BMB&A J/S Co.

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BMB.
Best Metal Buildings

DESIGN GUIDELINES BMB&A

Approval Design Department
R&D Team

BMB & A J/S CO.
148 Phan Xich Long St, W.7, Phu Nhuan Dist, HCMC, Viet Nam
Phone (84-28) 3 5172 509 . Fax (84-28) 3 5172512
Website: www.bmbsteel.com.vn

R&D TEAM

DESIGN GUIDELINES BMB&A

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Approved by:	Chief of Approval Design Mai Xuan Quang
Issued by:	R&D Team Nguyen An Phu Tran Kim Thanh Vu Nguyen Thi Tuyet Hang
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148 Phan Xich Long Street, Ward 7, Phu Nhuan District, Ho Chi Minh City, Vietnam
Phone: (84-28) 3 5172 509 • Fax (84-28) 3 5172 512
Website: bmbsteel.com.vn

With lots of outstanding advantages compared with other materials, steel is increasingly used in civil and industrial buildings. Over the past hundreds of years, a vast majority of researches on steel structure conducted in order to improve safety for the structure as well as decrease cost of the building. To pursue the dream of becoming the largest steel company in Vietnam and over the world, design department made effort continuously to both finish all projects on schedule and study and apply newest achievements over the world to buildings. Standards and documents from the most developed country in steel field America used to guarantee the stability of the structure as well as strict requirements of architects and weight from customers.
From the first design guide published in 2011 in this latest design guide, there are more additional theories and guidelines for constructing SAP model and calculating connection, mezzanine floors, loads for building, etc.… according to the newest standards. In addition, many practical pictures, tables, serving for computing and calculating supplemented as well as mistakes are also updated and revised. The object of this editing is to make a standard of designing for all designers and support the other department to understand more about the processing of the design department.
In the compilation process, mistakes cannot be avoided so it would be so helpful to receive any feedback from readers to more complete in the next edition.

Thanks and Best regards,

Chief of Approval Design,

Mai Xuan Quang

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GENERAL INFORMATION

Introduction

(1) The Design Department manual outlines the design process requirements of the BMB&A Standard in BMB&A J/S CO.

(2) The Design manual establishes rules and standards to ensure that parameters and figures are presented uniformly throughout in BMB&A J/S CO.

Scope of the design manual

(3) The design manual is binding for all Design Team, Estimator Team and Office in the BMB&A J/S CO.

CHAPTER 1. MATERIALS

1.1. Materials

Table 1 1 Material Specifications (unless indicated in drawings)

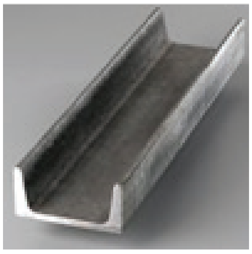
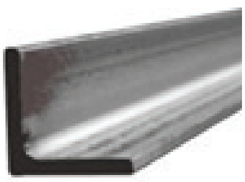
Materials	Specifications (or equivalent)	Minimum Strength
Built-up Member	ASTM-A572 GR50 (Q345) ASTM-A36 (Q235)	$Fy = 34.5kN / cm^2$ $Fy = 23.5kN / cm^2$
Steel Plate For Connection	ASTM-A572 GR50 (Q345)	$Fy = 34.5kN / cm^2$
Hot Rolled Member	JIS G3101/SS400	$Fy = 23.5kN / cm^2$
Cold Formed Light Gauge Shapes, Purlin & Girt Bracing	JIS G3302 (G450, Z275)	$Fy = 45.0kN / cm^2$
Roof Sheeting	ASTM A792M/755M	$Fy = 30.0kN / cm^2$
Decking	JIS G3302	$Fy = 23.5kN / cm^2$
Checker Plate	SS400	$Fy = 23.5kN / cm^2$
X - Bracing Rod	JIS G3101/SS400	$Fu = 40kN / cm^2$
X - Bracing Cable	ASTM A416	$Fu = 54kN / cm^2$
Bracing (Steel Pipe or Steel Angle)	JIS G3101/SS400	$Fy = 23.5kN / cm^2$
Anchor Bolt Grade 5.6	TCVN 1916-1995	$Fu = 50kN / cm^2$
Machine Bolts for Purlin/Girt Connections	DIN 933 TYPE 4.6	$Fu = 40kN / cm^2$
High Strength Bolt Grade 10.9	DIN 933 TYPE 10.9	$Fu = 100kN / cm^2$
High Strength Bolt Grade 8.8	DIN 933 TYPE 8.8	$Fu = 80kN / cm^2$
Minimum Flow Strength of Welding	AWS A5.17 & AWS A5.20 (E60XX)	$Fy \geq 34.5kN / cm^2$

1.2. Plates

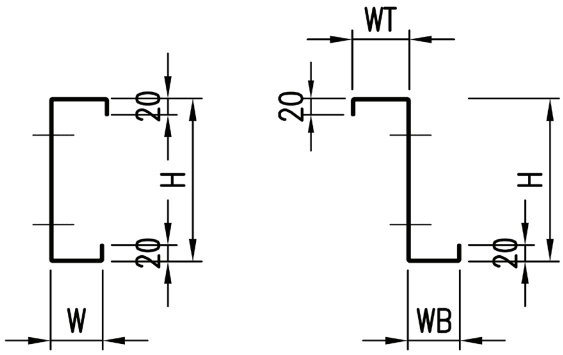
Thickness (mm)			**Order Size (m)**	**Specifications**
3	14	35	W = 1m; 1.5m; 2m; 2.5m; 3m	ASTM-A572 Gr50 (Q345)
4	16	40	L = 6m; 9m; 12m	ASTM-A36 (Q235)
5	18	50		
6	20	52		
8	22	55		
10	25			
12	30			

1.3. Hot Roll Members

Refer Appendix A for more sections.

Type (mm)	Order size (mm)	Usage
I section 	IPE 150x75x5x7 IPE 200x100x5.5x8 IPE 200x200x8x12 IPE 250x125x6x9 IPE 300x150x6.5x9 IPE 300x300x10x15 IPE 400x200x8x13	Wind column, end wall rafter, mezzanine joist, & sub-structure
Tube 	▯ 100x100x2.5 ▯ 150x150x2.8/3.5 ▯ 200x100x2.8	EB bracing, man door/ window/ shutter door frames, sub-structure
Channel 	U 100x50x5x7.5 U 125x65x6x8 U 150x75x6.5x10	Joist
	U 200x73x6.5x8 U 200x73x8.5x10 U 250x78x7x11	A Cap channel for crane beams, a stringer for the staircase
Angle 	V 50x50x5/6 V 60x60x5 V 63x63x5/6 V 75x75x6/7 V 90x90x6/7 V 100x100x8/10	Flange braces, X bracing & open web joist members
Pipe 	Ø 22x2.0 Ø 32x2.0 Ø 42x3.2	Steel handrail
	Ø 60x3.0 Ø 90x2.0/2.3/3.0/3.2/4.0 Ø 114x2.5/2.9/3.18/3.96 Ø 141x3.96 Ø 168x3.96 Ø 219x4.78 Ø 273x6.35	X bracing, Space frame, diagonal member

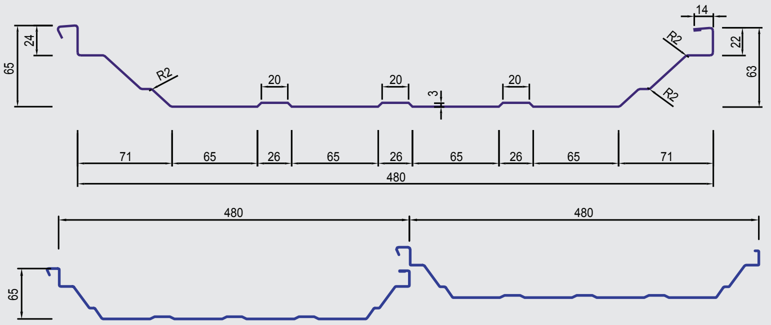
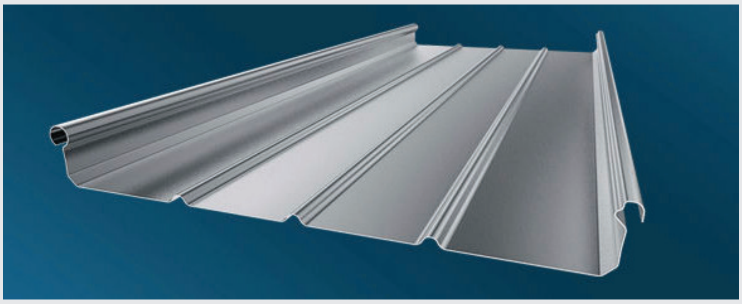
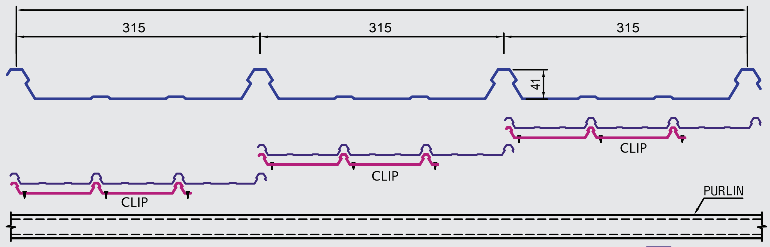
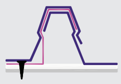
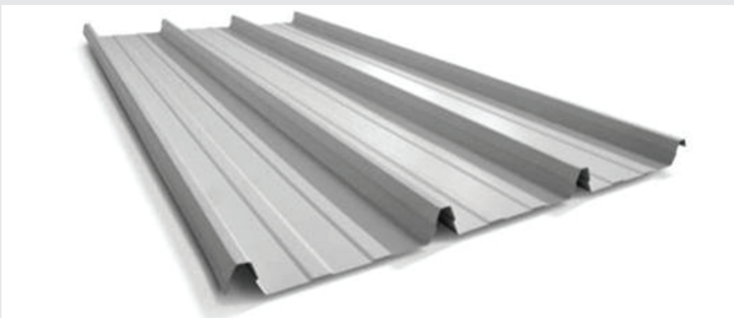
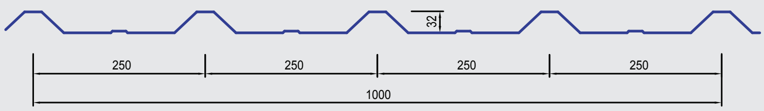
1.4. Cold Form Sections



Sym.	WB-WT	Thickness/Quantity (kg/m)						
		1.5	1.8	2	2.3	2.5	2.8	3
C100	50*	2.593	3.111	3.457	3.975	4.321	-	-
C120	50*	2.833	3.399	3.777	4.344	4.721	-	-
C150	50*	3.193	3.831	4.257	4.896	5.321	-	-
C150	65	3.553	4.263	4.737	5.448	5.922	-	-
C150	75*	3.793	4.552	5.057	5.816	6.322	-	-
C180	50*	3.553	4.263	4.737	5.448	5.922	-	-
C180	65	3.913	4.696	5.217	6.000	6.522	-	-
C200	50*	3.793	4.552	5.057	5.816	6.322	7.080	7.586
C200	65	4.153	4.984	5.537	6.368	6.922	7.752	8.306
C200	70*	4.273	5.128	5.697	6.552	7.122	7.976	8.546
C200	100*	4.993	5.992	6.658	7.656	8.322	9.321	9.987
C250	65	4.753	5.704	6.338	7.288	7.922	8.873	9.506
C300	65	-	6.424	7.138	8.208	8.922	9.993	10.707
Z150	50-56	3.265	3.918	4.353	5.006	5.441	-	-
Z150	62-68*	3.553	4.263	4.737	5.448	5.922	-	-
Z175	50-56	3.553	4.263	4.737	5.448	5.922	-	-
Z175	62-68*	3.853	4.624	5.137	5.908	6.422	-	-
Z200	50-56*	3.853	4.624	5.137	5.908	6.422	7.192	7.706
Z200	62-68	4.153	4.984	5.537	6.368	6.922	7.752	8.306
Z200	72-78*	4.393	5.272	5.857	6.736	7.322	8.200	8.786
Z250	62-68	4.753	5.704	6.338	7.288	7.922	8.873	9.506
Z250	72-78*	4.993	5.992	6.658	7.656	8.322	9.321	9.987
Z300	62-68	-	6.424	7.138	8.208	8.922	9.993	10.707
Z300	72-78*	-	6.712	7.458	8.577	9.322	10.441	11.187

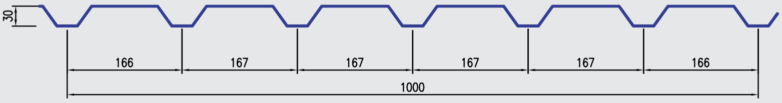
(*) Not available at Hung Yen Factory.

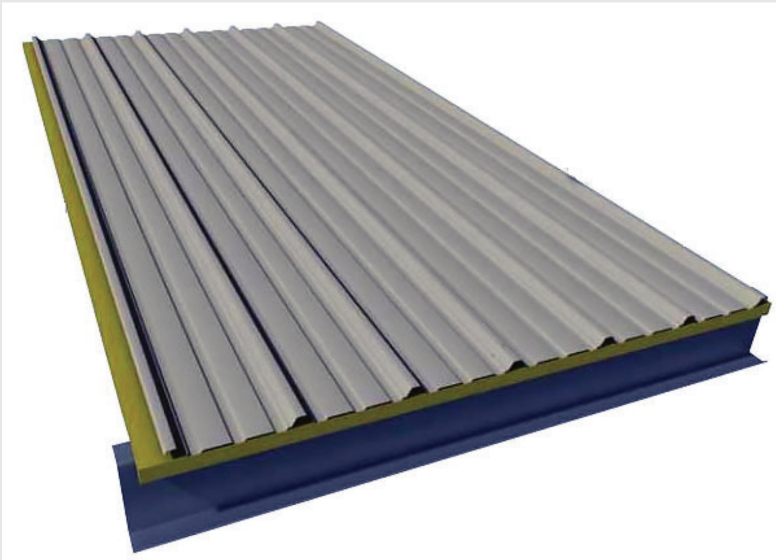
1.5. Sheeting

Type Usage	Order size (mm)	Specifications
Seam Roof panel		G300 $F_y = 30\,kN/cm^2$
Clicklock Roof + Wall		G500 $F_y = 50\,kN/cm^2$
5 Ribs Panel Roof panel Wall panel		G500 $F_y = 50\,kN/cm^2$

7 Ribs panel

Roof, wall panel



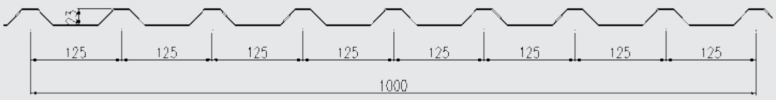


G500

$F_y = 50\,kN/cm^2$

9 ribs panel

Wall panel

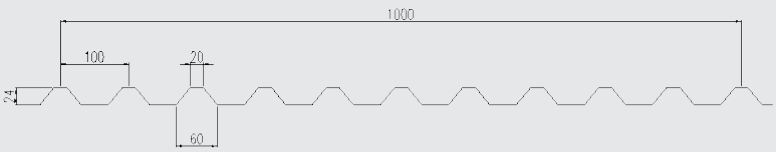


G500

$F_y = 50\,kN/cm^2$

11 ribs panel

Wall panel



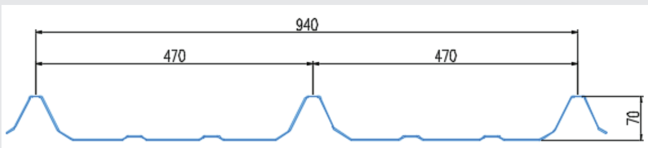
G300

$F_y = 30\,kN/cm^2$

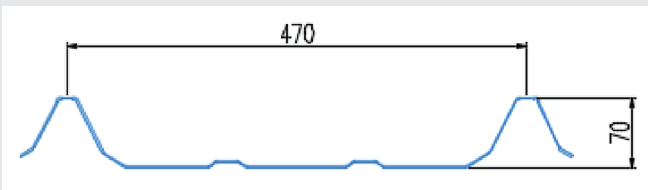
Skylight

Roof, wall panel

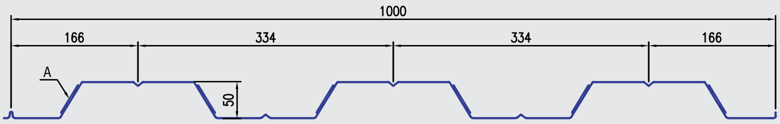
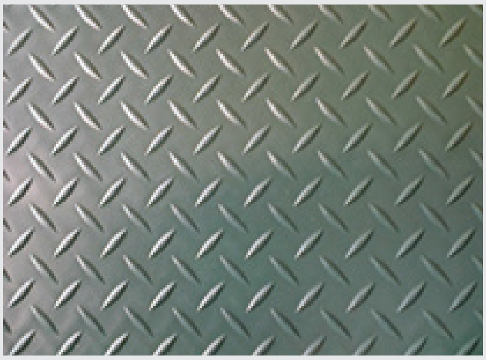
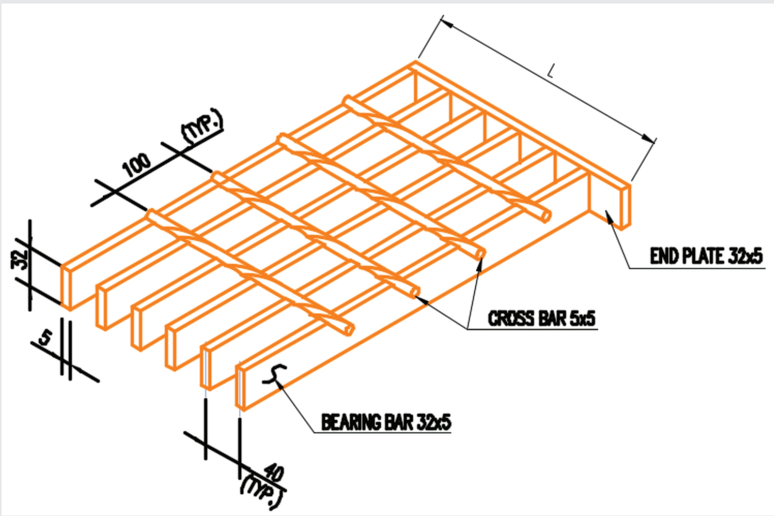
Size 940



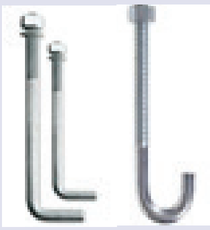
Size 470



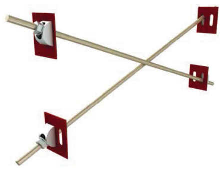
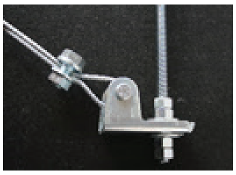
1.6. Slab

Type	Order size (mm)	Specifications
Decking	 Thickness: 0.75, 0.95, 1.15mm.	JIS G3302 $F_y = 23.5\,kN/cm^2$
Checker	 Thickness: 4, 5, 6, 8mm	SS400 $F_y = 23.5\,kN/cm^2$
Grating	**Typical size:** 32x5 space 40mm  Other sizes: 25×3 space 30mm; 25×4 space 40mm; 25×5 space 30mm 30×3 space 30mm; 30×5 space 30mm; 40×3 space 30mm; 40×5 space 30mm	

1.7. Bolts

Type		Bolt diameter (mm)	Order length (mm)	Usage	Specifications
Cast-in Anchor Bolt	**Headed bolt** 			Anchor bolt for end wall frame & partitions, column bases	TCVN 1916-1995 (Grade 5.6) $F_u = 50\, kN/cm^2$
	& Hooked bolt 	M16 M20 M24 M27 M30 M36 M42 M48	400 500 600 700 800 900 1000 1200	Anchor bolt for mainframe & mezzanine column bases	
Post-installed Anchor Bolt	**Chemical bolt** 	M16 M20 M24 M27 M30	190 260 300 340 380	Anchor bolt for partitions & stair column bases Connect to RC column	(Grade 4.6) $F_u = 40\, kN/cm^2$
	Expansion bolt 	M12 M16 M20	106 145 184		
Machine bolt		M12		Purlin/girt connection	DIN 933 Grade 4.6
High strength bolt		M16 M20 M24 M27 M30 M36	60 70 90 90 100 100	Mainframe connection	DIN 933 Grade 8.8 DIN 933 Grade 10.9

1.8. Bracing

Type (mm)	**Order size** (mm)	**Tension strength** (kN)	**Usage**	**Specifications**
Rod 	Ø16 Ø20 2Ø16 Ø25 2Ø20	30.16 47.12 60.32 73.63 94.25	X-bracing in roof and wall	JIS G3101/SS400 $F_u = 40\, kN/cm^2$
Cable 	Ø16 Ø20 2Ø16 Ø25 2Ø20	40.72 63.62 81.43 99.4 127.23		ASTM A416 $F_u = 54\, kN/cm^2$

CHAPTER 2. CODES AND LOADS

2.1. Codes and Manuals

The Pre-Engineered Building described in these calculations was designed according to the latest U.S.A. Buildings and Design Codes that have been referred to in the design:

1. "Minimum Design Loads for Buildings and Other Structures", ASCE 7-10.
2. "International Buildings Code", IBC– 2012.
3. "Metal Building Systems Manual 2012" issued by MBMA.
4. "American Institute of Steel Construction" - Allowable Stress Design, AISC 360-10.
5. Cold formed components have been designed in accordance with:
   "American Iron and Steel Institute" Cold Formed Steel Design Manual, AISI S100-2007
6. Welding has been applied in accordance with:
   "American Welding Society", AWS-2010.
7. Wind speed & live load for Vietnam Project has been applied in accordance with:
   "Load and Effects - Design Standard" TCVN 2737:1995.
8. Earthquake load has been considered in accordance with:
   "Uniform Building Code", UBC 1997 Edition.

The above codes are to be used for the design of buildings by BMB&A design engineers unless otherwise specified in the Contract Information Form.

2.2. Design Loads

BMB&A Pre-Engineered Buildings are designed according to ASCE 7-10.

2.2.1. Dead load (DL)

Dead Load on Roof Sheeting

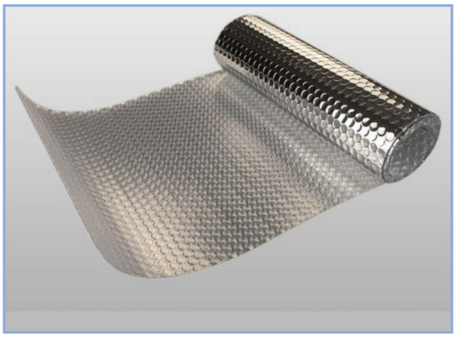
This includes the self-weight of rigid frames and imposed dead load due to secondary elements like roof sheeting, purlins, insulation…

Following are some standard dead loads (in kN/m2) as below:

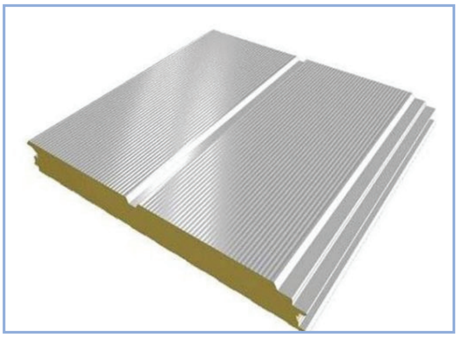
Purlin spacing	Insulation	DL (kN/m²)
$> 1.2m$	• 1 layer of (*Fiberglass/Air bubble/PE/Sandwich*)	0.10
$> 1.2m$	• 2 layers of (*Fiberglass/Air bubble/PE/Sandwich*) • PU (2 layers sheeting) • Rock Wool	0.15
≤1.2m	• 1 layer of (*Fiberglass/Air bubble/PE/Sandwich*)*	0.15

* Calculate manually dead load when using 2 layers insulation and purlin spacing is smaller than 1.2 m

Fiberglass

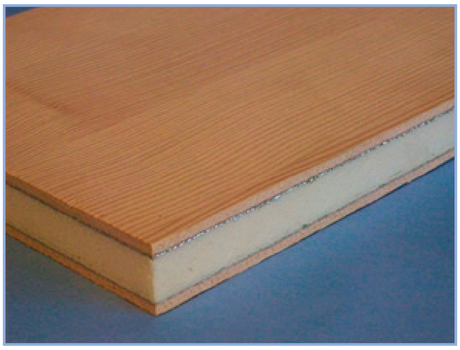


Air Bubble

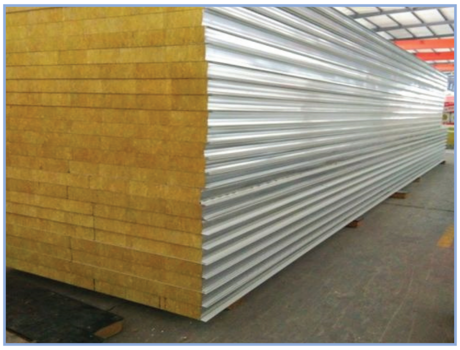


PU (Poly Urethane)

PE (Poly Ethylene)



Sandwich



Rock Wool

Figure 2.1 Types of Insulation

Mezzanine dead load

Concrete slab (in kN/m2)
Decking panel/ Checker plate/ Grating (in kN/m2)
Finishing (in kN/m2)
Wall load on beams, joists (in kN/m)
Stair/ elevator (in kN)
These values are determined depending on the specific weight & the sizes of material.
Following are some specific weight that BMB&A is used to calculate:

Material		Uniform Load
Reinforced concrete		25.00 kN/m^3
Brick wall		18.00 kN/m^3
Tempered glass		25.00 kN/m^3
Solar cell		0.23-0.27 kN/m^2
Grating		0.45-0.82 kN/m^2
Insulation	Fiberglass thickness 50mm/ 100mm	0.12 ÷ 0.24 kN/m^3
	Air bubble P2/A2	0.002 ÷ 0.004 kN/m^2
	PE foam thickness 5mm	0.20 ÷ 0.30 kN/m^3
	Sandwich	0.16 kN/m^3
	Rock wool	0.27-0.35 kN/m^3
	PU thickness 50mm	0.38-0.40 kN/m^3
Sheeting thickness 0.45mm		0.05 kN/m^2

2.2.2. Collateral load (AL)

Collateral load here means superimposed dead load (in kN/m2) includes:

Ceiling (Gypsum board)
HVAC duct
Lighting fixtures
Sprinklers

Besides, some collateral systems (such as HVAC duct, sprinklers) have live load; we calculate them into live load (LL).

2.2.3. Live load

Roof live load (LLr)

The roof live load is used to design purlin, depends on the tributary area of rigid frames. It includes the weight of labor & accessary to install, repair the roof.

Refer to *ASCE 7-10, Table 4-1 Minimum Uniformly Distributed Live Loads, L_o, and Minimum Concentrated Live Loads*. For built-up frames, minimum uniformly distributed live load on the roof is 1.0 kN/m². *ASCE 7-10 section 4.8.2* allows the use of 0.57kN/m² as live load for roof and purlins. Roof live loads as per other building codes should be verified before proceeding with your design. Some customers/consultants may require pattern loading in live load applications.

Live load on frame (LL)

Refer to *TCVN 2737:1995 (Load and Effects - Design Standard), section 4.3.1*, minimum uniformly distributed live load on the frame is 0.3 kN/m².

Country	Code	LL on roof (kN/m2)	LL on frame (kN/m2)
United States	ASCE 7-10	0.57 - 1	0.57 - 1
Vietnam	TCVN 2737:1995	0.57	0.3
Thailand	~ ASCE 7-10	0.57	0.3
Myanmar	MNBC 2016 ~ ASCE 7-10	0.57	0.3
Cambodia	~ ASCE 7-10	0.57	0.3
Indonesia	~ ASCE 7-10	0.57	0.3
Philippines	NSCP 2015 ~ ASCE 7-10	0.75-1.00	0.6

Mezzanine live load (FL)

For floor loads of different occupancy, refer to CHAPTER 6. MEZZANINE FLOOR DESIGN.

2.2.4. Wind load (WL)

The wind load pressure is determined in accordance with *Chapter 26-Chapter30, ASCE 7-10*. Wind loads are governed by wind speed, roof slope, wave height and open wall conditions of the building. BMB&A's steel buildings are not designed for a wind speed less than 110 km/h.

Wind load on frame W depends on importance factor I_w.

$$W = q_z \times \left(GC_{pf} - GC_{pi}\right) \times B$$

Where: - W = wind design pressure on frame

- q_z = velocity pressure evaluated at height z above ground (N/m²);

$q_z = 0.613 \times K_z \times K_{zt} \times K_d \times V^2 (N/m^2)$; V in m/s. *(27.3-1, ASCE 7-10)*

- GC_{pf} = product of the equivalent external pressure coefficient and gust-effect factor to be used in determination of wind loads for MWFRS of low-rise buildings

- GC_{pi} = product of the internal pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings
- B = bay spacing (m)

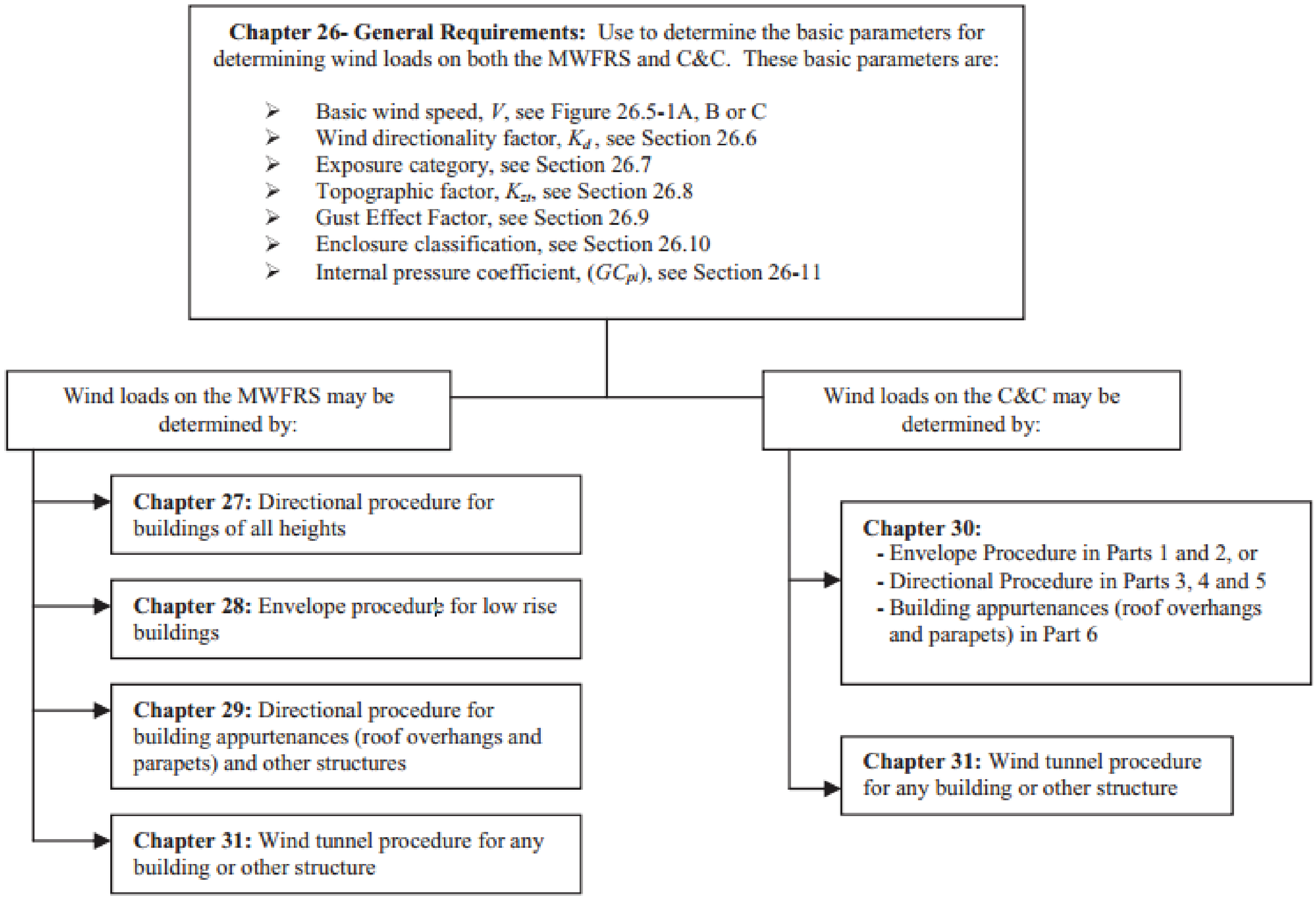


Figure 2.4 Outline of process for determining Wind load

Risk categorization

Building and other structures shall be classified, based on the risk to human life, health, and welfare associated with their damage or failure by nature of their occupancy or use, refer to *Table 1.5-1, ASCE 7-10* for purposes of applying flood, wind, snow, earthquake, and ice provisions. Each building or other structure shall be assigned to the highest applicable risk category or categories.

Table 2 1 Risk Category of Buildings and Other Structures in Flood, Wind, Snow, Earthquake, and Ice Loads (Table 1.5-1, ASCE 7-10)

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent a 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. 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 their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	III
Buildings and other structures designated as essential facilities. 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 exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released 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.	IV

[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 1.5.2 ASCE 7-10 that a release of the substances is commensurate with the risk associated with that Risk Category.

Table 2 2 Risk Category of Buildings for different codes

Country	Code	Risk category			
United States	ASCE 7-10 (Table 1.5-1)	I	II	III	IV
Vietnam	TCVN 9386:2012 (Appendixes E, F)	IV	III, II	I	Special
Thailand	~ ASCE 7-10	I	II	III	IV
Myanmar	MNBC 2016 (Table 3.1.2)	I	II	III	IV
Cambodia	~ ASCE 7-10 (Table 1.5-1)	I	II	III	IV
Indonesia	~ ASCE 7-10	I	II	III	IV
Philippines	NSCP 2015 (Table 103-1)	V	III, IV	II	I

Minimum design loads for structures shall incorporate the applicable importance factors given in *Table 1.5-2, ASCE 7-10.*

Table 2 3 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads [a]

Risk Category	Wind Importance Factor, I_w	Seismic Importance Factor, I_e
I	1.00	1.00
II	1.00	1.00
III	1.00	1.25
IV	1.00	1.50

Exposing categories [1]

Table 2 4 Exposure Categories

Exposure	Description
B	Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.
C	Open terrain with scattered obstructions having heights generally less than 30ft (9.1 m). This category includes flat open country and grasslands.
D	Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats, and unbroken ice.

For a site located in the transition zone between exposure categories, the category, resulting in the largest wind forces shall be used.

Table 2 5 Comparison of Exposure between TCVN2737 and ASCE7-10 Standard Codes

	Unobstructed areas	Rural areas	Urban areas
TCVN2737:1995	A	B	C
ASCE7-10	D	C	B

[1] ASCE 7-10, Section 26.7.3.

Basic wind speed $V_{50y,3s}$

Basic wind speed depends on every national standard. So we need to convert it into the value $V_{50y,3s}$ in ASCE 7-10 with design 3-second gust wind speeds (m/s) at 10m above ground for Exposure C. In this manual, we will summarize by below table.

