1 - PP1a

P9/16x9"13/16-11" (A913 Gr.50)



PP1 - PP1b

P9/16x9"13/16-11" (A913 Gr.50)



Symbol explanation

Symbol	Symbol explanation
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain
F_t	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
$\phi Rn_{Bearing}$	Bolt bearing resistance
Ut	Utilization
Ut_s	Utilization in shear
Ut_{ts}	Interaction of tension and shear EN 1993-1-8 – Tab. 3.4
$\phi Rn_{Tension}$	Bolt tension resistance - AISC 360-16 – J3.6
ϕRn_{Shear}	Bolt shear resistance - AISC 360-16 – J3.6
◢	Fillet weld
T_h	Throat thickness of weld
L_s	Leg size of weld
L	Length of weld
L_c	Length of critical weld element
F_n	Force in weld critical element
ϕRn_w	Weld resistance - AISC 360-16 – J2-4

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	No		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	MS/SK
Description	Beam to Beam Fin Plate Connection
Date	09-02-2024
Code	AISC/ACI

Material

Steel	A36, S 355, A913 Gr.50

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Beam to Beam Fin Plate Connection
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

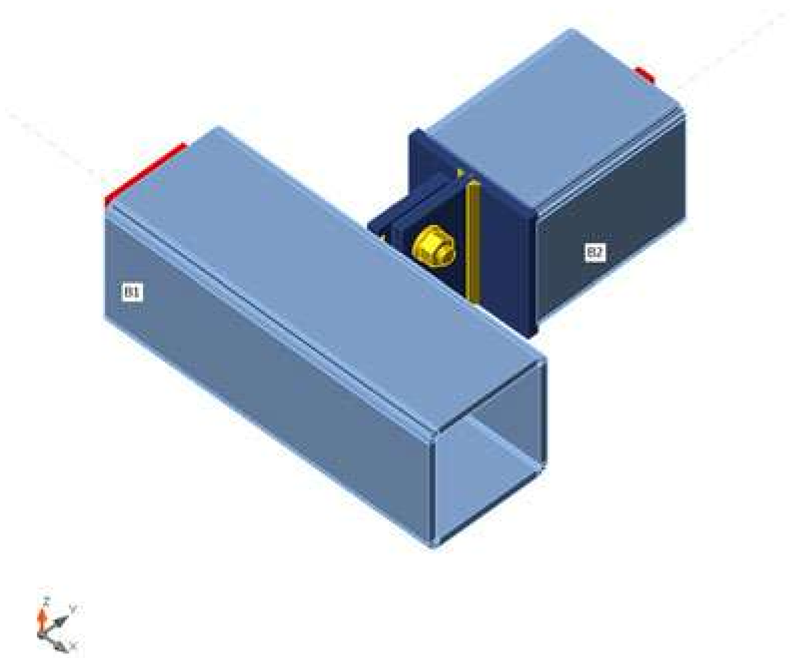
Members

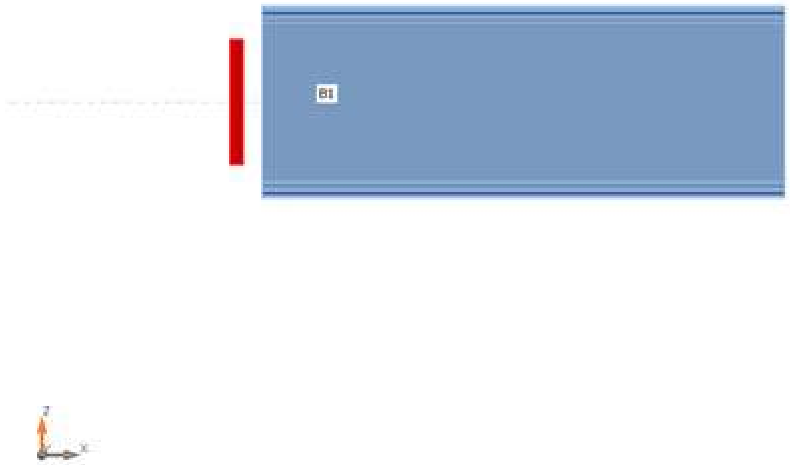
Geometry

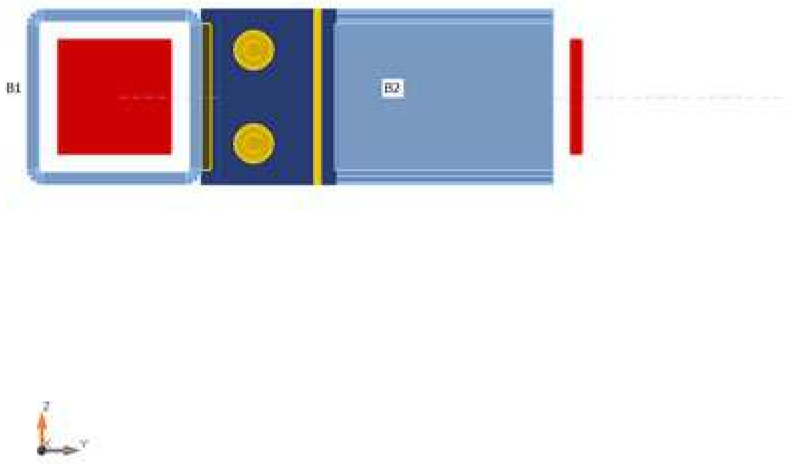
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
B1	13 - SHS150*150*10.0(RHS150x150)	0.0	0.0	-180.0	0"	-	
B2	13 - SHS150*150*10.0(RHS150x150)	90.0	0.0	-180.0	0"	-	

Supports and forces

Name	Support	Forces in	X [in]
B1 / begin	N-Vy-Vz-Mx-My-Mz	Node	0"
B1 / end		Node	0"
B2 / end	Mx-My-Mz	Bolts	0"



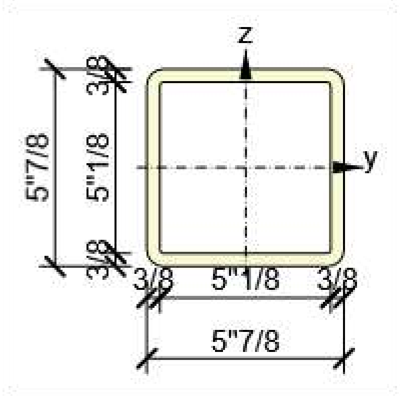




Cross-sections

Name	Material
13 - SHS150*150*10.0(RHS150x150)	A913 Gr.50

Cross-sections

Name	Material	Drawing
13 - SHS150*150*10.0(RHS150x150)	A913 Gr.50	

Bolts

Name	Bolt assembly	Diameter [in]	f_u [psi]	Gross area [in^2]
5/8 A325	5/8 A325	5/8	120000.0	0.3068

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	B1 / Begin	0.000	0.000	-2.700	0.00	0.00	0.00
	B1 / End	0.000	0.000	0.000	0.00	0.00	0.00
	B2 / End	0.000	0.000	2.700	0.00	0.00	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.000	0.000	0.000	0.00	0.00	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Bolts	8.7 < 100%	OK
Welds	19.8 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

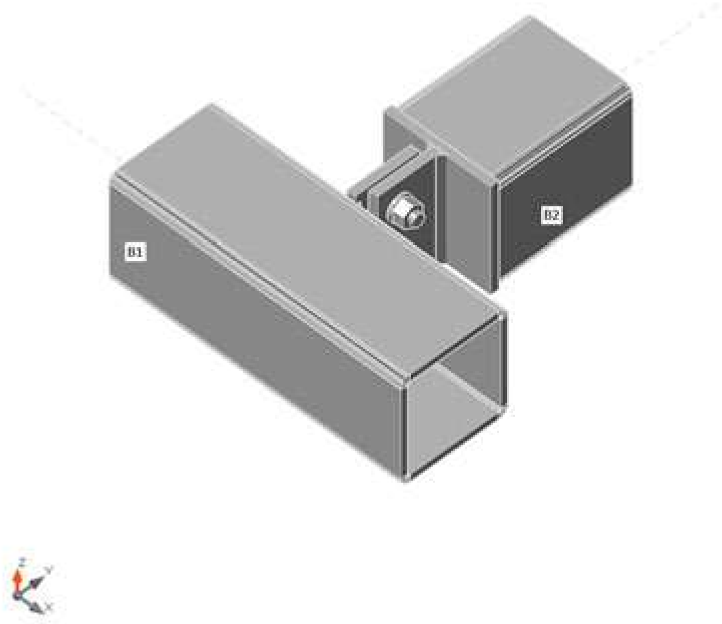
Name	t_p [in]	Loads	σ_{Ed} [psi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [psi]	Status
B1	3/8	LE1	5325.5	0.0	0.0	OK
B2	3/8	LE1	2996.0	0.0	0.0	OK
Plate 1	1/2	LE1	7320.5	0.0	0.0	OK
Plate 3	1/2	LE1	6590.8	0.0	0.0	OK
Plate 4	1/2	LE1	1559.3	0.0	0.0	OK

Design data

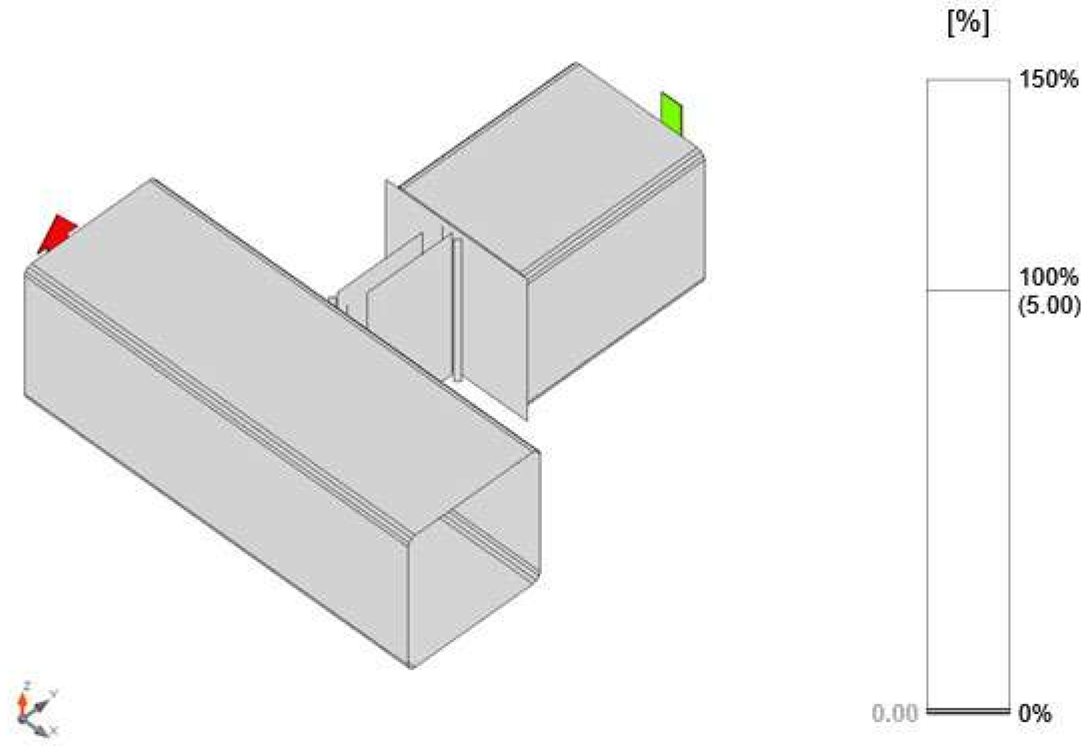
Material	F_y [psi]	ε_{lim} [%]
A913 Gr.50	50000.3	5.0

Symbol explanation

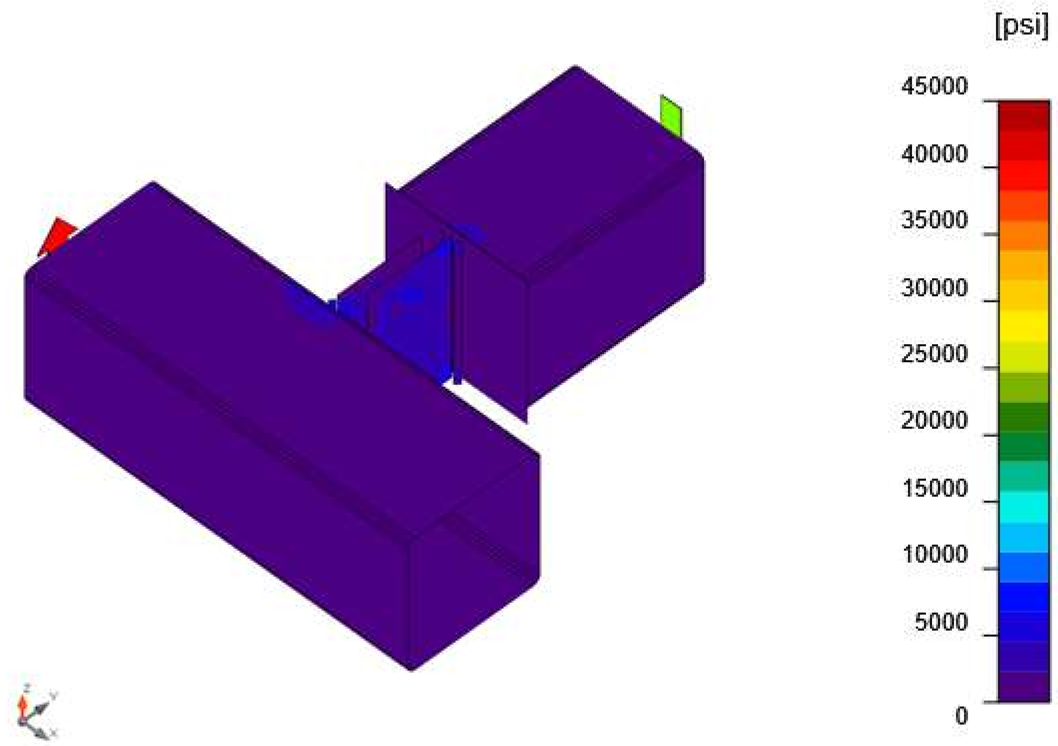
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain



Overall check, LE1

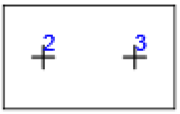


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B2	5/8 A325 - 1	LE1	0.000	1.368	29.090	0.0	8.7	-	OK	OK
	B3	5/8 A325 - 1	LE1	0.000	1.368	29.090	0.0	8.7	-	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
5/8 A325 - 1	20.691	15.652

Bolts: B2, B3 There is a gap between connected plates. Bolts should be designed as pins. Provided resistances of bolts in shear and plates in bearing may be incorrect

Symbol explanation

F_t	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
$\phi R_{n,bearing}$	Bolt bearing resistance
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$\phi R_{n,tension}$	Bolt tension resistance - AISC 360-16 – J3.6
$\phi R_{n,shear}$	Bolt shear resistance - AISC 360-16 – J3.6