Table 2 6 Wind return period for different codes

Country	Code	T years return period	Gust duration
United States	ASCE 7-10	50	3s
Viet Nam	TCVN 2737:1995	20	3s
Thailand	~ ASCE 7-10	50	1h
Myanmar	MNBC 2016 ~ ASCE 7-10	50	3s
Cambodia	~ ASCE 7-10	50	3s
Indonesia	~ ASCE 7-10	50	3s
Philippines	NSCP 2015 ~ ASCE 7-10	Search PHL map in NSCP 2015	

Example

Vietnam Location: converting 20 years return period to 50 years return period by following a formula:

$$\frac{V_T}{V_{50}} = 0.36 + 0.1 \times \ln(12T) \qquad \textit{(C26.5-2, ASCE 7-10)}$$

$$\frac{V_{20}}{V_{50}} = 0.36 + 0.1 \times \ln(12T) = 0.36 + 0.1 \times \ln(12 \times 20) = 0.908 \rightarrow V_{50} = \frac{V_{20}}{0.908}$$

Thailand Location: If converting gust duration from 1h to 3s.

See *Figure C26.5-1 (ASCE 7-10)*. Maximum speed average over ts to hourly wind speed

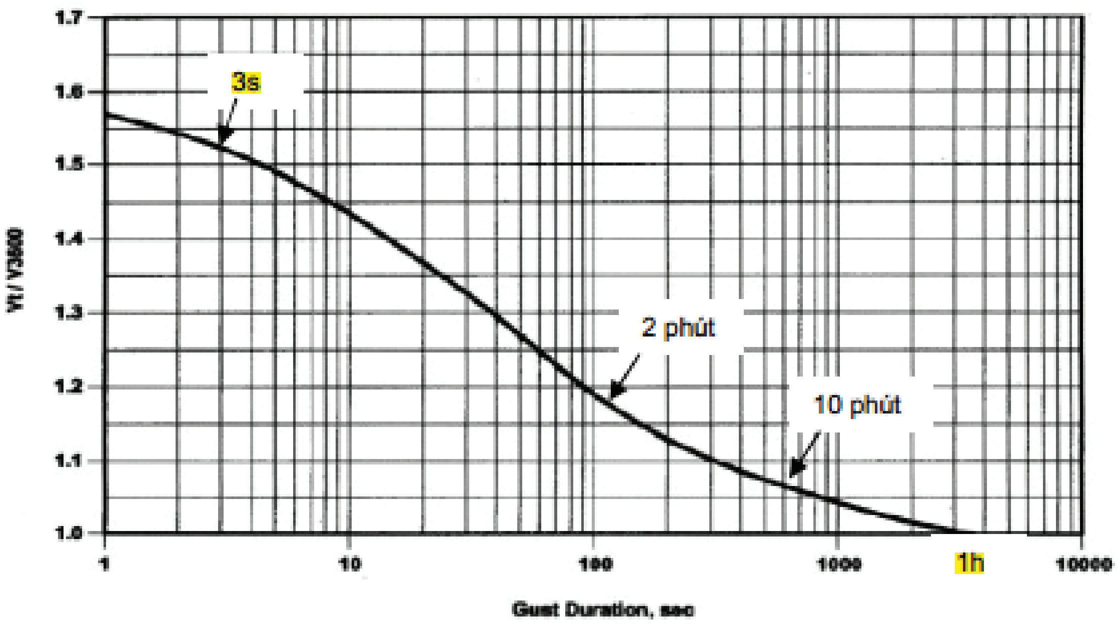


Figure 2.5 Converting gust duration diagram

According to the above diagram, we see $\frac{V_{3s}}{V_{1h}} = 1.52 \rightarrow V_{3s} = 1.52V_{1h}$

Design wind speed $V_{Ty,3s}$

Using the *Figure 26.5-1A, 1B, 1C ASCE 7-10* to determine the design wind speed, it depends on risk category of building

Table 2 7 Design wind returns period

Risk category of building	T year return period (ASCE 7-10)	Design T year return period	Gust duration
I	50	300	3s
II	50	700	3s
III, IV	50	1700	3s

Velocity pressure conversion factor:

$$\frac{V_{Ty,3s}}{V_{50\,y,3s}} = 0.36 + 0.1 \times \ln\left(12T\right) \qquad \textit{(C26.5-2, ASCE 7-10)}$$

→ Design wind speed: $V_{Ty,3s} = V_{50\,y,3s} \times \left[0.36 + 0.1 \times \ln\left(12T\right)\right]$

Velocity pressure exposure coefficient Kz

According to Table 27.3-1, ASCE 7-10, Velocity pressure exposure coefficient Kz depends on exposure and height of building above the ground.

Table 2 8 Velocity pressure exposure coefficients, Kh and Kz (Table 27.3-1, ASCE 7-10)

Height above ground level, z	Exposure		
(m)	B	C	D
0-4.6	0.57	0.85	1.03
6.1	0.62	0.90	1.08
7.6	0.66	0.94	1.12
9.1	0.70	0.98	1.16
12.2	0.76	1.04	1.22
15.2	0.81	1.09	1.27
18.0	0.85	1.13	1.31
21.3	0.89	1.17	1.34
24.4	0.93	1.21	1.38
27.4	0.96	1.24	1.40
30.5	0.99	1.26	1.43
36.6	1.04	1.31	1.48
42.7	1.09	1.36	1.52
48.8	1.13	1.39	1.55
54.9	1.17	1.43	1.58
61.0	1.20	1.46	1.61
76.2	1.28	1.53	1.68
91.4	1.35	1.59	1.73
106.7	1.41	1.64	1.78
121.9	1.47	1.69	1.82
137.2	1.52	1.73	1.86
152.4	1.56	1.77	1.89

NOTE

1. The velocity pressure exposure coefficient Kz may be determined from the following formula:

$$\text{For } 15\,ft. \le z < z_g, \quad K_z = 2.01\left(z/z_g\right)^{2/\alpha}$$

$$\text{For } z < 15\,ft., \quad K_z = 2.01\left(15/z_g\right)^{2/\alpha}$$

2. α and z_g are tabulated in Table 26.9.1 – ASCE 7-10.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposing categories are defined in Table 2 4.

Topographic factor Kzt

The topographic factor for the site K_{zt} is taken to be 1 in most cases. For more detail, see *Section 26.8.1 ASCE 7-10.*

Wind directionality factor Kd

Wind directionality factor K_d depends on type & shape of the building, see table below.

Table 2 9 Wind Directionality Factor, Kd (Table 26.6-1, ASCE 7-10)

Structure type	Directionality factor Kd
Buildings	
Main wind force resisting system	0.85
Components and cladding	0.85
Arched roofs	0.85
Chimneys, tanks, and similar structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid freestanding walls and solid freestanding and attached signs	0.85
Open signs and lattice framework	0.85
Trussed towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

Gust effect factor

The gust-effect factor for a rigid building or low-rise building is permitted to be taken as 0.85.
For flexible or dynamically sensitive building or high-rise building, calculate G value according to section 26.9 ASCE 7-10.

Enclosure classification

For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open

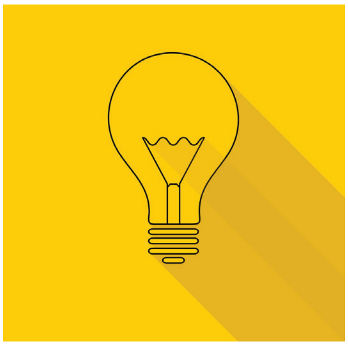
Enclosed Building	A building that does not comply with the requirements for open or partially enclosed buildings
Partially Enclosed Building	A building that complies with both of the following conditions: 1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10%. 2. The total area of openings in a wall that receives positive external pressure exceeds 4 ft^2 (0.37 m^2)
Open Building	A building having each wall at least 80% open. This condition is expressed for each wall by the equation $A_o > 0.8A_g$, where: A_o = total area of openings in a wall that receives positive external pressure. A_g = the gross area of that wall in which A_0 is identified.

Internal pressure coefficient (GCpi)

Internal pressure coefficients, (GC_{pi}), shall be determined from *Table 26.11-1 ASCE 7-10* based on building enclosure classifications.

Table 2 10 Internal pressure coefficients, GC_{pi} (Table 26.11-1, ASCE 7-10)

Enclosure classification	GC_{pi}
Open buildings	0.00
Partially Enclosed Buildings	+0.55
	-0.55
Enclosed Buildings	+0.18
	-0.18



NOTE:

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. The values of (GC_{pi}) shall be used with q_z or q_h as specified.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:

(i) A positive value of (GC_{pi}) applied to all internal surfaces
(ii) A negative value of (GC_{pi}) applied to all internal surfaces

2.2.5. Crane load (CR)

Crane loads are determined using the crane data available from the crane manufacturer. Crane data include wheel load, curb weight, crane weight, wheelbase, ends hook approach (used when two cranes operate in one aisle) and minimum vertical and horizontal clearances.
Refer CHAPTER 5. CRANE SYSTEMS DESIGN for more detail.

2.2.6. Earthquake

2.2.6.1 Seismic design concept

An effective seismic design generally includes:
1. Selecting an overall structural concept, including the layout of a lateral-force-resisting system that is appropriate to the anticipated level of ground shaking. This includes providing a redundant and continuous load path to ensure that a building responds as a unit when subjected to ground motion2. Determining code-prescribed forces and deformations generated by the ground motion, and distributing the forces vertically to the lateral-force-resisting system. The structural system, configuration, and site characteristics are all considered when determining these forces.
3. Analysis of the building of the combined effects of gravity and seismic loads to verify that adequate vertical and lateral strength and stiffness are achieved to satisfy the structural performance and acceptable deformation levels prescribed in the governing building code.
4. Providing details to assure that the structure has the sufficient inelastic deformability to undergo fairly large deformations when subjected to a major earthquake. Appropriately detailed members possess the necessary characteristics to dissipate energy by inelastic deformations.

2.2.6.2 Structural Response

If the base of a structure is suddenly moved, as in a seismic event, the upper part of the structure will not respond instantaneously but will lag because of the inertial resistance and flexibility of the structure. The resulting stresses and distortions in the building are the same as if the base of the structure was to remain stationary while time-varying horizontal forces are applied to the upper part of the building. These forces, called inertia forces, are equal to the product of the mass of the structure times acceleration, i.e., F = ma (the mass m is equal to weight divided by the acceleration of gravity, i.e., m = w/g). Because earthquake ground motion is three-dimensional (one vertical and two horizontal), the structure, in general, deforms in a three-dimensional manner. Generally, the inertia forces generated by the horizontal components of ground motion require greater consideration for seismic design since adequate resistance to vertical seismic loads is usually provided by the member capacities required for gravity load design. In the equivalent static procedure, the inertia forces are represented by equivalent static forces.

2.2.6.3 Load path

Buildings are generally composed of vertical and horizontal structural elements.
The vertical elements commonly used to transfer lateral forces on the ground are:

1) Shear walls;
2) Braced frames;
3) Moment-resisting frames.

The horizontal elements that distribute lateral forces to the vertical elements are:

1) Diaphragms, such as floor and roof slabs;
2) Horizontal bracing that transfers large shears from discontinuous walls or braces.

The seismic forces that are proportional to the mass of the building elements are considered to act as their centers of mass. All of the inertia forces originating from the masses on and off the structure must be transmitted to the lateral-force-resisting elements, and then to the base of the structure and into the ground.

A complete load path is a basic requirement for all buildings. There must be a complete lateral-force-resisting system that forms a continuous load path between the foundation, all diaphragm levels, and all portions of the building for proper seismic performance. The general load path is as follows. Seismic forces originating throughout the building, mostly in the heavier mass elements such as diaphragms, are delivered through connections to horizontal diaphragms; the diaphragms distribute these forces to vertical force-resisting elements such as shear walls and frames; the vertical elements transfer the forces into the foundation; and the foundation transfers the forces into the supporting soil.

If there is a discontinuity in the load path, the building is unable to resist seismic forces regardless of the strength of the elements. Interconnecting the elements needed to complete the load path is necessary to achieve good seismic performance. Examples of gaps in the load path would include a shear wall that does not extend to the foundation, a missing shear transfer connection between a diaphragm and vertical elements, a discontinuous chord at a diaphragm's notch, or a reentrant corner, or a missing collector.

A good way to remember this important design strategy is to ask yourself the question, "How does the inertia load get from here (meaning the point at which it is generated) to there (meaning the shear base of the structure, typically the foundations)?"

2.2.6.4 The design base shear according to UBC 97

The total design base shear in a given direction shall be determined following *section 1630.2 UBC 97:*

$$V = \frac{C_v \times I}{R \times T} \times W \ \text{(kN)}$$

Besides, V value must be satisfied below equation:

$$0.11C_a \times I \times W \leq V = \frac{C_v \times I}{R \times T} \times W \leq \frac{2.5C_a \times I}{R} \times W$$

For seismic zone 4, the total design base shear shall also not be less than the following:

$$V \geq \frac{0.8Z \times N_v \times I}{R} \times W$$

2.2.6.5 Seismic coefficient (Cv, Ca)

The seismic coefficient is determined by Table 2 11 and Table 2 12, it depends on soil profile type (Table 2 13) and seismic zone factor Z (Table 2 15).

Table 2 11 Seismic coefficients, C_v (Table 16-R, UBC 97)

Soil profile type	Seismic zone factor, Z				
	Z=0.075	Z=0.15	Z=0.2	Z=0.3	Z=0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	0.96Nv
S_F	Refer to site-specific Geotechnical investigation and dynamic site response analysis to determine C_v				

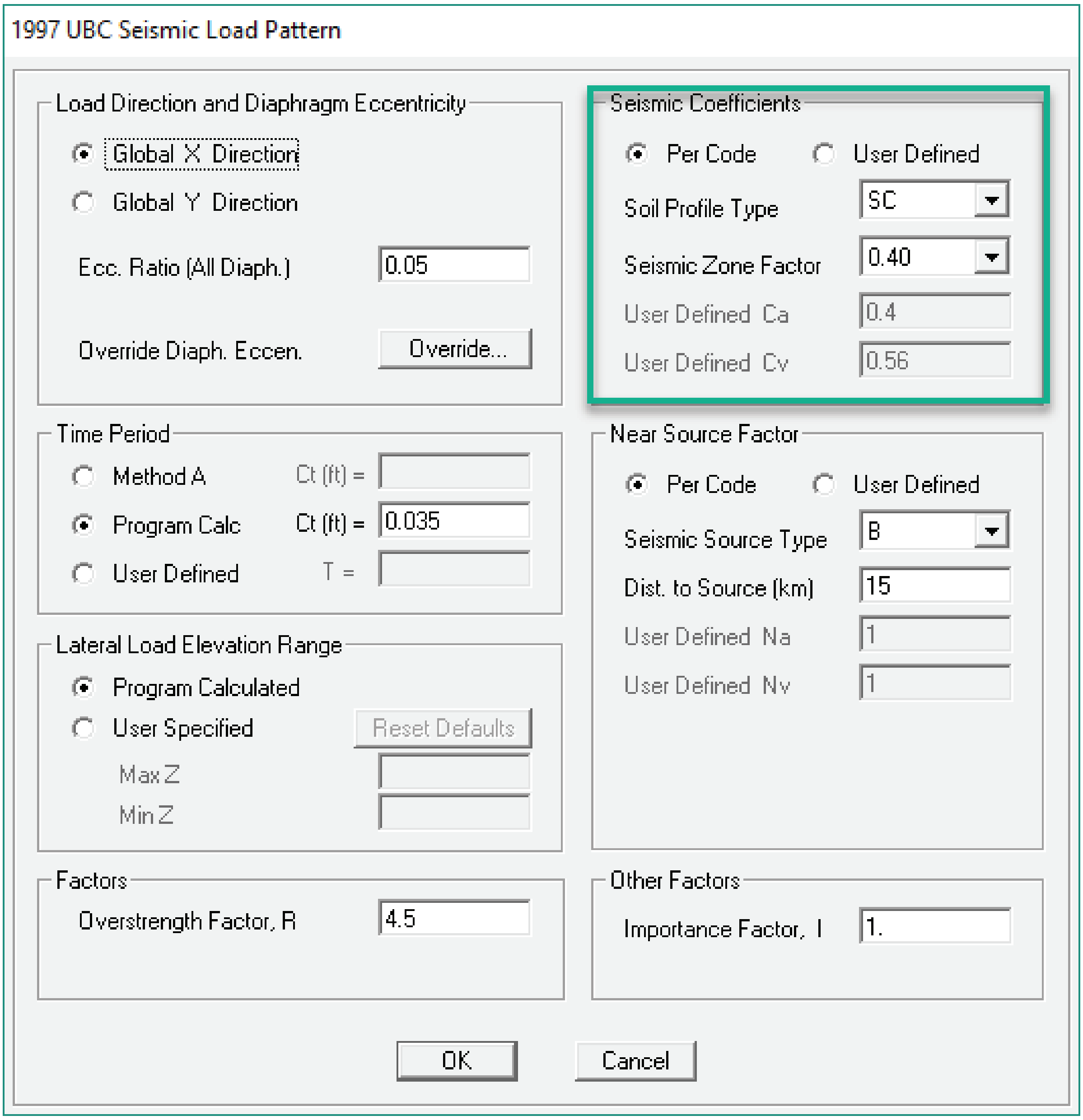
Table 2 12 Seismic coefficients, C_a (Table 16-Q, UBC 97)

Soil profile type	Seismic zone factor, Z				
	Z=0.075	Z=0.15	Z=0.2	Z=0.3	Z=0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.09	0.18	0.24	0.33	$0.40N_v$
S_D	0.12	0.22	0.28	0.36	$0.44N_v$
S_E	0.19	0.30	0.34	0.36	0.36Nv
S_F	Refer to site-specific Geotechnical investigation and dynamic site response analysis to determine C_v				

The seismic coefficients C_v and C_a, given in *Tables 16-R and 16-Q UBC 97*, are site-dependent ground motion coefficients that define the seismic response throughout the spectral range. They are measures of expected ground acceleration at a site.

For a given earthquake, a building on soft soil types such as S_C or S_D experiences a greater force than if the same building were located on a rock, type S_A or S_B. This is addressed in the UBC through the C_a and C_v coefficients, which are calibrated to soil type S_B with a value of unity. Instead of a single coefficient, two coefficients, C_a and C_v, are used to distinguish the response characteristics of short-period and long-period buildings. Long period buildings are more affected by soft soils than short-period buildings.

In SAP software, a designer needs to choose Soil profile type & Seismic zone factor, the program will calculate C_a, C_v automatically according to UBC 97.



2.2.6.6 Soil profile types

Table 2 13 Soil profile types (Table 16-J, UBC 97)

Soil profile type	Soil profile name/Generic description
S_A	Hard rock
S_B	Rock
S_C	Very dense soil and soft rock
S_D	Stiff soil profile
S_E	Soft soil profile
S_F	Soil requiring site – specific evaluation. See section 1629.3.1 UBC 97

NOTE
Use **Soil Types SD** if there is not any requirement from customers.

Table 2 14 Soil profile types for different codes

Country	Code	Soil profile types					
United States	UBC 97 (Table 16-J)	S_A	S_B	S_C	S_D	S_E	S_F
Vietnam	TCVN 9386:2012 (Table 3.1)	-	A	B	C	D	E,S1,S2
Thailand	UBC 97 (Table 16-J)	S_A	S_B	S_C	S_D	S_E	S_F
Myanmar	MNBC 2016 (Table 3.4.2)	A	B	C	D	E	F
Cambodia	UBC 97 (Table 16-J)	S_A	S_B	S_C	S_D	S_E	S_F
Indonesia	UBC 97 (Table 16-J)	S_A	S_B	S_C	S_D	S_E	S_F
Philippines	NSCP 2015	SA	SB	SC	SD	SE	SF

2.2.6.7 Seismic zone & Seismic zone factor (Z)

Table 2 15 Seismic zone factor, Z (Table 16-I, UBC 97)

Zone	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

Note: The zone shall be determined from the seismic zone map in Figure 16-2 UBC 97
The map accounts for the geographical variations in the expected levels of earthquake ground shaking and gives the estimated peak horizontal acceleration on a rock having a 10% chance of being exceeded in a 50-year period (or 500-year return period).
The value of the seismic zone coefficient Z can be considered the peak ground acceleration of the percentage of gravity.
It means, the peak ground acceleration: $a_g = Z \times g$
For example, Z = 0.4 indicates a peak ground acceleration of $0.4g$ equal to 40% of gravity.
For the buildings are not located in the United States, refer to *Appendix Chapter 16 UBC 97* to determine seismic zone.

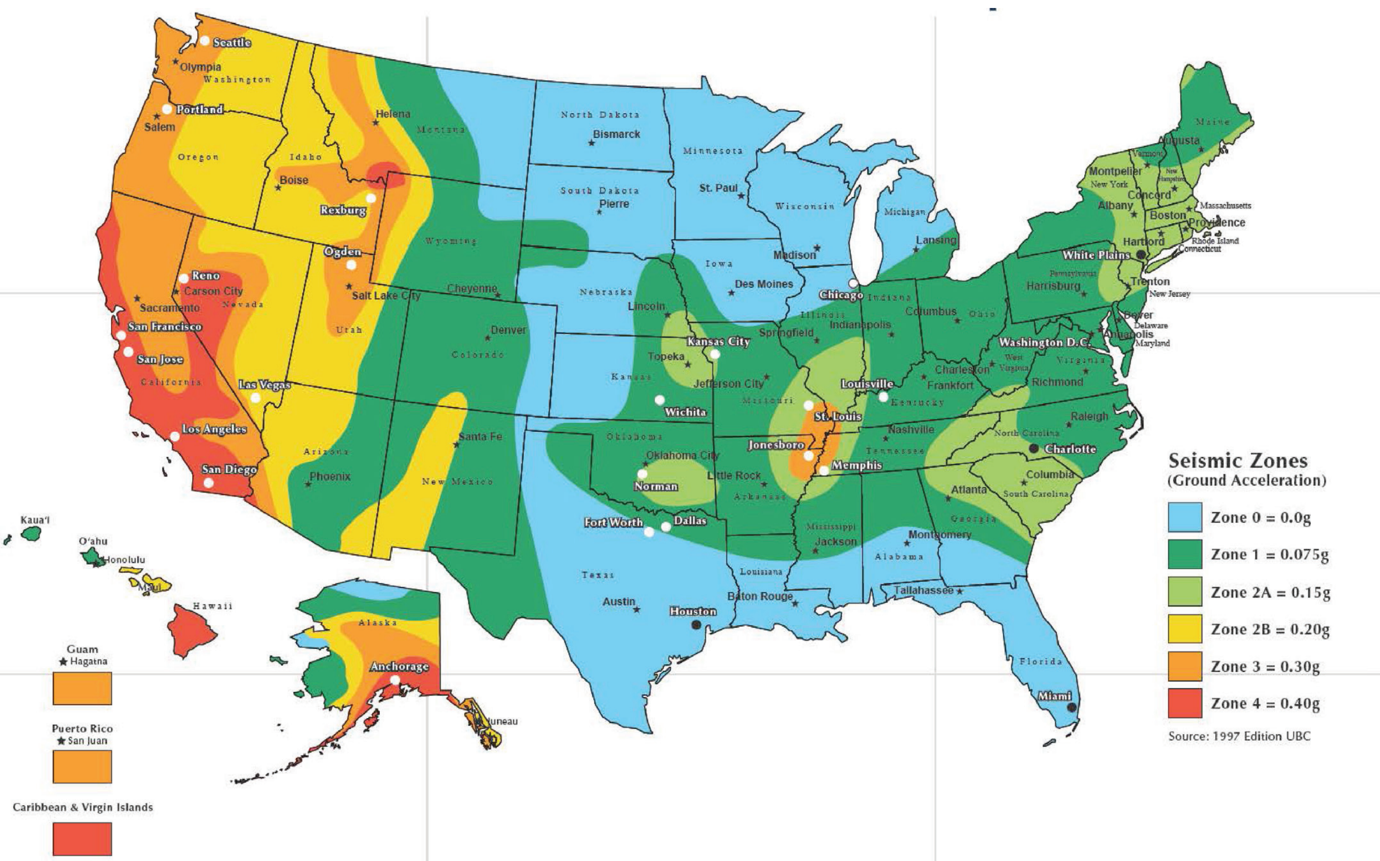


Figure 2.6 The United States Seismic zones map with 500-year return period

Table 2 16 Seismic zones for different codes

Country	Code	Seismic zones & peak ground acceleration				
United States	UBC 97 (Table 16-I)	1	2A	2B	3	4
	a_g =	0.075g	0.15g	0.2g	0.3g	0.4g
Vietnam	TCVN 9386:2012	1	2A	2B	-	-
Thailand	(Appendix G, H) ~ UBC 97	1	2A	2B	3	-
Myanmar	MNBC 2016	I	II	III	IV	V
Cambodia	(Fig. 3.4.1.5) ~ UBC 97	-	-	-	-	-
Indonesia	~ UBC 97	1	2A	2B	3	4
Philippines	NSCP 2015 (Table 208-3)	-	-	2B	-	4

Vietnam

Use *Appendix H - TCVN 9386:2012* to determine ground acceleration for every area in Vietnam.
In this standard, the peak ground acceleration was surveyed considered on rock, 500-year return period.

Thailand

According to Section 1653 UBC 97, we have a seismic zone for some areas in Thailand.

Table 2 17 Seismic zone in Thailand

Area	Zone
Bangkok	1
Chiang Mai	2A
Udorn	1

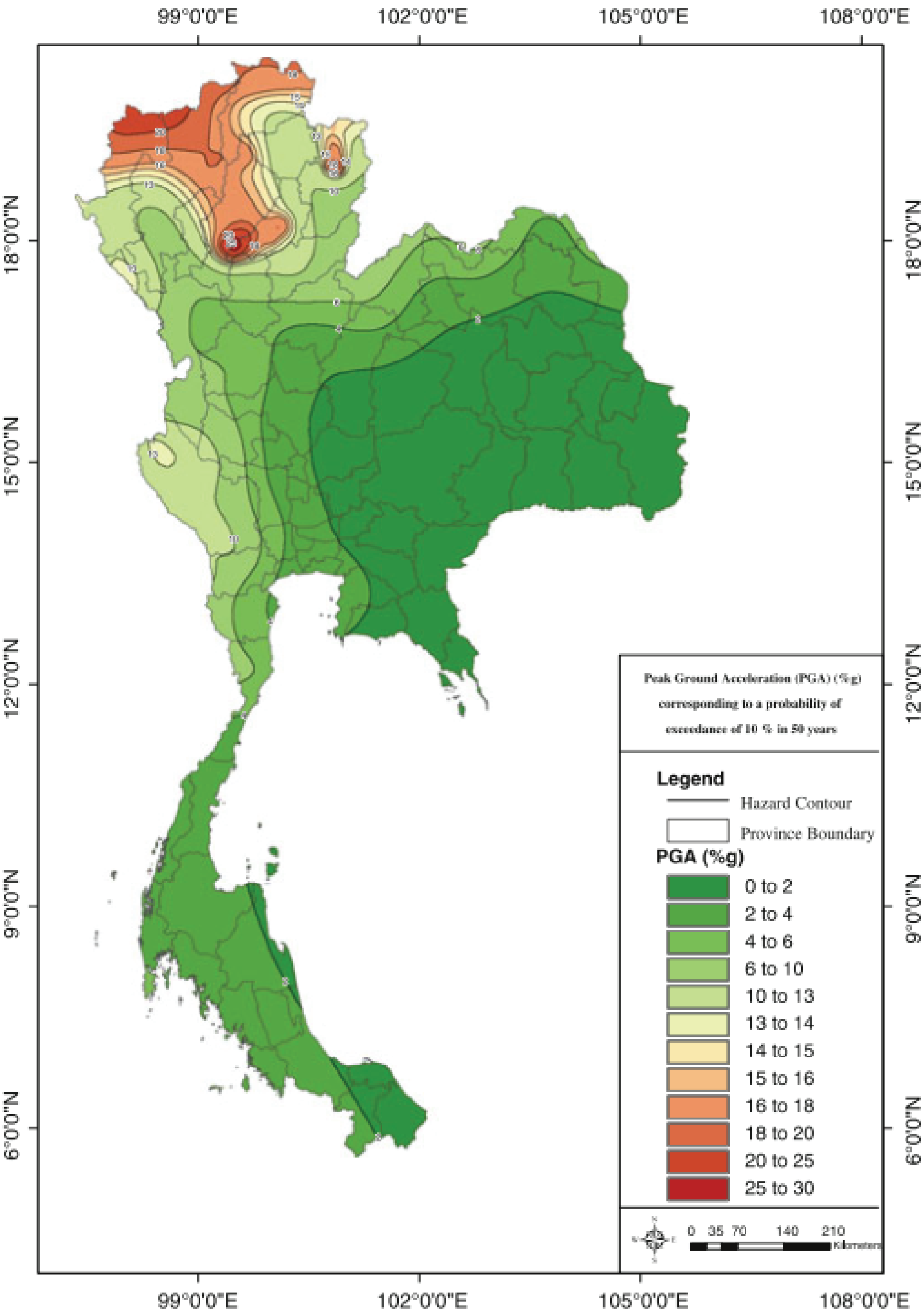
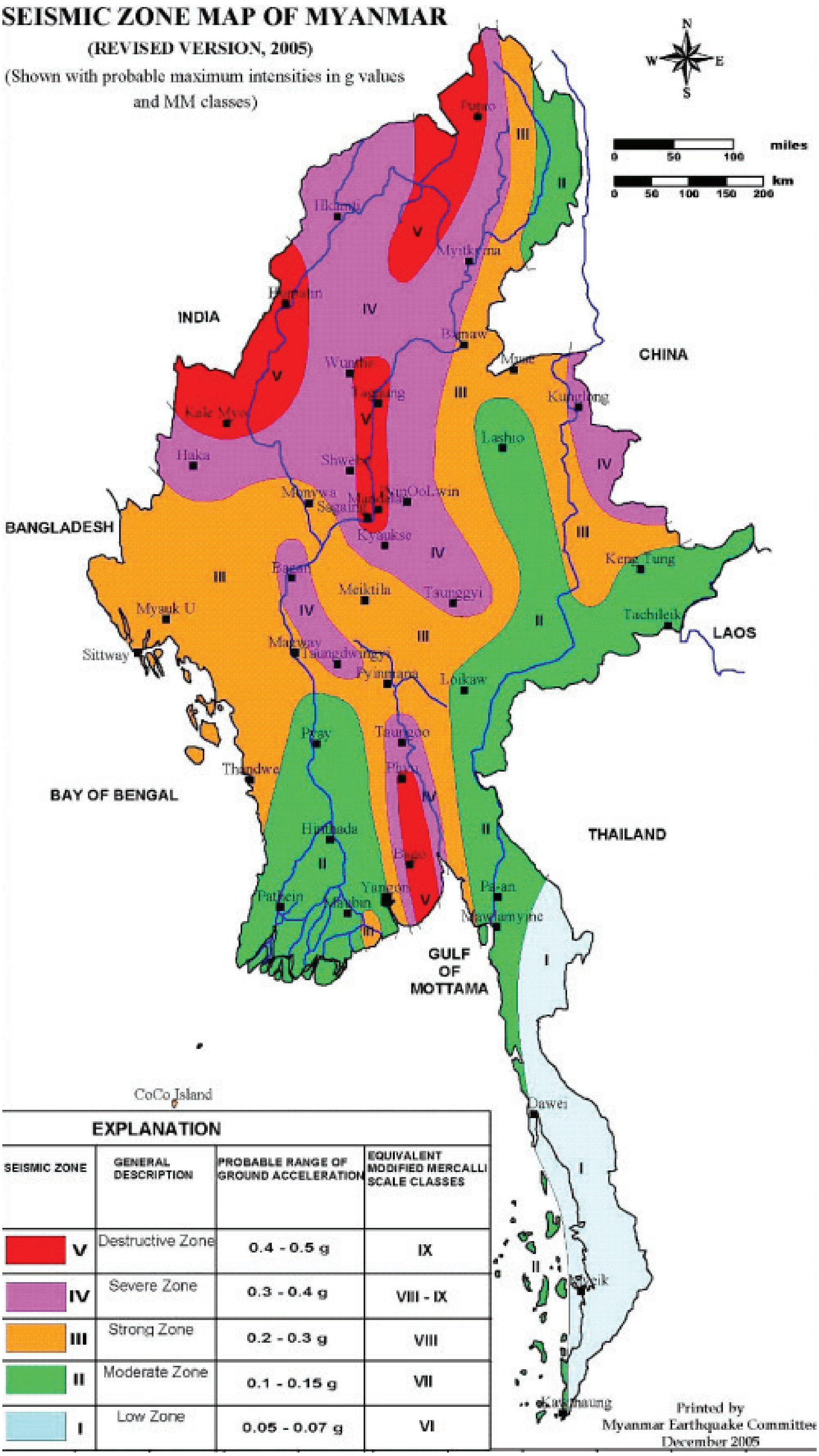


Fig. 8 Thailand hazard maps for PGA corresponding to a probability of exceedance of 10% in 50 years

Figure 2.7 Thailand Seismic zones map with 500-year return period

Myanmar

The seismic zone of Myanmar is divided into I, II, III, IV, V corresponding to seismic zone 2B, 3, 4 of UBC 97.



EXPLANATION

SEISMIC ZONE	GENERAL DESCRIPTION	PROBABLE RANGE OF GROUND ACCELERATION	EQUIVALENT MODIFIED MERCALLI SCALE CLASSES
V	Destructive Zone	0.4 - 0.5 g	IX
IV	Severe Zone	0.3 - 0.4 g	VIII - IX
III	Strong Zone	0.2 - 0.3 g	VIII
II	Moderate Zone	0.1 - 0.15 g	VII
I	Low Zone	0.05 - 0.07 g	VI

Figure 2.8 Myanmar Seismic zones map with 500-year return period (Figure 3.4.1.5 MNBC 2016)

Philippines

The Philippines archipelago is divided into 2 seismic zones only. Zone 2 covers the provinces of Palawan (except Busuanga), Sulu and Tawi-Tawi while the rest of the country is under zone 4.

Table 2 18 Seismic zone factor, Z in Philippines (Table 208-3, NSCP 2015)

Zone	2	4
Z	0.20	0.40

Figure 2.9 Philippines Seismic zones map with 500-year return period (Figure 208-1 NSCP 2015)

Cambodia

Use UBC 97 to design a building in Cambodia. Almost buildings no need to design earthquake.

Indonesia

According to Section 1653 UBC 97, we have a seismic zone for some areas in Indonesia.

Table 2 19 Seismic zone in Indonesia

Area	Zone
Bandung	4
Jakarta	4
Medan	3
Surabaya	4

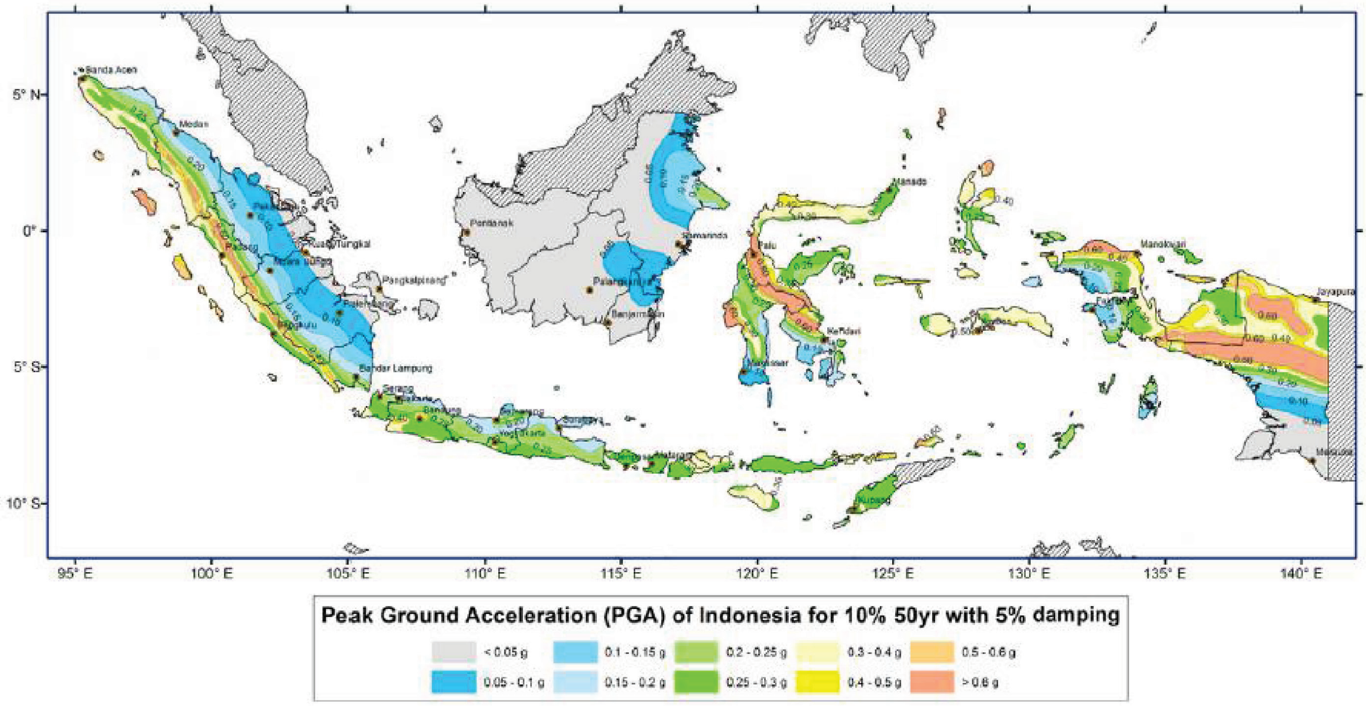


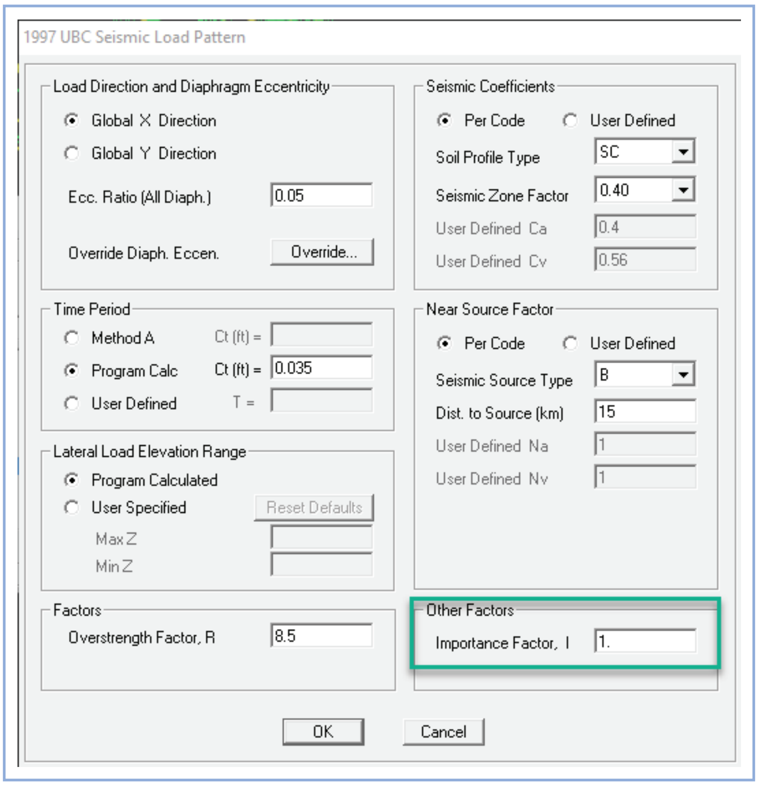
Figure 2.10 Indonesia Seismic zones map with 500-year return period

2.2.6.8 Important factor (I)

Table 2 20 Occupancy category (Table 16-K, UBC 97)

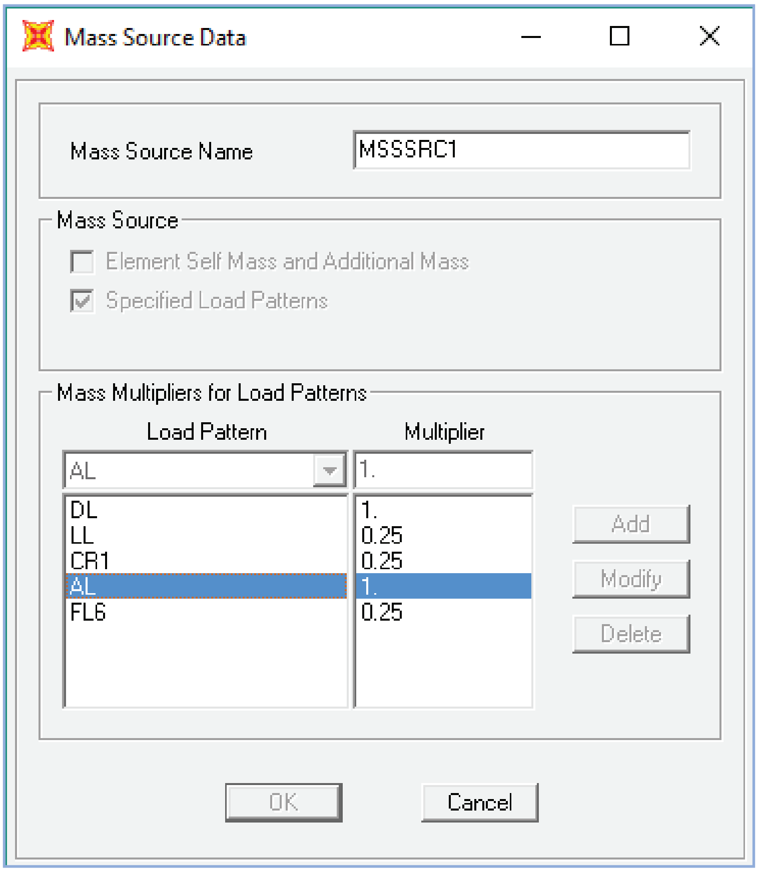
Occupancy category	Occupancy or functions of structure	Seismic importance factor, I	Seismic importance factor, Ip1	Wind importance factor, Iw
1. Essential facilities 2	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facility Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Non-building structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures3	Group A, Divisions 1, 2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more residents incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation	1.00	1.00	1.00
4. Standard occupancy structures3	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

For steel building, we usually use important factor $I = 1$.



2.2.6.9 The total seismic dead load (W)

According to *section 1630.1.1 UBC 97*, we can determine total seismic dead load W through mass source in SAP.



W = Mass source = DL + AL + 0.25 (LL + FL + CR) (kN)

NOTE

Load Pattern FL is fully applied floor load.

Each Crane will contribute one Crane Load Pattern.

2.2.6.10 Numerical coefficient representative of the inherent over strength and global ductility capacity of lateral force-resisting systems (R)

The coefficient R shown in *Table 16-N UBC 97* is a measure of ductility and over strength of a structural system, based primarily on performance of similar systems in past earthquakes.

A higher value of R has the effect of reducing the design base shear. For example, for a steel special moment-resisting frame, the factor has a value of 8.5, whereas for ordinary moment-resisting frame, the value is 4.5. This reflects the fact that a special moment-resisting frame performs better during an earthquake.

Table 2 21 Structural systems (Table 16-N UBC 97)

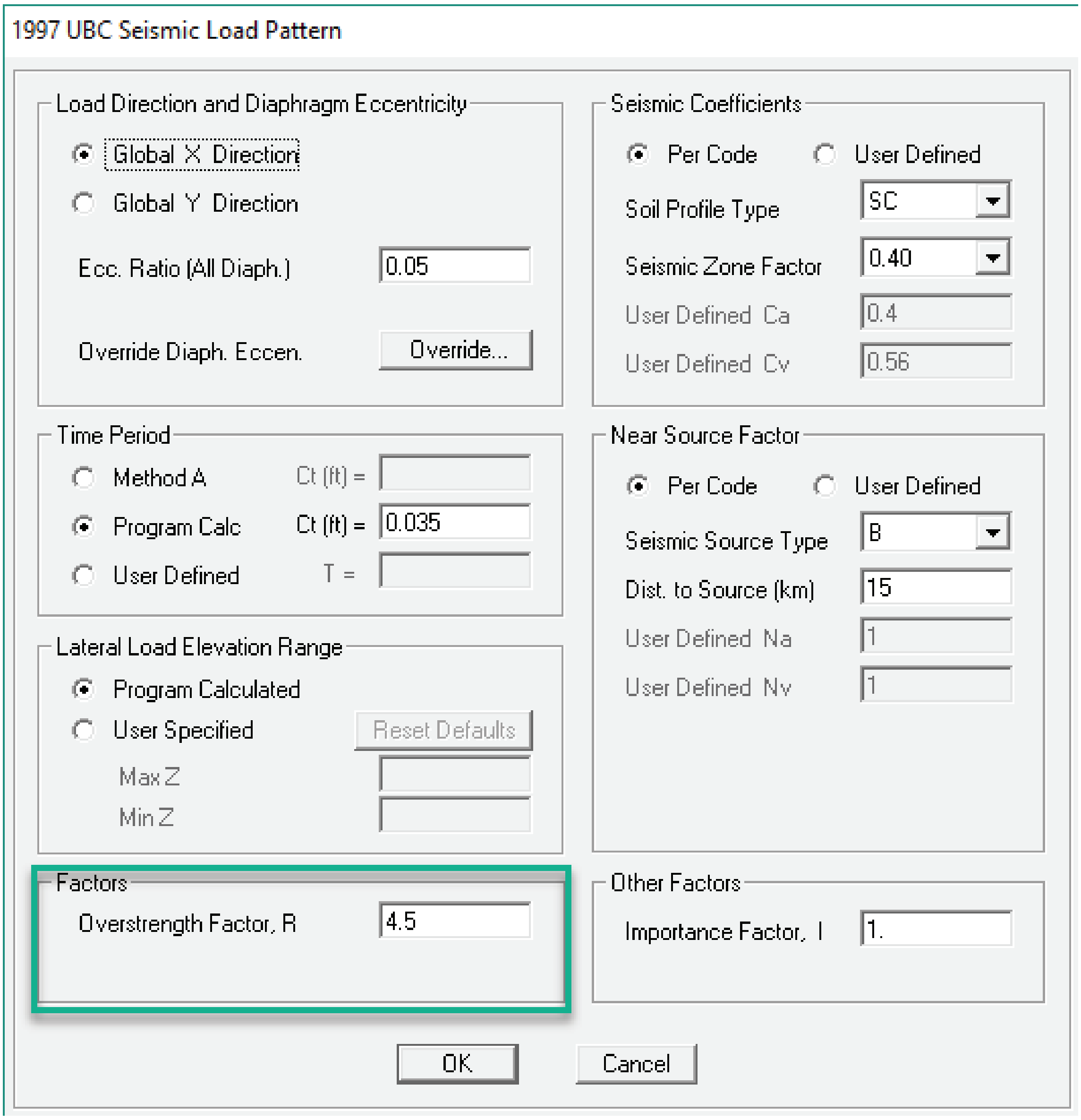
Basic structural system	Lateral force resisting system description	R	Ω_0	Height limit for seismic zones 3 and 4 (feet) x 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels a. Wood structural panel walls for structures three stories or less b. All other light-framed walls 2. Shear walls a. Concrete b. Masonry 3. Light steel-framed bearing walls with tension-only bracing 4. Braced frames where bracing carries gravity load a. Steel b. Concrete c. Heavy timber	5.5 4.5 4.5 4.5 2.8 4.4 2.8 2.8	2.8 2.8 2.8 2.8 2.2 2.2 2.2 2.2	65 65 160 160 65 160 - 65
2. Building frame system	1. Steel eccentrically braced frame (EBF) 2. Light-framed walls with shear panels a. Wood structural panel walls for structures three stories or less b. All other light-framed walls 3. Shear walls a. Concrete b. Masonry 4. Ordinary braced frames a. Steel b. Concrete c. Heavy timber 5. Special concentrically braced frames a. Steel	7.0 6.5 5.0 5.5 5.5 5.6 5.6 5.6 6.4	2.8 2.8 2.8 2.8 2.8 2.2 2.2 2.2 2.2	240 65 65 240 160 160 - 65 240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF) a. Steel b. Concrete 2. Masonry moment-resisting wall frame (MMRWF) 3. Concrete intermediate moment-resisting frame (IMRF) 4. Ordinary moment-resisting frame (OMRF) a. Steel b. Concrete 5. Special truss moment frames of steel (STMF)	8.5 8.5 6.5 5.5 4.5 3.5 6.5	2.8 2.8 2.8 2.8 2.8 2.8 2.8	N.L. N.L. 160 — 160 — 240
4. Dual systems	1. Shear walls a. Concrete with SMRF	8.5	2.8	N.L.

Basic structural system	Lateral force resisting system description	R	Ω_0	Height limit for seismic zones 3 and 4 (feet) x 304.8 for mm
	b. Concrete with steel OMRF	4.2	2.8	160
	c. Concrete with concrete IMRF	6.5	2.8	160
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF	4.2	2.8	160
	f. Masonry with concrete IMRF	4.2	2.8	-
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF	4.2	2.8	160
	3. Ordinary braced frames			
	a. Steel with steel SMRF	6.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
	c. Concrete with concrete SMRF	6.5	2.8	-
	d. Concrete with concrete IMRF	4.2	2.8	-
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35
6. Shear wall-frame interaction systems	1. Concrete	5.5	2.8	160
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2 UBC 97	-	-	-

Table 2 22 R and factors for nonbuilding structures (Table 16-P UBC 97)

Structure type	R	Ω_0
1. Vessels, including tanks and pressurized spheres, on braced or unbraced legs.	2.2	2.0
2. Cast-in-place concrete silos and chimneys having walls continuous to the foundations.	3.6	2.0
3. Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported	2.9	2.0
vertical vessels.		
4. Trussed towers (freestanding or guyed), guyed stacks and chimneys.	2.9	2.0
5. Cantilevered column-type structures.	2.2	2.0
6. Cooling towers.	3.6	2.0
7. Bins and hoppers on braced or unbraced legs.	2.9	2.0
8. Storage racks	3.6	2.0
9. Signs and billboards.	3.6	2.0
10. Amusement structures and monuments.	2.2	2.0
11. All other self-supporting structures not otherwise covered.	2.9	2.0

For steel building, we usually use R=4.5.


1997 UBC Seismic Load Pattern
Load Direction and Diaphragm Eccentricity
Global X Direction
Global Y Direction
Ecc. Ratio (All Diaph.)
0.05
Override Diaph. Eccen.
Override...
Seismic Coefficients
Per Code
User Defined
Soil Profile Type
SC
Seismic Zone Factor
0.40
User Defined Ca
0.4
User Defined Cv
0.56
Time Period
Method A
Ct (ft) =
Program Calc
Ct (ft) =
0.035
User Defined
T =
Near Source Factor
Per Code
User Defined
Seismic Source Type
B
Dist. to Source (km)
15
User Defined Na
1
User Defined Nv
1
Lateral Load Elevation Range
Program Calculated
User Specified
Reset Defaults
Max Z
Min Z
Factors
Overstrength Factor, R
4.5
Other Factors
Importance Factor, I
1.
OK
Cancel

2.2.6.11 Structure period (T)

Structure period or elastic fundamental period of vibration, of the structure in the direction under consideration.

Method A

For all buildings, the value T may be determined from *section 1630.2.2 UBC 97*

$$T = T_A = C_t (h_n)^{3/4} \text{ (s)}$$

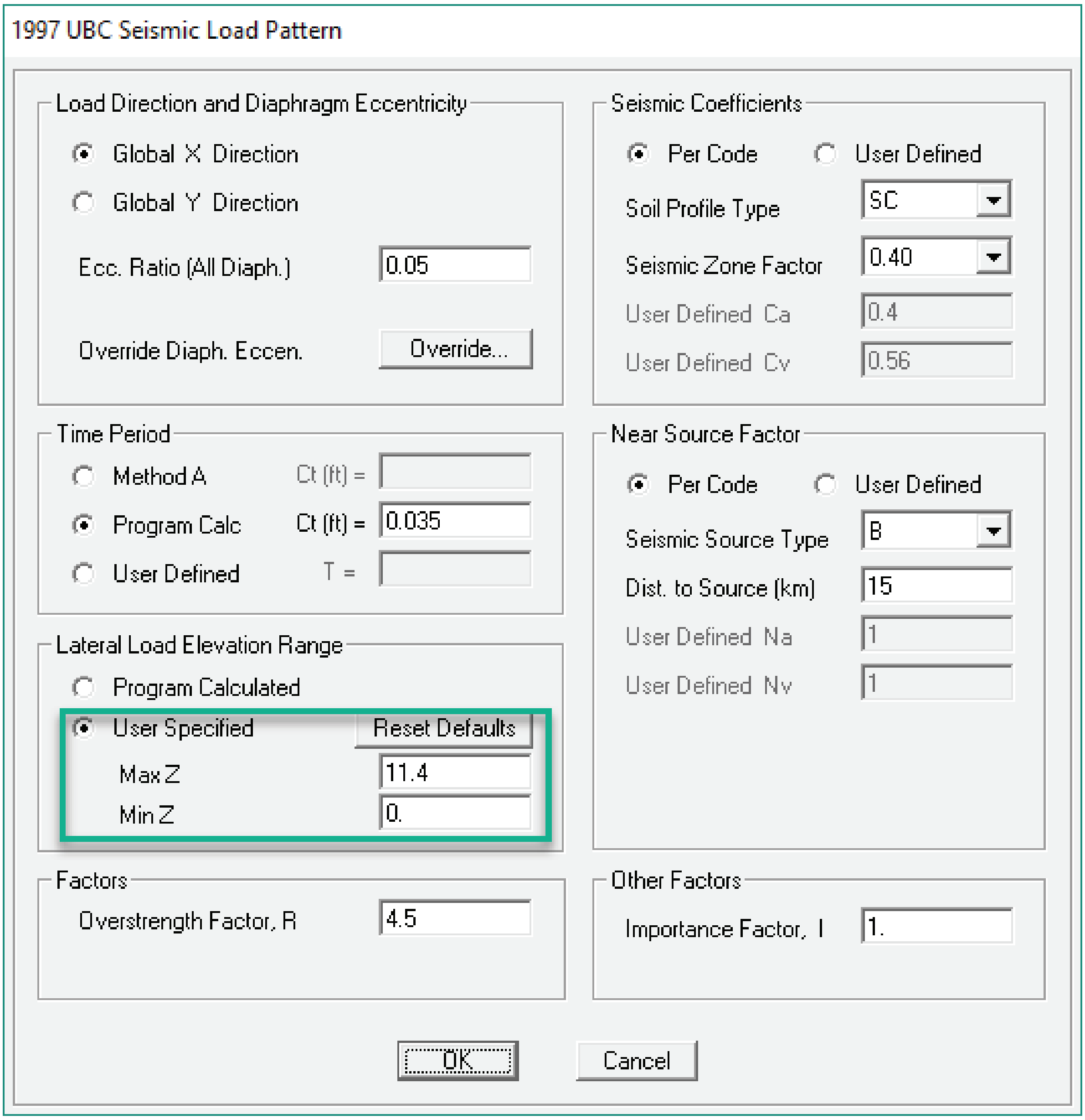
Where:

- C_t = the structural coefficient depends on the material of the frame, is always input in English unit in SAP software.

- $C_t = 0.035(0.0853)$ for steel moment-resisting frames.
- $C_t = 0.030(0.0731)$ for reinforced concrete moment-resisting frames and eccentrically braced frames.
- $C_t = 0.020(0.0488)$ for all other buildings.

- h_n = height above the base to level that is uppermost in the main portion of the structure. In SAP software, it's measured from the elevation of the specified bottom story/minimum elevation level to the (top of the) specified top story/maximum elevation level. (m)

The designer can modify h_n value by choosing an option "User specified" in Lateral load elevation range & fill in Min Z/ Max Z.



Method B

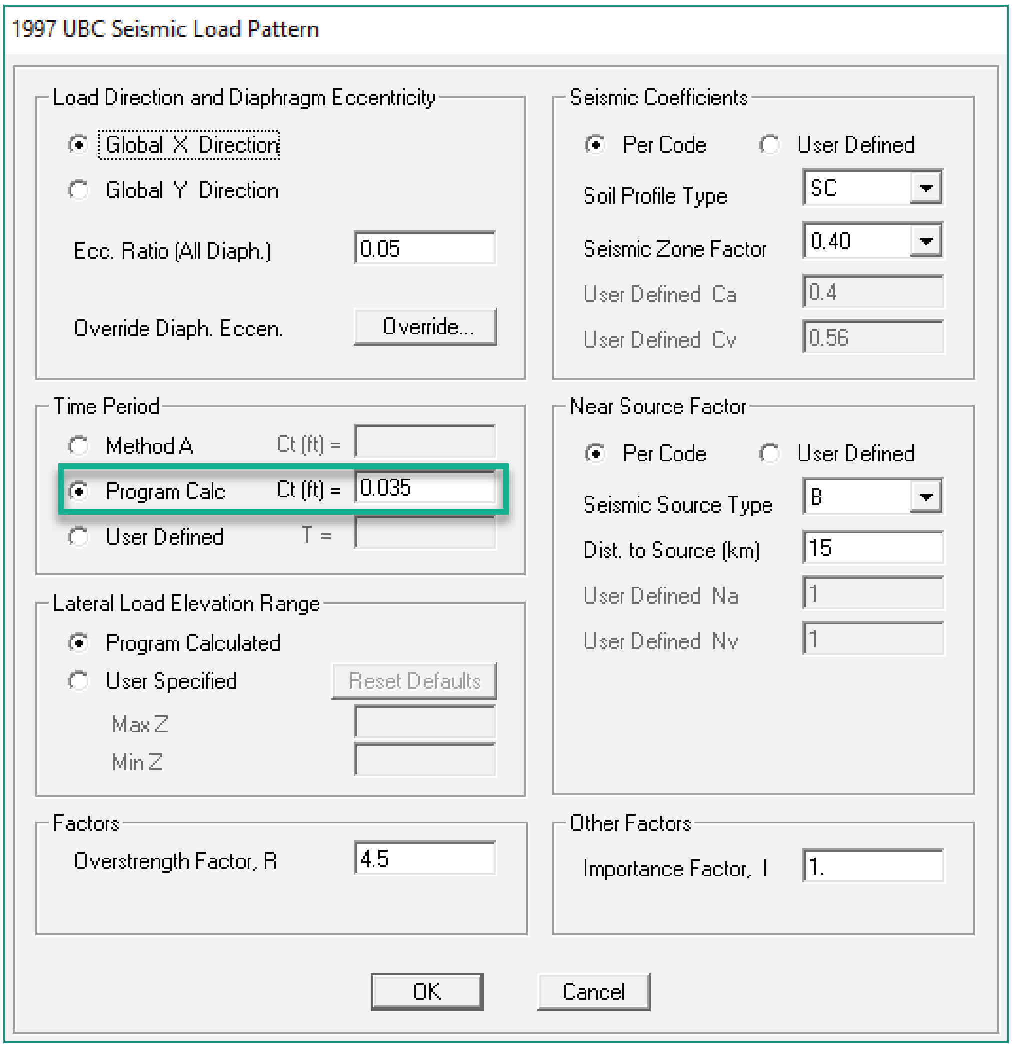
The fundamental period T may be calculated using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of *Section 1630.1.2 UBC 97*. The fundamental period T may be computed by using the following formula:

$$T_B = 2\pi\sqrt{\frac{m}{k}} = 2\pi\sqrt{\left(\sum_{i=1}^{n} \mathrm{w}_i\delta_i^2\right) \div \left(g\sum_{i=1}^{n} \mathrm{f}_i\delta_i\right)} \quad \text{(s)}$$

Where:

- f_i = lateral force at level i (kN)
- δ_i = horizontal displacement at Level i relative to the base due to applied lateral force f (m)
- g = acceleration due to gravity, $g = 9.81\left(m/s^2\right)$
- w_i = that portion of the total seismic dead load assigned to level i. (kN)

SAP software use this method to calculate T if designer choose "Program calc" in Time period.



- If the seismic zone is zone 4 then:

- If $T_B \le 1.3T_A \rightarrow T = T_B$
- If $T_B > 1.3T_A \rightarrow T = T_A$

- If the seismic zone is zone 1, 2A, 2B, 3 then:

- If $T_B > 1.4T_A \rightarrow T = T_A$
- If $T_B > 1.4T_A \rightarrow T = T_A$

2.2.6.12 Near-source factor (Na, Nv)

For seismic zone 4 (Z=0.4), we need near-source factor N_a, N_v to determine seismic coefficient C_a, C_v corresponding. They depend on seismic source type and the distance from the site to seismic source.

Table 2 23 Near-source factor Na (Table 16-S UBC 97)

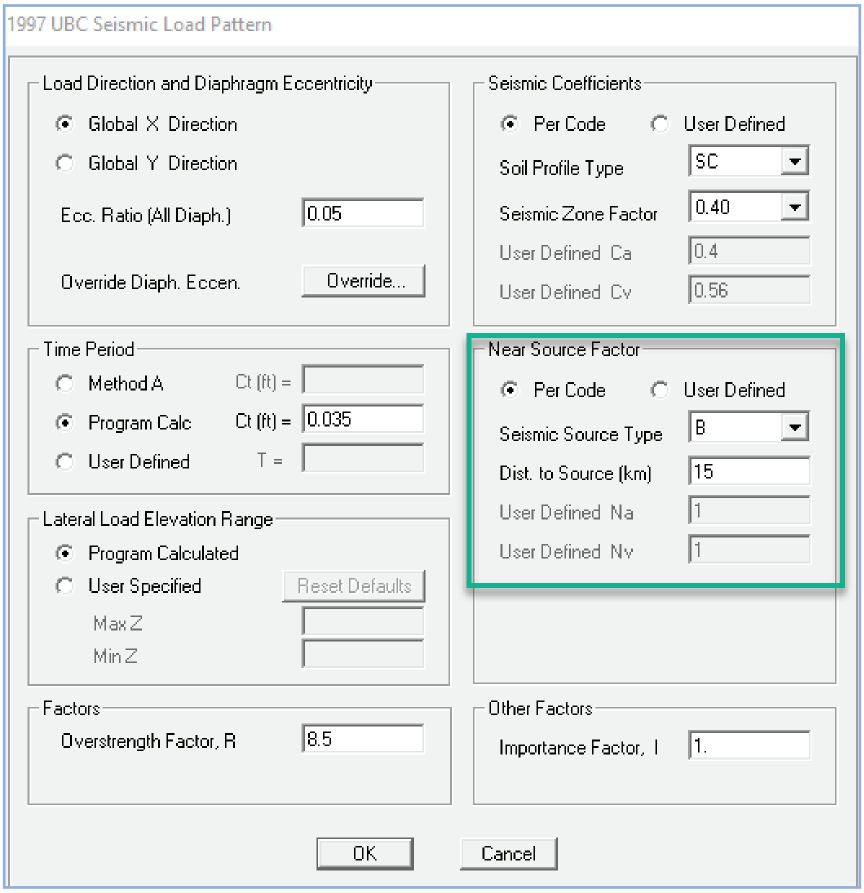
Seismic source type	Closest distance to known seismic source		
	≤ 2 km	5 km	≥ 10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

Table 2 24 Near-source factor NV (Table 16-T UBC 97)

Seismic source type	Closest distance to known seismic source			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

The purpose of N_a and N_v is to increase the soil-modified ground motion parameters, C_a and C_v, when there are active faults capable of generating large-magnitude earthquakes within 15 km of a seismic zone 4 site. So that $N_a, N_v \geq 1$.

In SAP software, a designer needs to choose "Seismic source type" & "Dist. to source", the program will be calculated N_a, N_v automatically according to UBC 97.



2.2.6.13 Seismic source types

Table 2 25 Seismic source type (Table 16-U UBC 97)

Seismic source type	Seismic source description	Seismic source definition	
		Maximum moment magnitude, M	Slip rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than types A and C	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$

The seismic source types labeled A, B, or C *(Table 16-U UBC 97)* is used to identify earthquake potential and activity of faults in the immediate vicinity of the structure.

They are defined in terms of the slip rate of the fault and the maximum magnitude of earthquake that may be generated at the fault. The highest seismic risk is posed by seismic source type A, which is defined by a maximum moment magnitude of 7.0 or greater and a slip rate of 5 mm/year or greater.

Moment magnitude (M) was introduced in 1979 by Hanks and Kanamori and has since become the most commonly used method of describing the size of a microseism. Moment magnitude measures the size of events in terms of how much energy is released.

The slip rate is how fast the two sides of a fault are slipping relative to one another, as determined from geodetic measurements, from offset man-made structures, or from offset geologic features whose age can be estimated. It is measured parallel to the predominant slip direction or estimated from the vertical or horizontal offset of geologic markers.

2.3. Load Combinations

Based on design method (LRFD or ASD), ASCE 7-10 define load combination coefficients for each method. BMB&A uses allowable stress design (ASD) method, see *section 2.4.1 ASCE 7-10*.

2.3.1. For Frame Structure

1. Dead Load (DL)
2. Dead Load (DL) + Live Load (LL)
3. Dead Load (DL) + Live Load (Floor/Crane)
4. Dead Load (DL) + 0.75 Live Load (LL) + 0.75 Live Load (Floor/Crane)
5. Dead Load (DL) + [0.6 Wind Load (WL) or 0.7 Earthquake load (EL)]

6a. Dead Load (DL) + 0.75 Live Load (LL) + 0.75 Live Load (Floor/Crane) + 0.45 Wind Load (WL)

6b. Dead Load (DL) + 0.75 Live Load (LL) + 0.525 Earthquake load (EL)

7. 0.6 Dead Load (DL) + 0.6 Wind Load (WL)
8. 0.6 Dead Load (DL) + 0.7 Earthquake load (EL)

2.3.2. For Cold-Formed Section

1. Dead Load (DL) + Roof Live Load (LLr)
2. Dead Load (DL) + 0.6 Wind Load (WL)
3. 0.6 Dead Load (DL) + 0.6 Wind Load (WL)

All of these are basic combination. For more loads (snow load, rain load, flood load…, etc.) see section 2.4 ASCE 7-10.

2.4. Serviceability Consideration

Table 2 26 Deflection Limitations

CONTRUCTION	LOAD & DEFORMALOAD		REMARK
1/ Rafter (Vertical deflection) [2]	**LL**	**DL + LL**	
a/ Supporting plaster ceiling	L/360	L/240	
b/ Supporting non-plaster ceiling	L/240	L/180	
c/ Not supporting ceiling	L/180	L/120	
2/ Column (Horizontal deflection) [3]	**DL + WL 10 yr.**	**H/60**	10 Yr. Wind
II/ Purlin1	**DL + LL**	**DL + WL 10 yr.**	10 Yr. Wind
1/ Purlin (Vertical deflection)	L/150	L/150	
2/ Girt (Horizontal deflection)	L/120	L/120	
III/ Floor members (Vertical deflection)	**LL**	**DL + LL**	
Beam, Joist, Decking, Checker Plate	L/360	L/240	
IV/ Top Running Cranes [4]	**DL + CR**		
1/ Runway Beam (Vertical deflection)	L/600		
2/ Runway Beam (Horizontal deflection)	L/400		
V/ Crane Bracket (Horizontal deflection) [3]	**DL + CR or WL 10 yr.**		Crane lateral
1/ Cab or Radio - Operator cranes	H/240 or ≤ 5.08cm		or 10 Yr.
2/ Pendant - Operator cranes	H/100		Wind

[2] International Building Code - IBC 2012, Table 1604.3, page 335.

[3] Metal Building Systems Manual 2012, Table 3.3, page 331.

[4] Metal Building Systems Manual 2012, Table 3.5, page 333.

CHAPTER 3. PRE-ENGINEERED BUILDING (PEB) SYSTEM

Planning of the pre-engineered buildings (low rise metal buildings) and arranging different building components is a very important step for the designer before proceeding with the design of each component. The Following building configurations are significantly affecting the building Stability and Cost:

1. Main Frame configuration (orientation, type, roof slope, eave height)
2. Roof purlins spacing
3. Wall girts (connection & spacing)
4. End wall system
5. Expansion joints
6. Bay spacing
7. Bracing systems arrangement
8. Mezzanine floor beams/columns (orientation & spacing)
9. Crane systems

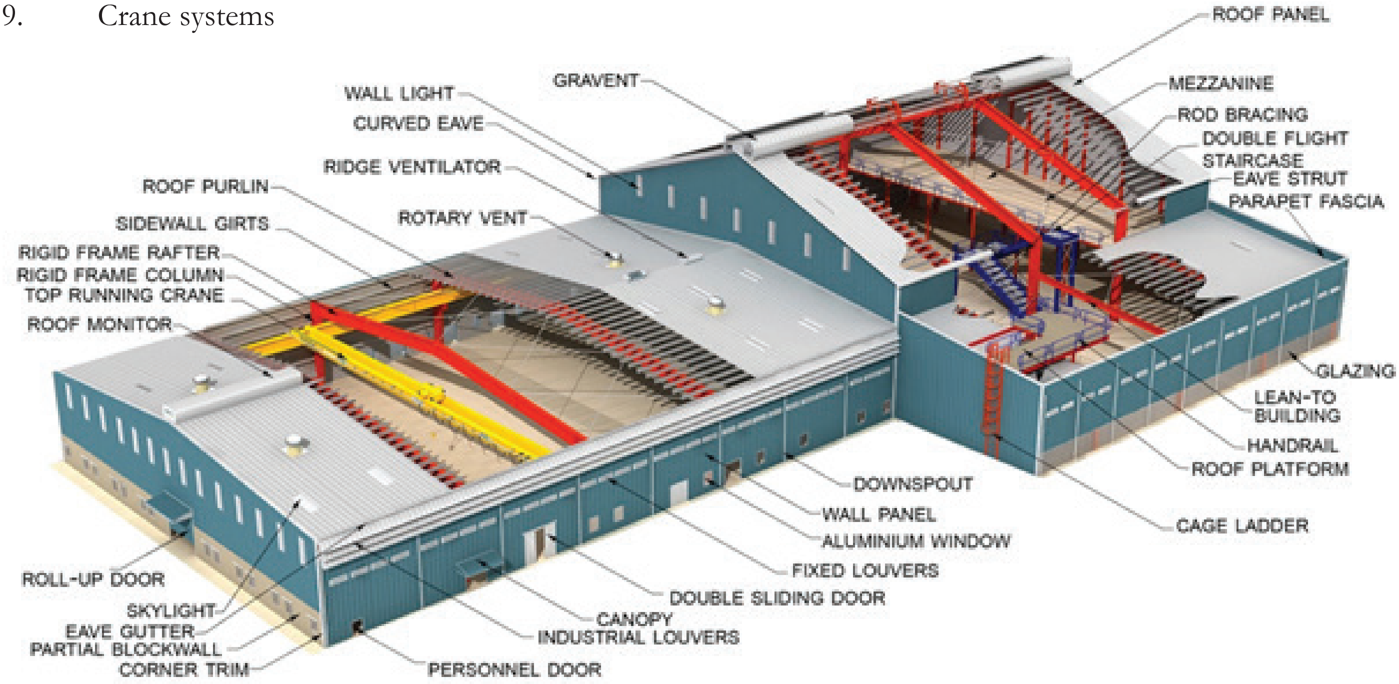


Figure 3.1 Pre-engineered building system

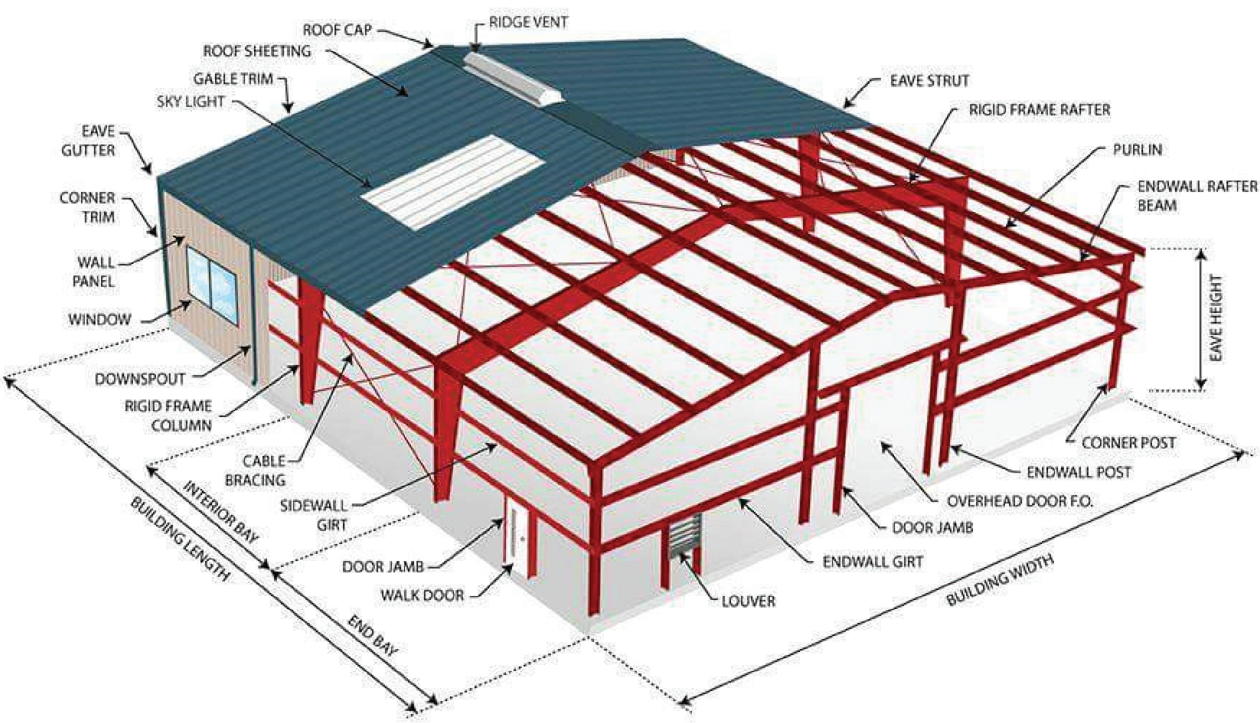


Figure 3.2 Pre-engineered building system

3.1. Main Frame Configuration

Main frame is the basic supporting component in the PEB systems; main frames provide the vertical support for the whole building plus providing the lateral stability for the building in its direction while lateral stability in the other direction is usually achieved by a bracing system.