Detailed result for B2

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.000 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 15.652 kip ≥ $V =$ 1.368 kip

Where:

F_{nv} = 68022.7 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 29.090 kip ≥ $V =$ 1.368 kip

Where:

l_c = 1"1/16 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 1/2 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Detailed result for B3

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 20.691 kip ≥ $F_t =$ 0.000 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 15.652 kip ≥ $V =$ 1.368 kip

Where:

F_{nv} = 68022.7 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.3068 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \quad \leq \quad 2.40 \cdot d \cdot t \cdot F_u$$

$\phi R_n =$ 29.090 kip $\geq$ $V =$ 1.368 kip

Where:

l_c = 1"1/16 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 1/2 in – thickness of the plate

d = 5/8 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
B1-w 4	Plate 1	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	5/16	LE1	0.628	3.170	19.8	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4"7/8	5/16	LE1	0.478	3.010	15.9	OK	OK
Plate 4	B2-w 1	E70xx	-	-	4"15/16	-	-	-	-	-	OK	OK
Plate 4	B2-arc 1	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-arc 2	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-arc 3	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-w 2	E70xx	-	-	4"15/16	-	-	-	-	-	OK	OK
Plate 4	B2-arc 4	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-arc 5	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-arc 6	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-w 3	E70xx	-	-	4"15/16	-	-	-	-	-	OK	OK
Plate 4	B2-arc 7	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-arc 8	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-arc 9	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-w 4	E70xx	-	-	4"15/16	-	-	-	-	-	OK	OK
Plate 4	B2-arc 10	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-arc 11	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	B2-arc 12	E70xx	-	-	1/8	-	-	-	-	-	OK	OK
Plate 4	Plate 3	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5"13/16	5/16	LE1	0.167	2.772	6.0	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5"7/8	5/16	LE1	0.326	3.239	10.1	OK	OK

Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
◢	Fillet weld

Detailed result for B1-w 4 / Plate 1 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.170 kip ≥ $F_n =$ 0.628 kip

Where:

F_{nw} = 62108.6 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 76.3° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0681 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for B1-w 4 / Plate 1 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.010 kip ≥ $F_n =$ 0.478 kip

Where:

F_{nw} = 58969.9 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 60.2° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0681 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Plate 4 / B2-w 1 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 1 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 3 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-w 2 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 5 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 6 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-w 3 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 7 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 8 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 9 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-w 4 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 10 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 11 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / B2-arc 12 - 1

Butt welds are not checked. Their resistance is assumed to be the same as that of the welded member.

Detailed result for Plate 4 / Plate 3 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 2.772 kip ≥ $F_n =$ 0.167 kip

Where:

F_{nw} = 54069.1 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 43.7° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0683 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Plate 4 / Plate 3 - 2

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 3.239 kip ≥ $F_n =$ 0.326 kip

Where:

F_{nw} = 62788.2 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 83.3° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0688 in^2 – effective area of weld critical element

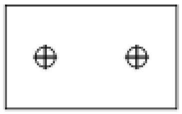
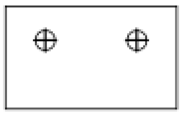
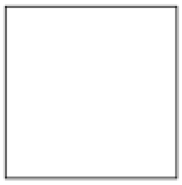
ϕ = 0.75 – resistance factor for welded connections

Buckling

Buckling analysis was not calculated.

Bill of material

Manufacturing operations

Name	Plates [in]	Shape	Nr.	Welds [in]	Length [in]	Bolts	Nr.
Cut 3							
Cut 4							
Plate 1	P1/2x5"7/8-3"9/16 (A913 Gr.50)		1			5/8 A325	2
Plate 3	P1/2x5"7/8-3"9/16 (A913 Gr.50)		1			5/8 A325	2
Plate 4	P1/2x5"7/8-5"7/8 (A913 Gr.50)		1				

Welds

Type	Material	Throat thickness [in]	Leg size [in]	Length [in]
Double fillet	E70xx	1/4	5/16	10"13/16
Butt	E70xx	-	-	1'-9"1/2

Bolts

Name	Grip length [in]	Count
5/8 A325	1"1/8	2

Drawing

Plate 1

P1/2x3"9/16-5"7/8 (A913 Gr.50)

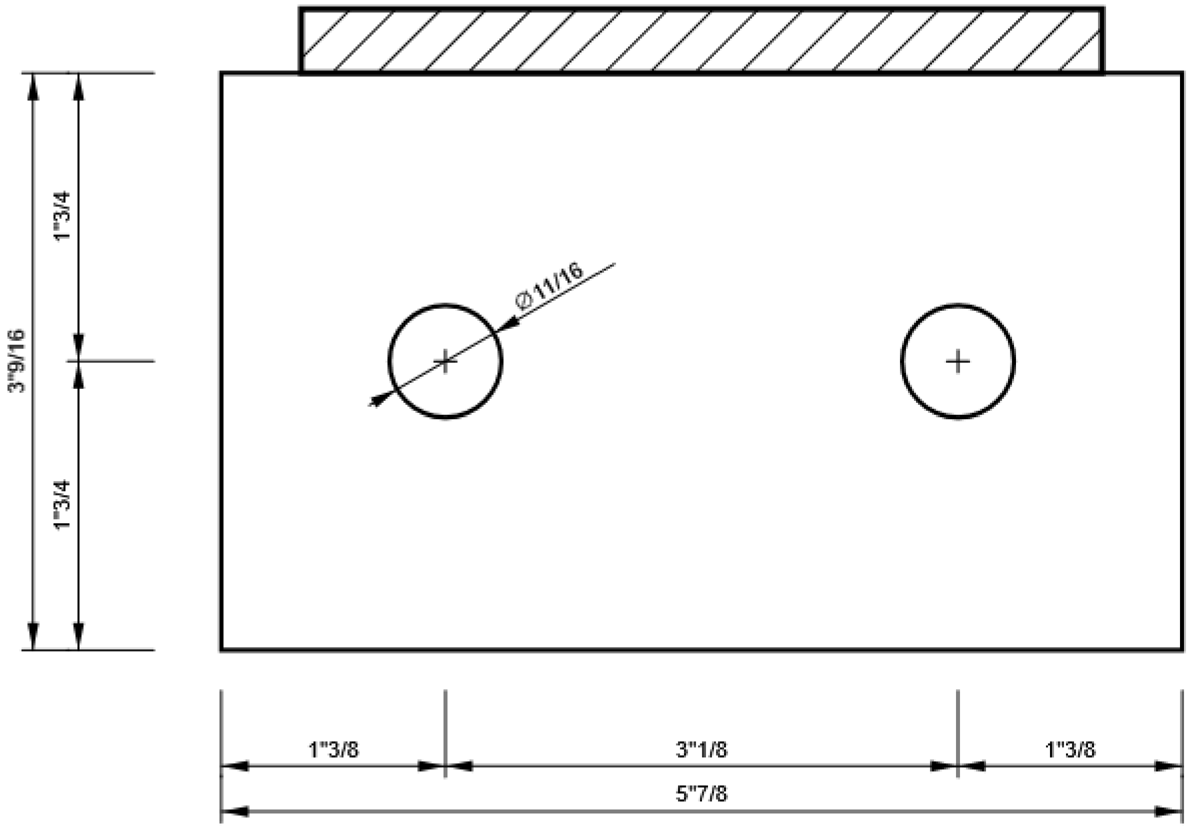


Plate 3

P1/2x3"9/16-5"7/8 (A913 Gr.50)

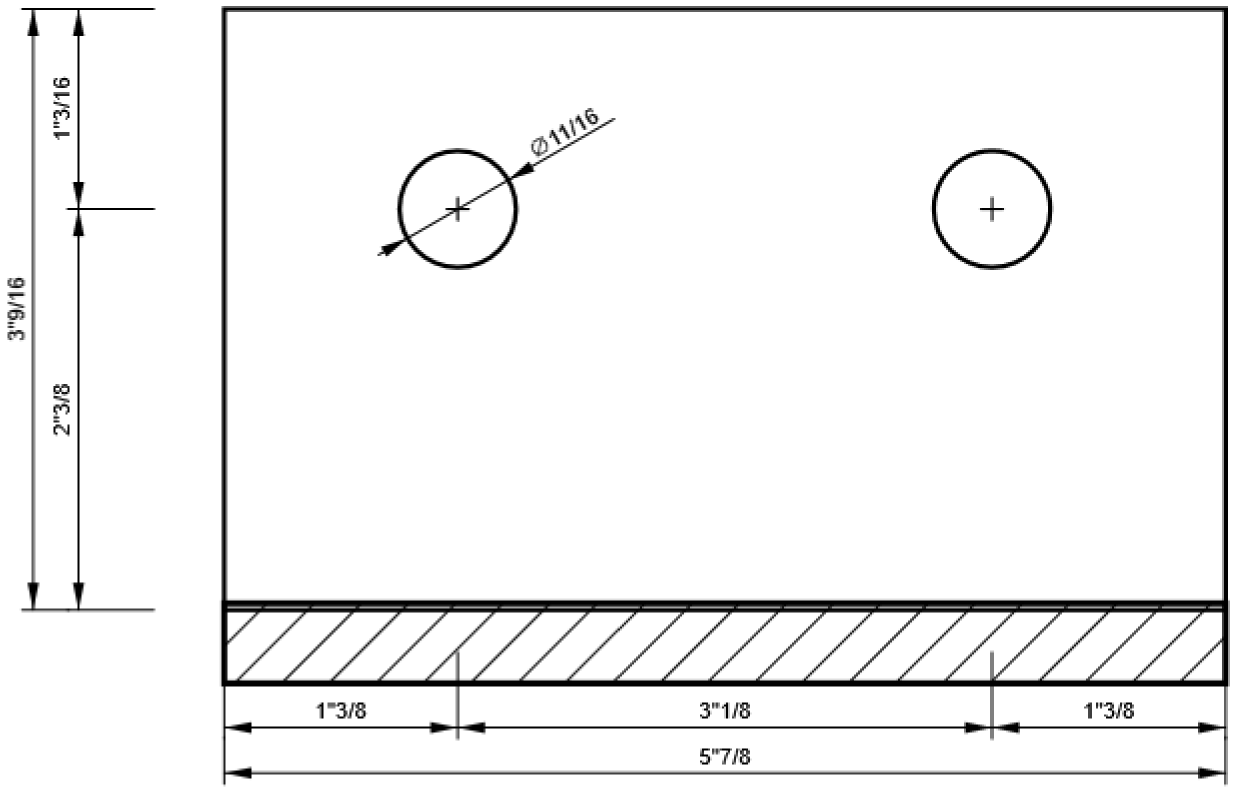
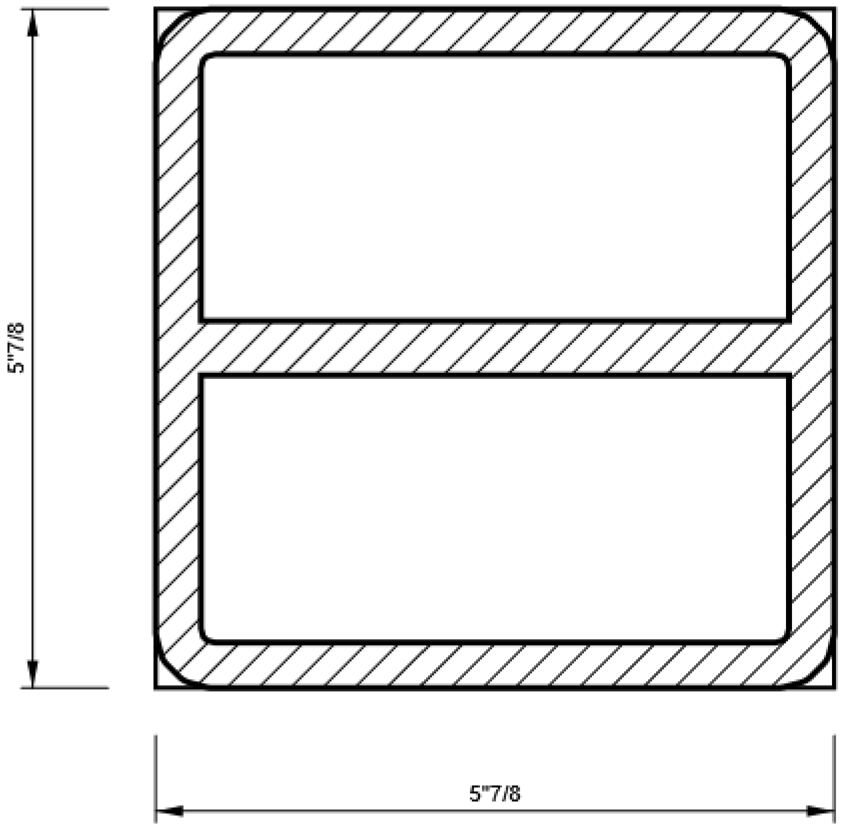


Plate 4

P1/2x5"7/8-5"7/8 (A913 Gr.50)



Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	No		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	MS/SK
Description	Overhang Beam Rest On Centre Column
Date	12-02-2024
Code	AISC/ACI

Material

Steel	A529. Gr. 50, A913 Gr.50

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Overhang Beam Rest On Centre Column
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

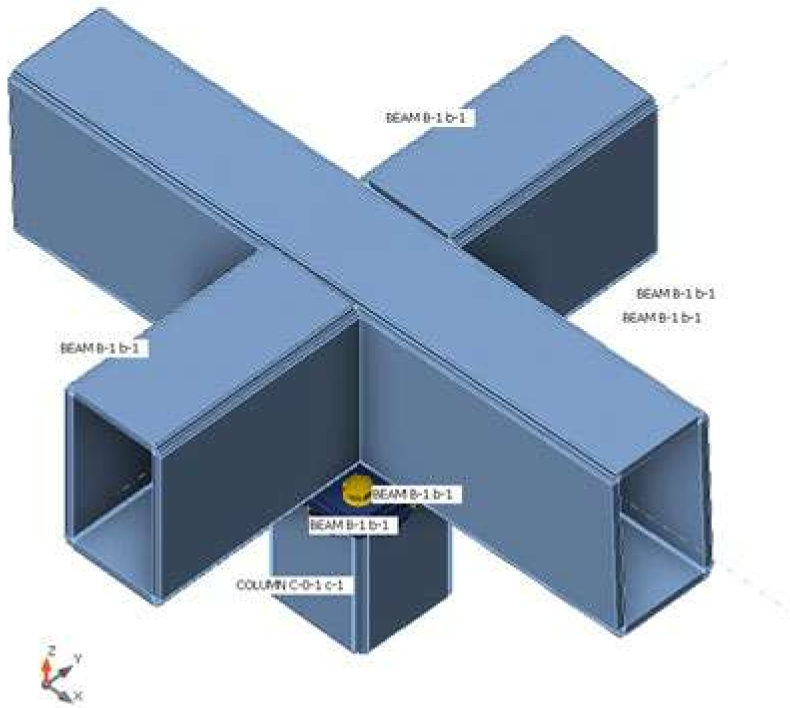
Members

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
BEAM B-1 b-1	13 - RHS250*150*12.0(RHS250x150)	-180.0	-2.0	-180.0	0.00	0.00	0.00
BEAM B-1 b-1	13 - RHS250*150*12.0(RHS250x150)	90.0	0.0	-180.0	0.00	0.00	-0.10
BEAM B-1 b-1	13 - RHS250*150*12.0(RHS250x150)	90.0	0.0	-180.0	0.00	0.00	-0.10
COLUMN C-0-1 c-1	6 - SHS150*150*12.0(RHS150x150)	0.0	90.0	-180.0	0.00	0.00	0.00