The width of the building is defined as the out-to-out dimensions of girts/eave struts and these extents define the sidewall steel lines. Eave height is the height measured from bottom of the column base plate to top of the eave strut. Rigid frame members are tapered using built-up sections following the shape of the bending moment diagram. Columns with fixed base are straight. Also the interior columns are always maintained straight.

3.1.1. Main frame orientation

Building should be oriented in such a way that the length is greater than the width. This will result in more number of lighter frames rather than less number of heavy frames. This also will reduce the wind bracing forces results in lighter bracing systems.

3.1.2. Main frame types

3.1.2.1 Clear span

Clear Span rigid frames are single gable frames and offer full-width clear space inside the building without interior columns. This type of frame is extensively used anywhere an unobstructed working area is desired in diverse applications such as auditoriums, gymnasiums, aircraft hangars, showrooms and recreation facilities.

The deepest part of the frame is the knee, the joint between the rafter and the column, which is generally designed as horizontal knee connection. An alternate design of knee joint is as vertical knee connection that is employed for flush side-wall construction. Clear Span rigid frames are appropriate and economical when:

i) Frame width is in the range 24m-30m.

ii) Headroom at the exterior walls is not critical.

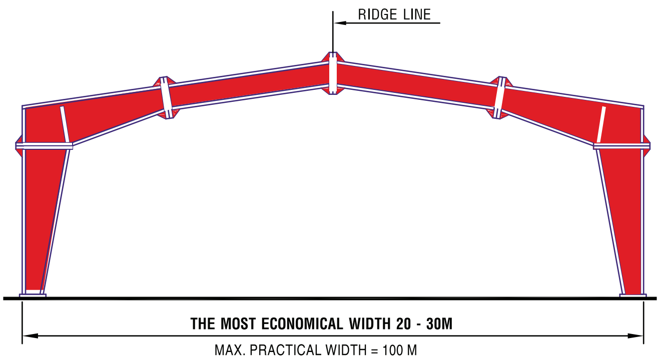


Figure 3.3 Clear span frame

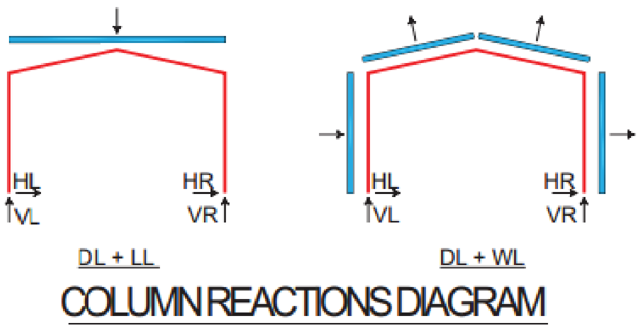


Figure 3.4 Column reaction of clear span frame

3.1.2.2 Multi – span

When clear space inside the building is not the crucial requirement then Multi-Span rigid frames offer greater economy and theoretically unlimited building size. Buildings wider than around 90m experience a build-up of temperature stresses and require temperature load analysis and design. Multi-span rigid frames have straight interior columns, generally hot-rolled tube sections pin connected at the top with the rafter. When lateral sway is critical, the interior columns may be moment connected at the top with the rafter, and in such a situation built-up straight columns are more viable than hot-rolled tube columns.

Multi-Span rigid frame with an interior column located at ridge requires the rafter at ridge to have a horizontal bottom flange in order to accommodate horizontal cap plate. Multi-Span rigid frame is the most economical solution for wider buildings (width > 24m) and is used for buildings such as warehouses, distribution centers and factories. The most economical modular width in multi-span buildings is in the range 18m-24m.

The disadvantages of such a framing system include:

- The susceptibility to differential settlement of column supports.
- Locations of the interior columns are difficult to change in future.
- Longer un-braced interior columns especially for wider buildings.
- Horizontal sway may be critical and governing the design in case of internal columns pined with rafter.

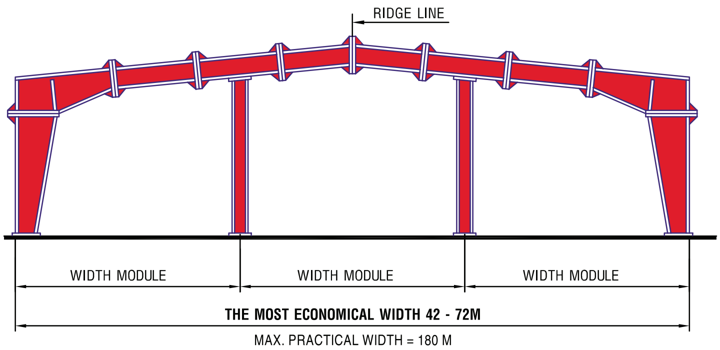


Figure 3.5 Multi - span frame

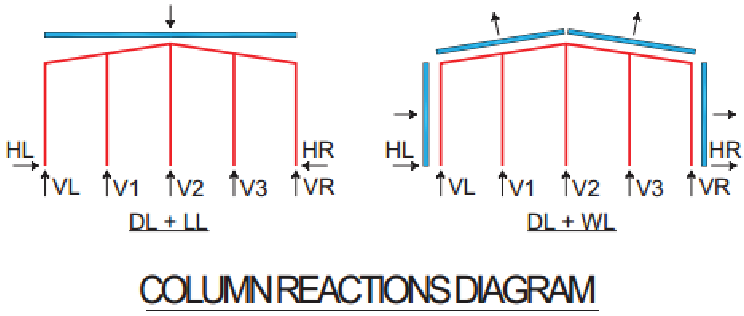


Figure 3.6 Column reaction of multi - span frame

3.1.2.3 Lean – to

Lean-To is not a self-contained and stable framing system rather an add-on to the existing building with a single slope. This type of frame achieves stability when it is connected to an existing rigid framing. Usually column rafter connection at knee is pinned type, which results in lighter columns. In general, columns and rafters are straight except that rafters are tapered for larger widths (> 12m). For clear widths larger than 18m, tapered columns with moment resisting connections at the knee are more economical. Lean-To framing is typically used for building additions, equipment rooms and storage.

For larger widths “Multi-Span-Lean-To” framing can be adopted with exterior column tapered and moment connected at the knee.

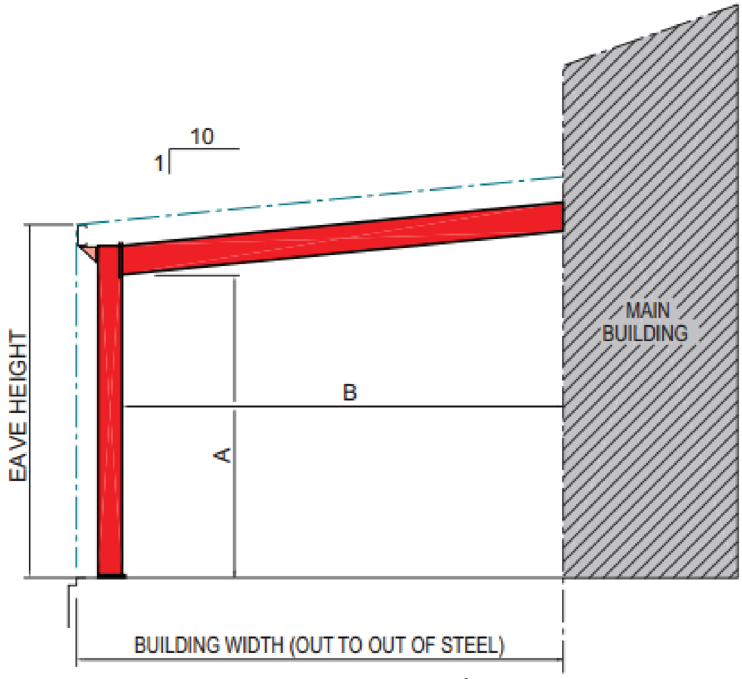


Figure 3.7 Lean - to frame

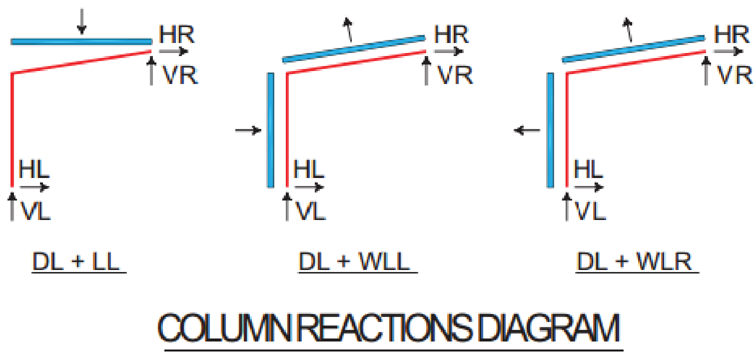


Figure 3.8 Column reaction of lean - to frame

3.1.2.4 Mono-slope

Mono-slope or single-slope framing system is an alternative to gable type of frame that may be either Clear Span or multi-span. Mono-Slope configuration results in more expensive framing than the gable type.

Mono-slope framing system is frequently adopted where:

- Rainwater needs to be drained away from the parking areas or from the adjacent buildings
- Larger headroom is required at one sidewall
- A new building is added directly adjacent to an existing building and it is required to avoid:
• The creation of a valley condition along the connection of both buildings.
• The imposition of additional loads on the columns and foundations of the existing building.

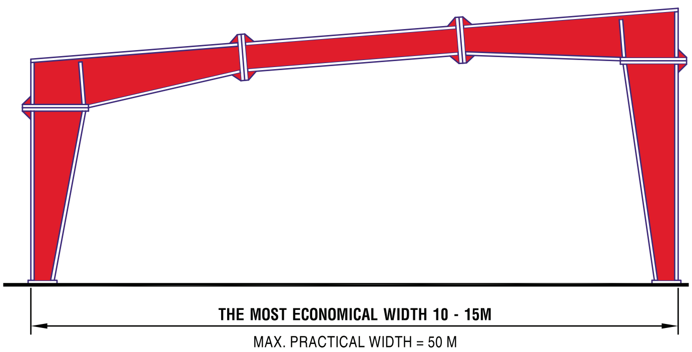


Figure 3.9 Mono-slope frame

For larger widths "mono-slope-multi-span" framing will be more economical when column free area inside the building is not an essential requirement.

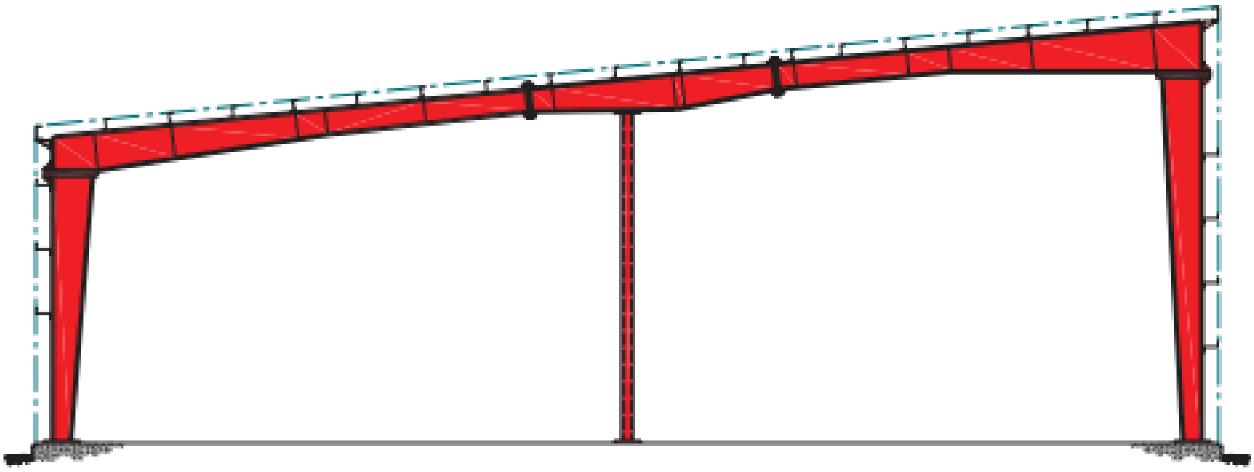


Figure 3.12 Mono-slope frame with 2 spans

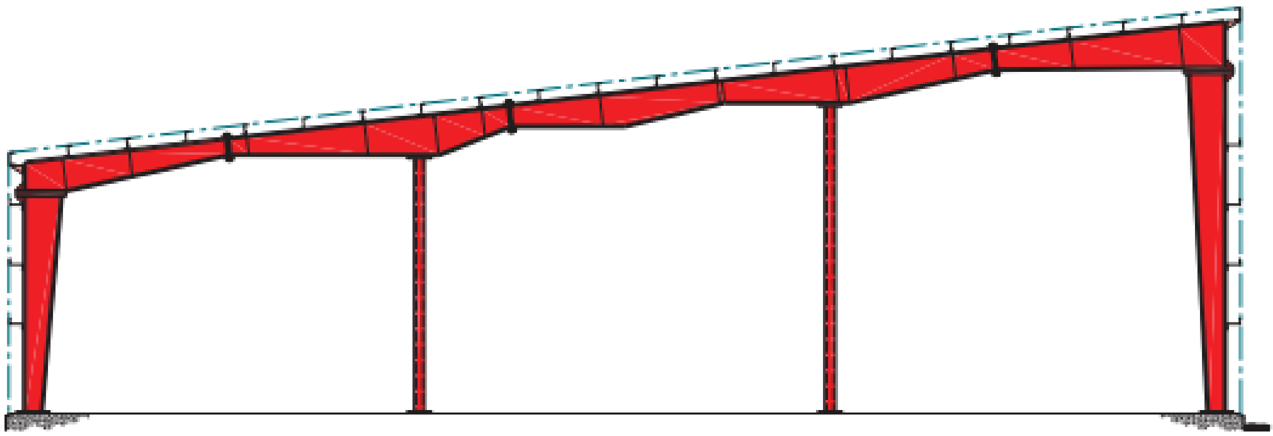


Figure 3.13 Mono-slope frame with 3 spans

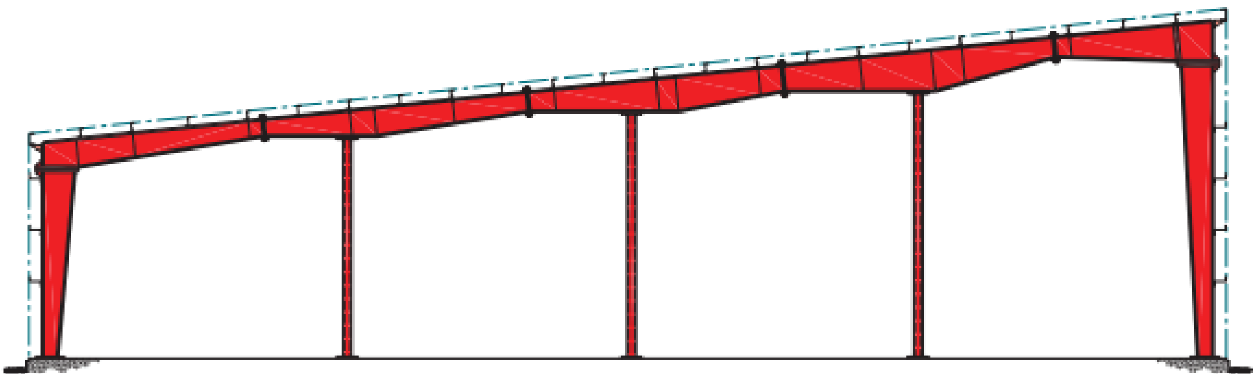


Figure 3.14 Mono-slope frame with 4 spans

3.1.2.5 Space saver

Space Saver framing system offers straight columns, keeping the rafter bottom flange horizontal for ceiling applications with rigid knee connection. Selection of Space Saver is appropriate when:

- The frame width is between 6m to 18m and eave height does not exceed 6m.
- Straight columns are desired.
- Roof slope $\leq 5\%$ are acceptable.
- Customer requires minimum air volume inside the building especially in cold storage warehouses

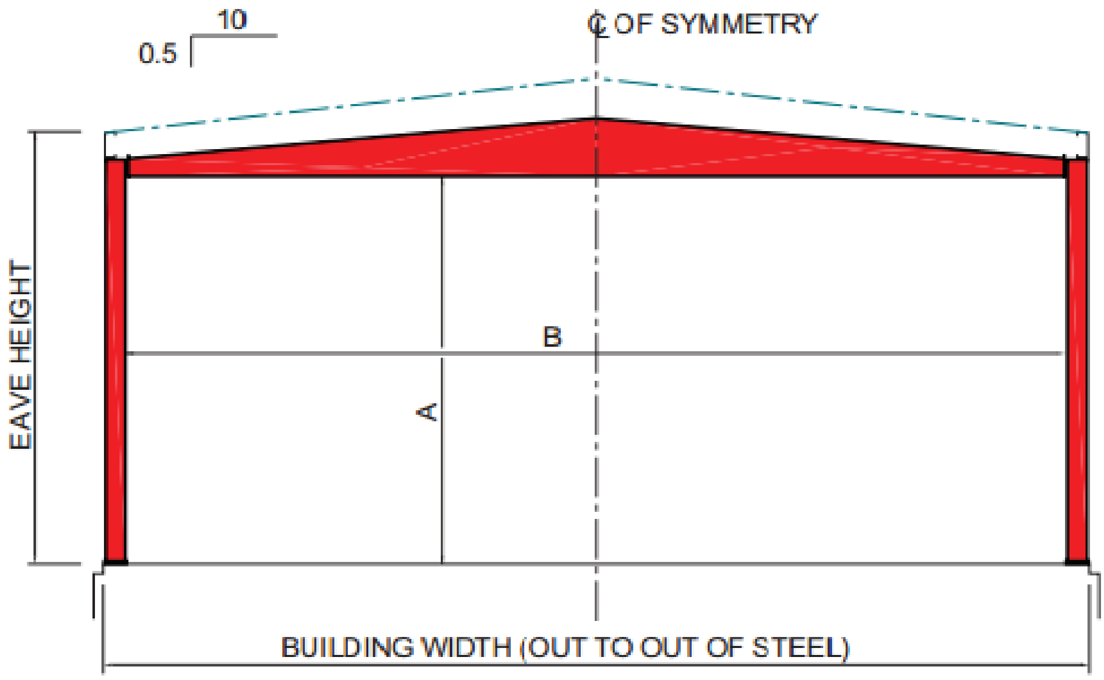


Figure 3.15 Space saver frame

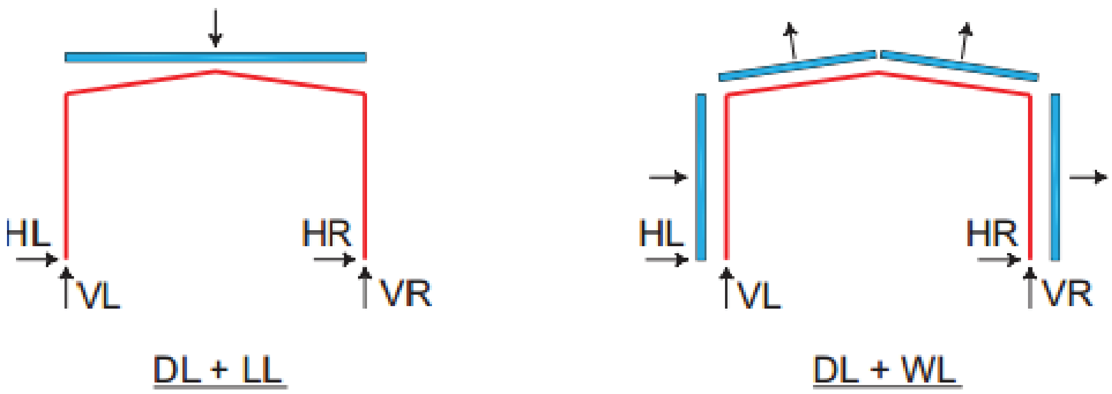


Figure 3.16 Column reaction of space saver frame

3.1.2.6 Roof system

A Roof System framing consists of beam (rafter) resting onto a planned or an existing substructure. The substructure is normally made of concrete or masonry. The rafter is designed in such a way to result in only vertical reaction (no horizontal reaction) by prescribing a roller support condition at one end. The roller supports are provided at one end by means of roller rods.

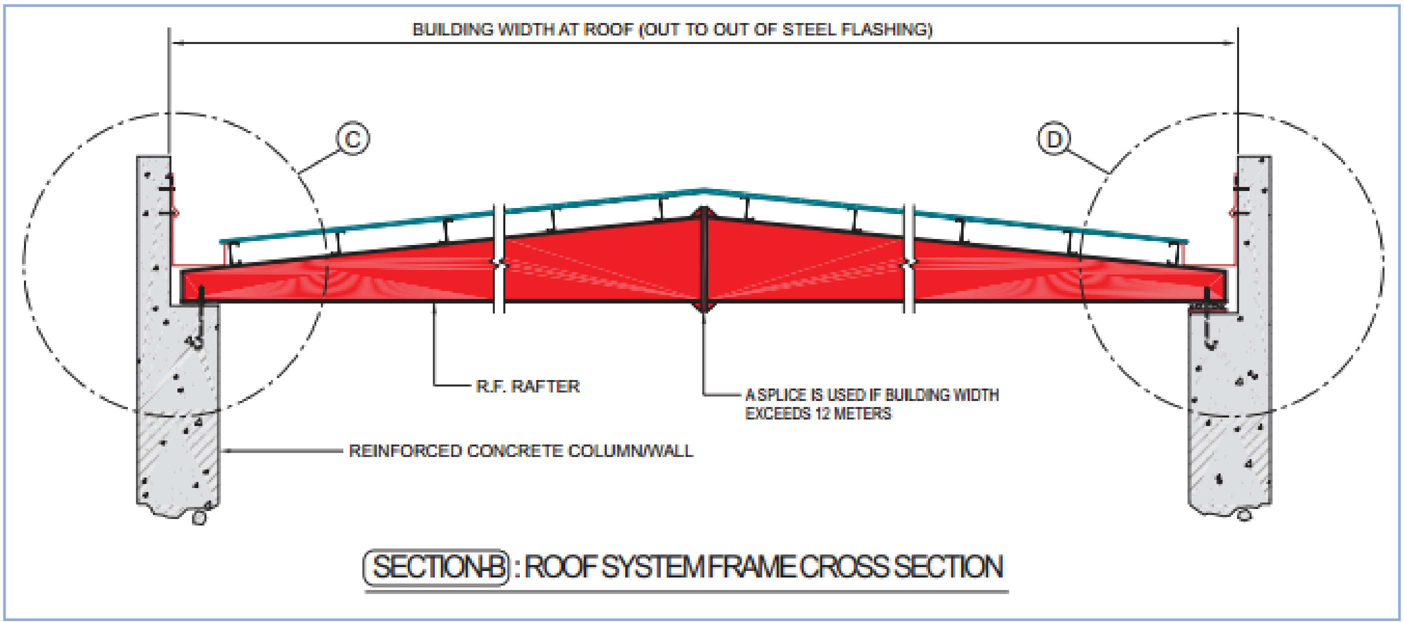


Figure 3.17 Roof systems connect to RC column

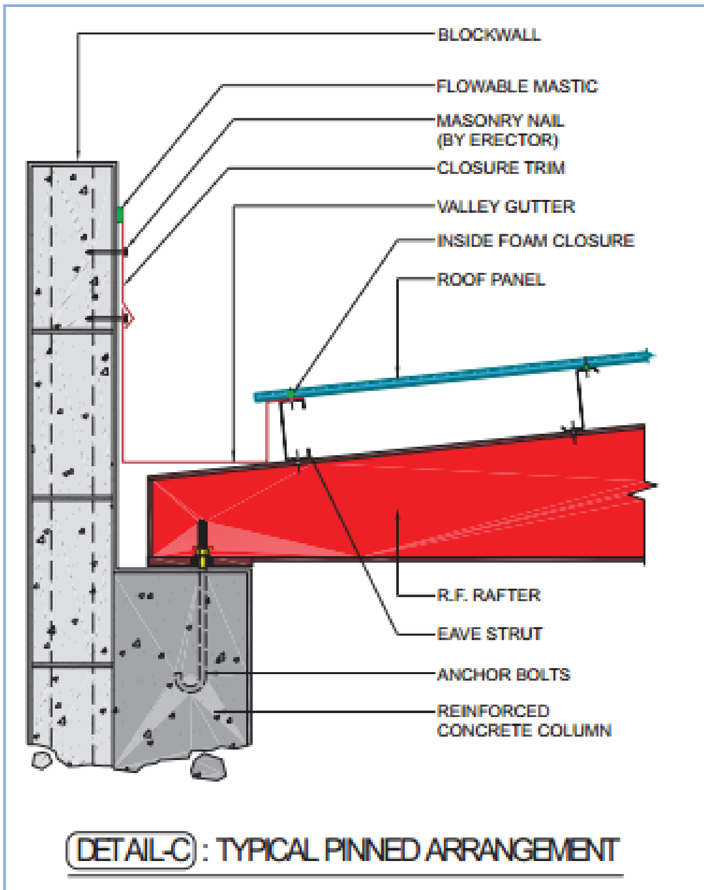


Figure 3.18 Detail C – typical pinned arrangement

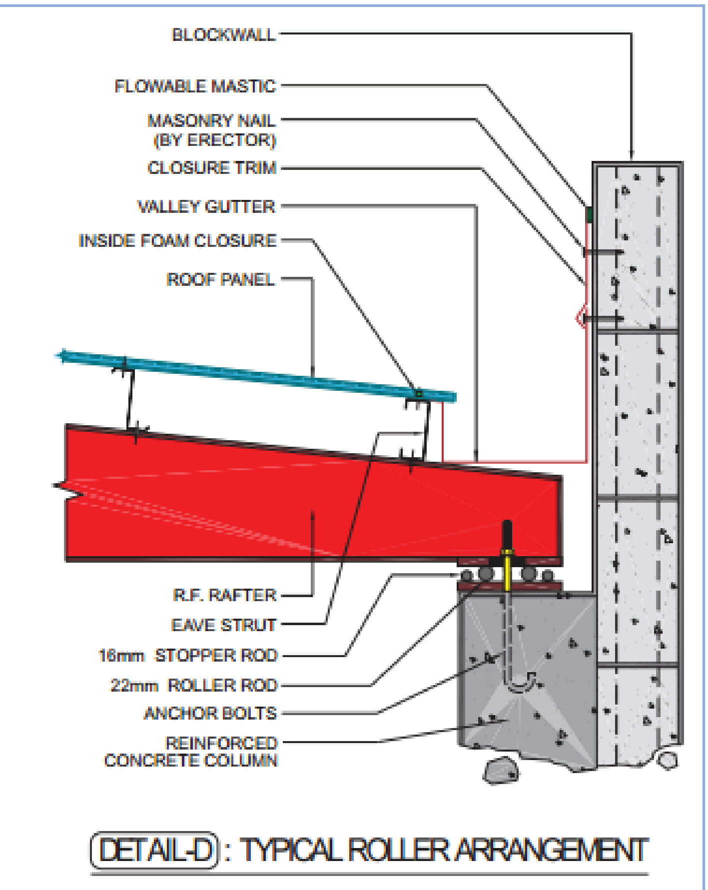


Figure 3.19 Detail D – typical rolled arrangement

A Roof System is generally not economical for spans greater than 12m although it can span as large as 36m. This is due to fact that the Roof System stresses are concentrated at mid-span rather than at the knees.

3.1.2.7 Multi – gable

Multi-Gable buildings are not recommended due to maintenance requirement of valley region, internal drainage and bracing requirement inside the building at columns located at valley. Especially in snow areas, Multi-Gable framing should be discouraged. However, for very wide buildings this type of framing offers a viable solution due to:

- Reduced height of ridge and thus the reduced height of interior columns
- Temperature effects can be controlled by dividing the frame into separate structural segments

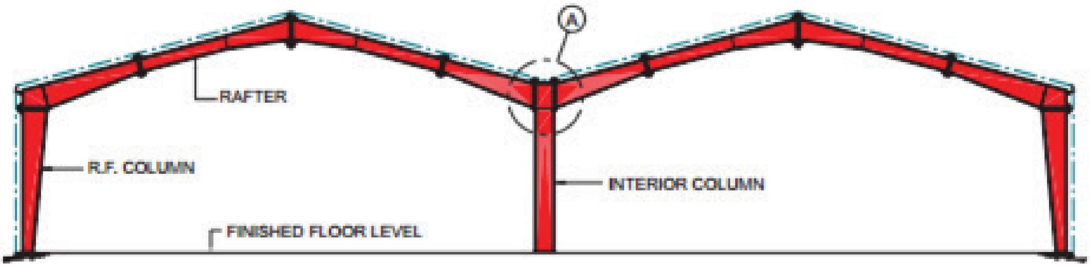


Figure 3.20 Multi – gable frame with 2 clear spans

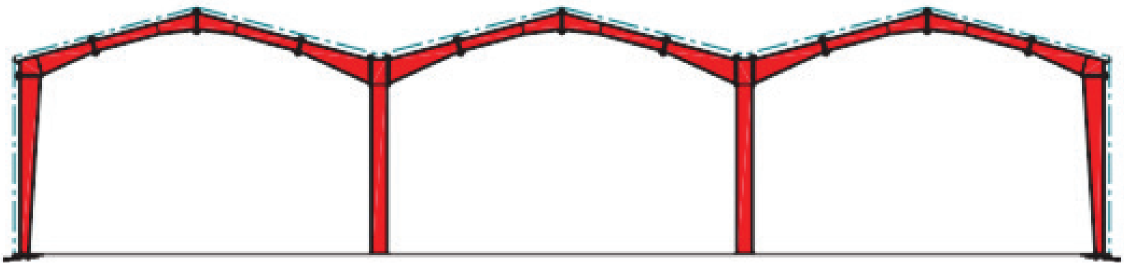


Figure 3.21 Multi – gable frame with 3 clear spans

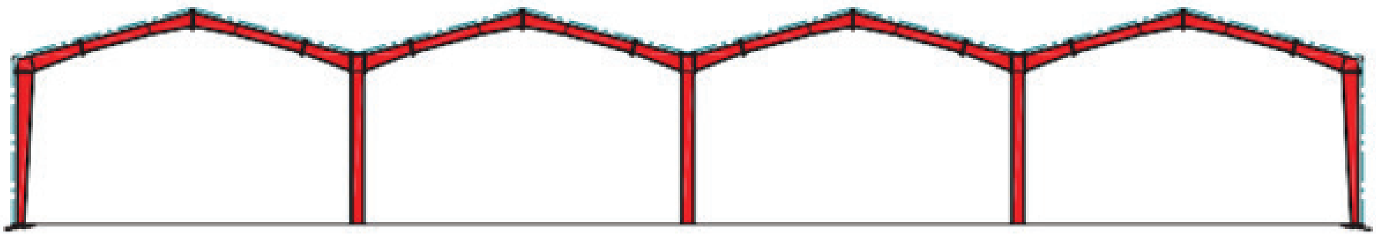


Figure 3.22 Multi – gable frame with 4 clear spans

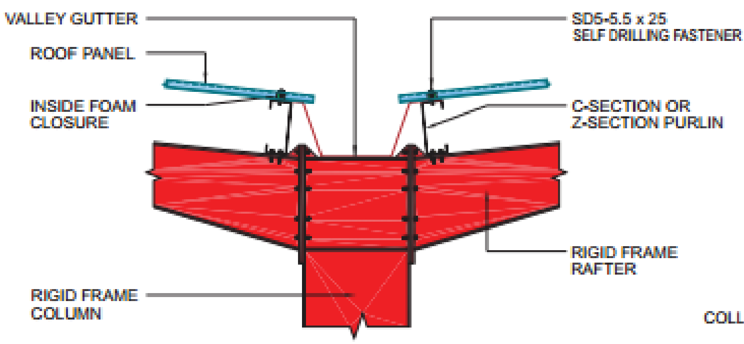


Figure 3.23 Typical detail at valley

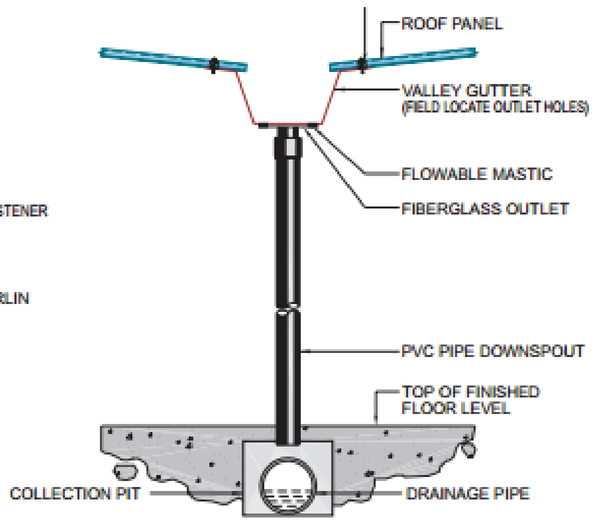


Figure 3.24 Recommended interior drainage arrangement

Thus, Multi-Gable buildings are more economical than Multi-Span buildings for very wide buildings. Multi-Gable frames may be either Clear Spans or Multi-Spans. The columns at the valley location should be designed as rigidly connected to rafters on either side using a vertical type of connection.