Supports and forces

Name	Support	Forces in	X [in]
BEAM B-1 b-1 / begin		Node	0.00
BEAM B-1 b-1 / end		Node	0.00
BEAM B-1 b-1 / end		Node	0.00
BEAM B-1 b-1 / begin		Node	0.00
COLUMN C-0-1 c-1 / begin	N-Vy-Vz-Mx-My-Mz	Node	0.00



Cross-sections

Name	Material
13 - RHS250*150*12.0(RHS250x150)	A913 Gr.50
6 - SHS150*150*12.0(RHS150x150)	A913 Gr.50

Bolts

Name	Bolt assembly	Diameter [in]	f_u [ksi]	Gross area [in^2]
20 A325	20 A325	0.79	120.0	0.49

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	BEAM B-1 b-1 / End	0.00	0.00	0.00	0.00	0.00	0.00
	BEAM B-1 b-1 / End	0.00	0.00	10.59	0.00	-58.63	0.00
	BEAM B-1 b-1 / Begin	0.00	0.00	10.59	0.00	58.63	0.00
	COLUMN C-0-1 c-1 / Begin	26.96	0.00	-0.20	0.00	45.56	0.00
	BEAM B-1 b-1 / Begin	0.00	0.00	5.78	0.00	45.56	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.00	0.00	0.00	0.00	0.00	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	3.4 < 5.0%	OK
Loc. deformation	0.2 < 3%	OK
Bolts	90.4 < 100%	OK
Welds	96.1 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

Name	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
BEAM B-1 b-1	1/2	LE1	45.8	2.7	0.0	OK
BEAM B-1 b-1	1/2	LE1	46.0	3.4	0.0	OK
BEAM B-1 b-1	1/2	LE1	46.0	3.4	0.0	OK
COLUMN C-0-1 c-1	1/2	LE1	45.2	0.7	0.0	OK
Plate 1	1/2	LE1	45.2	0.8	0.0	OK
Plate 2	1/2	LE1	45.2	0.8	0.0	OK
Plate 3	1/2	LE1	45.7	2.2	0.0	OK
Plate 4	1/2	LE1	22.8	0.0	0.0	OK
Plate 5	1/2	LE1	45.7	2.3	0.0	OK
Plate 6	1/2	LE1	45.0	0.1	0.0	OK
Plate 7	1/2	LE1	45.0	0.1	0.0	OK
Plate 8	9/16	LE1	45.3	0.9	20.6	OK
Plate 9	1/2	LE1	23.5	0.0	0.0	OK
Plate 10	9/16	LE1	45.8	2.7	20.6	OK

Design data

Material	F_y [ksi]	ε_{lim} [%]
A913 Gr.50	50.0	5.0

Symbol explanation

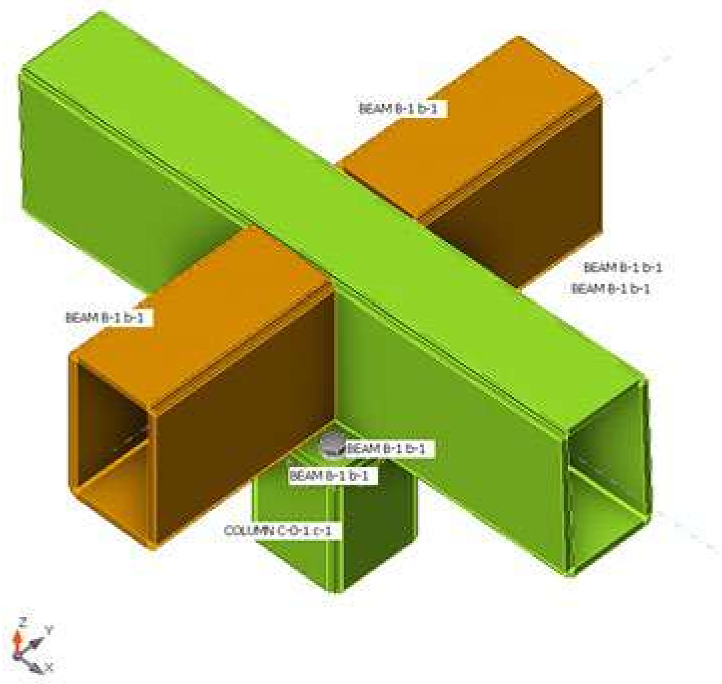
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain

Loc. deformation

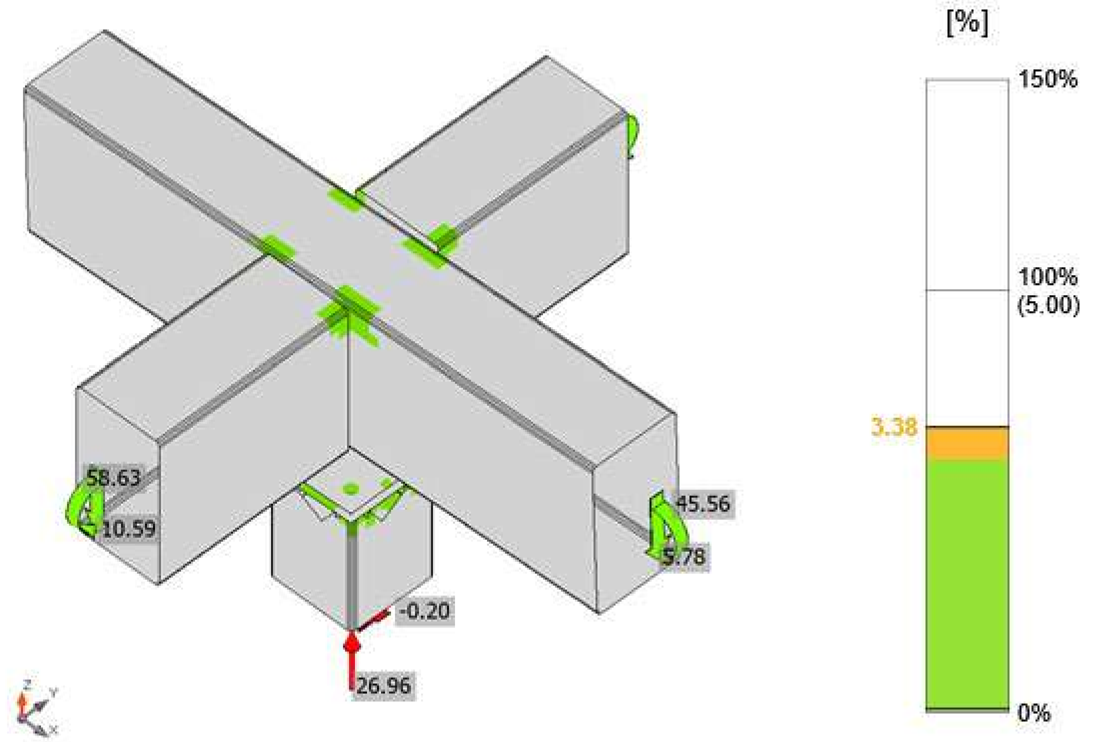
Name	d_0 [in]	Loads	δ [in]	$δ_{lim}$ [in]	$δ/d_0$ [%]	Check status
BEAM B-1 b-1	5.91	LE1	0.01	0.18	0.2	OK
BEAM B-1 b-1	5.91	LE1	0.01	0.18	0.1	OK
BEAM B-1 b-1	5.91	LE1	0.01	0.18	0.1	OK
COLUMN C-0-1 c-1	5.91	LE1	0.01	0.18	0.2	OK

Symbol explanation

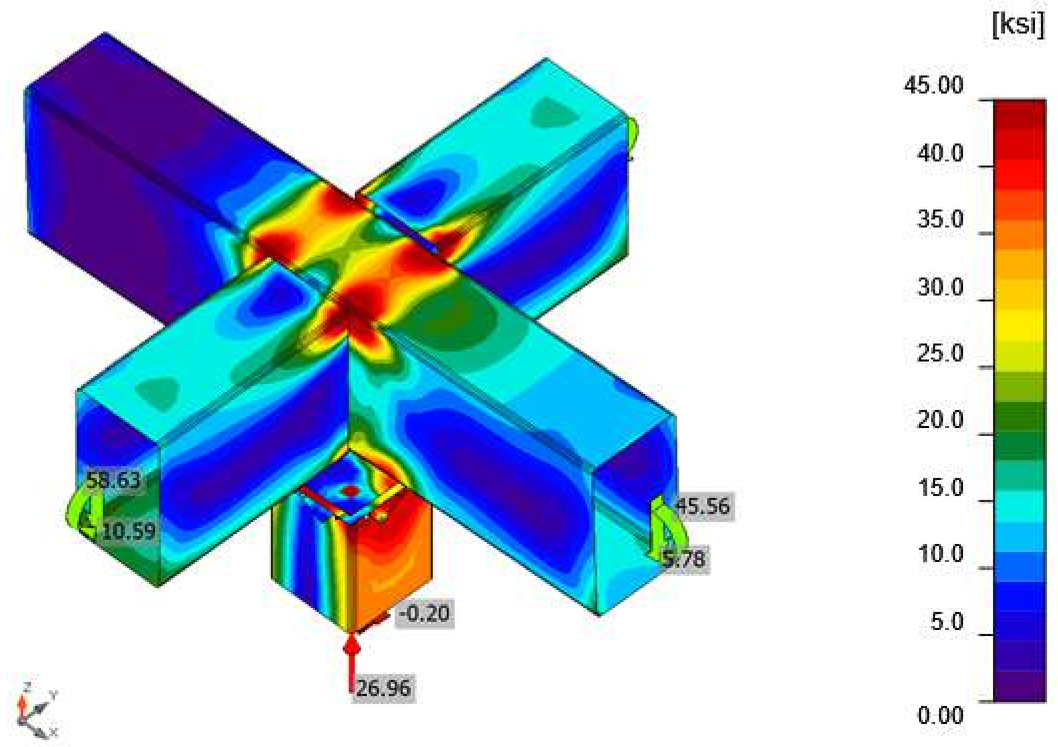
d_0	Cross-section size
δ	Local cross-section deformation
$δ_{lim}$	Allowed deformation



Overall check, LE1

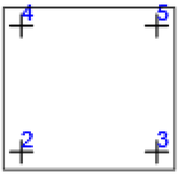


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B2	20 A325 - 1	LE1	3.93	0.65	28.55	12.0	2.6	-	OK	OK
	B3	20 A325 - 1	LE1	29.69	1.84	31.59	90.4	7.4	-	OK	OK
	B4	20 A325 - 1	LE1	3.74	0.65	29.30	11.4	2.6	-	OK	OK
	B5	20 A325 - 1	LE1	29.70	1.88	31.76	90.4	7.6	-	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
20 A325 - 1	32.84	24.84

Symbol explanation

F_t	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
$\phi R_{n,bearing}$	Bolt bearing resistance
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$\phi R_{n,tension}$	Bolt tension resistance - AISC 360-16 – J3.6
$\phi R_{n,shear}$	Bolt shear resistance - AISC 360-16 – J3.6

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
COLUMN C-0-1 c-1-w 1	Plate 1	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.45	0.31	LE1	2.00	2.65	75.5	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.45	0.31	LE1	2.26	2.92	77.1	OK	OK
COLUMN C-0-1 c-1-w 1	Plate 2	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.45	0.31	LE1	2.26	2.93	77.2	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.45	0.31	LE1	2.01	2.66	75.6	OK	OK
COLUMN C-0-1 c-1-w 3	Plate 3	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.68	0.30	LE1	2.27	2.84	79.9	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.68	0.30	LE1	2.32	2.85	81.2	OK	OK
COLUMN C-0-1 c-1-w 4	Plate 4	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.61	0.33	LE1	0.85	2.35	36.3	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.62	0.33	LE1	0.53	2.58	20.5	OK	OK
COLUMN C-0-1 c-1-w 3	Plate 5	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.68	0.30	LE1	2.32	2.86	81.2	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.68	0.30	LE1	2.27	2.84	79.9	OK	OK
COLUMN C-0-1 c-1-w 2	Plate 6	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.50	0.31	LE1	2.03	2.83	71.7	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.50	0.31	LE1	1.77	2.64	67.0	OK	OK
COLUMN C-0-1 c-1-w 4	Plate 7	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.50	0.31	LE1	1.70	2.67	63.5	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.50	0.31	LE1	2.00	2.87	70.0	OK	OK
COLUMN C-0-1 c-1-w 2	Plate 9	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.62	0.33	LE1	0.52	2.58	20.0	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.61	0.33	LE1	0.88	2.34	37.5	OK	OK
Plate 10	COLUMN C-0-1 c-1-w 1	E70xx	-	-	4.88	-	-	-	-	-	OK	OK

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
Plate 10	COLUMN C-0-1 c-1-arc 1	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 2	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 3	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-w 2	E70xx	-	-	4.88	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 4	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 5	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 6	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-w 3	E70xx	-	-	4.88	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 7	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 8	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 9	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-w 4	E70xx	-	-	4.88	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 10	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 11	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 10	COLUMN C-0-1 c-1-arc 12	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 8	BEAM B-1 b-1-w 3	E70xx	-	-	11.30	-	-	-	-	-	OK	OK
Plate 8	BEAM B-1 b-1-w 3	E70xx	-	-	11.30	-	-	-	-	-	OK	OK
Plate 10	Plate 1	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.64	0.33	LE1	2.57	3.16	81.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.66	0.33	LE1	2.08	3.39	61.3	OK	OK
Plate 10	Plate 2	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.64	0.33	LE1	1.78	2.43	73.1	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.64	0.33	LE1	2.59	3.18	81.6	OK	OK
Plate 10	Plate 3	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	3.11	3.24	96.0	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	3.08	3.23	95.4	OK	OK

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
Plate 10	Plate 4	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	0.39	2.57	15.1	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.92	0.32	LE1	1.31	3.04	43.1	OK	OK
Plate 10	Plate 5	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	3.08	3.23	95.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	3.12	3.24	96.1	OK	OK
Plate 10	Plate 6	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	1.77	2.47	71.8	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	2.16	2.81	76.7	OK	OK
Plate 10	Plate 7	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	2.19	2.86	76.8	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	1.39	2.52	55.2	OK	OK
Plate 10	Plate 9	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.92	0.32	LE1	1.33	3.14	42.4	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	0.41	3.20	12.7	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 1	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 2	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 3	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-w 2	E70xx	-	-	8.82	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 4	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 5	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 6	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-w 3	E70xx	-	-	4.88	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 7	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 8	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 9	E70xx	-	-	0.14	-	-	-	-	-	OK	OK

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
BEAM B-1 b-1-w 2	BEAM B-1 b-1-w 4	E70xx	-	-	8.82	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 2	BEAM B-1 b-1-arc 10	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 1	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 2	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 3	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-w 2	E70xx	-	-	8.82	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 4	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 5	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 6	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-w 3	E70xx	-	-	4.88	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 7	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 8	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 9	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-w 4	E70xx	-	-	8.82	-	-	-	-	-	OK	OK
BEAM B-1 b-1-w 4	BEAM B-1 b-1-arc 10	E70xx	-	-	0.14	-	-	-	-	-	OK	OK

Design data

Material	F_{exx} [ksi]
E70xx	70.0

Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
◢	Fillet weld
F_{exx}	Ultimate strength as rated by electrode classification number

Buckling

Buckling analysis was not calculated.