3.1.3. Roof slope

Optimum roof slope:

Type of building	Recommended slope (%)
Multi - span buildings	5
Clear span, width up to 45m	10
Clear span, width up to 60m	15
Clear span, width > 60m	20

3.1.4. Eave height

Eave height is governed by:

- Clear height at eave (head clearance)
- Mezzanine clear heights below beam and above joist
- Crane beam/ Crane hook heights

Minimize eave height to the bare minimum requirement since the eave height affects the price of the building by adding to the price of sheeting, girts and columns. If columns are unbraced eave height affects the frame weight significantly. Also higher eave heights increase the wind loads on the building.

If eave height to width ratio becomes more than 0.8 then the frame may have a fixed based design in order to control the lateral deflection.

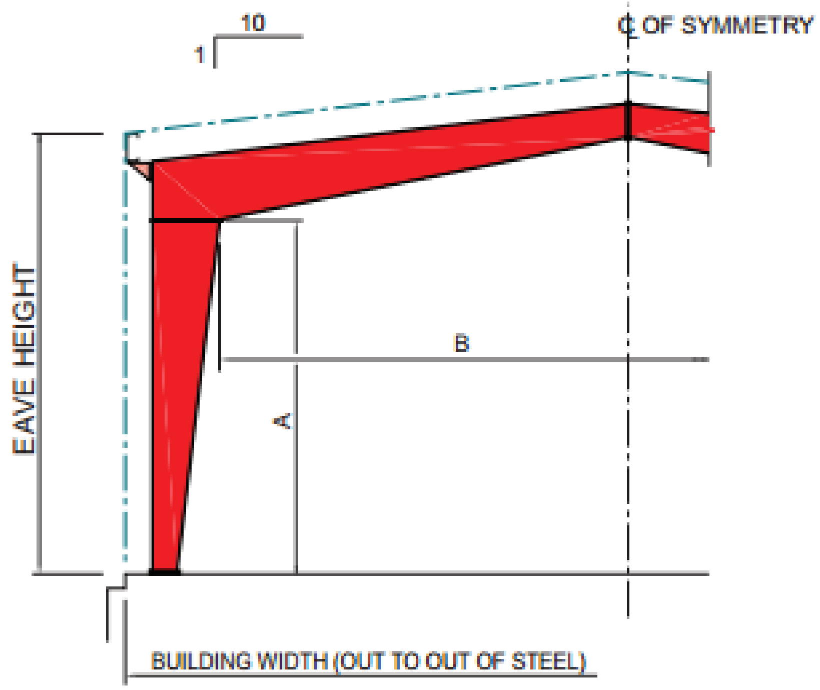


Figure 3.25 Eave height in the building

3.2. End Wall Systems

3.2.1. Generals

The standard end wall is designed as post & beam (all connections are pinned) the lateral stability is provided by the diaphragm action, in the absence of this shear diaphragm wind bracing are required.

End rigid frame are used in case of:

- Future extension is intended; in this case only wind posts are required.
- Crane running to the end wall
- Open for access condition prevails at the end wall
- X-bracing is not allowed at end wall in the case of by-framed end wall.

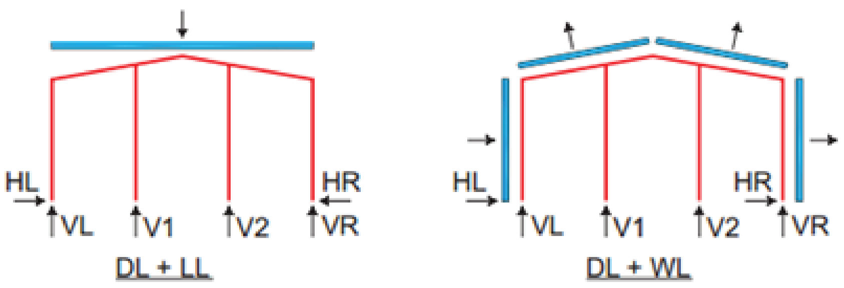
3.2.2. Post & Beam End Wall Rafter

All end wall rafters are designed as simple beams over the end wall posts. They are comprised of built-up sections or hot rolled sections.

Design concept

Under gravity load (Dead load + Live Load): The top flange of the end wall rafters is under compression and it is braced against lateral torsional buckling at every purlin location.

Under uplift loads (Dead + Wind Load): The bottom flange of the end wall rafters is under compression and it is unbraced against lateral torsional buckling. The buckling length is the distance between end wall columns, this may significantly reduce the section bending capacity or flange braces are to be used.



COLUMN REACTIONS DIAGRAM

3.2.3. End Wall Post

All end wall posts are flush with the end wall structural line in end post & beam gable type and they are supporting end wall rafters. In all cases end wall posts are oriented so that end wall wind pressure is producing bending moments about the column major axis. End wall posts are comprised of built-up sections or hot rolled sections.

Design concept

End wall posts are the supporting elements for end wall girts/block walls for wind loads (pressure or suction) which produce bending moment about posts major axes, end wall posts are designed as simple beam supported at foundation level (base plate) and are connected with Spanner at roof purlins level.

For Post & Beam end walls additional vertical loads from end wall rafters are transmitted to posts producing axial loads (compression or tension).

End wall post buckling length about major axis is the column length.

Flanges unsupported length is depending on the end wall type and position.

3.3. Expansion Joints

The purpose to add the expansion joint is separating the building into many areas in order that every area has allowable temperature deformation. When separating the foundation into 2 parts, expansion joint become settlement joint.

Section L7 AISC 360-10 recommends that the length of building needs to be added expansion joint or research NRC 1974 for more.

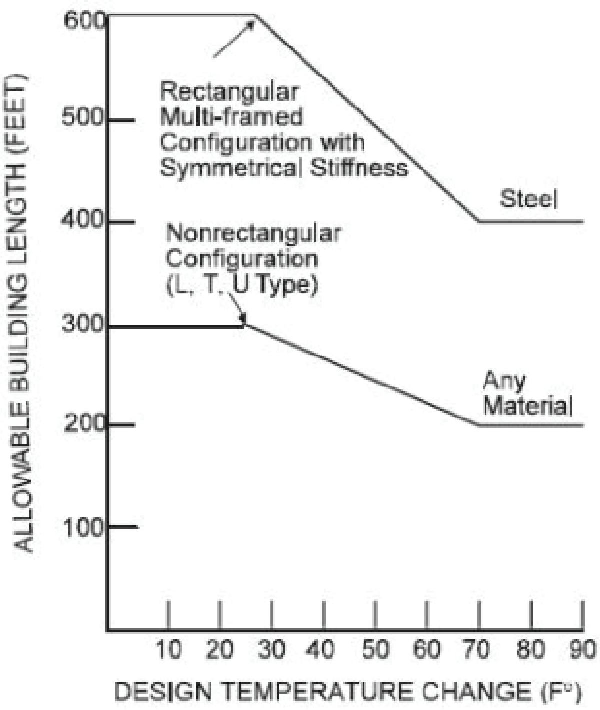


Fig. 1: Maximum allowable building length without expansion joints for various design temperature changes.

Unless more specifc site information is available, most engineers assume a range of 50° to 70° F (10° to 21° C) for continuously heated and air-conditioned buildings. Using that assumption, most steel, rectangular, framed confguration buildings with symmetrical stiffness can tolerate 460 ft (140 m) between expansion joints.

Or the designer can refer to section 11.1.2 TCVN 5575:2012 to know the length & width of building with expansion joint to be added.

Type of building	Maximum distance (m)		
	Between 2 expansion joints		
	Longitudinal direction	Lateral direction	From expansion joist or the end of building to axis of nearest wall bracing system
Building with insulation	230	150	150
Building without insulation & heating factory	200	120	120
Viaduct	130	-	-
Note: If there are 2 wall bracing systems in every expansion area, the distance of these not exceeding from 40m to 50m for building; from 25m to 30m for viaduct.			

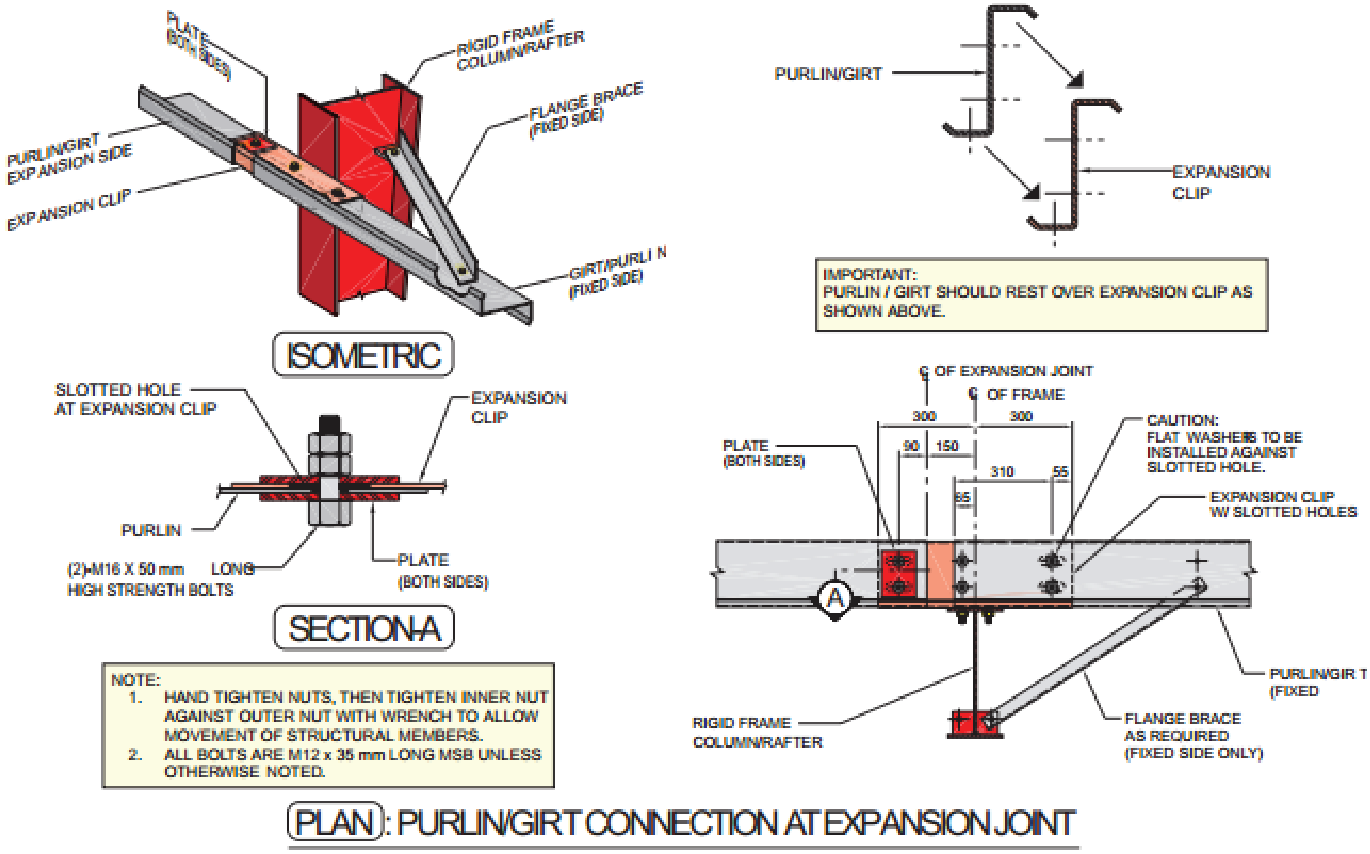


Figure 3.26 Purlin/girt connection at expansion joint

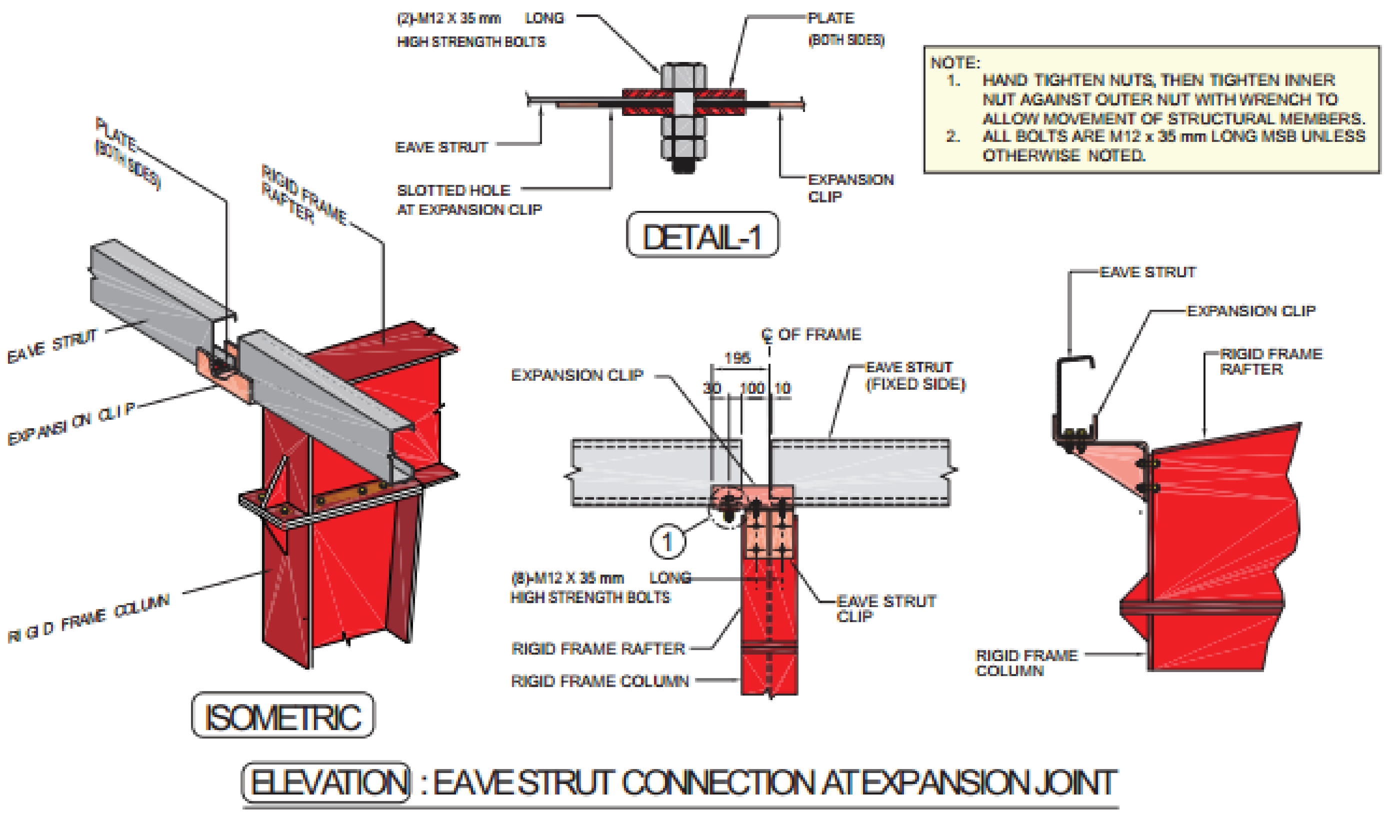


Figure 3.27 Eave strut connection at expansion joint

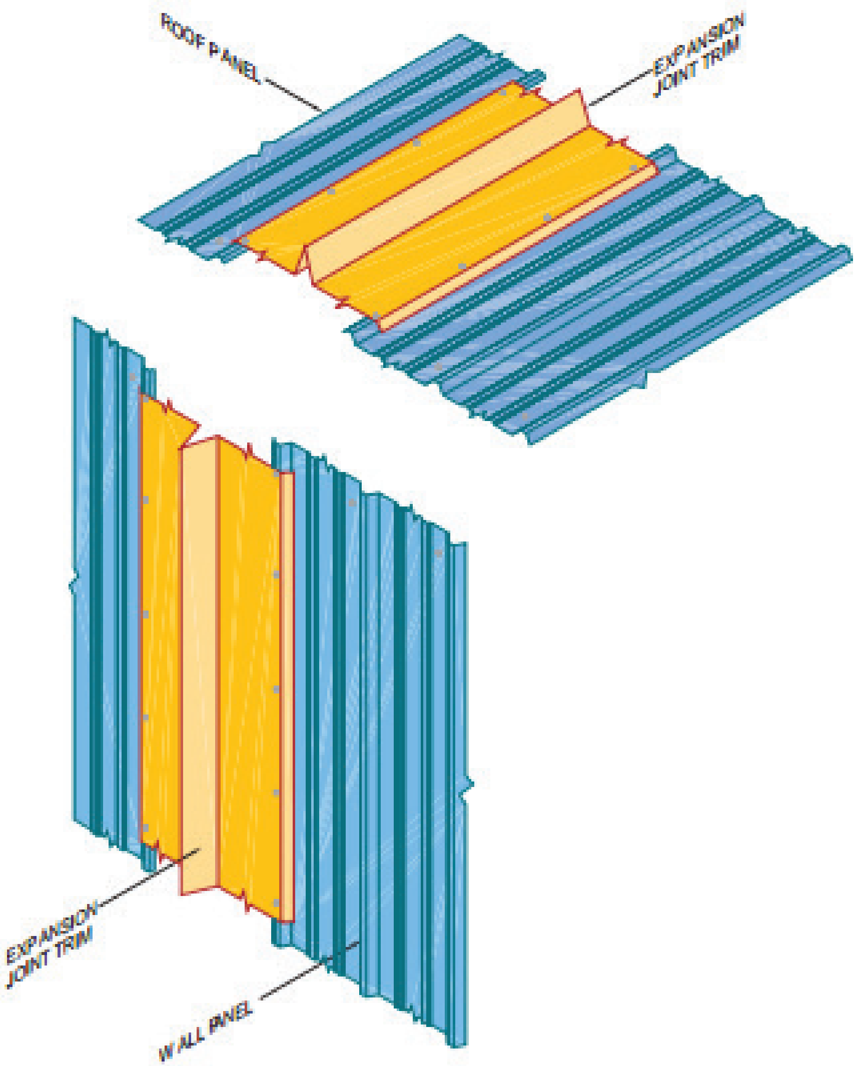


Figure 3.28 Roof & wall panels at expansion joint

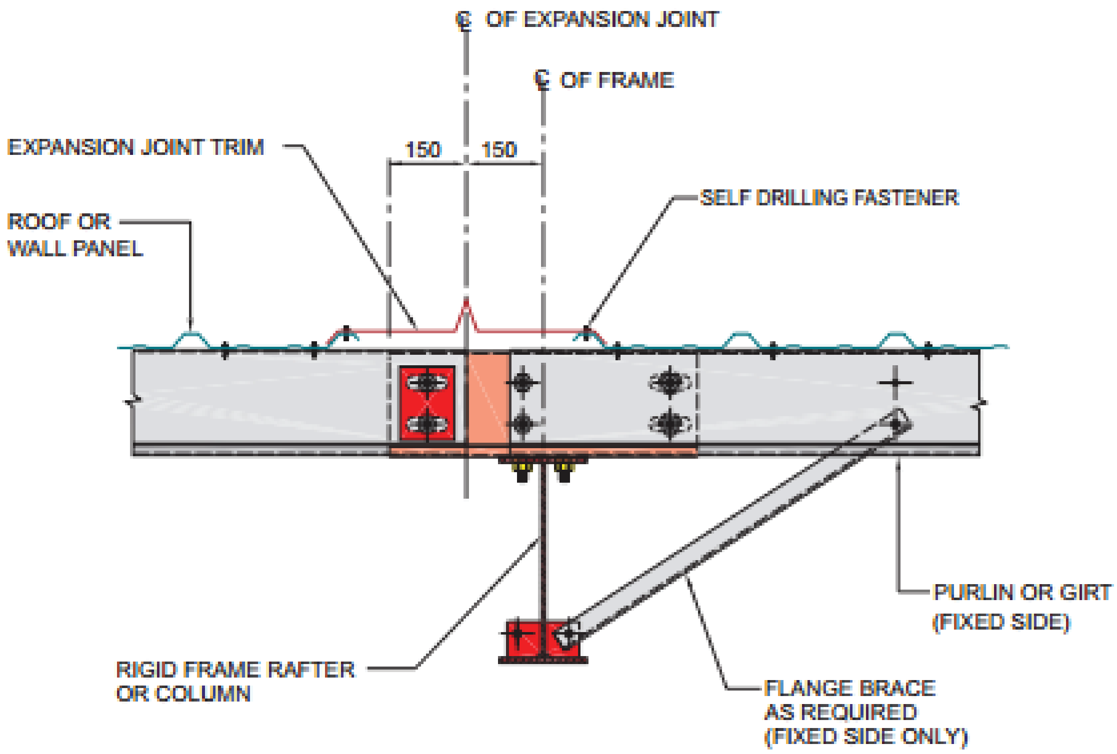


Figure 3.29 Panel connection at expansion joint

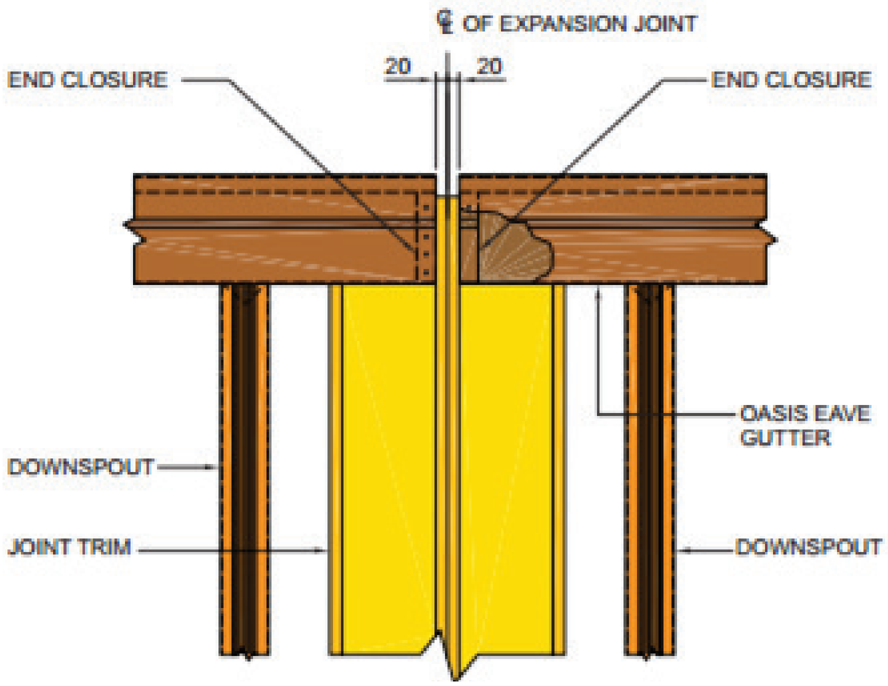


Figure 3.30 Gutter at expansion joint

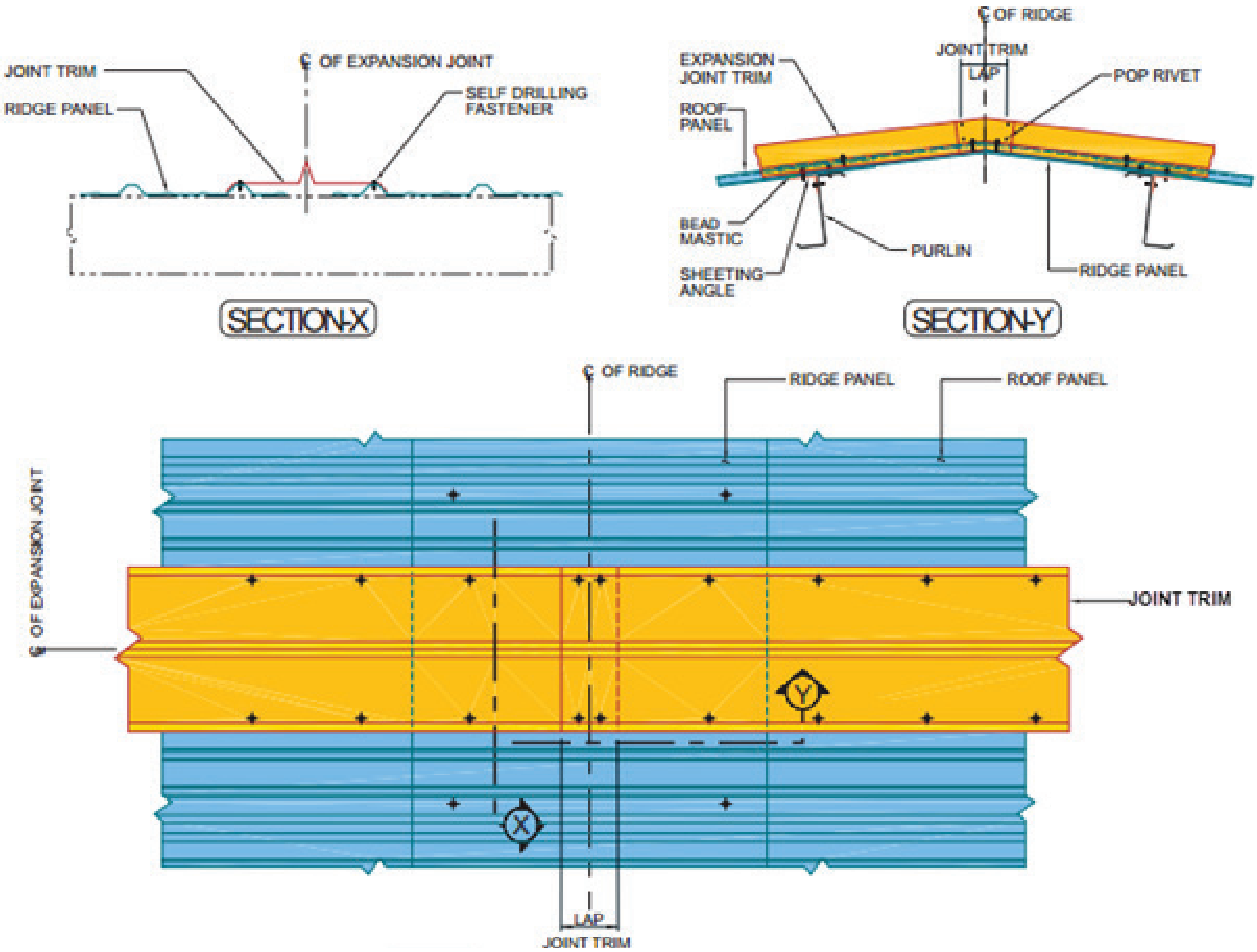


Figure 3.31 Expansion joint at ridge

3.4. Bay Spacing

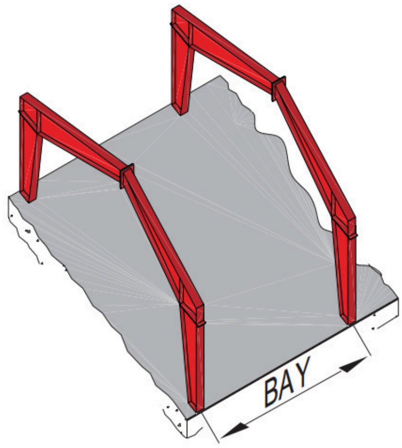


Figure 3.32 Bay spacing

The most economical bay spacing is around 8m for the standard loads as following:

Live Loads on roof and frame (kN/m2)	Wind Speed (km/h)
0.57	130

For greater loads than standard loads the economical bay spacing tends to decrease.
For buildings with heavy cranes (crane capacity > 10 Tons) the economical bay spacing ranges between 6m and 7m.
Smaller end bays than interior bays will taper off the effect of higher deflection and bending moment in end bays as compared to interior bays and help reduce the weights of purlins/girts in the end bays. This will avoid the need of nested purlins/girts in the end bays and result in uniform size of purlin/girt sizes.
Some buildings require bay spacing more than 10m in order to have a greater clear space at the interior of the building in Multi-Span buildings. Such a situation can be handled by providing jack beams that support the intermediate frames without interior columns. Thus the exterior columns will have bay spacing of say 6m while the interior columns are spaced at 12m. Intermediate frames allow the purlin to span for 6m.