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Appendix- 21

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	MS/SK
Description	Overhand Beam Rest On End Column
Date	12-02-2024
Code	AISC/ACI

Material

Steel	A529. Gr. 50, A913 Gr.50

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Overhang Beam rest On End Column
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

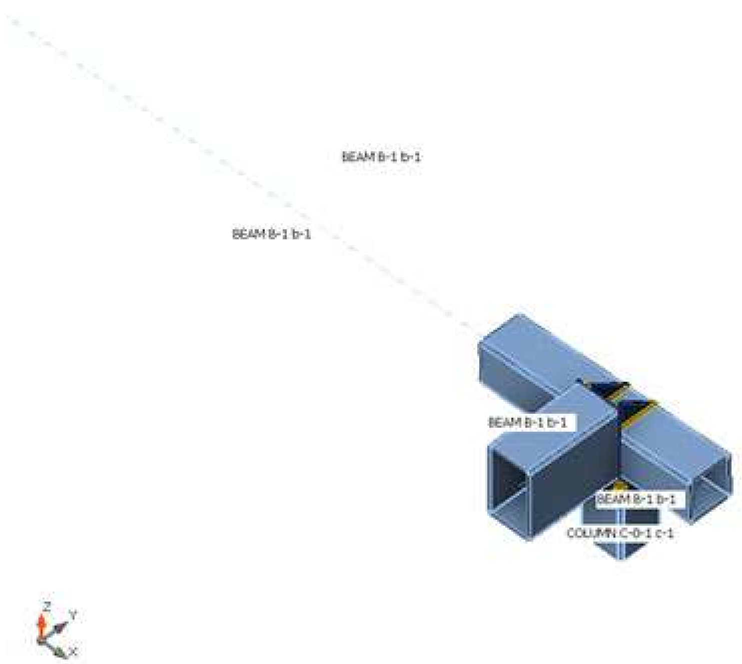
Members

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
BEAM B-1 b-1	6 - SHS150*150*12.0(RHS150x150)	-180.0	-2.0	-180.0	0.00	0.00	3.30
COLUMN C-0-1 c-1	6 - SHS150*150*12.0(RHS150x150)	0.0	90.0	-180.0	0.00	0.00	94.49
BEAM B-1 b-1	13 - RHS250*150*12.0(RHS250x150)	90.0	0.0	-180.0	0.00	94.49	-2.07

Supports and forces

Name	Support	Forces in	X [in]
BEAM B-1 b-1 / begin		Node	0.00
COLUMN C-0-1 c-1 / begin	N-Vy-Vz-Mx-My-Mz	Node	0.00
BEAM B-1 b-1 / begin		Node	0.00



Cross-sections

Name	Material
6 - SHS150*150*12.0(RHS150x150)	A913 Gr.50
13 - RHS250*150*12.0(RHS250x150)	A913 Gr.50

Bolts

Name	Bolt assembly	Diameter [in]	f_u [ksi]	Gross area [in^2]
20 A325	20 A325	0.79	120.0	0.49

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	BEAM B-1 b-1 / Begin	0.00	0.00	1.00	0.00	6.75	0.00
	COLUMN C-0-1 c-1 / Begin	2.70	0.00	-0.03	0.00	-1.12	-1.97
	BEAM B-1 b-1 / Begin	0.00	0.00	1.70	0.00	1.97	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.00	0.00	0.00	0.00	0.00	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Loc. deformation	0.0 < 3%	OK
Bolts	7.5 < 100%	OK
Welds	54.4 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

Name	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
BEAM B-1 b-1	1/2	LE1	9.0	0.0	0.0	OK
COLUMN C-0-1 c-1	1/2	LE1	3.7	0.0	0.0	OK
BEAM B-1 b-1	1/2	LE1	10.1	0.0	0.0	OK
Plate 1	9/16	LE1	6.7	0.0	1.1	OK
Plate 2	9/16	LE1	5.9	0.0	1.1	OK
Plate 3	1/2	LE1	3.4	0.0	0.0	OK
Plate 4	1/2	LE1	0.8	0.0	0.0	OK

Name	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
Plate 5	1/2	LE1	1.9	0.0	0.0	OK
Plate 6	1/2	LE1	4.0	0.0	0.0	OK
Plate 7	1/2	LE1	4.3	0.0	0.0	OK
Plate 8	1/2	LE1	1.8	0.0	0.0	OK
Plate 9	1/2	LE1	0.9	0.0	0.0	OK
Plate 10	1/2	LE1	2.8	0.0	0.0	OK
Plate 11	3/8	LE1	28.6	0.0	0.0	OK
Plate 12	3/8	LE1	30.4	0.0	0.0	OK
Plate 13	3/8	LE1	11.2	0.0	0.0	OK

Design data

Material	F_y [ksi]	ε_{lim} [%]
A913 Gr.50	50.0	5.0

Symbol explanation

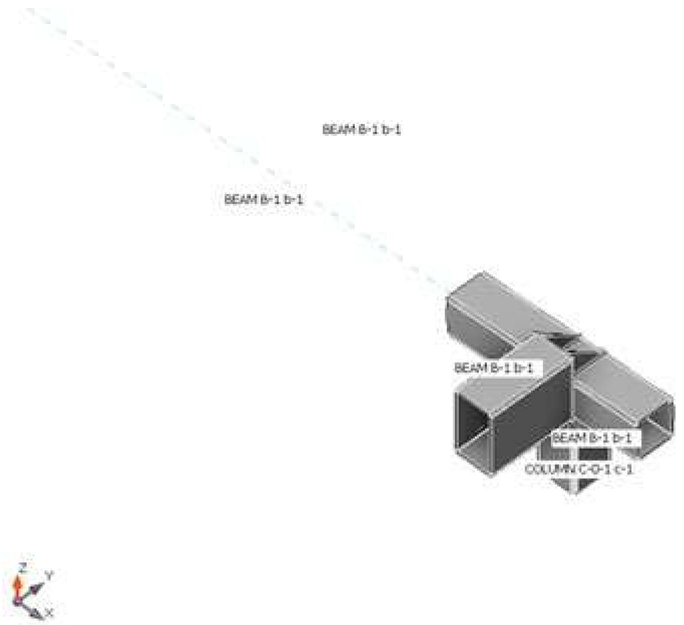
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain

Loc. deformation

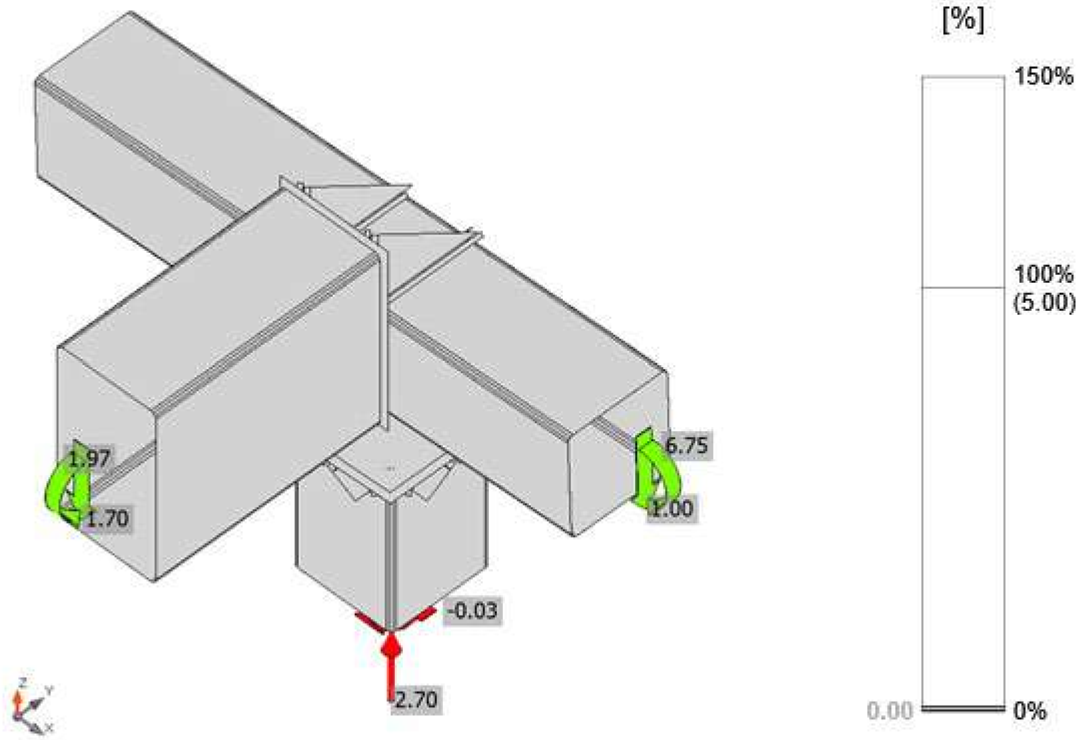
Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
BEAM B-1 b-1	5.91	LE1	0.00	0.18	0.0	OK
COLUMN C-0-1 c-1	5.91	LE1	0.00	0.18	0.0	OK
BEAM B-1 b-1	5.91	LE1	0.00	0.18	0.0	OK

Symbol explanation

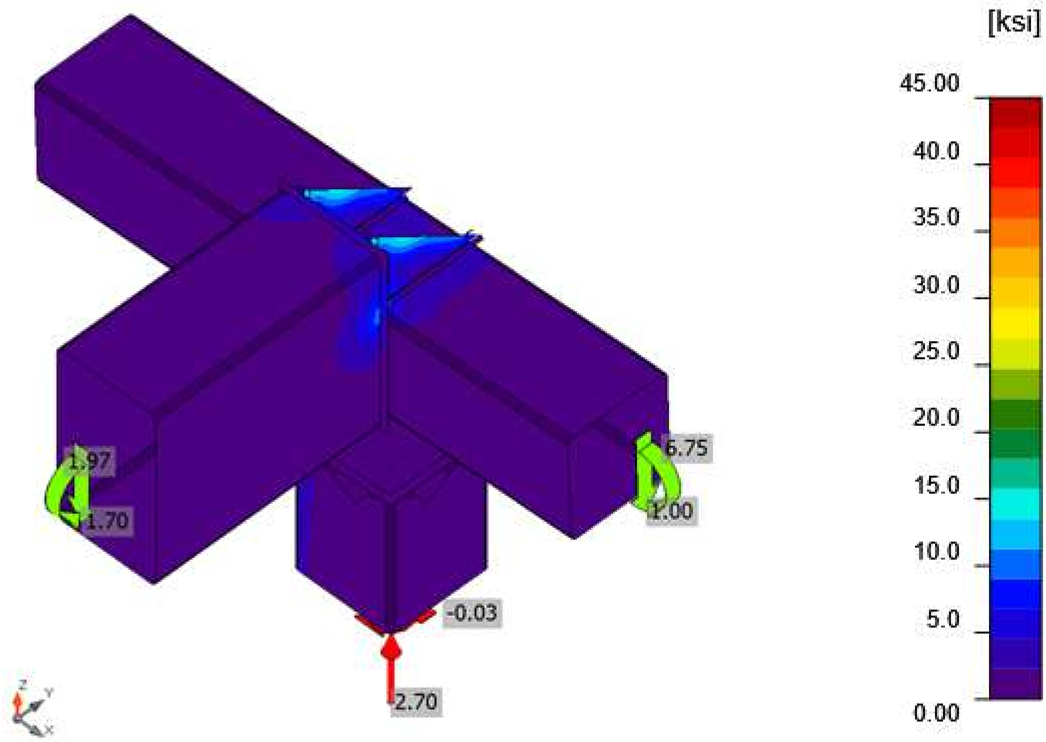
d_0	Cross-section size
δ	Local cross-section deformation
δ_{lim}	Allowed deformation



Overall check, LE1

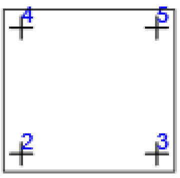


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Detailing	Status
	B2	20 A325 - 1	LE1	2.46	0.07	34.75	7.5	0.3	-	OK	OK
	B3	20 A325 - 1	LE1	1.08	0.06	32.66	3.3	0.3	-	OK	OK
	B4	20 A325 - 1	LE1	0.08	0.06	29.61	0.2	0.2	-	OK	OK
	B5	20 A325 - 1	LE1	0.02	0.03	30.23	0.1	0.1	-	OK	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
20 A325 - 1	32.84	24.84

Symbol explanation

F_t	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
$\phi R_{n,bearing}$	Bolt bearing resistance
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$\phi R_{n,tension}$	Bolt tension resistance - AISC 360-16 – J3.6
$\phi R_{n,shear}$	Bolt shear resistance - AISC 360-16 – J3.6

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
COLUMN C-0-1 c-1-w 1	Plate 3	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.45	0.31	LE1	0.22	2.70	8.3	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.45	0.31	LE1	0.23	2.77	8.5	OK	OK
COLUMN C-0-1 c-1-w 3	Plate 4	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.68	0.30	LE1	0.03	2.54	1.1	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.68	0.30	LE1	0.02	2.53	0.9	OK	OK
COLUMN C-0-1 c-1-w 1	Plate 5	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.45	0.31	LE1	0.10	2.94	3.5	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.45	0.31	LE1	0.07	2.87	2.6	OK	OK
COLUMN C-0-1 c-1-w 3	Plate 6	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.68	0.30	LE1	0.12	2.69	4.6	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.68	0.30	LE1	0.14	2.77	4.9	OK	OK
COLUMN C-0-1 c-1-w 4	Plate 7	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.62	0.33	LE1	0.14	3.01	4.7	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.62	0.33	LE1	0.12	2.91	4.2	OK	OK
COLUMN C-0-1 c-1-w 4	Plate 8	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.50	0.31	LE1	0.04	2.66	1.7	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.50	0.31	LE1	0.06	2.78	2.2	OK	OK
COLUMN C-0-1 c-1-w 2	Plate 9	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.62	0.33	LE1	0.06	2.83	2.3	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.62	0.33	LE1	0.06	2.79	2.0	OK	OK
COLUMN C-0-1 c-1-w 2	Plate 10	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.50	0.31	LE1	0.15	2.67	5.7	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.50	0.31	LE1	0.15	2.61	5.9	OK	OK
BEAM B-1 b-1-w 1	Plate 11	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.38	0.32	LE1	1.48	3.29	44.9	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	5.38	0.32	LE1	1.56	3.29	47.3	OK	OK
BEAM B-1 b-1-w 1	Plate 12	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4.85	0.32	LE1	1.75	3.30	53.1	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	4.85	0.32	LE1	1.59	3.33	47.7	OK	OK
Plate 13	BEAM B-1 b-1-w 1	E70xx	-	-	4.88	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 1	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 2	E70xx	-	-	0.14	-	-	-	-	-	OK	OK

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
Plate 13	BEAM B-1 b-1-arc 3	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-w 2	E70xx	-	-	8.82	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 4	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 5	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 6	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-w 3	E70xx	-	-	4.88	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 7	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 8	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 9	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-w 4	E70xx	-	-	8.82	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 10	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 11	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	BEAM B-1 b-1-arc 12	E70xx	-	-	0.14	-	-	-	-	-	OK	OK
Plate 13	Plate 11	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	3.55	0.32	LE1	1.71	3.36	50.8	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	3.55	0.32	LE1	1.14	3.34	34.0	OK	OK
Plate 13	Plate 12	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	3.68	0.31	LE1	1.13	3.18	35.6	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	3.68	0.31	LE1	1.74	3.20	54.4	OK	OK
Plate 1	BEAM B-1 b-1-w 3	E70xx	-	-	11.30	-	-	-	-	-	OK	OK
Plate 1	BEAM B-1 b-1-w 3	E70xx	-	-	11.30	-	-	-	-	-	OK	OK
Plate 2	Plate 3	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.66	0.33	LE1	0.17	3.01	5.5	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.66	0.33	LE1	0.07	2.66	2.7	OK	OK
Plate 2	Plate 4	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.92	0.32	LE1	0.04	2.92	1.5	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	0.01	2.91	0.4	OK	OK
Plate 2	Plate 5	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.64	0.33	LE1	0.06	2.50	2.5	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	2.64	0.33	LE1	0.13	2.68	4.8	OK	OK

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Detailing	Status
Plate 2	Plate 6	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	0.23	2.82	8.0	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	0.10	2.84	3.7	OK	OK
Plate 2	Plate 7	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	0.09	2.27	3.8	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.92	0.32	LE1	0.22	2.87	7.8	OK	OK
Plate 2	Plate 8	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	0.09	2.70	3.5	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.90	0.32	LE1	0.06	2.49	2.3	OK	OK
Plate 2	Plate 9	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.92	0.32	LE1	0.05	2.90	1.8	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.92	0.32	LE1	0.05	2.96	1.6	OK	OK
Plate 2	Plate 10	E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.92	0.32	LE1	0.12	3.14	3.9	OK	OK
		E70xx	◢ 1/4 ◣	◢ 5/16 ◣	1.92	0.32	LE1	0.20	3.13	6.2	OK	OK
Plate 2	COLUMN C-0-1 c-1	E70xx	◢ 1/4	◢ 5/16	21.16	0.14	LE1	0.33	1.49	21.9	OK	OK

Design data

Material	F_{exx} [ksi]
E70xx	70.0

Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
◢	Fillet weld
F_{exx}	Ultimate strength as rated by electrode classification number

Buckling

Buckling analysis was not calculated.

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	Yes		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	MS/SK
Description	Incline Column
Date	12-02-2024
Code	AISC/ACI

Material

Steel	A529. Gr. 50, A913 Gr.50

Project:	**Exuma,**
Project no:	**Bahamas**
Author:	**242201**

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Incline Column
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

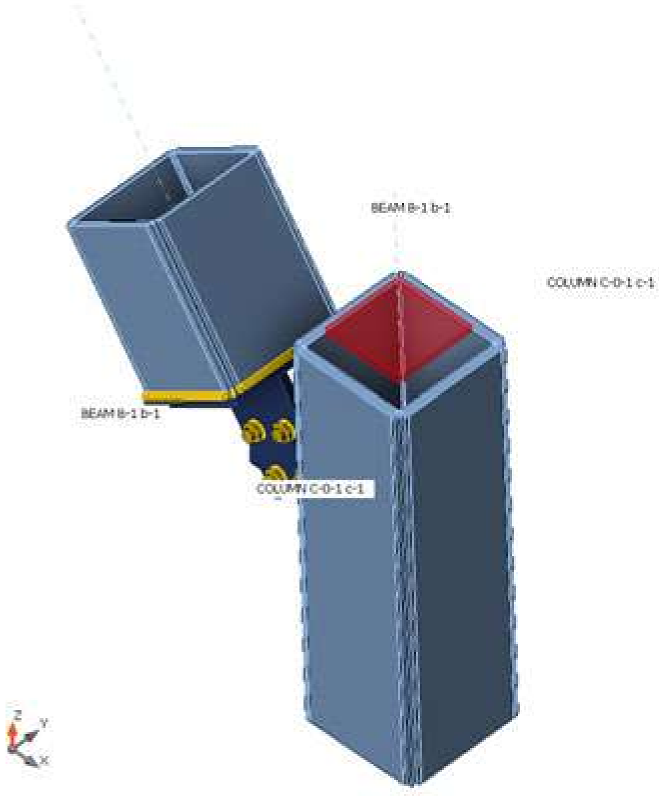
Members

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
COLUMN C-0-1 c-1	6 - SHS150*150*12.0(RHS150x150)	-180.0	88.0	-180.0	0.00	0.00	0.00
BEAM B-1 b-1	280 - SHS150*150*10.0(RHS150x150)	-180.0	54.4	-180.0	0.00	0.00	5.21

Supports and forces

Name	Support	Forces in	X [in]
COLUMN C-0-1 c-1 / end	N-Vy-Vz-Mx-My-Mz	Node	0.00
BEAM B-1 b-1 / end		Bolts	0.00



Cross-sections

Name	Material
6 - SHS150*150*12.0(RHS150x150)	A913 Gr.50
280 - SHS150*150*10.0(RHS150x150)	A913 Gr.50

Bolts

Name	Bolt assembly	Diameter [in]	f_u [ksi]	Gross area [in^2]
12 A325	12 A325	0.47	120.0	0.18

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	COLUMN C-0-1 c-1 / End	-3.16	0.00	-2.10	-0.69	-1.65	-0.47
	BEAM B-1 b-1 / End	3.80	0.00	0.00	0.85	0.00	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE1	0.00	0.00	0.00	0.00	0.00	0.02

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.3 < 5.0%	OK
Loc. deformation	0.1 < 3%	OK
Bolts	21.0 < 100%	OK
Welds	76.2 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

Name	t_p [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [ksi]	Status
COLUMN C-0-1 c-1	1/2	LE1	31.4	0.0	0.0	OK
BEAM B-1 b-1	3/8	LE1	14.0	0.0	0.0	OK
Plate 1	3/8	LE1	45.1	0.2	13.5	OK
Plate 2	3/8	LE1	45.1	0.3	12.8	OK
Plate 3	3/8	LE1	22.1	0.0	0.0	OK

Project:	Exuma,
Project no:	Bahamas
Author:	242201

Design data

Material	F_y [ksi]	ε_{lim} [%]
A913 Gr.50	50.0	5.0

Symbol explanation

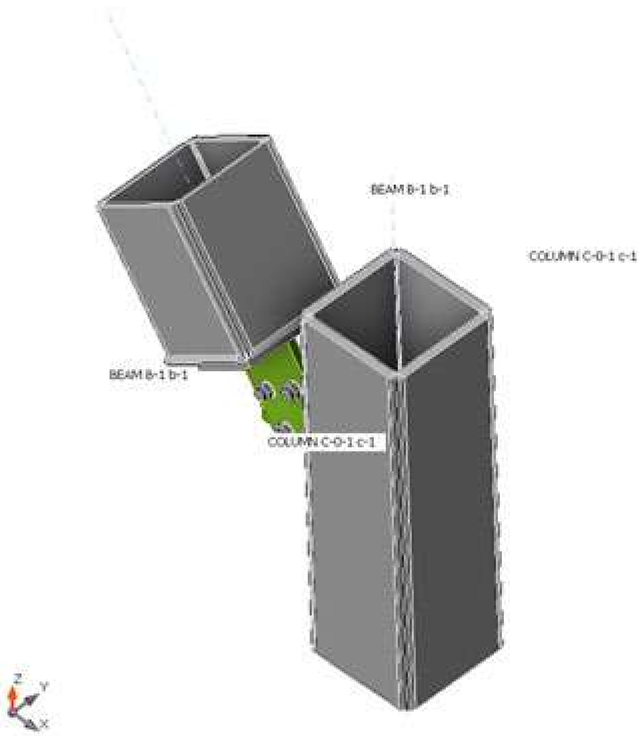
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain

Loc. deformation

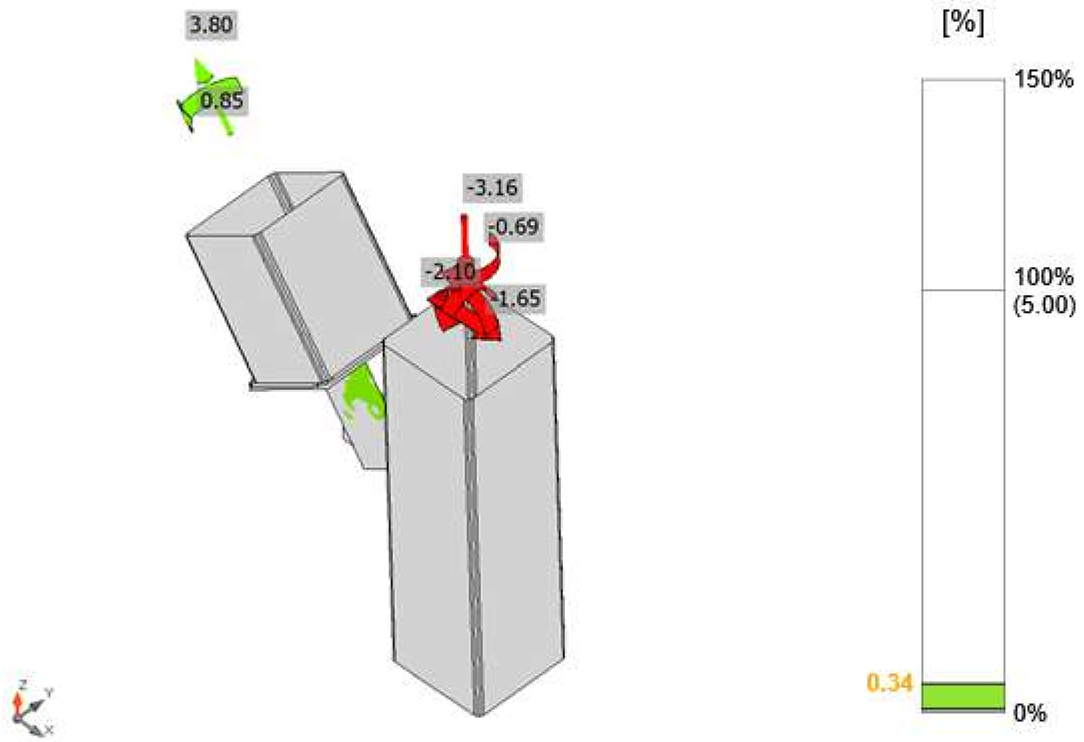
Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
COLUMN C-0-1 c-1	5.91	LE1	0.00	0.18	0.1	OK
BEAM B-1 b-1	5.91	LE1	0.00	0.18	0.1	OK

Symbol explanation

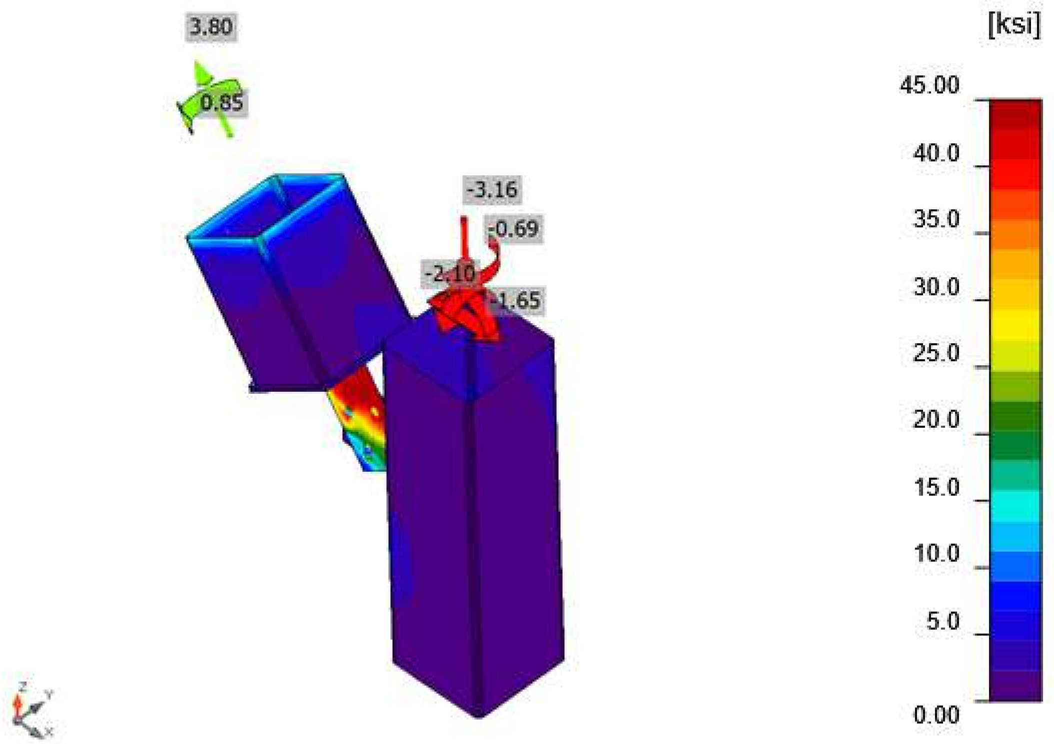
d_0	Cross-section size
δ	Local cross-section deformation
δ_{lim}	Allowed deformation



Overall check, LE1

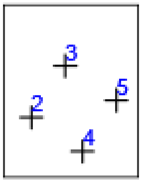


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B2	12 A325 - 1	LE1	0.89	1.13	16.78	7.5	12.7	-	OK
	B3	12 A325 - 1	LE1	2.01	1.04	19.39	17.0	11.6	-	OK
	B4	12 A325 - 1	LE1	2.48	1.16	18.23	21.0	13.0	-	OK
	B5	12 A325 - 1	LE1	1.23	0.64	15.81	10.4	7.1	-	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
12 A325 - 1	11.82	8.94

Symbol explanation

F_t	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
$\phi R_{n,bearing}$	Bolt bearing resistance
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$\phi R_{n,tension}$	Bolt tension resistance - AISC 360-16 – J3.6
$\phi R_{n,shear}$	Bolt shear resistance - AISC 360-16 – J3.6

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Status
COLUMN C-0-1 c-1-w 3	Plate 2	E70xx	◢ 3/16 ◣	◢ 1/4 ◣	4.30	0.31	LE1	1.78	2.34	76.2	OK
		E70xx	◢ 3/16 ◣	◢ 1/4 ◣	4.30	0.31	LE1	0.92	1.91	48.3	OK
Plate 3	Plate 1	E70xx	◢ 3/16 ◣	◢ 1/4 ◣	3.91	0.33	LE1	1.91	2.53	75.4	OK
		E70xx	◢ 3/16 ◣	◢ 1/4 ◣	3.91	0.33	LE1	1.89	2.51	75.2	OK
Plate 3	BEAM B-1 b-1	E70xx	◢ 3/16	◢ 1/4	21.27	0.30	LE1	0.30	2.38	12.5	OK

Design data

Material	F_{exx} [ksi]
E70xx	70.0

Symbol explanation

t_w	Throat thickness of weld
w	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance - AISC 360-16 – J2-4
Ut	Utilization
◢	Fillet weld
F_{exx}	Ultimate strength as rated by electrode classification number

Buckling

Buckling analysis was not calculated.