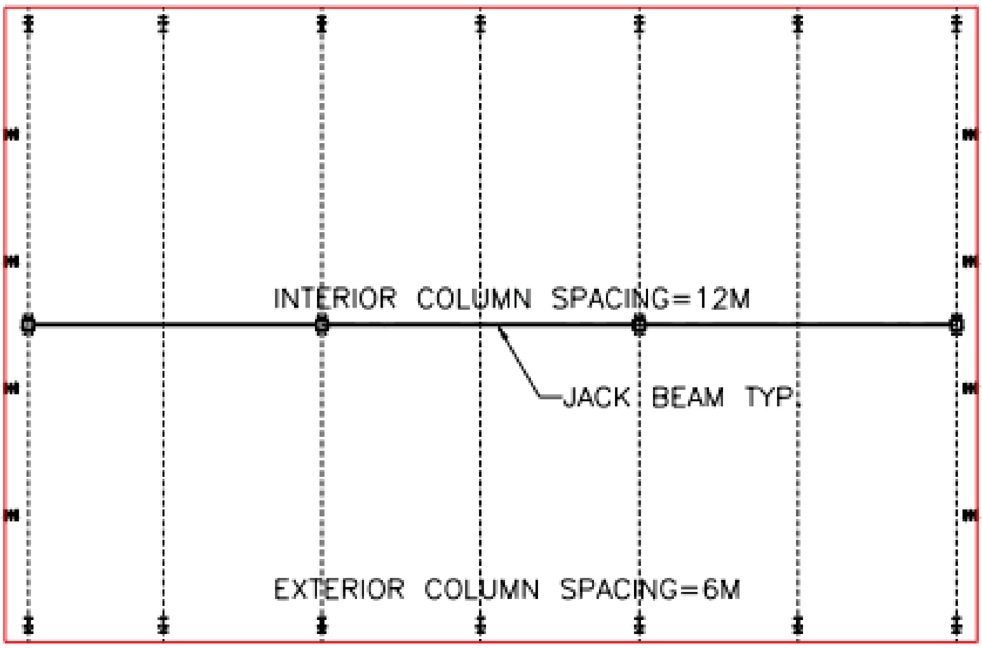


Figure 3.33 Jack beam plan with bay Spacing 12m

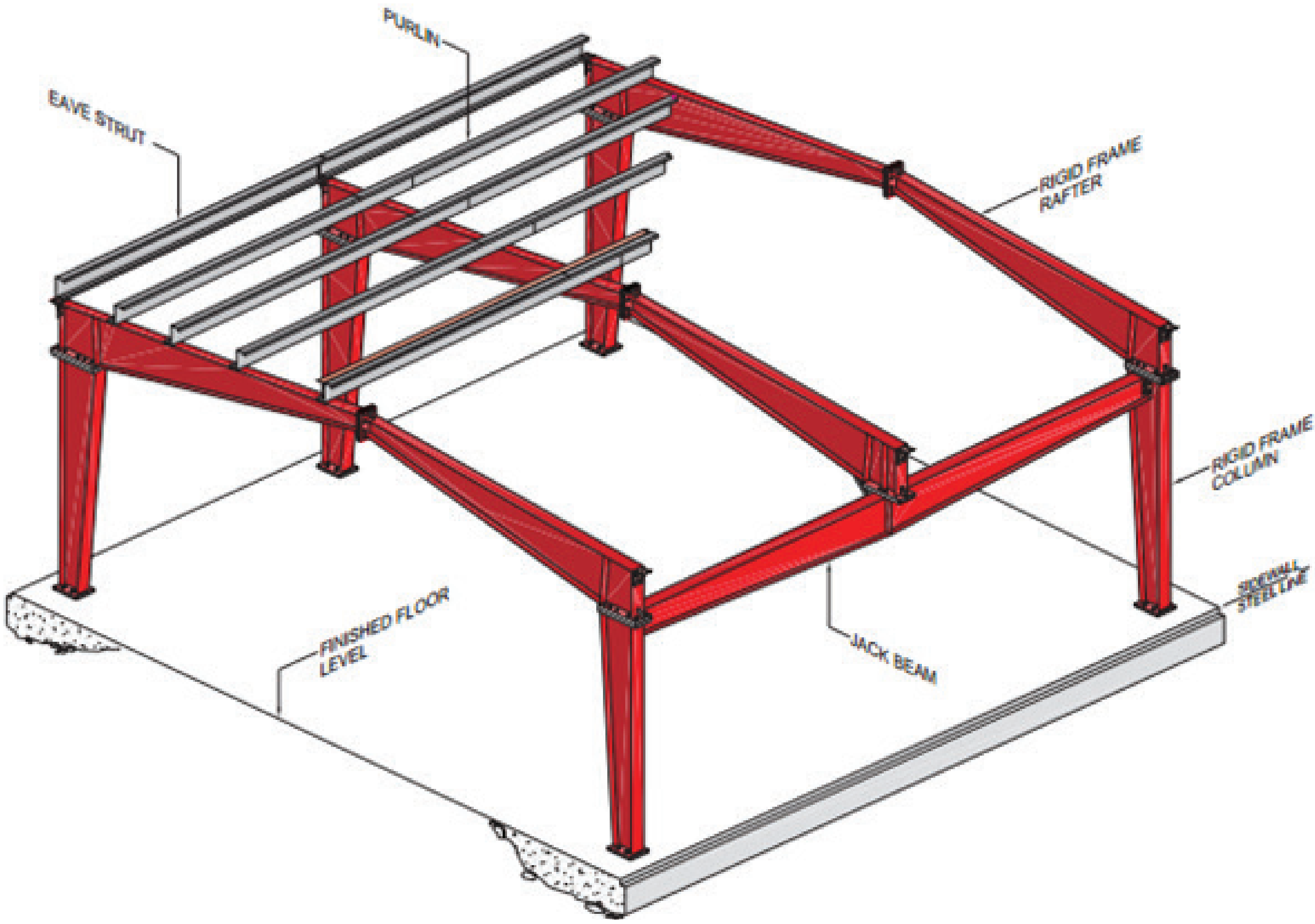


Figure 3.34 Jack beam at side wall

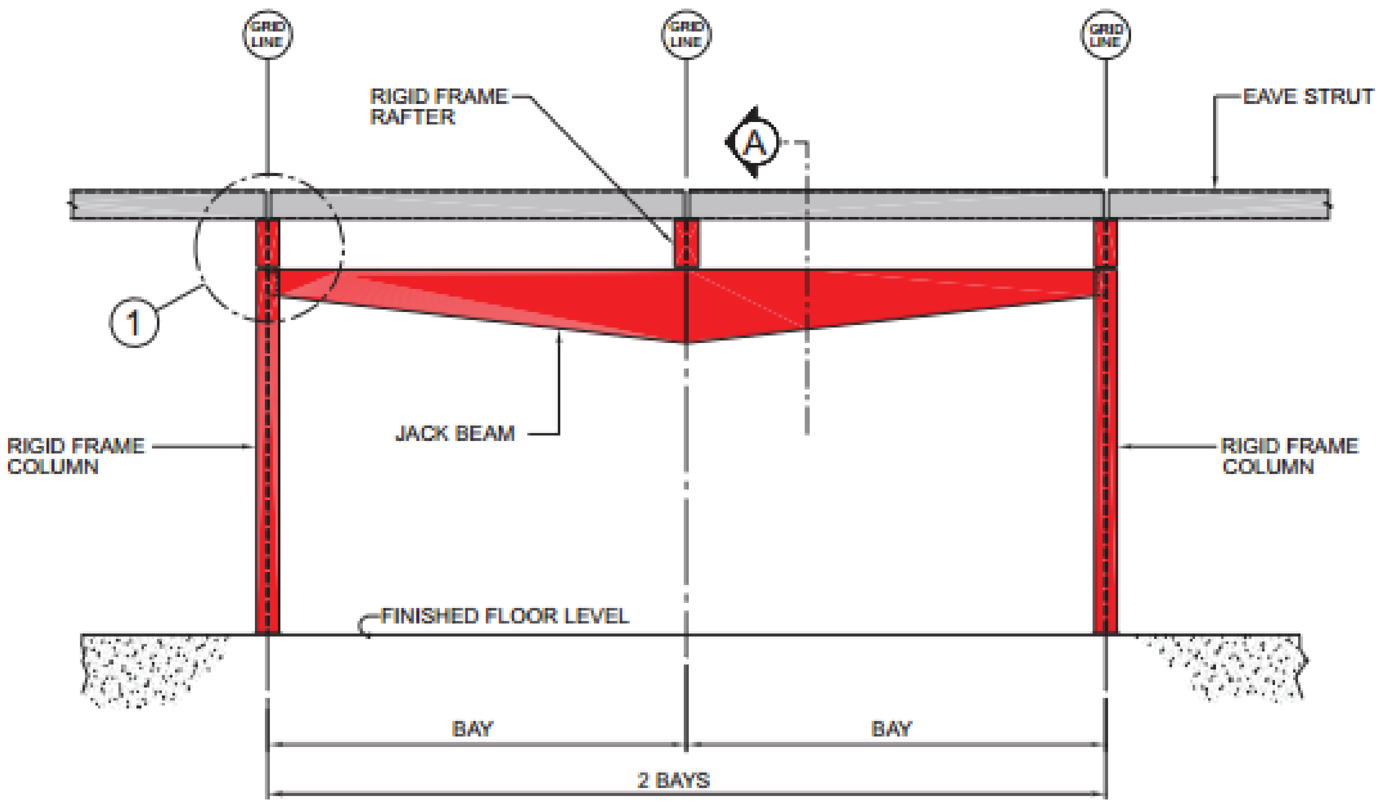


Figure 3.35 Elevation of jack beam at side wall

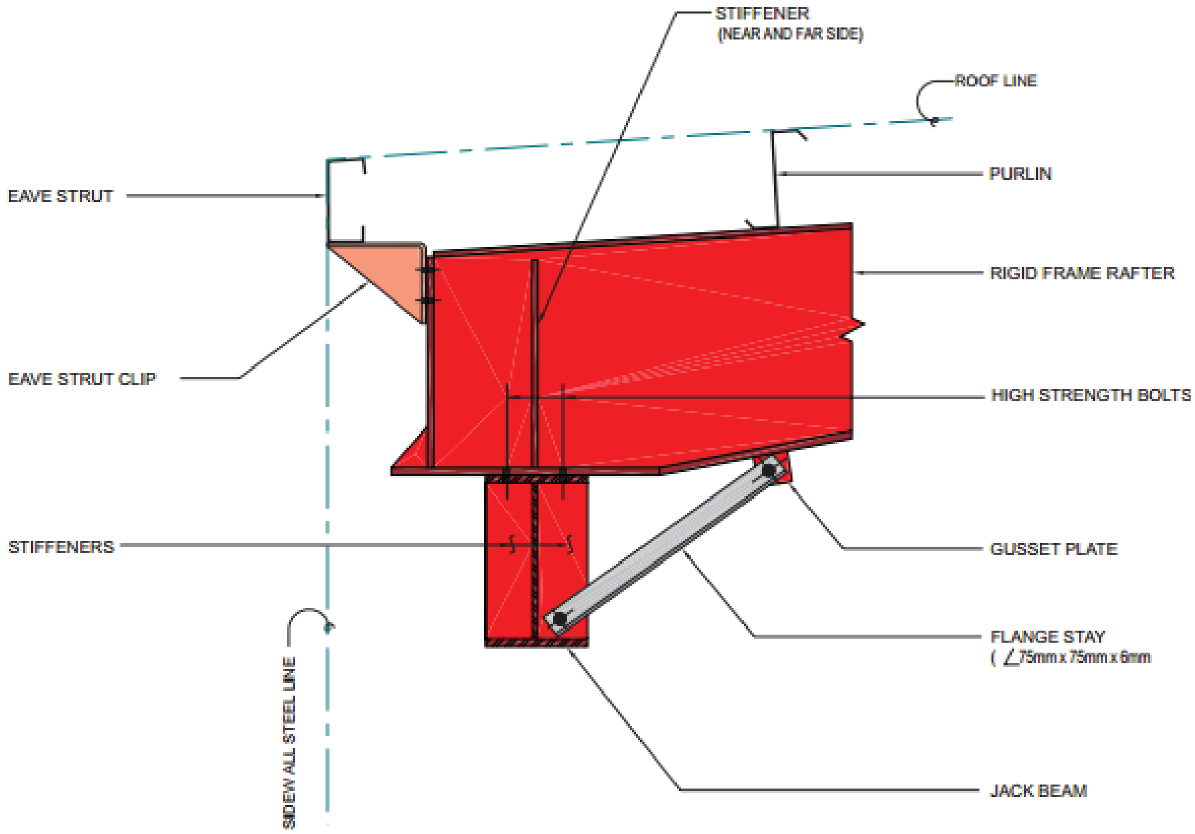


Figure 3.36 Section A - Jack beam at side wall

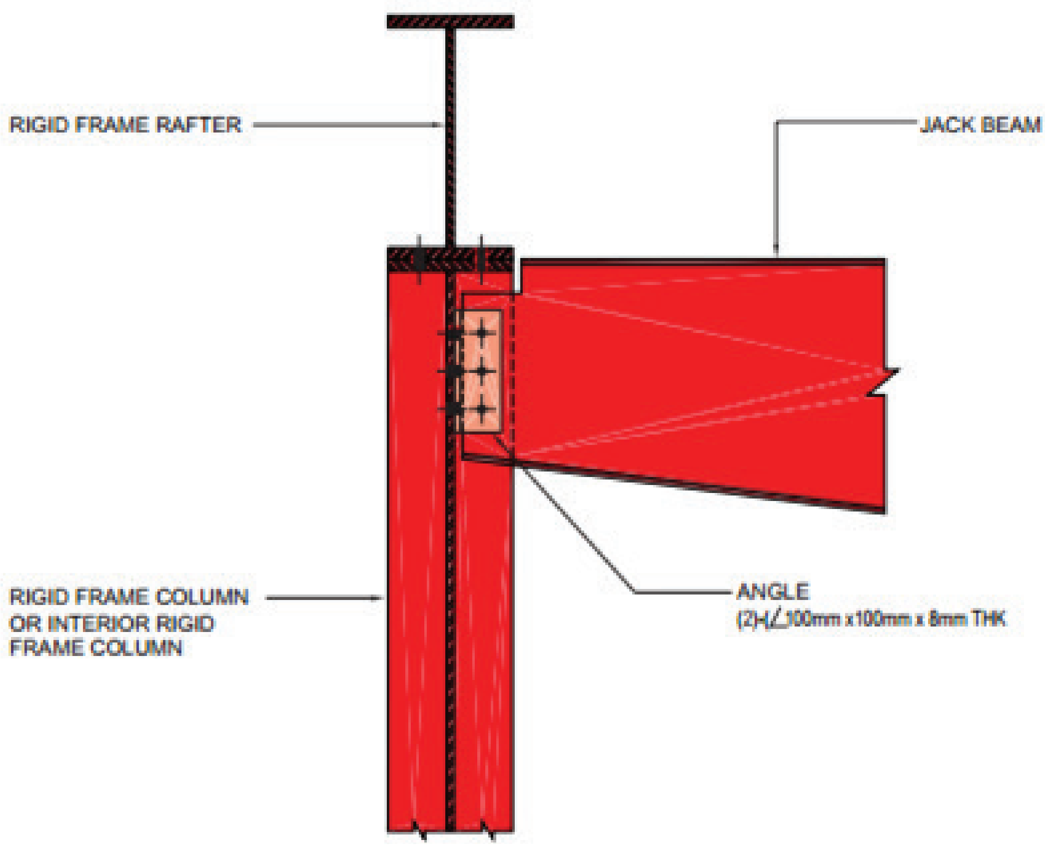


Figure 3.37 Detail 1 - Jack beam at side wall

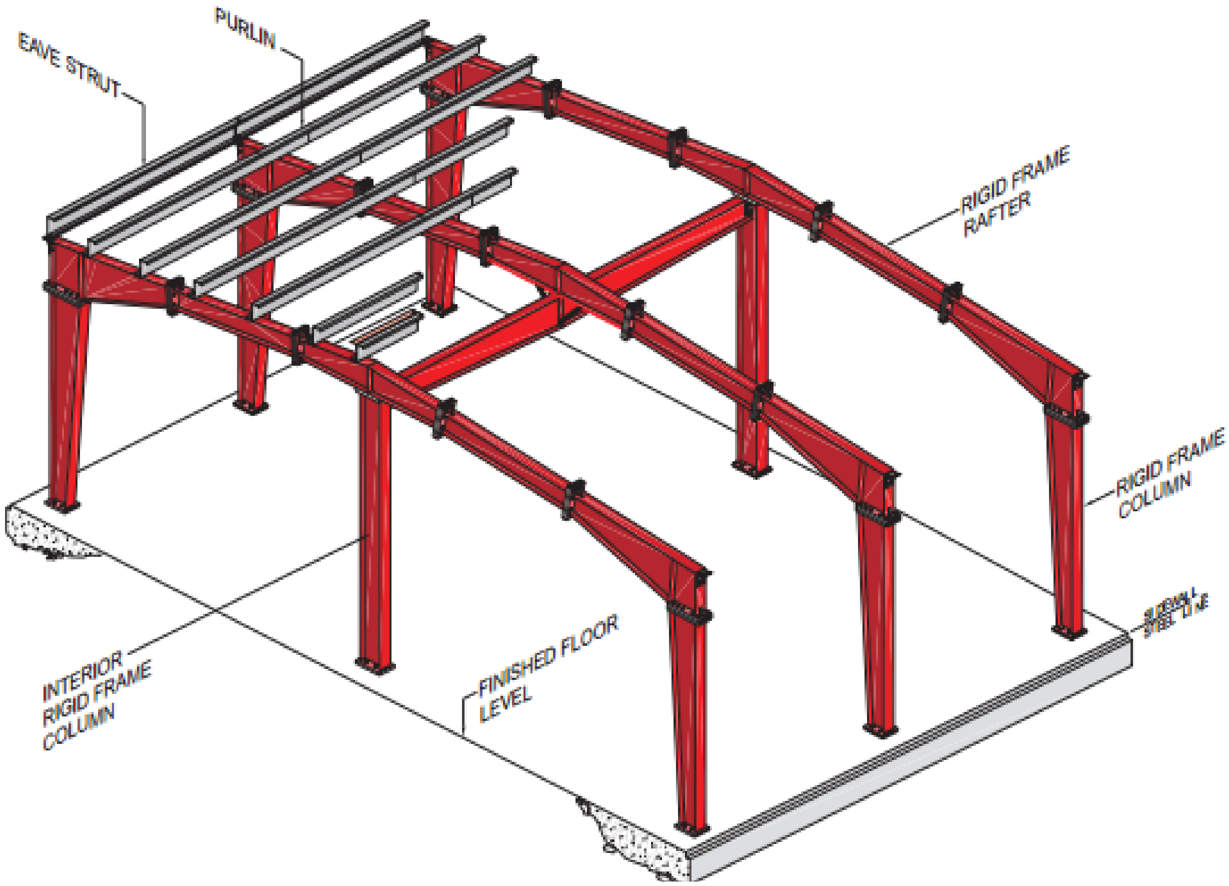


Figure 3.38 Interior jack beam

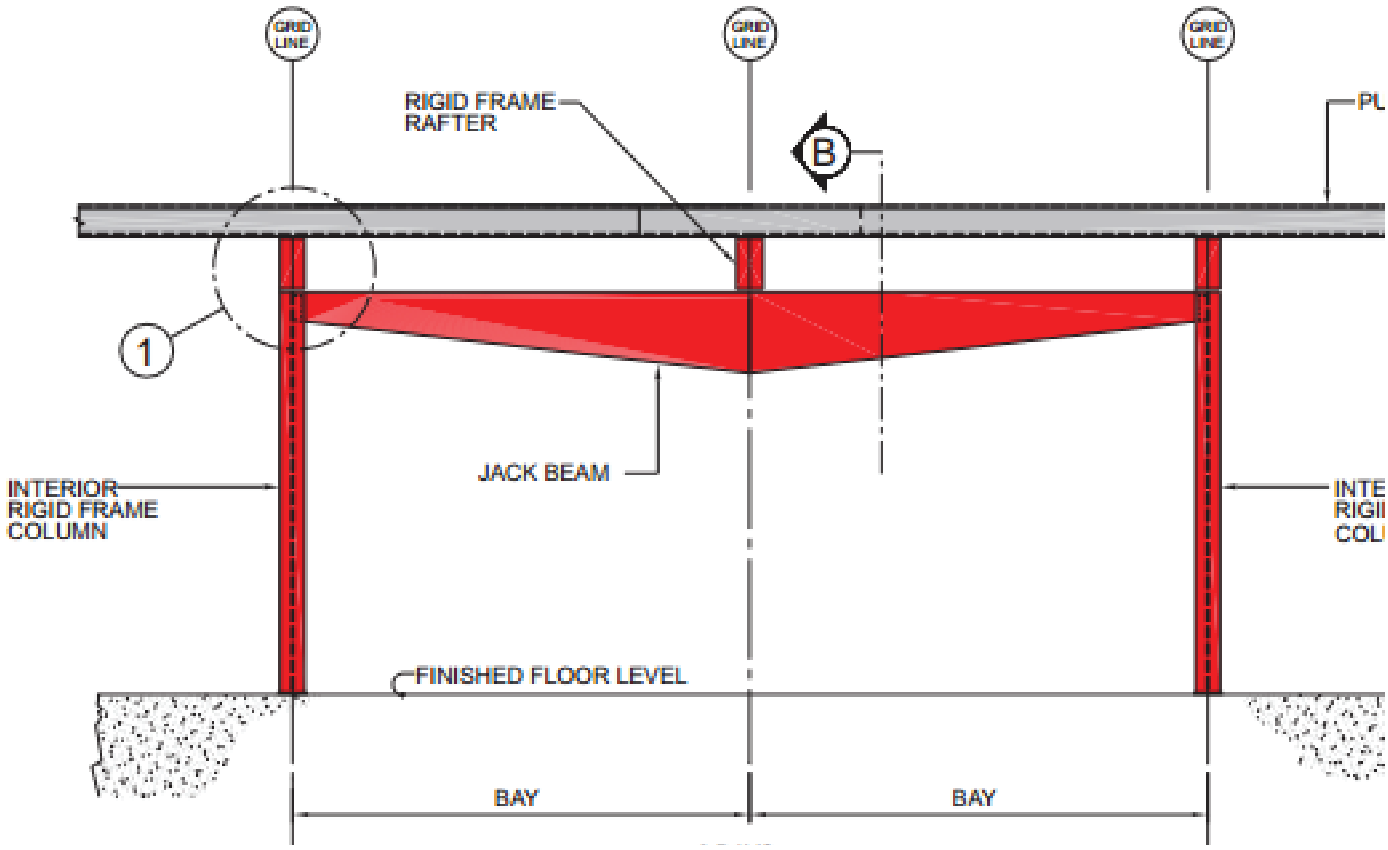


Figure 3.39 Elevation - Interior jack beam

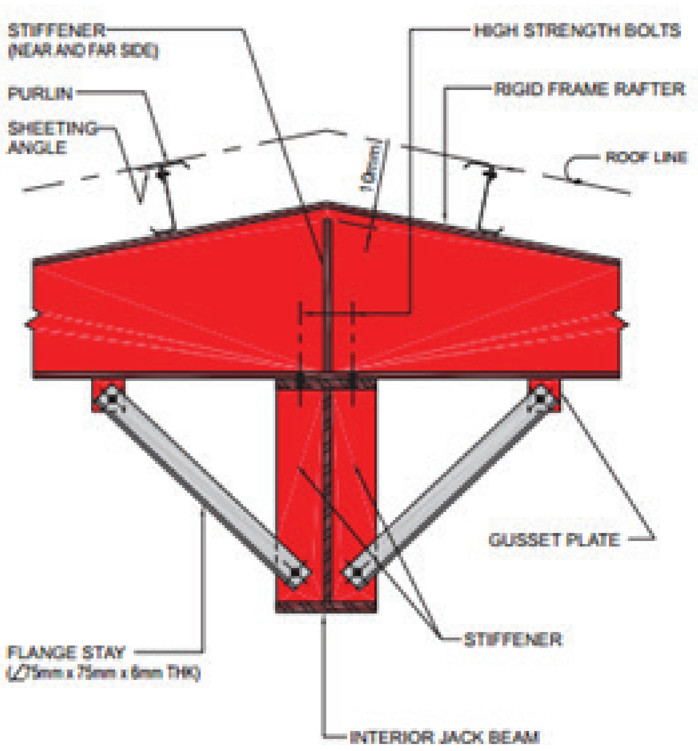


Figure 3.40 Jack beam at middle span of rafter

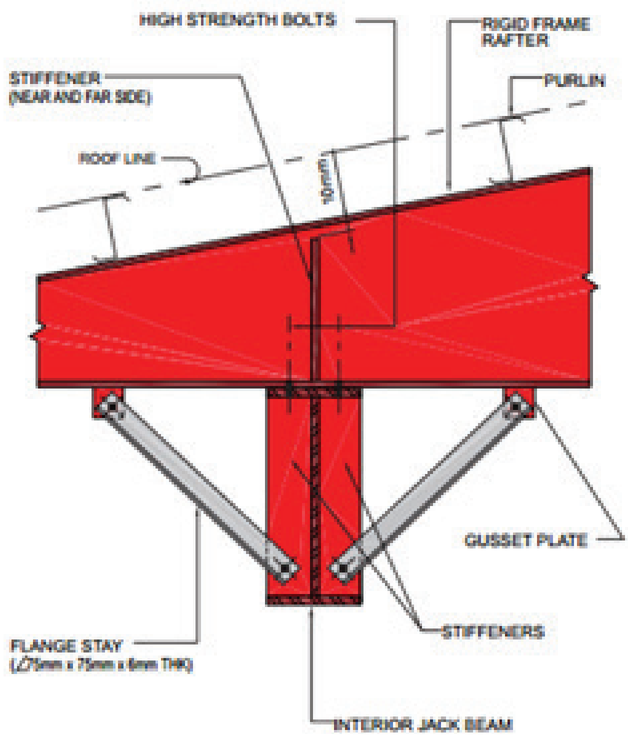


Figure 3.41 Jack beam at intermediate span of rafter

3.5. Bracing Systems Arrangement

Bracing is a structural system used to provide stability in a structure in a direction where applied forces on that structure would otherwise make it unstable. Whether it is a force due to wind, crane or seismic applications, the bracing system will always eventually transmit that load down to the column base and then to the foundations. The rules of arranging different types of bracing systems are as follows:

3.5.1. Bracing for wind and seismic loads in the longitudinal direction

1. In long buildings, braced bays shall be provided in intervals not to exceed 5 bays.
2. Sidewall bracing shall be generally placed in the same bays of roof bracing. This may not be possible at times due to openings in the sidewalls. In such cases, sidewall bracing shall be placed in bays adjacent to those containing the roof bracing with a consideration that load transfers to the adjacent bays.
3. Roof rod bracing shall not cross the ridgeline.
4. Cables/rods braces shall not exceed 15m in length. If a cross bracing contains rods longer than 15m, then the bracing should be broken to two sets of bracings with a strut member between them so that the rod/cable lengths shall not exceed 15 m.
5. Sidewall bracing shall be comprised of any one of the following types:
- Cables
- Rods or angles.
- Portal frame with/without rods or angles.
6. There shall be only one type of bracing in the same sidewall. Do not mix different types/materials in the same sidewall.
7. It is preferable to use only one type of wall bracing in the whole building otherwise the lateral loads (especially seismic loads) will not be divided equally between bracing lines. For cases when this will result in excessive weight for bracing system advanced calculation is to be done to determine the force that will be carried by each type depending on its stiffness and location.
8. Do not use rod/cable Ø25mm for roof bracing because its weight make a large vertical deflection.

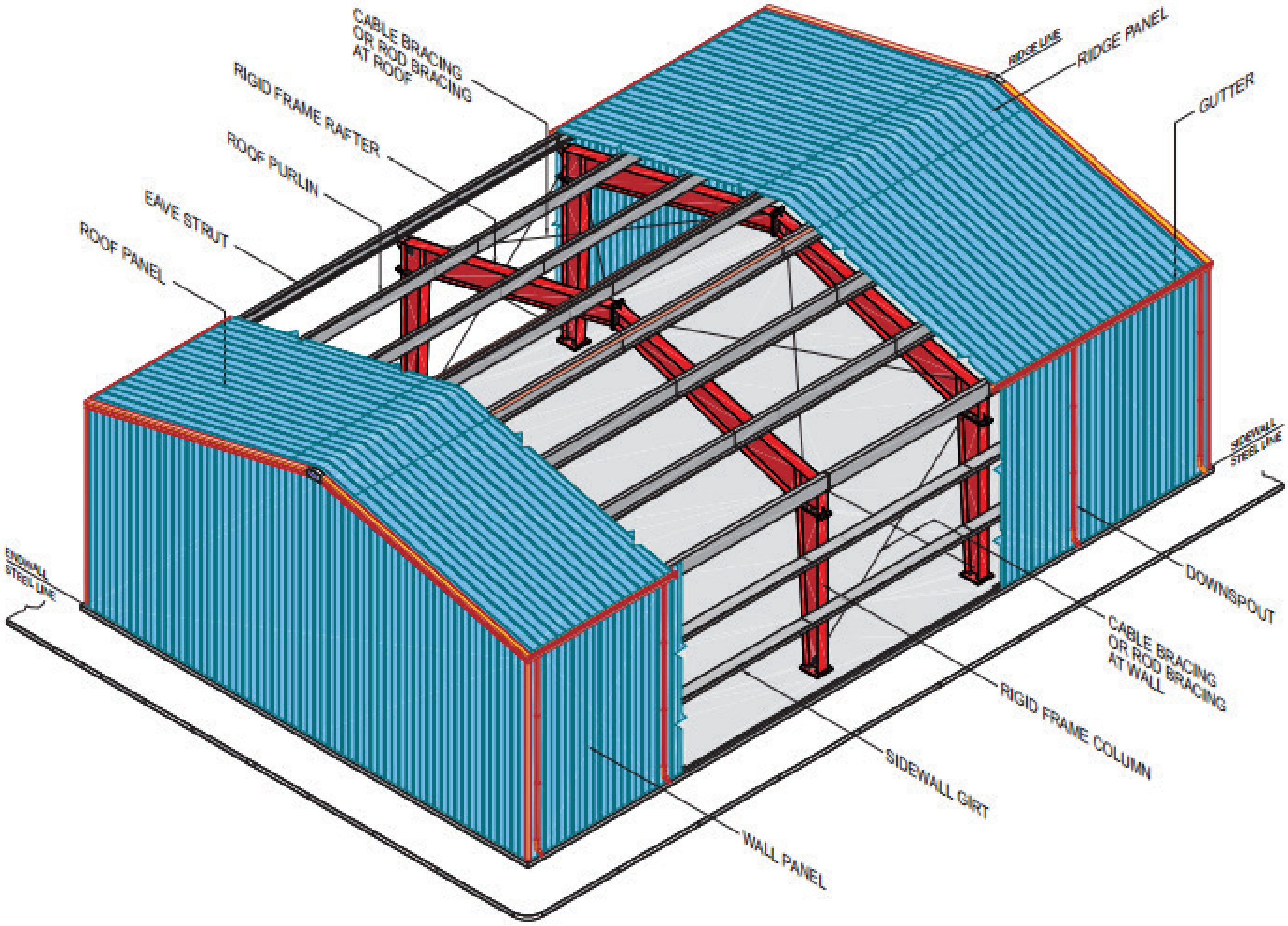


Figure 3.42 Cable or rod bracing at roof & wall of a braced bay

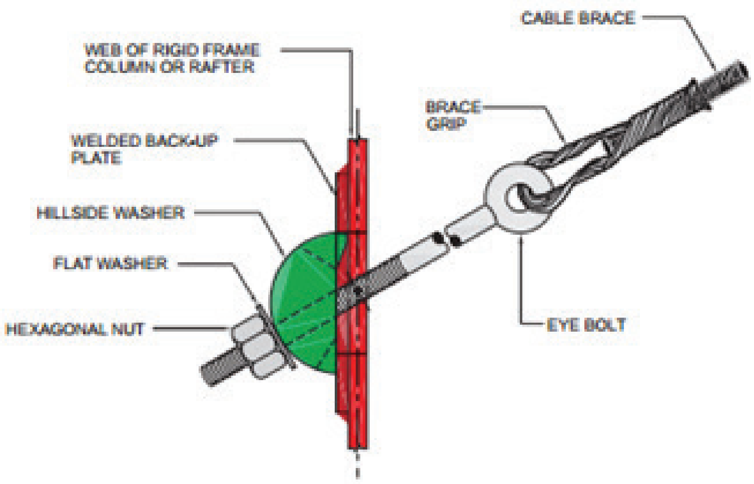


Figure 3.43 Cable bracing

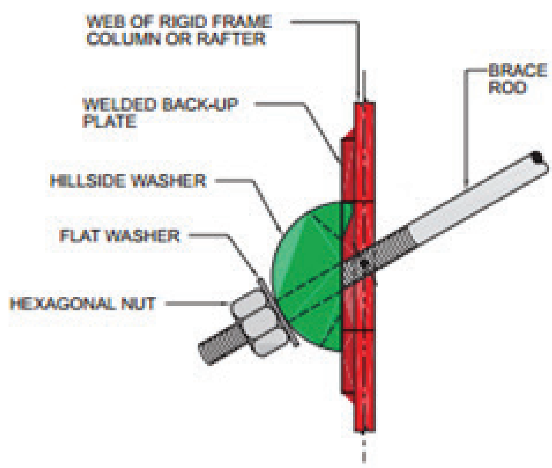


Figure 3.44 Rod bracing

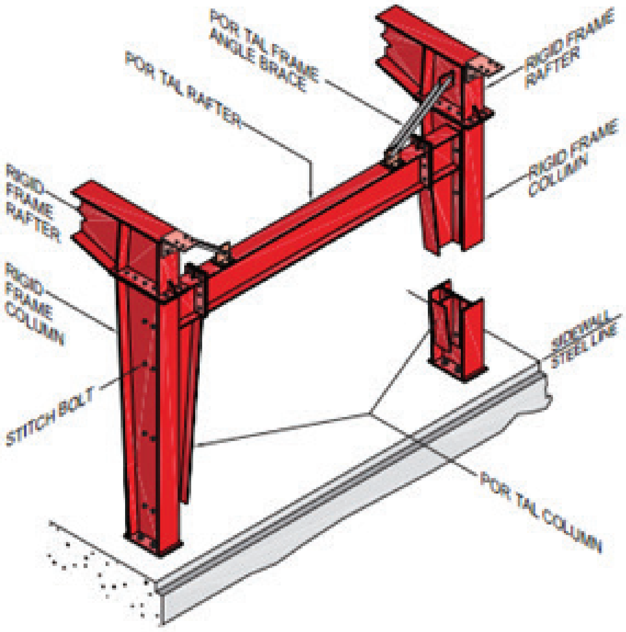


Figure 3.45 Portal frame

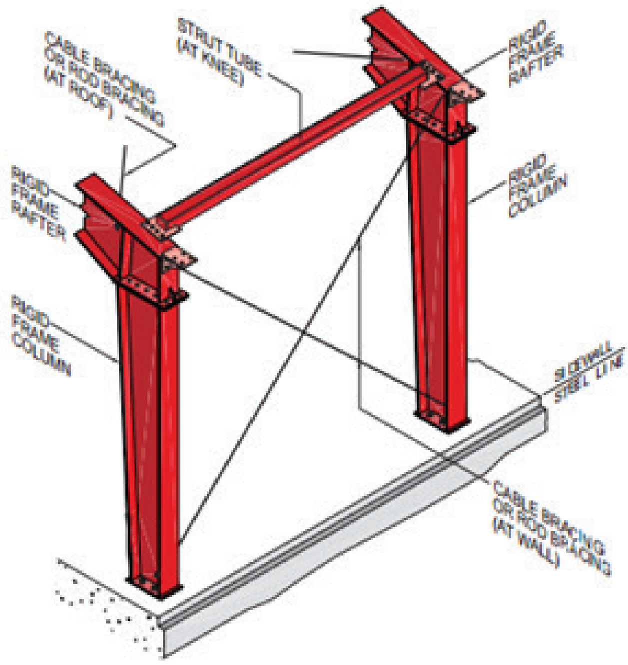


Figure 3.46 Cable or rod bracing with strut tube

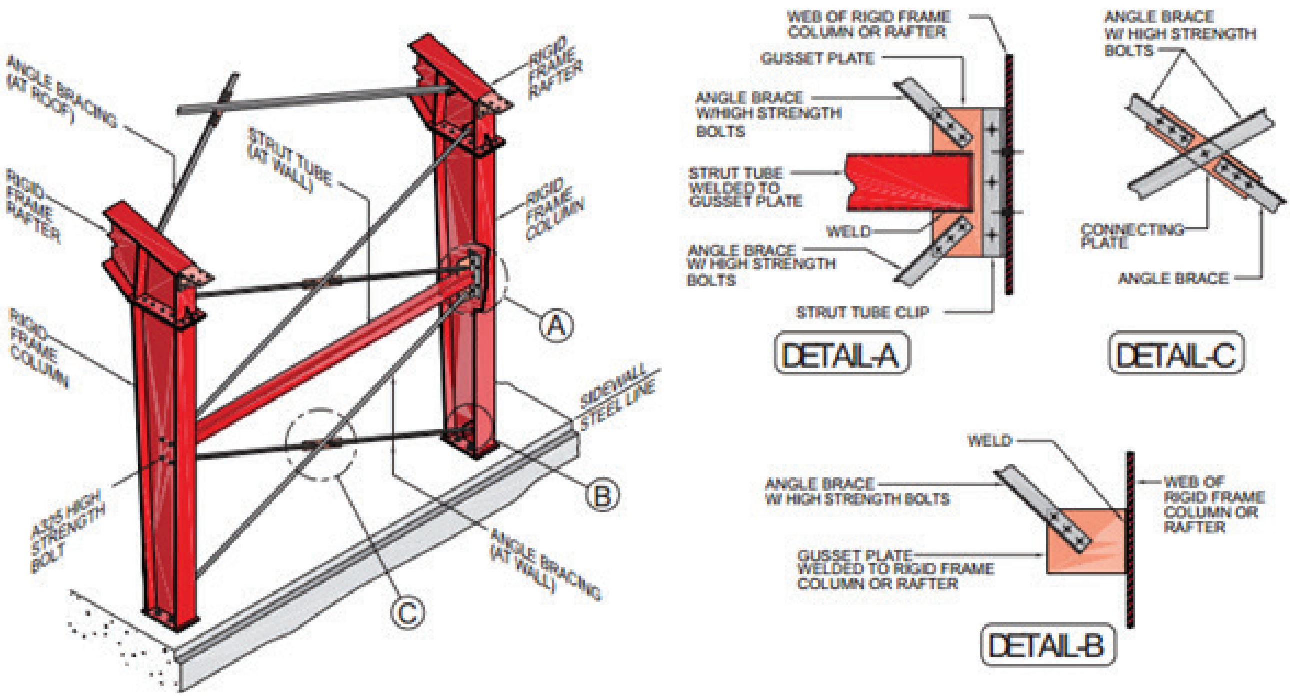


Figure 3.47 Angle bracing with strut tube

3.5.2. Wind and seismic bracing in P&B end wall

1. End wall bracing is not required for a fully sheeted P&B end wall with flush girt construction. If P&B end walls have by-framed girts then this end wall needs bracing.
2. If required, bracing in P&B end walls shall comprise cables or rods, unless otherwise specified by the customer. In such a case the end wall members shall be either built-up or hot-rolled members.
3. If an end wall requires bracing and the customer requests that no bracing to be placed in the plane of the end wall, then it is recommended that the load in the plane of the end wall is transferred back to the first rigid frame through additional roof bracing in the end bay.

3.5.3. Crane Bracing

1. In crane buildings, bracing has to be designed for longitudinal crane loads for top running or underhung cranes. The bracing shall be placed in intervals not to exceed 5 bays.
2. Longitudinal bracing for top running cranes shall be comprised of any one of the following types.
- Angles or pipes;
- Portal frame with rods (or angles);
- Portal frame without rods (or angles).
3. Longitudinal bracing for top running cranes shall be of only one type in the same longitudinal plane of a building.
4. Longitudinal bracing for underhung cranes shall consist of either rods or angles.
5. Lateral bracing for underhung cranes (attached to crane brackets), if any shall consist of either rods or angles.
6. Whenever a brace rod is used for crane bracing, the minimum diameter of that rod shall be 20mm.
7. A brace rod shall not exceed 15m in length. If angles are used the critical slenderness ratio of a bracing angle shall not exceed 300.

CHAPTER 4. MAIN FRAME AND CONNECTION

4.1. Main frame design procedure and constraints

4.1.1. Design procedure

4.1.1.1 Stress Unity Checks

Combined Stress Unity Check

$$\frac{f_a}{F_a}+\frac{f_{bx}}{F_{bx}}+\frac{f_{by}}{F_{by}}\leq 1.0$$

Where fa, fbx and fby are actual axial, major axis bending and minor axis bending stresses respectively. Fa, Fbx and Fby are corresponding allowable stresses. If section fails in combined unity check then check the allowable stresses:

(1) If **Mn/Omega Capacity** or **Pnc/Omega Capacity** are much lower than **Mn/Omega No LTB** or **Pnt/Omega Capacity** then it implies that the member is not properly braced then try one of the following:

	Pr Force	Pnc/Omega Capacity	Pnt/Omega Capacity
Axial	1252.377	2184.504	3250.850
	Mr Moment	Mn/Omega Capacity	Mn/Omega No LTB
Major Moment	3.762	327.434	327.434
Minor Moment	0.000	187.005	

Figure 4 1 Stress check in SAP

- For rafters and exterior columns (with sheeted side walls) adding flange braces with roof purlins or wall girts will adjust the allowable stresses for the unbraced flange.
- For exterior columns (without sheeted side walls) then providing EB (strut tubes) adequately connected to bracing system at an appropriate height would reduce the unbraced length and adjust the allowable stress.
- For interior I-section columns they can also be braced by means of EB if allowed and adequately connected to bracing system.
- For interior I-section columns that brace points cannot be added in the design then stress ratios can be improved by increasing flanges width or by minor adjustment in the flange thickness.
- For columns connected with mezzanine beams/joists columns are considered braced at mezzanine level.
- For columns supporting top running crane beam the columns are considered laterally braced at the level of carne beam top flange.

(2) If allowable stresses are sufficiently high and still the section is failing in unity check, then unity check ratio can be improved by increasing the following in the given order:

- Increasing the web depth
- Increasing the flange width
- Increasing the flange thickness

(3) If Shear stress unit ratio fv/Fv > 1.0 increase web thickness.

4.1.1.2 Controlling Deflections

Refer Table 2 26 Deflection Limitations for Deflection Limitations.

If lateral deflection exceeds the prescribed limit (normally H/60) then check the H/Width ratio. If H/B > 0.75 then fixing the base would result in more economical frame. If H/B<0.75 then increase the web depth at knee of both column and rafter (difference between knee depth of column and rafter < 200mm)

In multi-span frames before going for the option of fixing the exterior column at base, check whether fixing the tops of interior columns control the lateral sway. If not then fix the exterior column bases.

If Vertical deflection Δv exceeds the prescribed limit (normally Span/180), increase the web depth at knee of both column and rafter. A slight increase in the rafter depth at ridge will also help control the vertical deflection.

4.1.2. Design constraints

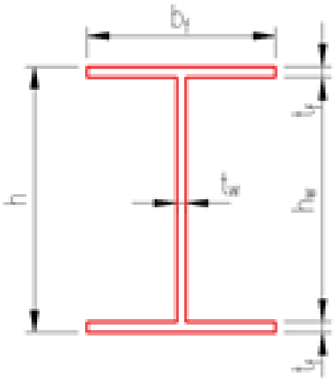
4.1.2.1 Standard

$$\frac{h_w}{t_w} < 180\ ;\ \frac{b_f}{t_f} < 31$$

$$\frac{h}{b_f} < 5;\ \frac{t_f}{t_w} < 2.5$$

Compression element: $\frac{kL}{r} < 200$

Tension element: $\frac{kL}{r} < 300$



4.1.2.2 Fabrication limitation for built-up section

Web thickness	Minimum	4mm: beam, rafter
		5mm: column
	Maximum	50mm but give priority to 12mm
Flange thickness	Minimum	5mm: beam, rafter
		6mm: column
	Maximum	50mm but give priority to 12mm
Web depth	Minimum	200mm
	Maximum	1500mm
Flange width	Minimum	134mm
	Maximum	746mm

NOTE

- Width of continuous flange should be constant along the one welded piece.
- Variation of thickness at any butt weld splice of continuous flange/web within the one welded piece should be limited to maximum 6mm.
- Width flange/web having constant width, use steel plate gouge like below table.

Table 4 1 Size of plate

Width (mm)	÷													
	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1200	600	400	300	240	200	171	150	133						
	596	397	298	238	198	169	148	131						
1500	750	500	375	300	250	214	188	167	150	136				
	746	496	372	298	248	212	184	164	148	134				
2000	1000	667	500	400	333	286	250	222	200	182	167	154	143	133
	996	663	496	397	330	284	248	220	198	180	165	152	141	131

4.1.2.3 Shipping limitation

Maximum fabricated out-to-out length of the piece is **14m** for transportation by truck (in Vietnam), and **11.7m** for transportation by dry cargo container (foreign) except **13.5m** for Cambodia.

4.1.2.4 Other guidelines

(1) At knee connection, maximum difference between column depth and rafter depth is 200mm.
(2) In a tapered section, the minimum difference in web depth at start and end should be 100mm.
(3) Minimum base plate thickness = 14 mm.
(4) Minimum base plate width = 164 mm.
(5) Minimum splice plate thickness = 12 mm.
(6) Minimum splice plate width = 164 mm.
(7) Minimum anchor bolt diameter = M20 (except end-wall post M16).
(8) Minimum splice bolt = M16.

4.1.2.5 Optimization

To produce the most economical frame profiles, let apply the following rules:

(1) Minimize number of splices in the columns and rafters by providing maximum possible lengths regardless of the material savings that can be produced otherwise. Section lengths should be multiple of 3m i.e., 3m, 6m, 9m and 12m in order to reduce the scrap.

(2) In case of different bay spacing avoid using more than 3 frames.

(3) Different frame should be adopted if saving of 5% on all frames with a minimum of 1.0ton is ascertained.

(4) When different frames have to be used due to different bay spacing, maintain the same web cuts for all such frames.

(5) Minimize the number of different flange widths in a frame. Maximum different widths of flanges in all the frames should preferably be less than three.

(6) As much as possible maintain uniformity in the base plate detail and anchor bolt sizes for all the frames.

(7) Try to locate the splices at the locations where the bending moment is least and/or where the depth is least in a frame.

(8) Try to follow the shape of bending moment diagram for the controlling load combination in the configuration of the frame by maintaining the stress unity check ratios closer to 1.

4.1.3. Instruction to build Sap2000 Model

This section provides step-by-step instructions for building a basic SAP2000 model.

Defining

Units and geometry

Material and member section properties

Load patterns, load cases and load combinations

↓

Modeling

Draw/Edit Frame Objects and Bracing System

Assigning/Edit Section Properties, Releases & Restraints.

Assign Load

Assign Unbraced Length Ratio (ULR)

↓

Analyzing

Set Analysis Option

Set Load Cases to Run form

↓

Display the result

↓

Design Steel Frame

Check Deflection Limitation

4.1.3.1 Defining

In this Step, the basic grid that will serve as a template for developing the model will be defined. Then a material will be defined and sections will be selected.

a. Setting Default Units (KN, m, C) through: **File > New Model Form.**

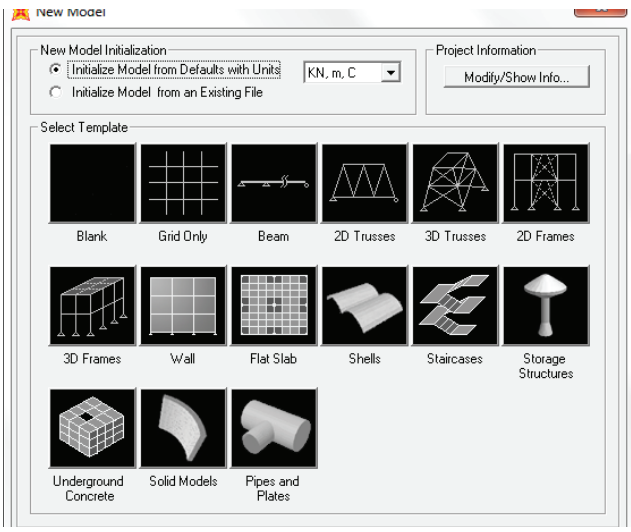


Figure 4 2 New model form

b. Setting up geometry in 2 ways:

(1) Creating Generally Grid System: **New Model form > Grid Only**

(2) Define Grid System Data: **Define>Coordinate Systems/Grids>GLOBAL>Modify/Show System**

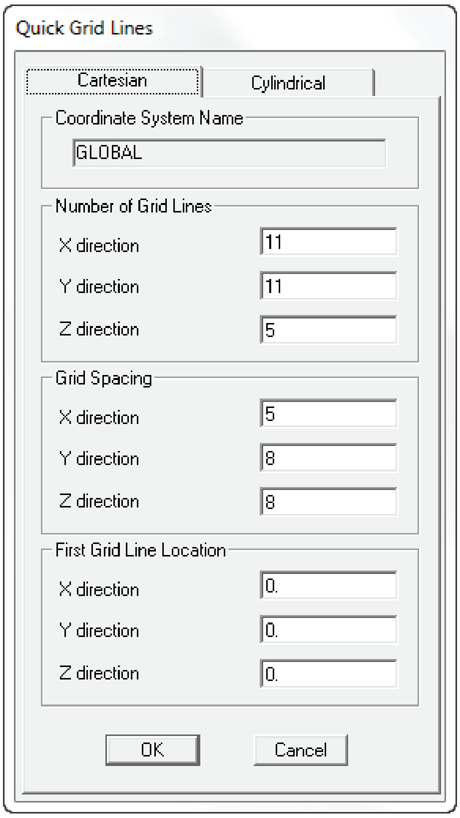


Figure 4 3 Quick Grid Lines form

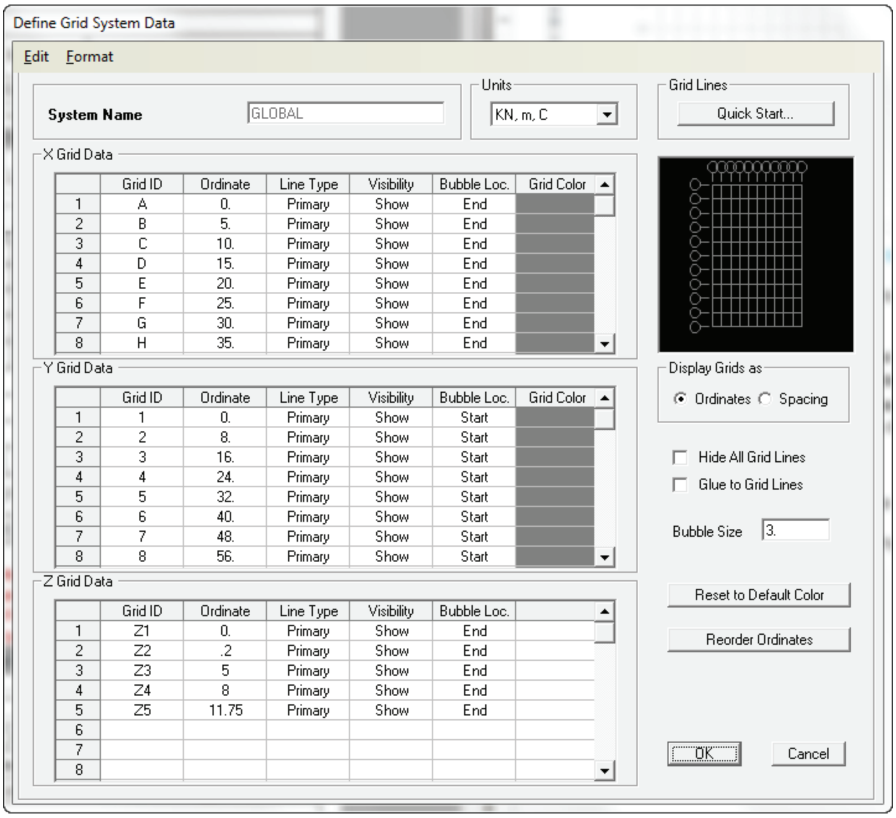
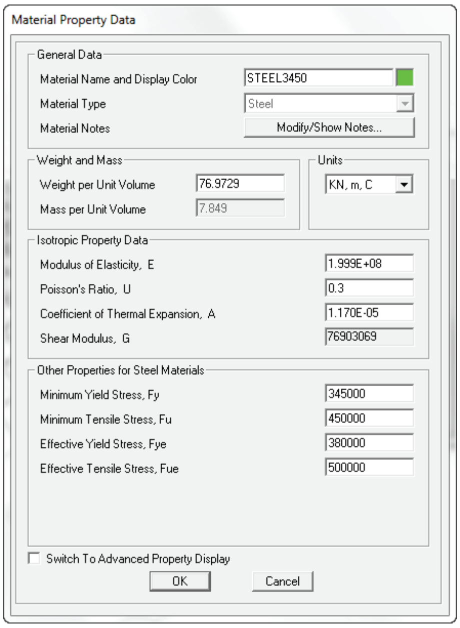
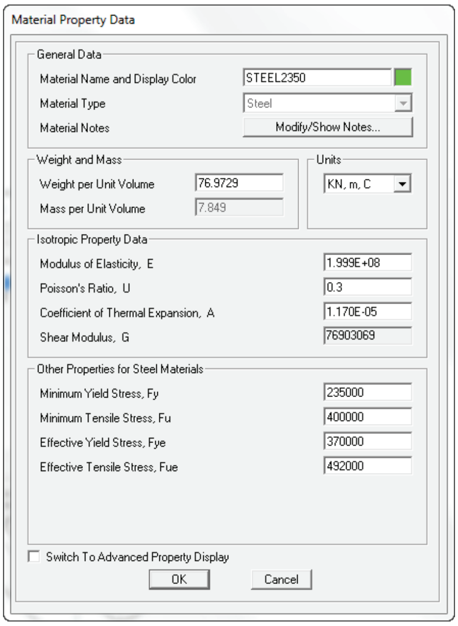


Figure 4 4 Define Grid System Date form

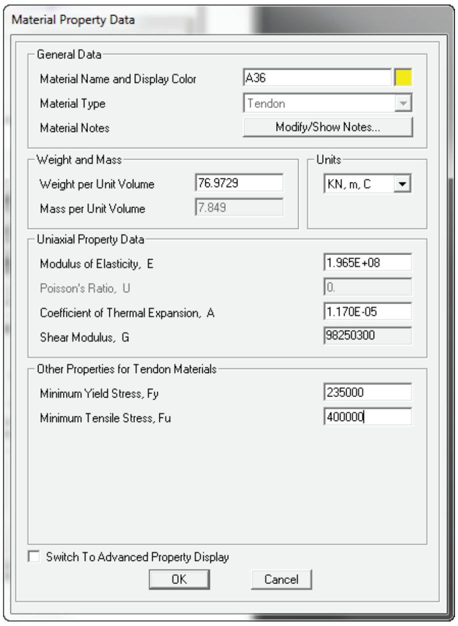
c. Define Materials through: **Define > Materials**



STEEL3450



STEEL2350



A36 (Tendon)

Figure 4 5 Material Property Data

d. Define Frame Sections

Define Frame Section through: **Define/Section Properties/Frame Sections**

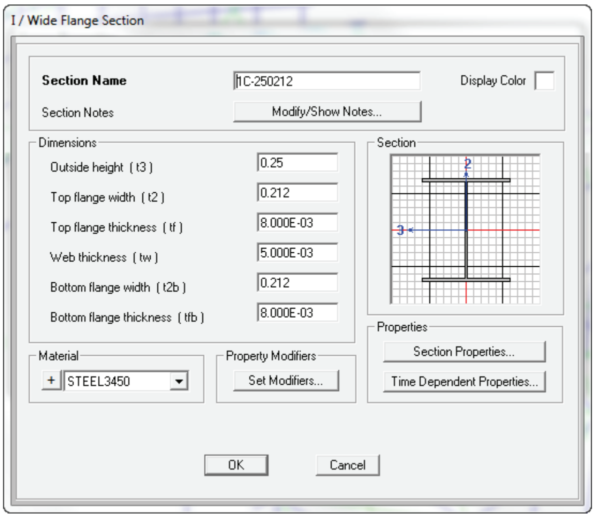


Figure 4 6 I/Wide Flange Section form

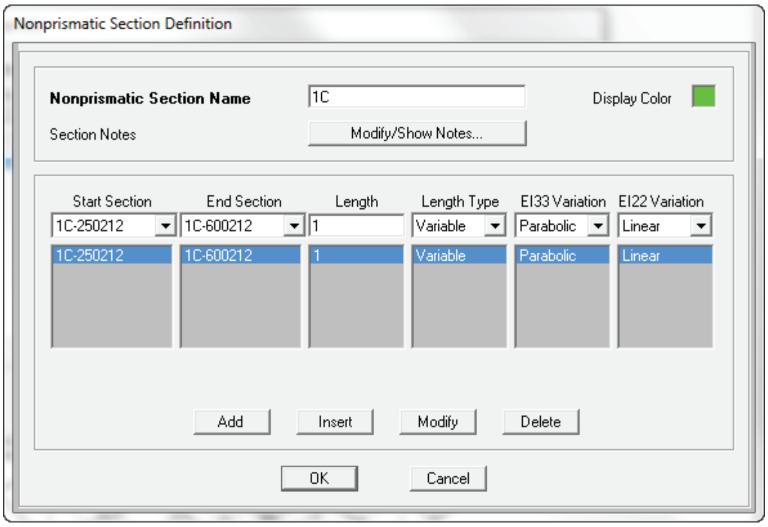


Figure 4 7 Nonprismatic Section Definition form

Define Tendon Section through: **Define/Section Properties/Tendon Sections**

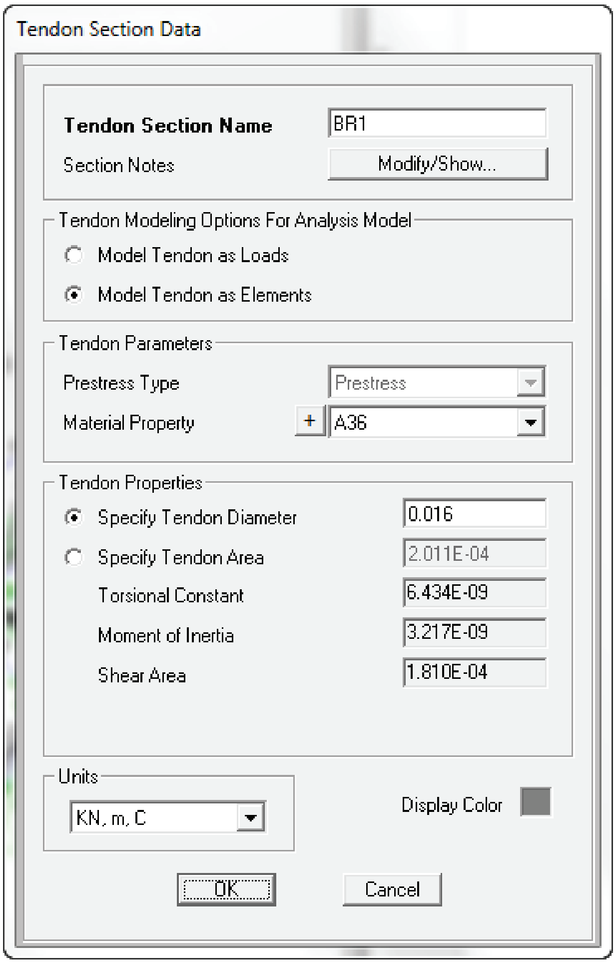


Figure 4 8 Tendon Section Data form (Diameter 16mm)

e. Define Load patterns, load cases and load combinations

- Define Load Patterns through: **Define > Load Patterns**
- Define Load Cases through: **Define > Load Cases**
- Define Load Combinations through: **Define > Load Combinations** or using ***.s2k** text file

In "**1A - LOAD APPLICATION AND PURLIN - Under 18m**" file, go to sheet "2" and enter the number for each load pattern, click **COMBINATION** then click **EXPORT TO .TXT** which will display a window to specify position for saving file. Click the Save button to save file.

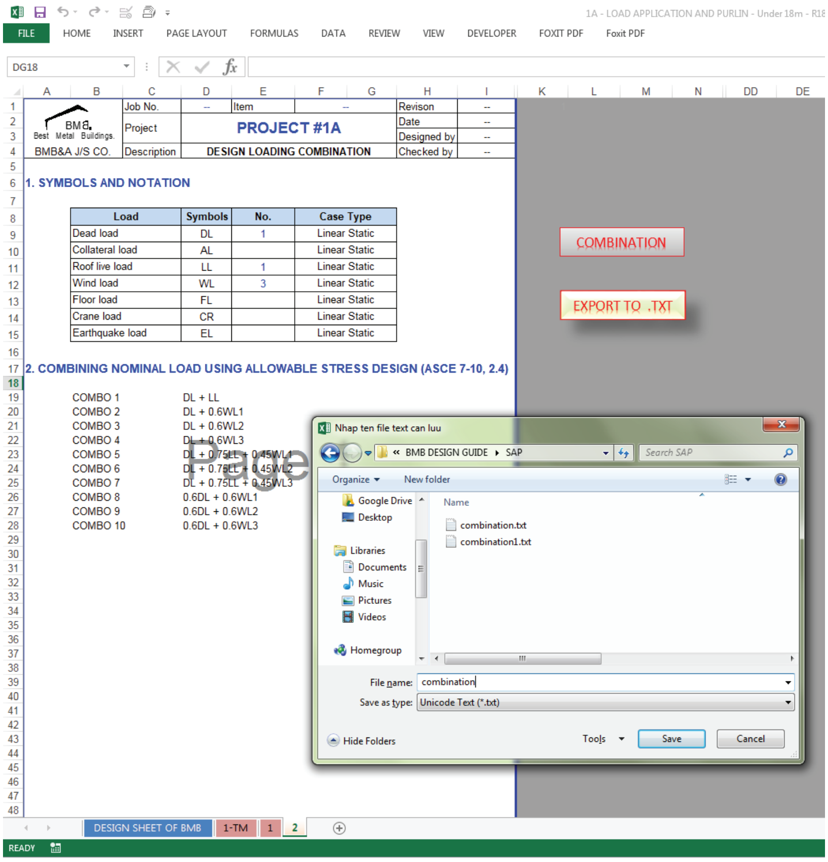


Figure 4 9 Design loading combination sheet

In SAP2000, you have to define at least one load combination before export .s2k text file. Click the **Define > Load Combinations** command, in the **Define Load Combinations** form which has just appeared, click the **Add New Combo** button and create an combination as shown in figure below.

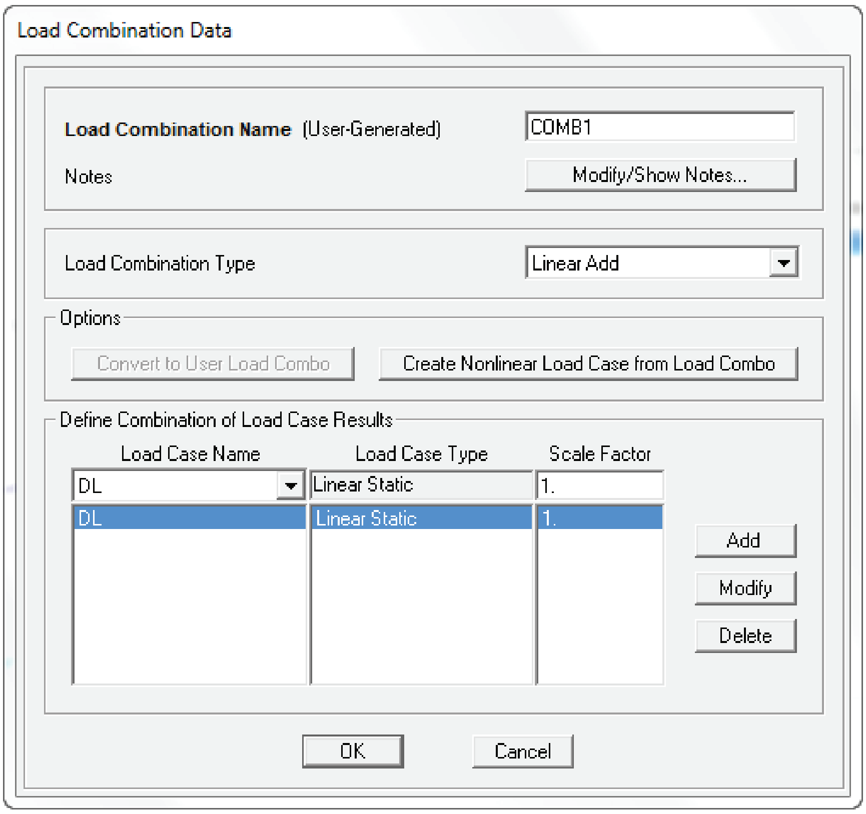


Figure 4 10 Create load combination COMB1

After create any combination, click the **File > Export > Sap2000 .s2k Text File** command to display the **Choose Tables for Export to Text File** form shown in figure below.

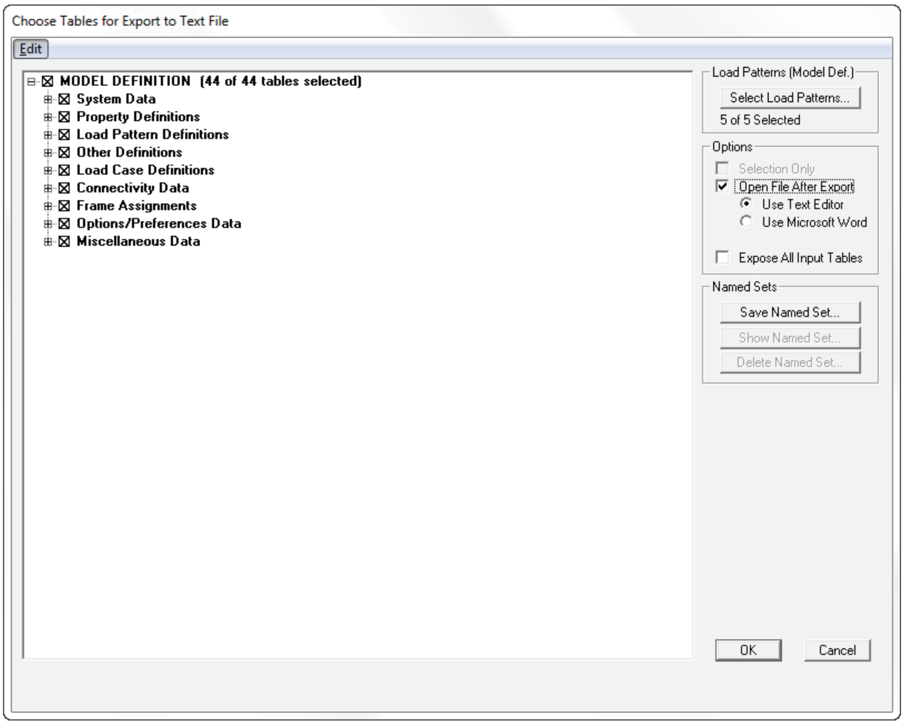
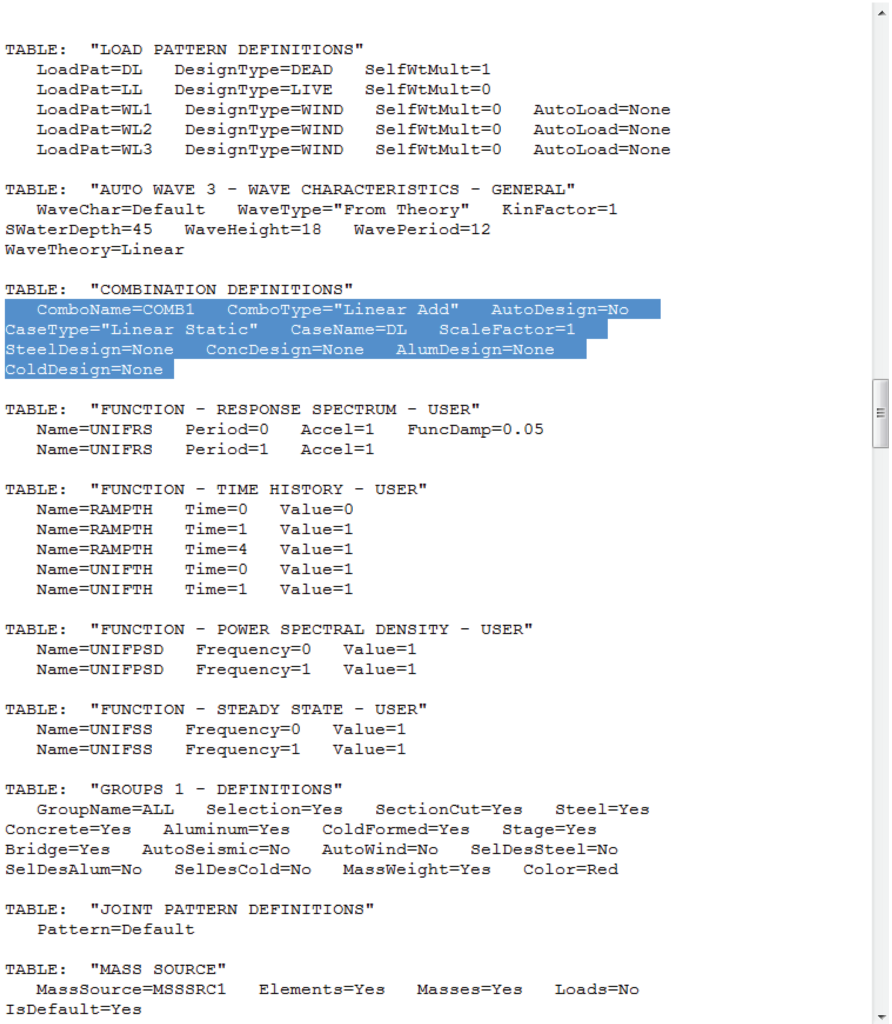


Figure 4 11 Choose Tables for Export to Text File form

Make sure that all tables, load patterns, and Open File After Export option are selected before click the **OK** button. Copy all data in combination.txt text file and paste to .s2k text file at highlight text shown in figure below.



```
TABLE:  "LOAD PATTERN DEFINITIONS"
   LoadPat=DL   DesignType=DEAD   SelfWtMult=1
   LoadPat=LL   DesignType=LIVE   SelfWtMult=0
   LoadPat=WL1   DesignType=WIND   SelfWtMult=0   AutoLoad=None
   LoadPat=WL2   DesignType=WIND   SelfWtMult=0   AutoLoad=None
   LoadPat=WL3   DesignType=WIND   SelfWtMult=0   AutoLoad=None

TABLE:  "AUTO WAVE 3 - WAVE CHARACTERISTICS - GENERAL"
   WaveChar=Default   WaveType="From Theory"   KinFactor=1
SWaterDepth=45   WaveHeight=18   WavePeriod=12
WaveTheory=Linear

TABLE:  "COMBINATION DEFINITIONS"
   ComboName=COMB1   ComboType="Linear Add"   AutoDesign=No
CaseType="Linear Static"   CaseName=DL   ScaleFactor=1
SteelDesign=None   ConcDesign=None   AlumDesign=None
ColdDesign=None

TABLE:  "FUNCTION - RESPONSE SPECTRUM - USER"
   Name=UNIFRS   Period=0   Accel=1   FuncDamp=0.05
   Name=UNIFRS   Period=1   Accel=1

TABLE:  "FUNCTION - TIME HISTORY - USER"
   Name=RAMPTH   Time=0   Value=0
   Name=RAMPTH   Time=1   Value=1
   Name=RAMPTH   Time=4   Value=1
   Name=UNIFTH   Time=0   Value=1
   Name=UNIFTH   Time=1   Value=1

TABLE:  "FUNCTION - POWER SPECTRAL DENSITY - USER"
   Name=UNIFPSD   Frequency=0   Value=1
   Name=UNIFPSD   Frequency=1   Value=1

TABLE:  "FUNCTION - STEADY STATE - USER"
   Name=UNIFSS   Frequency=0   Value=1
   Name=UNIFSS   Frequency=1   Value=1

TABLE:  "GROUPS 1 - DEFINITIONS"
   GroupName=ALL   Selection=Yes   SectionCut=Yes   Steel=Yes
Concrete=Yes   Aluminum=Yes   ColdFormed=Yes   Stage=Yes
Bridge=Yes   AutoSeismic=No   AutoWind=No   SelDesSteel=No
SelDesAlum=No   SelDesCold=No   MassWeight=Yes   Color=Red

TABLE:  "JOINT PATTERN DEFINITIONS"
   Pattern=Default

TABLE:  "MASS SOURCE"
   MassSource=MSSSRC1   Elements=Yes   Masses=Yes   Loads=No
IsDefault=Yes
```

Figure 4 11 Choose Tables for Export to Text File form

Save and close file. Return to SAP and import this file. Click the **File > Import > SAP2000 .s2k Text File** command to display **Import Tabular Database** shown in figure below. Click the OK button, specify created .s2k text file in appeared window and click the Done button to import file.

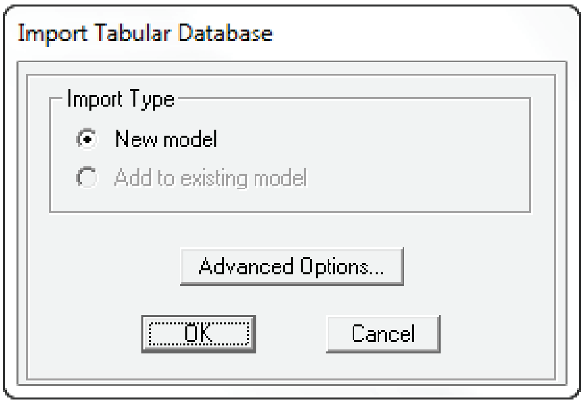


Figure 4 13 Import Tabular Database form

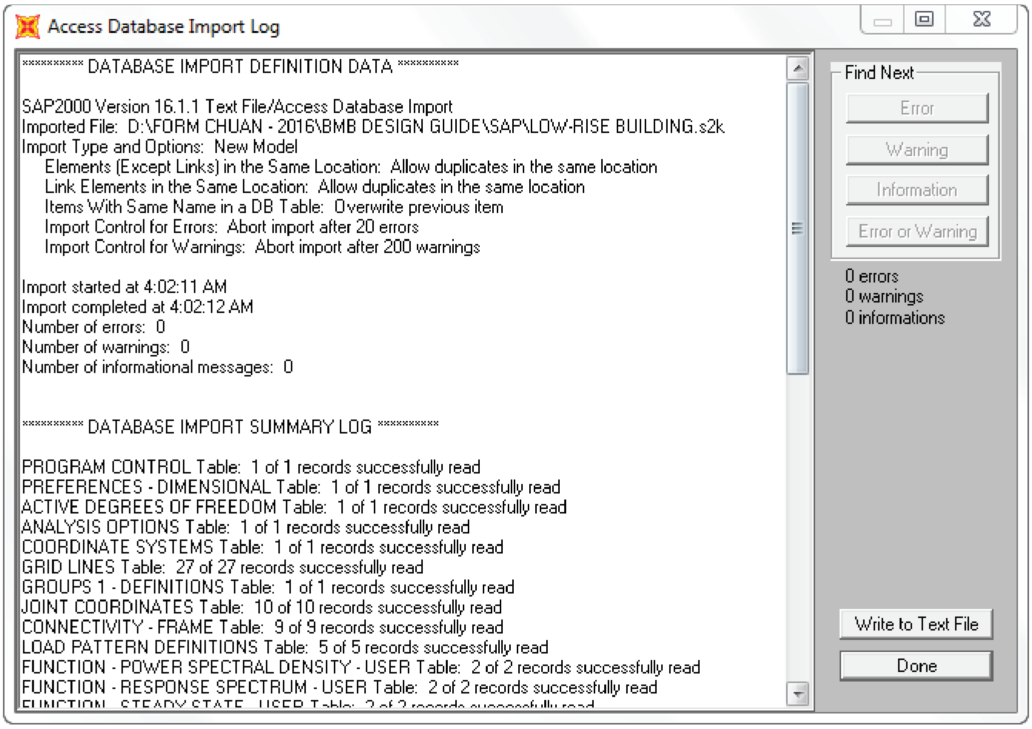


Figure 4 14 Click the Done button to finish importing .s2k text file

4.1.3.2 Modeling

a. Draw Frame Objects

In this step, frame objects with the associated sections previously defined are drawn using the grids and snap-to options, and generated using **Edit>Draw Frame/Cable.**

The **Properties of Object** pop-up form for frames will appear as shown in figure below.

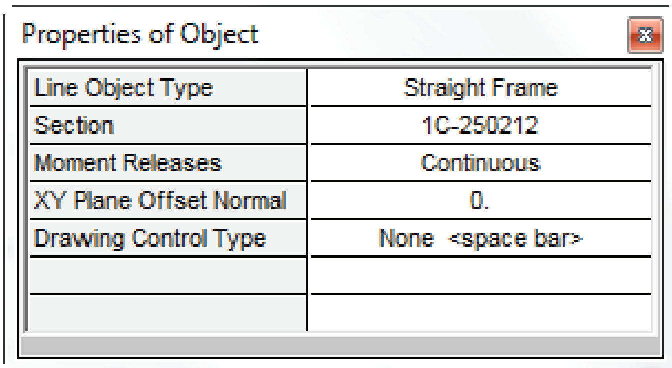


Figure 4 15 Properties of Object form

Use default values as seen in figure above to draw because we will assign information to frames after dividing them with appropriate length.

b. Draw Bracing System

To draw EB and Tendon/Cable system, using Draw Frame/Cable button.