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	No		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Project data

Project name	Exuma, Bahamas
Project number	242201
Author	SK/MS
Description	Top Track to HRS Plate Connection
Date	12-02-2024
Code	AISC/ACI

Material

Steel	A36, A913 Gr.50
Concrete	4000 psi

Project item Exuma, Bahamas

Design

Name	Exuma, Bahamas
Description	Top Track to HRS Plate Connection
Analysis	Stress, strain/ loads in equilibrium
Design code	AISC - LRFD (2016)

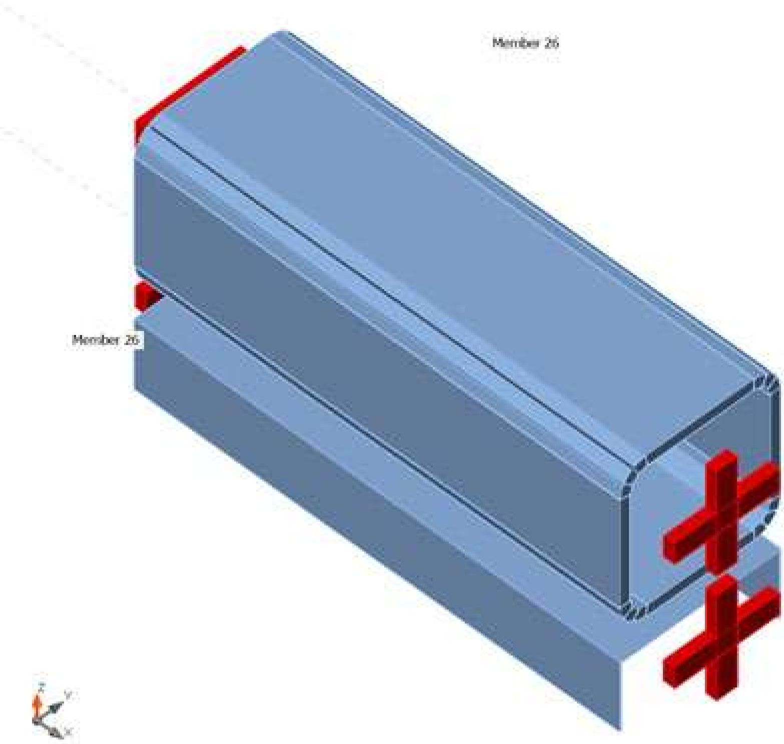
Members

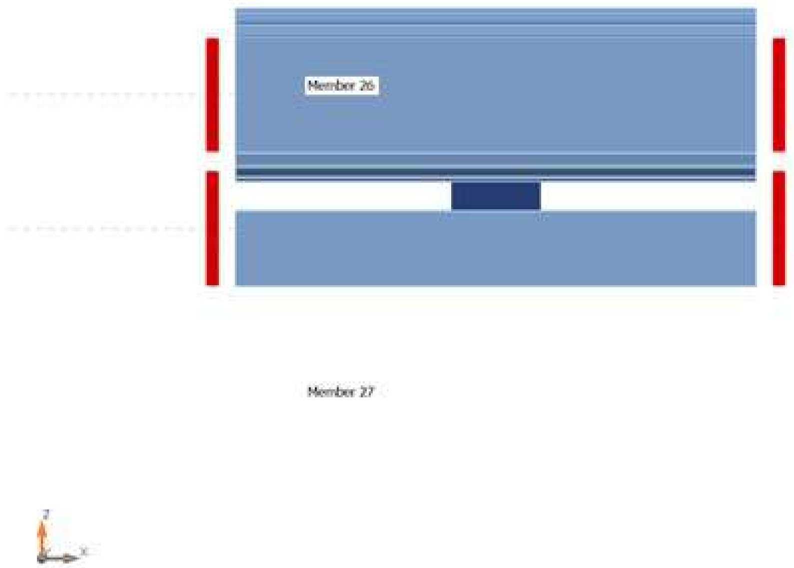
Geometry

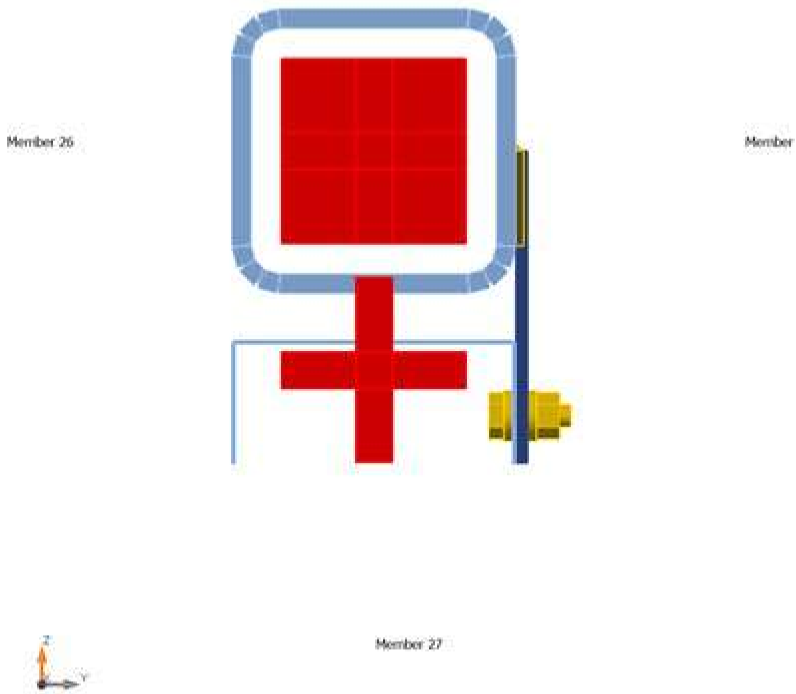
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]
Member 26	8 - SHS150/150/10.0	0.0	0.0	180.0	0"		
Member 27	7 - C.150.50.15(U150)	0.0	0.0	-90.0	0"	4"9/16	-

Supports and forces

Name	Support	Forces in	X [in]
Member 26 / begin	N-Vy-Vz-Mx-My-Mz	Bolts	0"
Member 26 / end		Bolts	0"
Member 27 / begin	Mx-My-Mz	Bolts	0"
Member 27 / end	Mx-My-Mz	Bolts	0"



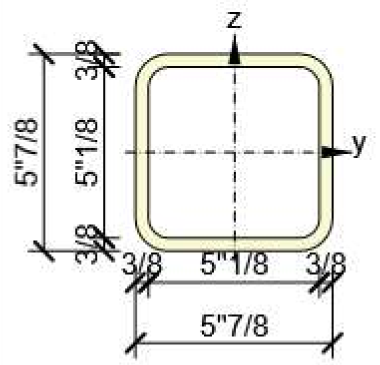
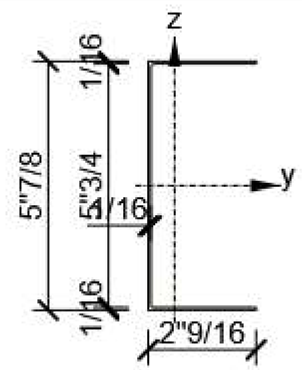




Cross-sections

Name	Material
8 - SHS150/150/10.0	A913 Gr.50
7 - C.150.50.15(U150)	A913 Gr.50

Cross-sections

Name	Material	Drawing
8 - SHS150/150/10.0	A913 Gr.50	
7 - C.150.50.15(U150)	A913 Gr.50	

Bolts

Name	Bolt assembly	Diameter [in]	f_u [psi]	Gross area [in^2]
12 A325M	12 A325M	1/2	120381.3	0.1752

Load effects (forces in equilibrium)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE2	Member 26 / Begin	0.000	1.272	0.000	0.00	0.00	0.00
	Member 26 / End	0.000	0.000	0.000	0.00	0.00	0.00
	Member 27 / Begin	0.000	0.000	0.636	0.00	0.00	0.00
	Member 27 / End	0.000	0.000	0.636	0.00	0.00	0.00

Unbalanced forces

Name	X [kip]	Y [kip]	Z [kip]	Mx [kip.ft]	My [kip.ft]	Mz [kip.ft]
LE2	0.000	0.000	0.000	0.48	0.00	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	1.3 < 5.0%	OK
Loc. deformation	0.0 < 3%	OK
Bolts	7.7 < 100%	OK
Welds	96.4 < 100%	OK
Buckling	Not calculated	
GMNA	Calculated	

Plates

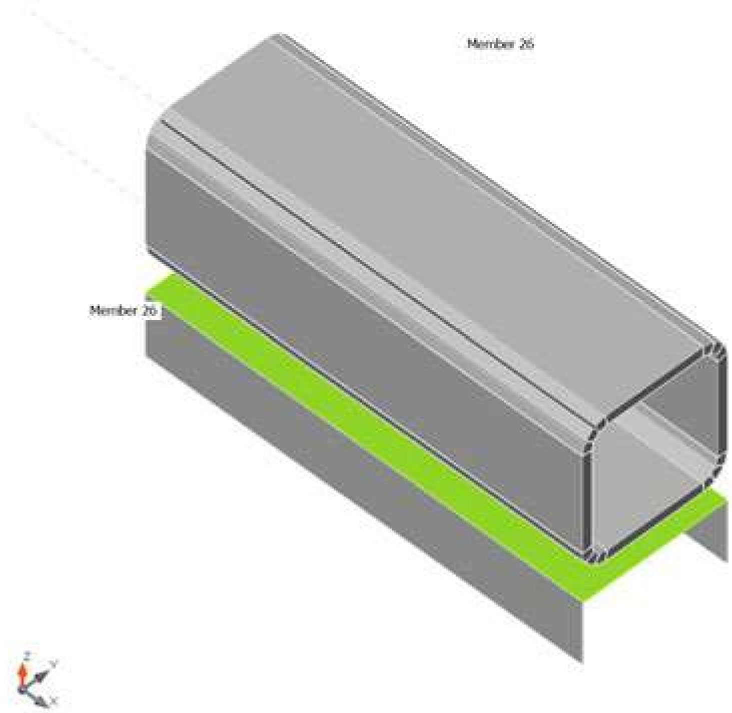
Name	t_p [in]	Loads	σ_{Ed} [psi]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [psi]	Status
Member 26	3/8	LE2	15202.0	0.0	0.0	OK
Member 27-bfl 1	1/16	LE2	10094.1	0.0	0.0	OK
Member 27-tfl 1	1/16	LE2	38009.8	0.0	3690.8	OK
Member 27-w 1	1/16	LE2	44901.7	0.0	0.0	OK
Plate 1	1/4	LE2	45385.0	1.3	4554.3	OK

Design data

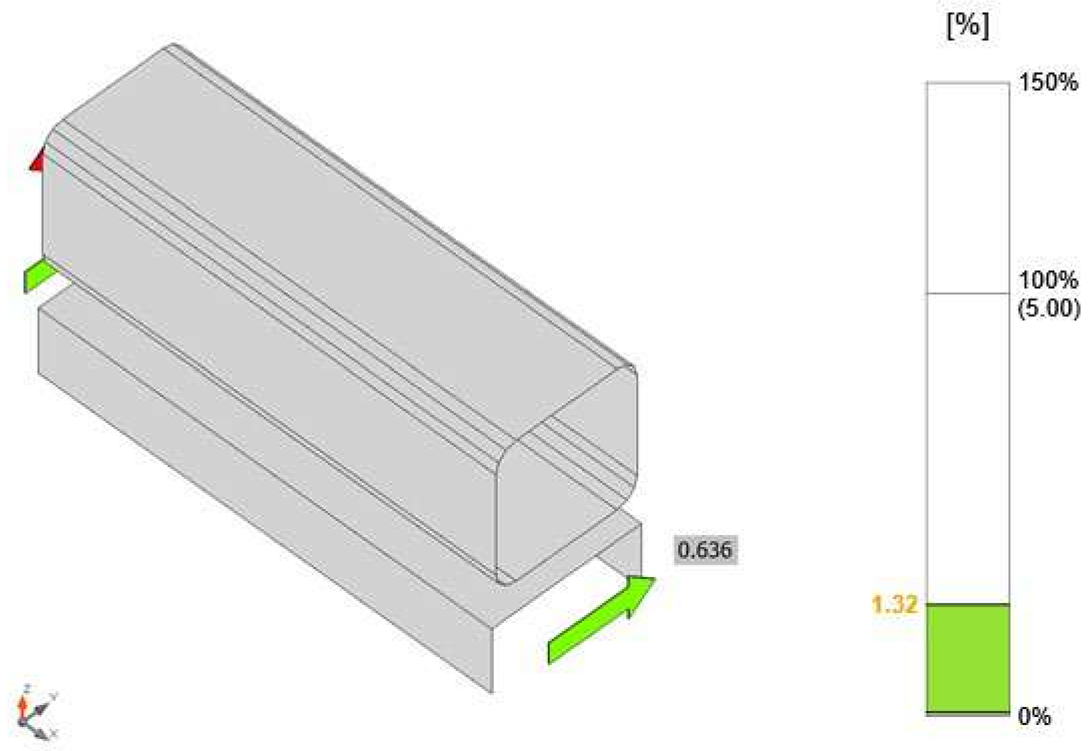
Material	F_y [psi]	ε_{lim} [%]
A913 Gr.50	50000.3	5.0

Loc. deformation

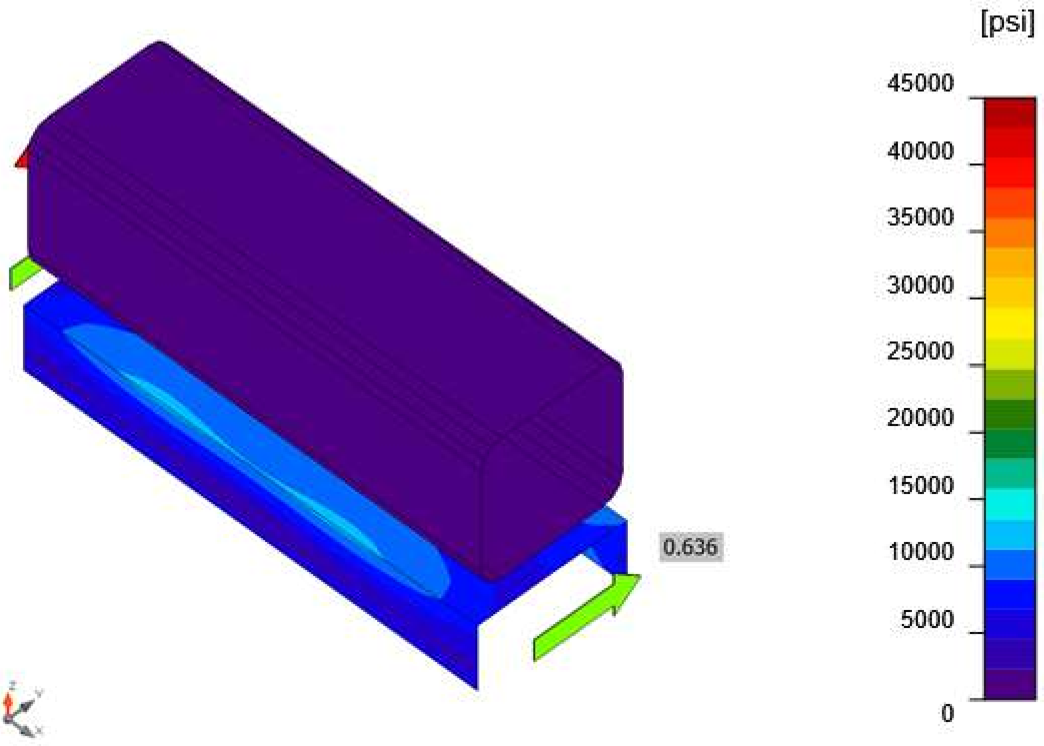
Name	d_0 [in]	Loads	δ [in]	δ_{lim} [in]	δ/d_0 [%]	Check status
Member 26	5"7/8	LE2		3/16	0.0	OK



Overall check, LE2



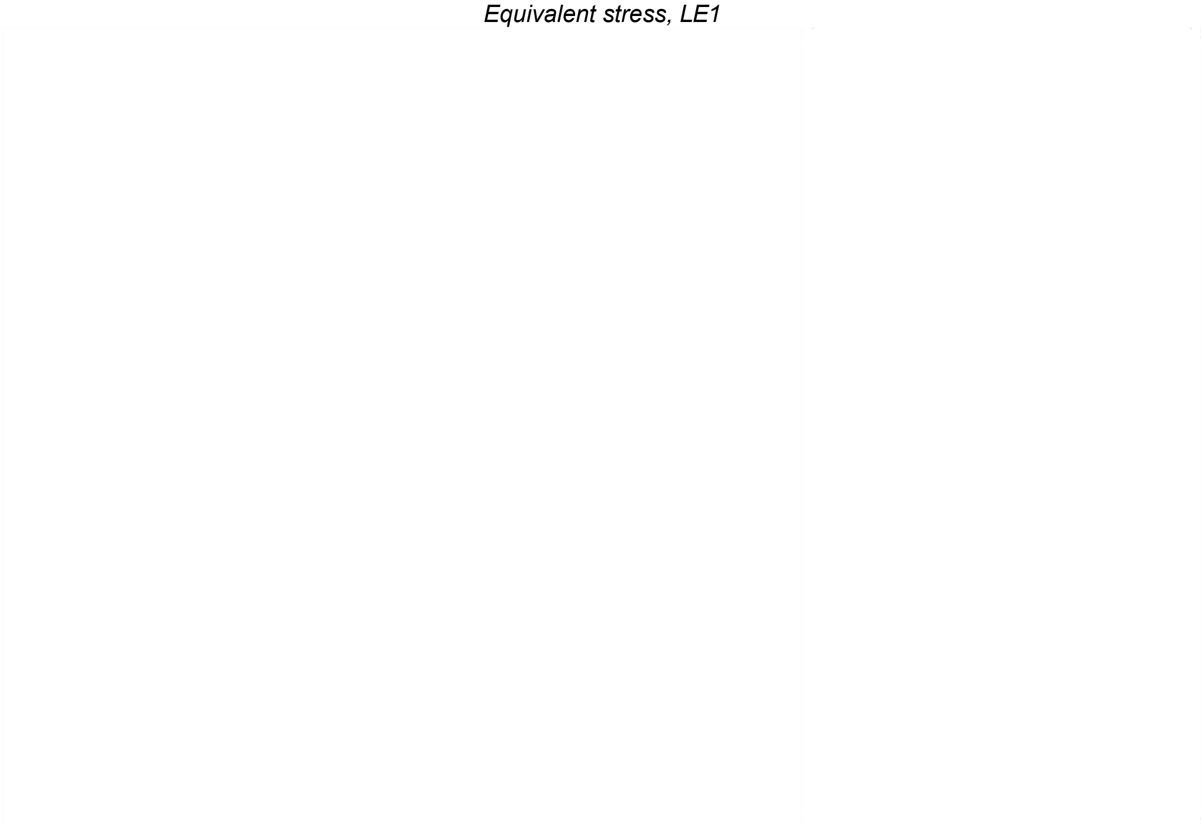
Strain check, LE2



Equivalent stress, LE2

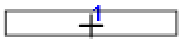
Overall check, LE1

Equivalent stress, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	12 A325M - 1	LE2	0.908	0.000	3.482	7.7	0.0	-	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
12 A325M - 1	11.813	7.088

Detailed result for B1

Tension resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nt} \cdot A_b =$ 11.813 kip ≥ $F_t =$ 0.908 kip

Where:

F_{nt} = 89923.4 psi – nominal tensile stress AISC 360-16 – Table J3.2

A_b = 0.1752 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Shear resistance check (AISC 360-16 – J3-1)

$\phi R_n = \phi \cdot F_{nv} \cdot A_b =$ 7.088 kip ≥ $V =$ 0.000 kip

Where:

F_{nv} = 53954.0 psi – nominal shear stress AISC 360-16 – Table J3.2

A_b = 0.1752 in^2 – gross bolt cross-sectional area

ϕ = 0.75 – resistance factor

Bearing resistance check (AISC 360-16 – J3-6)

$R_n = 1.20 \cdot l_c \cdot t \cdot F_u \leq 2.40 \cdot d \cdot t \cdot F_u$

$\phi R_n =$ 3.482 kip ≥ $V =$ 0.000 kip

Where:

l_c = 3'-3"1/8 in – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material

t = 1/16 in – thickness of the plate

d = 1/2 in – diameter of a bolt

F_u = 65000.1 psi – tensile strength of the connected material

ϕ = 0.75 – resistance factor for bearing at bolt holes

Interaction of tension and shear check (AISC 360-16 – J3-2)

The required stress, in either shear or tension, is less than or equal to 30% of the corresponding available stress and the effects of combined stresses need not to be investigated.

Welds

Item	Edge	Xu	t_w [in]	w [in]	L [in]	L_c [in]	Loads	F_n [kip]	ϕR_n [kip]	Ut [%]	Status
Member 26-w 4	Plate 1	E70xx	◢ 1/8	◢ 3/16	1"15/16	3/8	LE2	1.735	1.804	96.2	OK
Member 26-w 4	Plate 1	E70xx	◢ 1/8	◢ 3/16	1"15/16	3/8	LE2	1.740	1.804	96.4	OK
Member 26-w 4	Plate 1	E70xx	◢ 1/8	◢ 3/16	3"	3/8	LE2	0.258	1.718	15.0	OK

Detailed result for Member 26-w 4 / Plate 1 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 1.983 kip ≥ $F_n =$ 1.735 kip

Where:

F_{nw} = 60604.0 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
    - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
    - $\theta =$ 67.3° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0436 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 1.804 kip ≥ $F_n =$ 1.735 kip

Where:

F_{nBM} = 39000.1 psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
    - $F_u =$ 65000.1 psi – tensile strength of the connected material

A_{BM} = 0.0617 in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot \sqrt{2}$, where:
    - $A_{we} =$ 0.0436 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Member 26-w 4 / Plate 1 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 1.983 kip ≥ $F_n =$ 1.740 kip

Where:

F_{nw} = 60612.7 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  -
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength

    $\theta =$ 67.3° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0436 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 1.804 kip ≥ $F_n =$ 1.740 kip

Where:

F_{nBM} = 39000.1 psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
  - $F_u =$ 65000.1 psi – tensile strength of the connected material

A_{BM} = 0.0617 in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot \sqrt{2}$, where:
  - $A_{we} =$ 0.0436 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Detailed result for Member 26-w 4 / Plate 1 - 1

Weld resistance check (AISC 360-16 – J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 1.757 kip ≥ $F_n =$ 0.258 kip

Where:

F_{nw} = 56410.4 psi – nominal stress of weld material:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, where:
  - $F_{EXX} =$ 70000.0 psi – electrode classification number, i.e. minimum specified tensile strength
  - $\theta =$ 51.1° – angle of loading measured from the weld longitudinal axis

A_{we} = 0.0415 in^2 – effective area of weld critical element

ϕ = 0.75 – resistance factor for welded connections

Base metal capacity check (AISC 360-16 – J2-2)

$\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} =$ 1.718 kip ≥ $F_n =$ 0.258 kip

Where:

F_{nBM} = 39000.1 psi – nominal stress of the base metal:

- $F_{nBM} = 0.6 \cdot F_u$, where:
  - $F_u =$ 65000.1 psi – tensile strength of the connected material

A_{BM} = 0.0587 in^2 – cross-sectional area of base metal:

- $A_{BM} = A_{we} \cdot \sqrt{2}$, where:
  - $A_{we} =$ 0.0415 in^2 – effective area of weld critical element

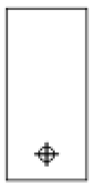
ϕ = 0.75 – resistance factor for welded connections

Buckling

Buckling analysis was not calculated.

Bill of material

Manufacturing operations

Name	Plates [in]	Shape	Nr.	Welds [in]	Length [in]	Bolts	Nr.
Plate 1	P1/4x3"-6"1/2 (A913 Gr.50)		1			12 A325M	1

Welds

Type	Material	Throat thickness [in]	Leg size [in]	Length [in]
Fillet	E70xx	1/8	3/16	6"15/16

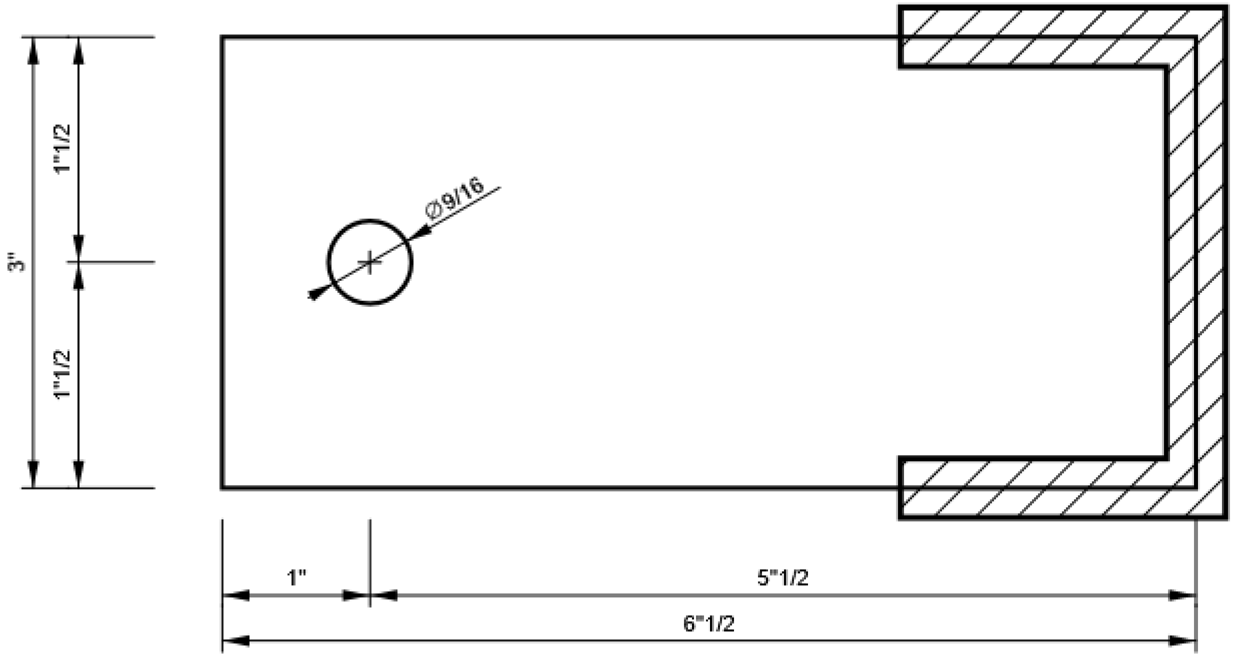
Bolts

Name	Grip length [in]	Count
12 A325M	5/16	1

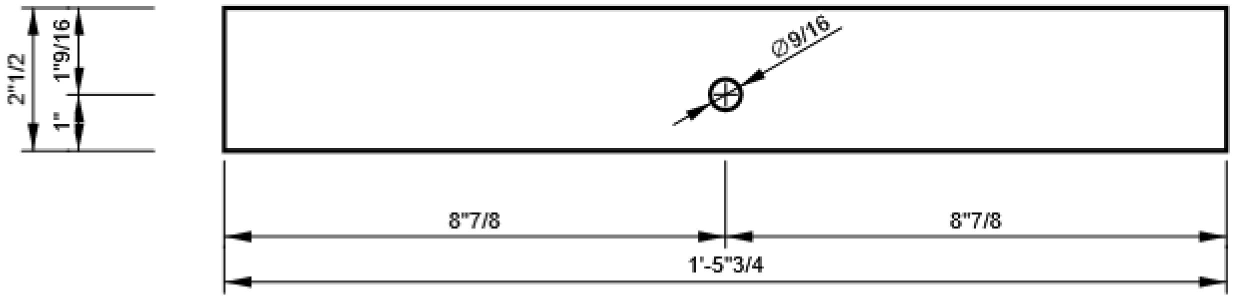
Drawing

Plate 1

P1/4x6"1/2-3" (A913 Gr.50)



Member 27, C.150.50.15(U150) - Top flange 1:



Symbol explanation

Symbol	Symbol explanation
t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
F_y	Yield strength
ε_{lim}	Limit of plastic strain
F_t	Tension force
V	Resultant of bolt shear forces Vy and Vz in shear planes
$\phi Rn_{Bearing}$	Bolt bearing resistance
Ut	Utilization
Ut_s	Utilization in shear
Ut_{ts}	Interaction of tension and shear EN 1993-1-8 – Tab. 3.4
$\phi Rn_{Tension}$	Bolt tension resistance - AISC 360-16 – J3.6
ϕRn_{Shear}	Bolt shear resistance - AISC 360-16 – J3.6
◢	Fillet weld
T_h	Throat thickness of weld
L_s	Leg size of weld
L	Length of weld
L_c	Length of critical weld element
F_n	Force in weld critical element
ϕRn_w	Weld resistance - AISC 360-16 – J2-4

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349-01 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Detailing	No		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	Yes		AISC 360-16 – J2-2
Deformation at bolt hole at service load is design consideration	Yes		AISC 360-16 – J3.10
Cracked concrete	Yes		ACI 318-14 – 17
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	4.Base track to concrete	Date:	15/02/2024
Fastening point:			

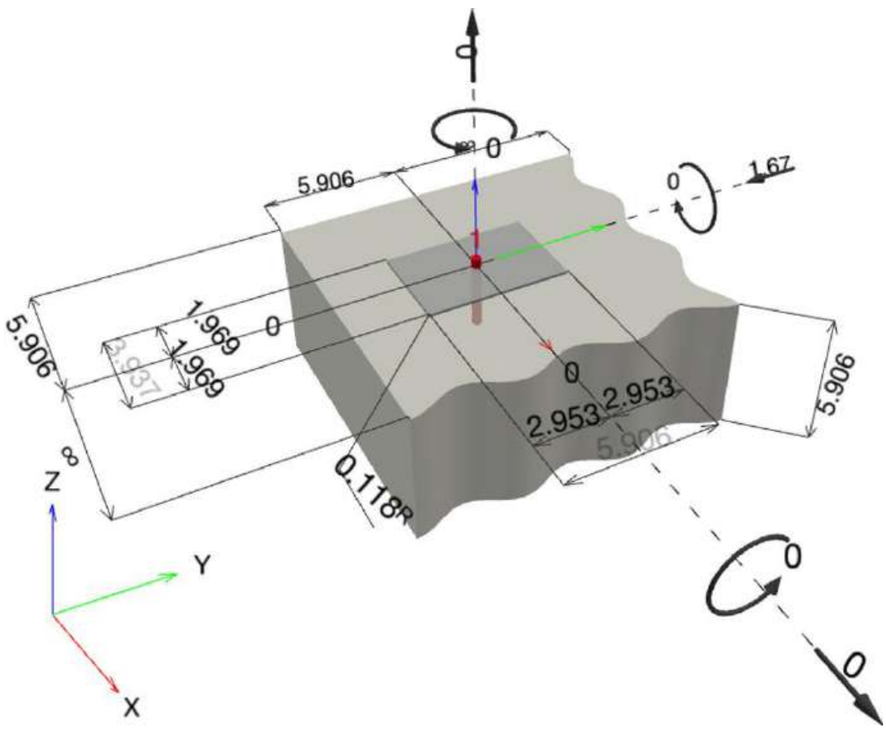
Specifier's comments:

1 Input data

Anchor type and diameter:	**HUS4-HF 10 h_nom3**
Return period (service life in years):	50
Item number:	2293596 HUS4-HF 10x100 45/25/15
Effective embedment depth:	h_{ef} = 2.677 in. ($h_{ef,ETA}$ = 2.677 in.), h_{nom} = 3.819 in.
Material:	Carbon Steel
Evaluation Service Report:	ETA-20/0867
Issued I Valid:	14/07/2022 | -
Proof:	Design Method ETAG (No. 001 Annex C/2010)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.118 in.
Anchor plate^R :	l_x x l_y x t = 3.937 in. x 5.906 in. x 0.118 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, C20/25, $f_{c,cube}$ = 3,626 psi; h = 5.906 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)
	no longitudinal edge reinforcement

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

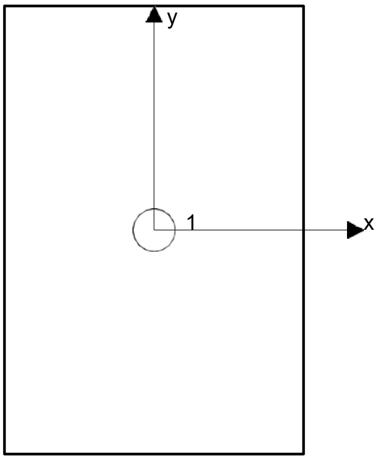
Geometry [in.] & Loading [kip, ft.kip]



1.1 Load combination

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Fire	Max. Util. Anchor [%]
1	Combination 1	N = 0.000; V_x = 0.000; V_y = -1.670; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	no	no	79

2 Load case/Resulting anchor forces



Anchor reactions [kip]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	1.670	0.000	-1.670

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0.000 [kip]
resulting compression force in (x/y)=(0.000/0.000): 0.000 [kip]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load (ETAG, Annex C, Section 5.2.2)

	Load [kip]	Capacity [kip]	Utilization b_N [%]	Status
Steel Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Splitting failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

4 Shear load (ETAG, Annex C, Section 5.2.3)

	Load [kip]	Capacity [kip]	Utilization b_V [%]	Status
Steel Strength (without lever arm)*	1.670	5.755	30	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1.670	6.051	28	OK
Concrete edge failure in direction y-**	1.670	2.139	79	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength (without lever arm)

$V_{Sd} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{M,s}}$ ETAG 001 Annex C, Table 5.2.3.1

$V_{Rk,s}$ [kip]	$\gamma_{M,s}$	$V_{Rd,s}$ [kip]	V_{Sd} [kip]
7.194	1.250	5.755	1.670

4.2 Pryout Strength

$V_{Sd} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{M,c,p}}$ ETAG 001 Annex C, Table 5.2.3.1

$V_{Rk,cp} = k \cdot N_{Rk,c}$ ETAG 001 Annex C, Eq. (5.6)

$N_{Rk,c} = N^0_{Rk,c} \cdot \frac{A_{c,N}}{A^0_{c,N}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec1,N} \cdot \psi_{ec2,N}$ ETAG 001 Annex C, Eq. (5.2)

$N^0_{Rk,c} = k_1 \cdot \sqrt{f_{ck,cube}} \cdot h_{ef}^{1,5}$ ETAG 001 Annex C, Eq. (5.2a)

$A^0_{c,N} = s_{cr,N} \cdot s_{cr,N}$ ETAG 001 Annex C, Eq. (5.2b)

$\psi_{s,N} = 0.7 + 0.3 \cdot \frac{c}{c_{cr,N}} \leq 1.00$ ETAG 001 Annex C, Eq. (5.2c)

$\psi_{re,N} = 0.5 + \frac{h_{ef}}{200} \leq 1.00$ ETAG 001 Annex C, Eq. (5.2d)

$\psi_{ec1,N} = \frac{1}{1 + \frac{2 \cdot e_{c1,V}}{s_{cr,N}}} \leq 1.00$ ETAG 001 Annex C, Eq. (5.2e)

$\psi_{ec2,N} = \frac{1}{1 + \frac{2 \cdot e_{c2,V}}{s_{cr,N}}} \leq 1.00$ ETAG 001 Annex C, Eq. (5.2e)

$A_{c,N}$ [in.^{2}]	$A^0_{c,N}$ [in.^{2}]	$c_{cr,N}$ [in.]	$s_{cr,N}$ [in.]	k-factor
64.50	64.50	4.016	8.031	2.000

$e_{c1,V}$ [in.]	$\psi_{ec1,N}$	$e_{c2,V}$ [in.]	$\psi_{ec2,N}$	$\psi_{s,N}$	$\psi_{re,N}$
0.000	1.000	0.000	1.000	1.000	1.000

$N^0_{Rk,c}$ [kip]	$\psi_{M,c,p}$	$V_{Rd,cp}$ [kip]	V_{Sd} [kip]
4.538	1.500	6.051	1.670

Group anchor ID
1

4.3 Concrete edge failure in direction y-

$V_{Sd} \le V_{Rd,c} = \frac{V_{Rk,c}}{\psi_{M,c}}$	ETAG 001 Annex C, Table 5.2.3.1
$V_{Rk,c} = V^0_{Rk,c} \cdot \frac{A_{c,V}}{A^0_{c,V}} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$	ETAG 001 Annex C, Eq. (5.7)
$V^0_{Rk,c} = k_1 \cdot d^{\alpha}_{nom} \cdot l^{\beta}_f \cdot \sqrt{f_{ck,cube}} \cdot c_1^{1.5}$	ETAG 001 Annex C, Eq. (5.7a)
$\alpha = 0.1 \cdot \left(\frac{l_f}{c_1}\right)^{0,5}$	ETAG 001 Annex C, Eq. (5.7b)
$\beta = 0.1 \cdot \left(\frac{d_{nom}}{c_1}\right)^{0,2}$	ETAG 001 Annex C, Eq. (5.7c)
$A^0_{c,V} = 4.5 \cdot c_1^2$	ETAG 001 Annex C, Eq. (5.7d)
$\psi_{s,V} = 0.7 + 0.3 \cdot \frac{c_2}{1.5 \cdot c_1} \le 1.00$	ETAG 001 Annex C, Eq. (5.7e)
$\psi_{h,V} = \left(\frac{1.5 \cdot c_1}{h}\right)^{0,5} \ge 1.00$	ETAG 001 Annex C, Eq. (5.7f)
$\psi_{\alpha,V} = \sqrt{\frac{1}{(\cos \alpha_V)^2 + \left(\frac{\sin \alpha_V}{2.5}\right)^2}} \ge 1.00$	ETAG 001 Annex C, Eq. (5.7g)
$\psi_{ec,V} = \frac{1}{1 + \frac{2 \cdot e_{c,V}}{3 \cdot c_1}} \le 1.00$	ETAG 001 Annex C, Eq. (5.7h)

l_f [in.]	d_{nom} [in.]	k_1	α	β
2.677	0.3937	1.700	0.067	0.058

c_1 [in.]	$A_{c,V}$ [in.^{2}]	$A^0_{c,V}$ [in.^{2}]
5.906	87.19	156.94

$\psi_{s,V}$	$\psi_{h,V}$	α_V [°]	$\psi_{\alpha,V}$	$e_{c,V}$ [in.]	$\psi_{ec,V}$	$\psi_{re,V}$
0.900	1.225	0.00	1.000	0.000	1.000	1.000

$V^0_{Rk,c}$ [kip]	$\psi_{M,c}$	$V_{Rd,c}$ [kip]	V_{Sd} [kip]
5.240	1.500	2.139	1.670

5 Displacements (highest loaded anchor)

Short term loading:

N_{Sk}	=	0.000 [kip]	δ_N	=	0.000000 [in.]
V_{Sk}	=	1.237 [kip]	δ_V	=	0.300689 [in.]
			δ_{NV}	=	0.300689 [in.]

Long term loading:

N_{Sk}	=	0.000 [kip]	δ_N	=	0.000000 [in.]

V_{Sk}	=	1.237 [kip]	8_{V}	=	0.451034 [in.]
			8_{NV}	=	0.451034 [in.]

Comments: Tension displacements are valid with half of the required installation torque moment for uncracked concrete! Shear displacements are valid without friction between the concrete and the anchor plate! The gap due to the drilled hole and clearance hole tolerances are not included in this calculation!

The acceptable anchor displacements depend on the fastened construction and must be defined by the designer!

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- In general, the conditions given in ETAG 001, Annex C, section 4.2.2.1 and 4.2.2.3 b) are not fulfilled because the diameter of the clearance hole in the fixture acc. to Annex 3, Table 3 is greater than the values given in Annex C, Table 4.1 and AS5126 for the corresponding diameter of the anchor. Therefore the design resistance for anchor groups is limited to twice the steel resistance (of a single anchor) in accordance with the approval.
- Checking the transfer of loads into the base material is required in accordance with ETAG 001, Annex C(2010)Section 7! The software considers that the grout is installed under the anchor plate without creating air voids and before application of the loads.
- The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 4.1 of ETAG 001, Annex C! For larger diameters of the clearance hole see Chapter 1.1. of ETAG 001, Annex C!
- The accessory list in this report is for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The characteristic bond resistances depend on the return period (service life in years): 50

Fastening meets the design criteria!

7 Installation data

Anchor plate, steel: S 235; E = 30,457,925 psi; f_{yk} = 34,084 psi

Profile: no profile

Hole diameter in the fixture: d_f = 0.551 in.

Plate thickness (input): 0.118 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Clean the drill hole. Under the conditions - according to fastener size and drilling direction - given in the ETA and MPII (IFU), the cleaning of the drill hole may be omitted.

Anchor type and diameter: HUS4-HF 10 h_nom3

Item number: 2293596 HUS4-HF 10x100 45/25/15

Maximum installation torque: Hilti SIW 22T-A

Hole diameter in the base material: 0.394 in.

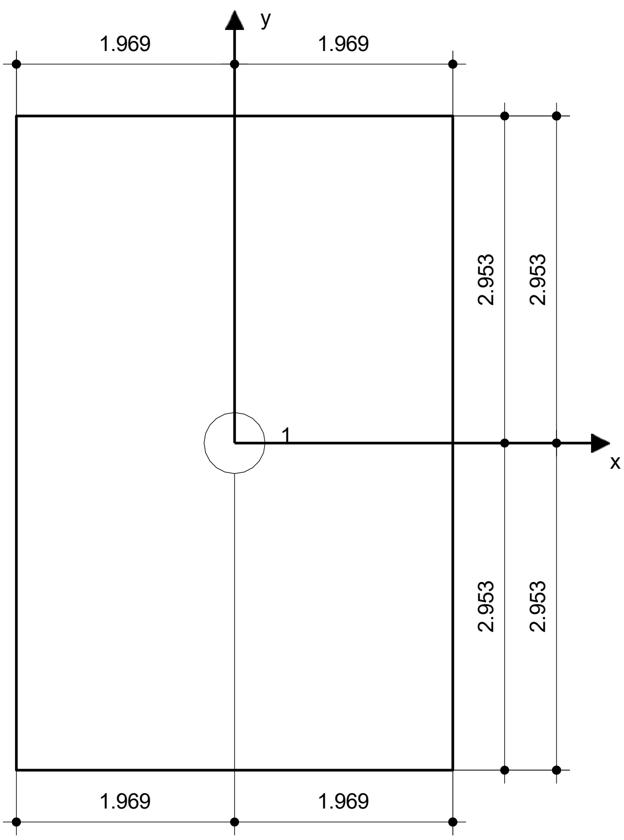
Hole depth in the base material: 4.213 in.

Minimum thickness of the base material: 5.394 in.

Hilti HUS screw anchor with 3.346457 in embedment, 10 h_nom3, Hot dip galvanized, installation per ETA-20/0867

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Hilti SIW 22T-A impact screw driver



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}

1	0.000	0.000	5.906	-	5.906	-

8 Remarks; Your Cooperation Duties

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