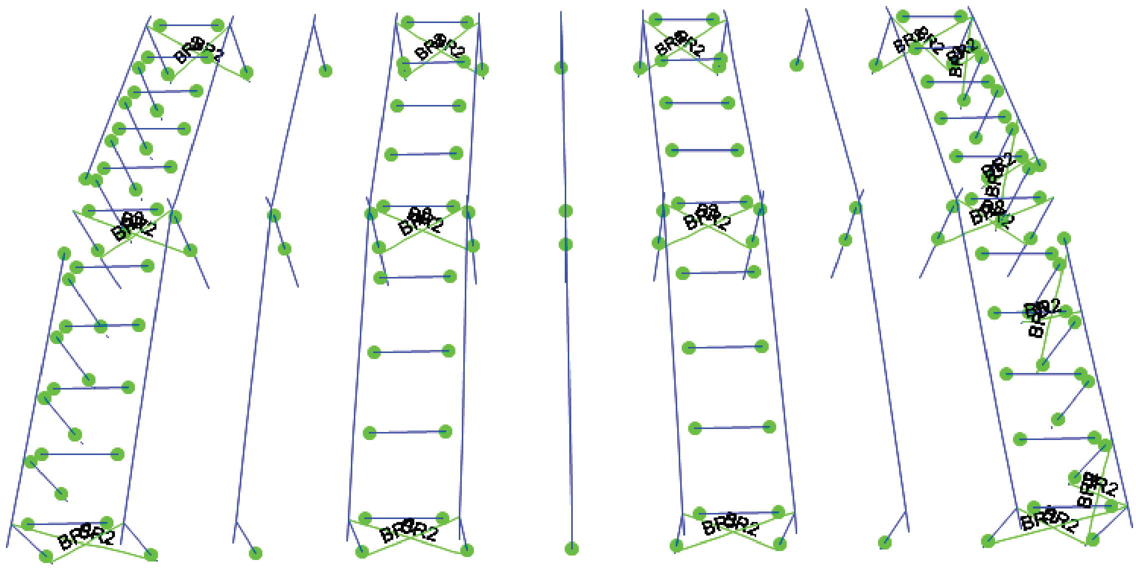


Figure 4 16 Bracing System

c. Edit Modeling

Divide frame objects

SAP2000 allows members to be sub-divided into multiple objects after they are drawn to accommodate changes in geometry (this differs from the internal meshing done during analysis where the number of objects remains the same).

Select rafters and vertical lines, then click the **Edit > Edit Lines > Divide Frames** command to show Divide Selected Frames form. In this form, make sure that Break at intersection with selected Joints, Frames, Area Edges and Solid Edges is selected before click the OK button.

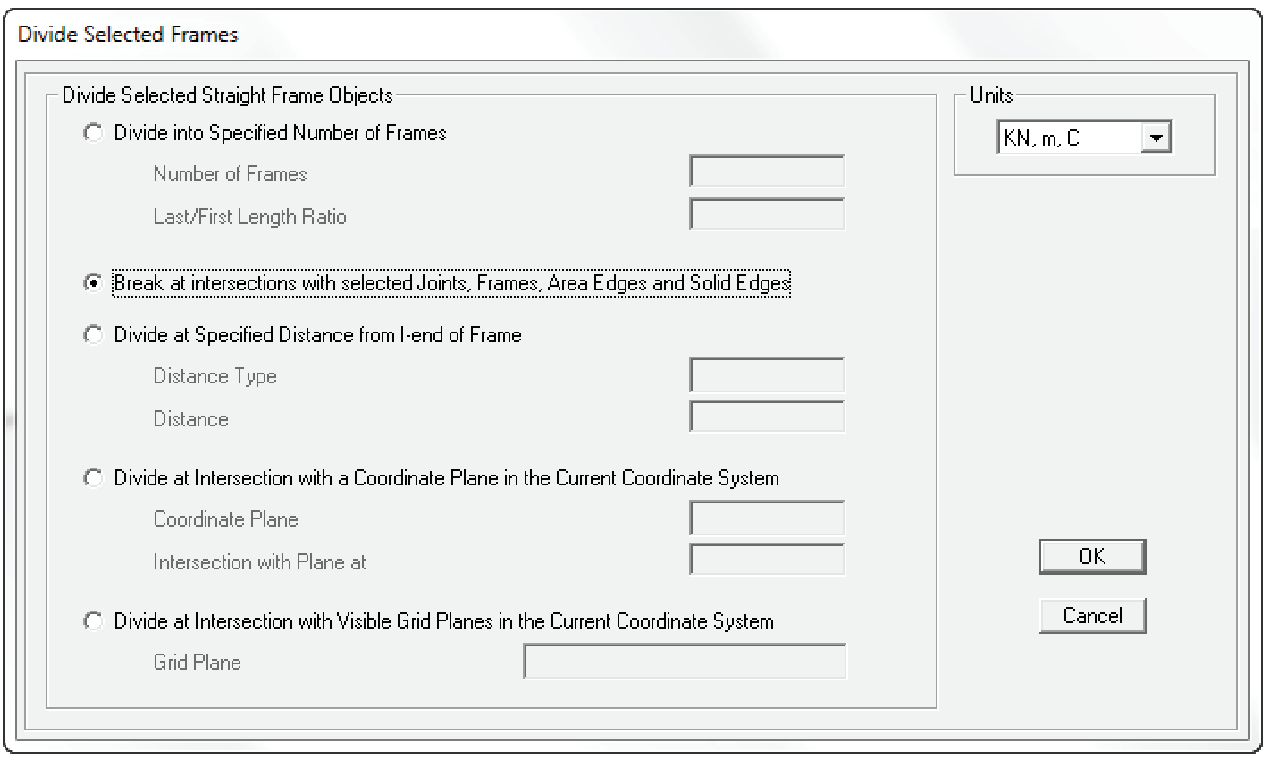


Figure 4 17 Divide Selected Frames form

Replicate Frame through Edit > Replicate.

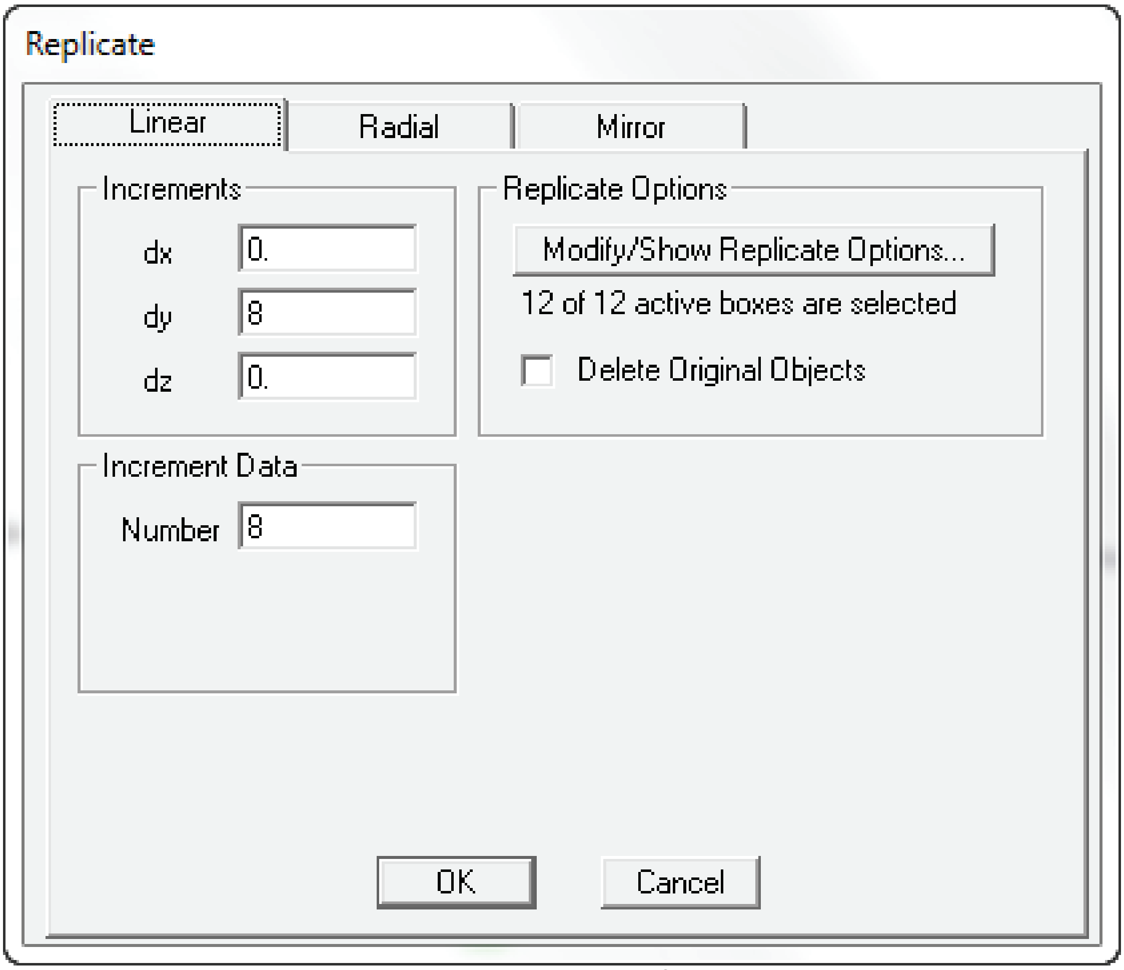


Figure 4 18 Replicate form

Assign member end releases and restraint

- Assign Frame Releases through: **Assign > Frame > Releases/Partial Fixity**
- Assign Joint Restraints through: **Assign > Joint > Restraints**
- Assign Frame Sections through: **Assign > Frame > Frame Sections**
- Assign Unbraced Length Ratio (ULR):

ULR is specified as a fraction of the frame object length. Multiplying these factor times the frame object length gives the unbraced length for the object. There are three types of ULR include ULR in major axis, minor axis, and lateral torsional buckling.

(1) **ULR (major)** is ULR for buckling about the frame object major axis (occur when object is compressed). This item is taken to be the maximum distance between two major bracing divided by actual length of object and it is usually specified as Program Determined.

(2) **ULR (minor)** is ULR for buckling about the frame object minor axis (occur when object is compressed). This item is taken to be the maximum distance between two bracing points (bracing member frame into web or flange of the braced member) divided by actual length of object.

(3) **ULR (LTB)** is ULR for lateral-torsional buckling for the frame object (occur when object is flexed). This item is taken to be the maximum distance between two bracing points (bracing member frame into flange of the braced member) divided by actual length of object.

NOTE

Assume purlin spacing and girt spacing is **1.5m.**

With rafters, bracing members are purlins and flange braces. To conservative, just locations that have both purlin and flange brace are considered as bracing point.

Therefore, unbraced length is spacing between flange braces.

With outer columns, bracing members are girts and flange braces.

Therefore, unbraced length is the greater value of spacing between flange brace or distance from ground to the first flange brace.

With inner columns, bracing members are longitudinal steel members and portal frame. Unbraced length is the largest of unbraced lengths.

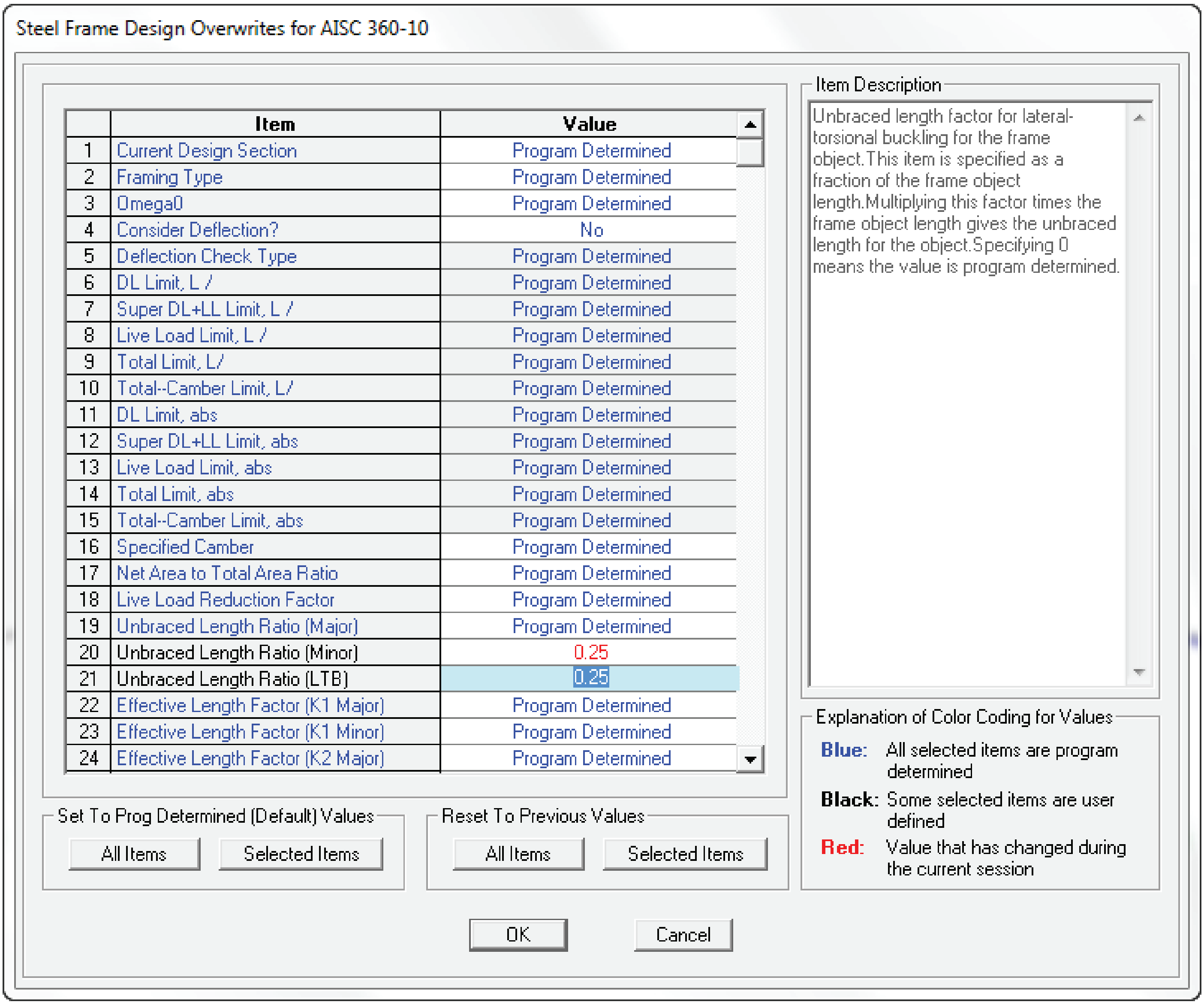


Figure 4 19 Steel frame design overwrites form

d. Assign loads through Assign/Frame Loads.

In this step, the dead, live and wind loads will be applied to the model.

Calculate frame loads which will be assigned to frame using "**1A - LOAD APPLICATION AND PURLIN - Under 18m**" file.

NOTE

It will be easier to assign frame load when frames are not be divided into different length members.

4.1.3.3 Analyzing

In this part SAP2000 will assemble and solve the global matrix. The following steps are needed:
Setting Analysis Option through: **Analysis > Set Option:** check the available DOFs. If you are analyzing a plane truss, check UX and UY, leave the UZ, RX, RY and RZ blank.
From the **Analysis** menu, select **Run**.Click the **Run Now** button to start running.

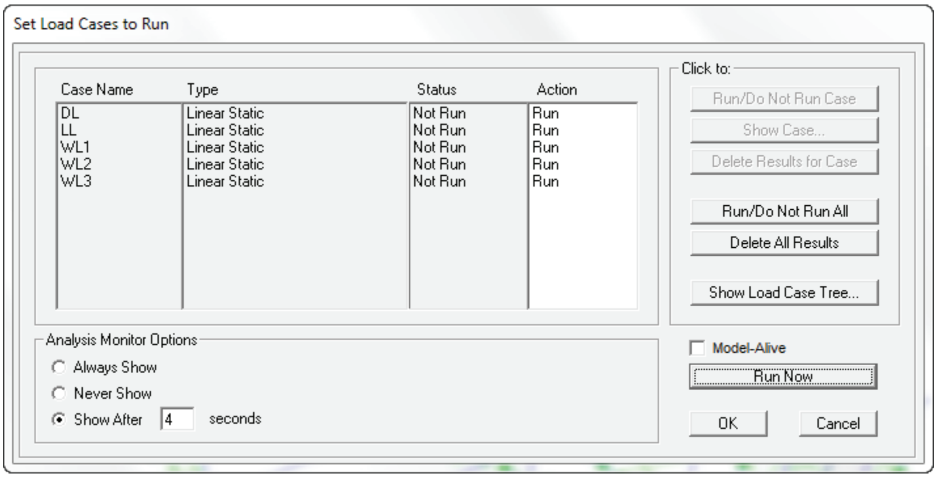


Figure 4 20 Set Load Cases to Run form

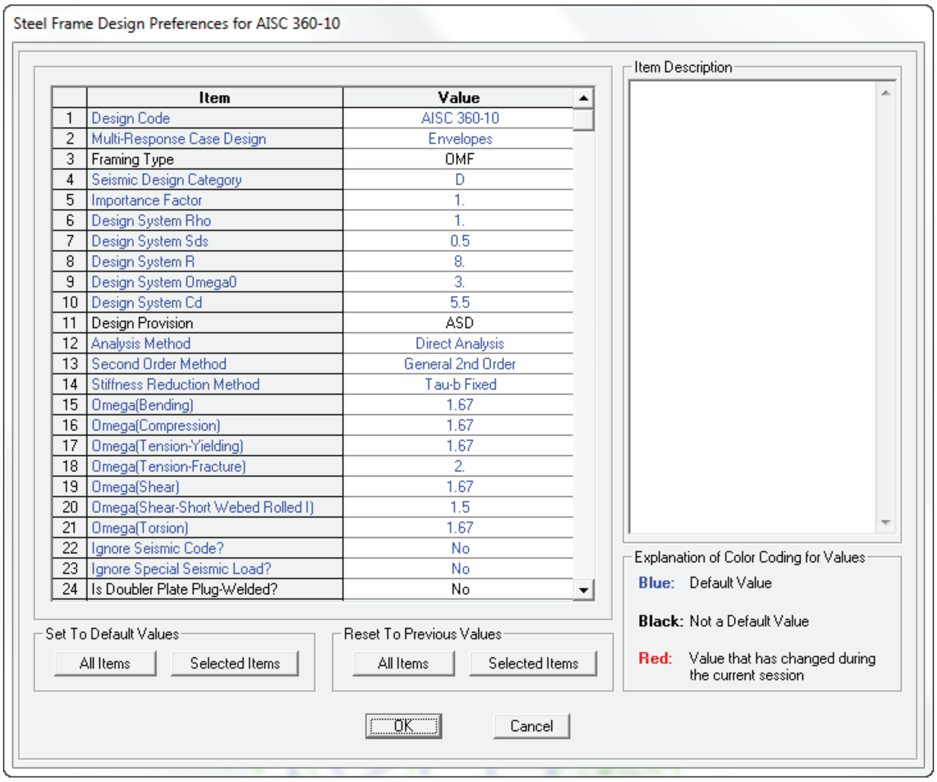
When the analysis is finished, the program automatically displays a deformed shape view of the model, and the model is locked. Check the general stability of model by clicking the Start Animation button at the lower right hand of monitor. If any member is deformed unusually, you should unlock model and revise it.

4.1.3.4 Display the result

- Displaying the deformed shape through: **Display > Show Deformed Shape**
- Return normal model through: F4 or the **Display > Show Undeformed Shape**
- Show Model Definition or Analysis Result through: Ctrl+T or the **Display > Show Table**

4.1.3.5 Design Steel Frame

Click the **Design menu > Steel Frame Design > View/Revise Preferences** command.
The **Steel Frame Design Preferences** form shown in figure below displays.



	Item	Value
1	Design Code	AISC 360-10
2	Multi-Response Case Design	Envelopes
3	Framing Type	OMF
4	Seismic Design Category	D
5	Importance Factor	1.
6	Design System Rho	1.
7	Design System Sds	0.5
8	Design System R	8.
9	Design System Omega0	3.
10	Design System Cd	5.5
11	Design Provision	ASD
12	Analysis Method	Direct Analysis
13	Second Order Method	General 2nd Order
14	Stiffness Reduction Method	Tau-b Fixed
15	Omega(Bending)	1.67
16	Omega(Compression)	1.67
17	Omega(Tension-Yielding)	1.67
18	Omega(Tension-Fracture)	2.
19	Omega(Shear)	1.67
20	Omega(Shear-Short Webed Rolled I)	1.5
21	Omega(Torsion)	1.67
22	Ignore Seismic Code?	No
23	Ignore Special Seismic Load?	No
24	Is Doubler Plate Plug-Welded?	No

Figure 4 21 Steel Frame Design Preferences form

Make sure that the Design Code is set to **AISC 360-10**, the Framing Type is set to **OMF**(ordinary-moment-frame) and the Design Provision is set to **ASD**.

Review the information contained in the other items and then click OK to accept the selections.

Click the **Design menu > Steel Frame Design > Select Design Combos** command to access the Design Load Combinations Selection form. Uncheck **Automatically Generate Code-Based Design Load Combinations**, then click OK to accept the changes.

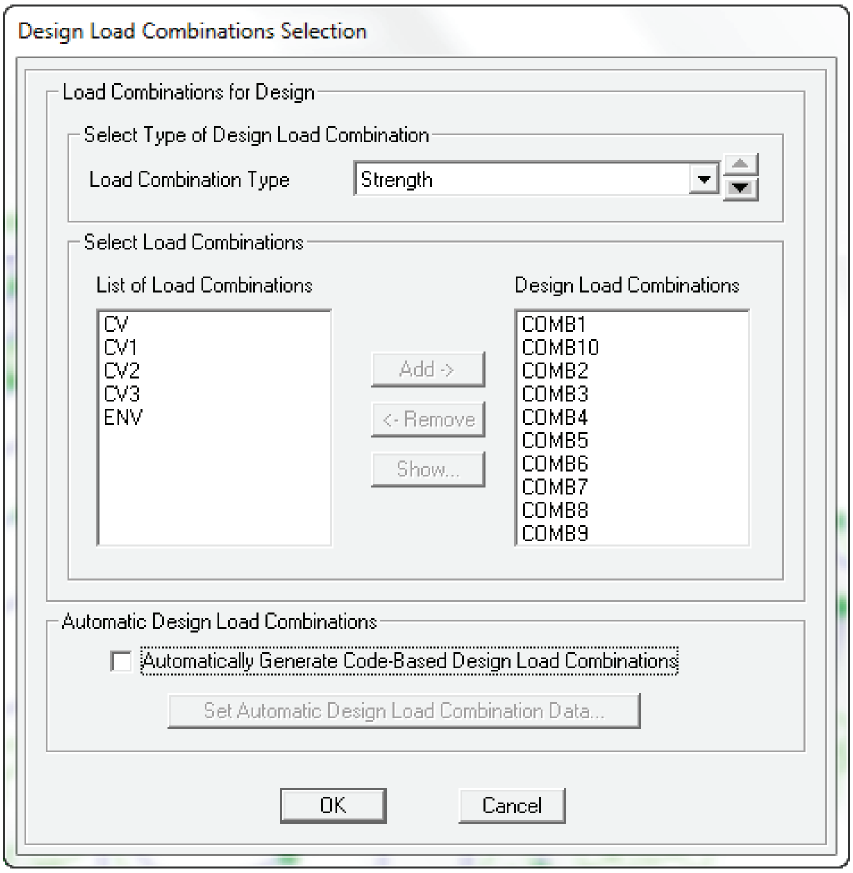


Figure 4 22 Design Load Combinations Selection form

Click the **Design menu > Steel Frame Design > Start Design/ Check of Structure** command to start the steel frame design process. When the design is complete, stress ratio are displayed on the model as a color pallet.

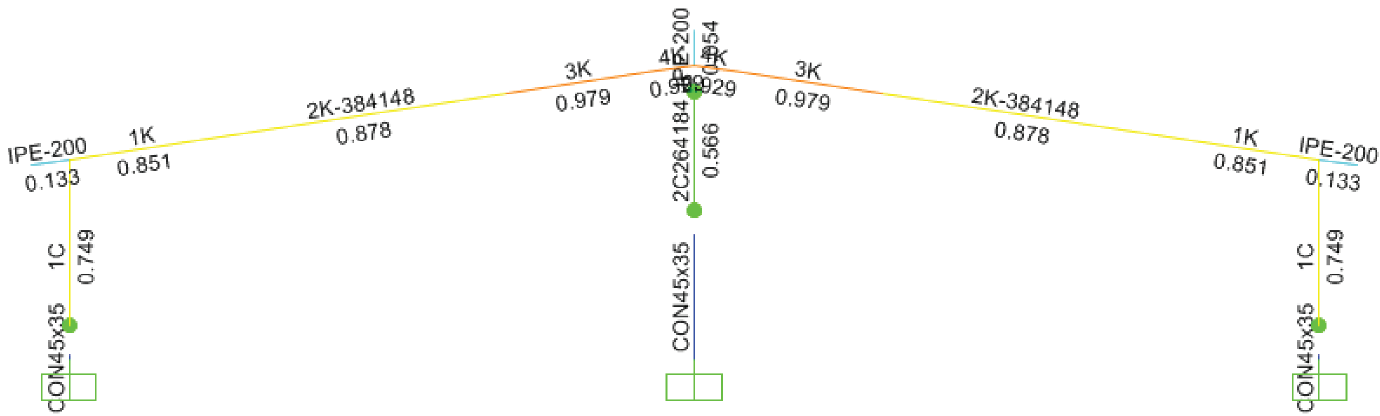
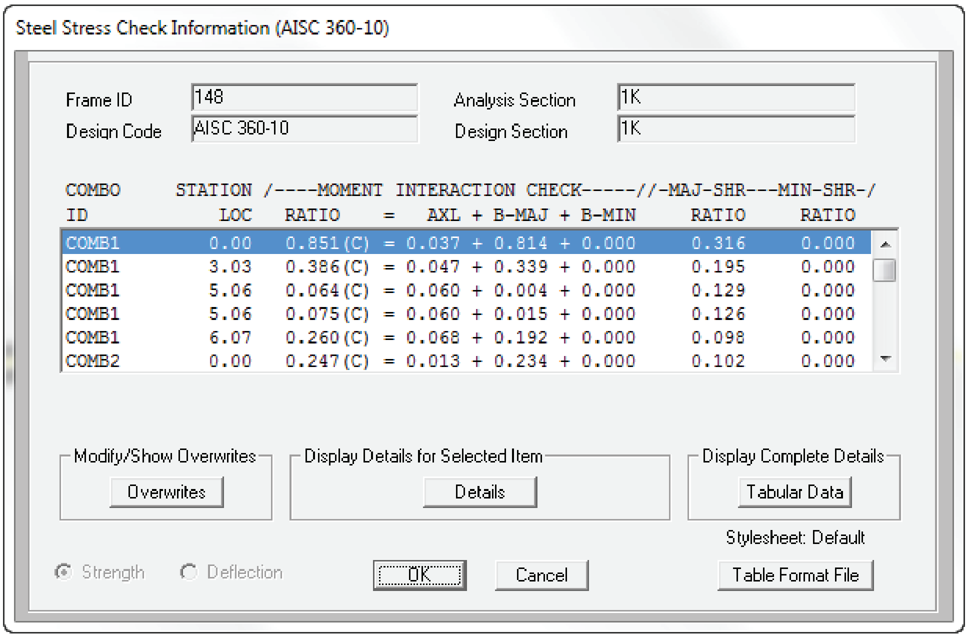


Figure 4 23 Model after checking

Click the **Design menu > Steel Frame Design > Display Design Info** command to show **Display Steel Design Results (AISC 360-10)** form. In the **Design Output** drop-down list, select P-M Ratio Colors & Values, then click OK to accept.

To see detail information of checking strength, right click on the **1K** member to show the **Steel Stress Check Information** form.



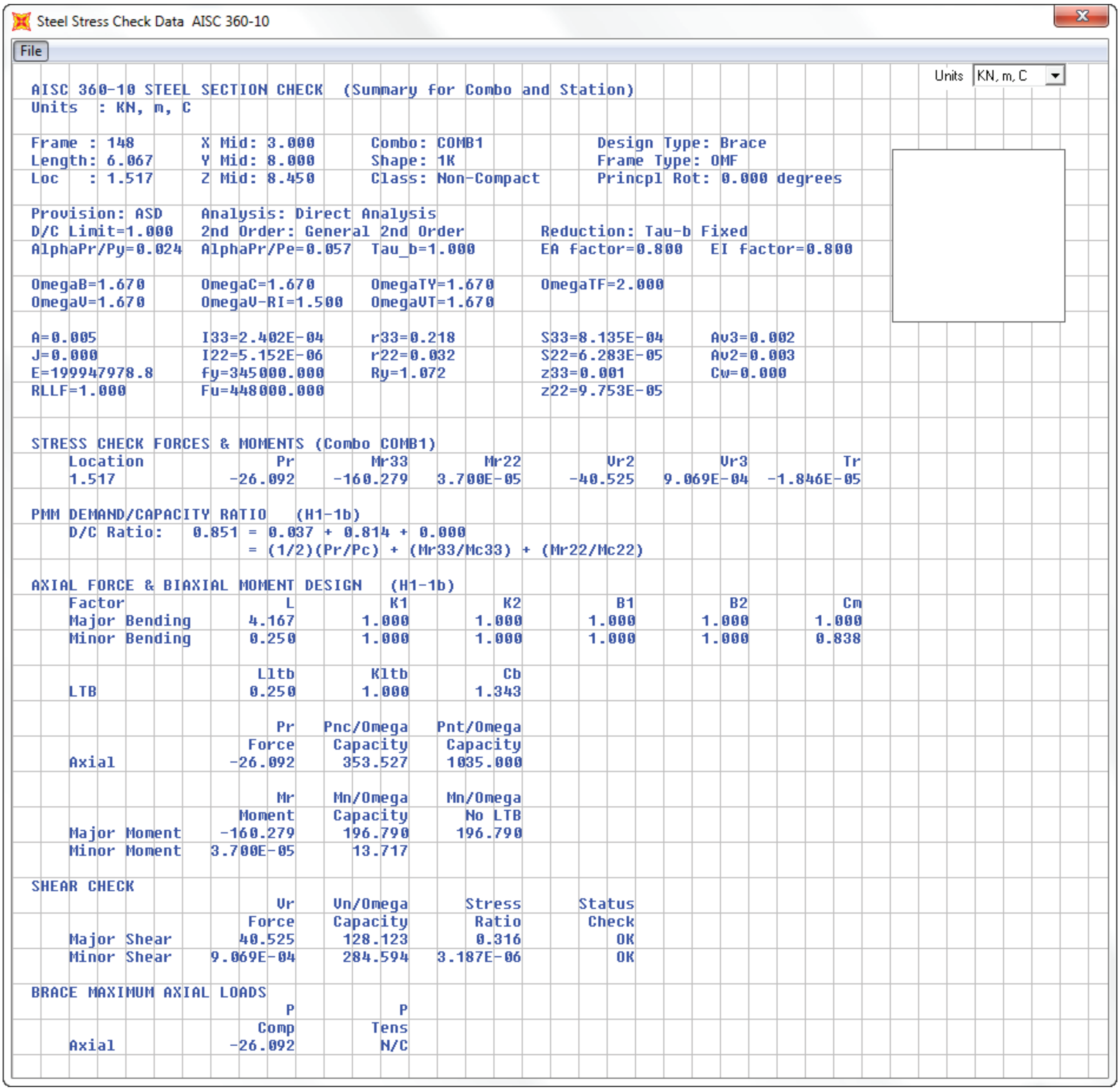
Steel Stress Check Information (AISC 360-10)

Frame ID 148 | Analysis Section 1K
Design Code AISC 360-10 | Design Section 1K

COMBO ID	STATION LOC	/----MOMENT INTERACTION CHECK-----/ RATIO	=	AXL + B-MAJ + B-MIN	/-MAJ-SHR- RATIO	--MIN-SHR-/ RATIO
COMB1	0.00	0.851 (C)	=	0.037 + 0.814 + 0.000	0.316	0.000
COMB1	3.03	0.386 (C)	=	0.047 + 0.339 + 0.000	0.195	0.000
COMB1	5.06	0.064 (C)	=	0.060 + 0.004 + 0.000	0.129	0.000
COMB1	5.06	0.075 (C)	=	0.060 + 0.015 + 0.000	0.126	0.000
COMB1	6.07	0.260 (C)	=	0.068 + 0.192 + 0.000	0.098	0.000
COMB2	0.00	0.247 (C)	=	0.013 + 0.234 + 0.000	0.102	0.000

Modify/Show Overwrites: Overwrites
Display Details for Selected Item: Details
Display Complete Details: Tabular Data
Stylesheet: Default
Strength Deflection OK Cancel Table Format File

Figure 4 24 Steel Stress Check Information form



Steel Stress Check Data AISC 360-10
File
Units KN, m, C

AISC 360-10 STEEL SECTION CHECK (Summary for Combo and Station)
Units : KN, m, C

Frame : 148 X Mid: 3.000 Combo: COMB1 Design Type: Brace
Length: 6.067 Y Mid: 8.000 Shape: 1K Frame Type: OMF
Loc : 1.517 Z Mid: 8.450 Class: Non-Compact Princpl Rot: 0.000 degrees

Provision: ASD Analysis: Direct Analysis
D/C Limit=1.000 2nd Order: General 2nd Order Reduction: Tau-b Fixed
AlphaPr/Py=0.024 AlphaPr/Pe=0.057 Tau_b=1.000 EA factor=0.800 EI factor=0.800

OmegaB=1.670 OmegaC=1.670 OmegaTY=1.670 OmegaTF=2.000
OmegaV=1.670 OmegaV-RI=1.500 OmegaVT=1.670

A=0.005 I33=2.402E-04 r33=0.218 S33=8.135E-04 Av3=0.002
J=0.000 I22=5.152E-06 r22=0.032 S22=6.283E-05 Av2=0.003
E=199947978.8 fy=345000.000 Ry=1.072 z33=0.001 Cw=0.000
RLLF=1.000 Fu=448000.000 z22=9.753E-05

STRESS CHECK FORCES & MOMENTS (Combo COMB1)

Location	Pr	Mr33	Mr22	Vr2	Vr3	Tr
1.517	-26.092	-160.279	3.700E-05	-40.525	9.069E-04	-1.846E-05

PMM DEMAND/CAPACITY RATIO (H1-1b)
D/C Ratio: 0.851 = 0.037 + 0.814 + 0.000
= (1/2)(Pr/Pc) + (Mr33/Mc33) + (Mr22/Mc22)

AXIAL FORCE & BIAXIAL MOMENT DESIGN (H1-1b)

Factor	L	K1	K2	B1	B2	Cm
Major Bending	4.167	1.000	1.000	1.000	1.000	1.000
Minor Bending	0.250	1.000	1.000	1.000	1.000	0.838

	Lltb	Kltb	Cb
LTB	0.250	1.000	1.343

	Pr Force	Pnc/Omega Capacity	Pnt/Omega Capacity
Axial	-26.092	353.527	1035.000

	Mr Moment	Mn/Omega Capacity	Mn/Omega No LTB
Major Moment	-160.279	196.790	196.790
Minor Moment	3.700E-05	13.717	

SHEAR CHECK

	Vr Force	Vn/Omega Capacity	Stress Ratio	Status Check
Major Shear	40.525	128.123	0.316	OK
Minor Shear	9.069E-04	284.594	3.187E-06	OK

BRACE MAXIMUM AXIAL LOADS

	P Comp	P Tens
Axial	-26.092	N/C

Figure 4 25 Steel Stress Check Data form

Click the Details button to show more detail information.

NOTE

Unbraced length ratio for major bending is 4.167 which is taken to be distance between two bracing point in major axis (25m) divided by actual length of member (6m). You should carefully review this ratio to ensure that the design process is consistent with your expectations.

To check whether all steel frames passed the stress check, click the **Design menu > Steel Frame Design > Verify All Members Passed** command to display as figure below.

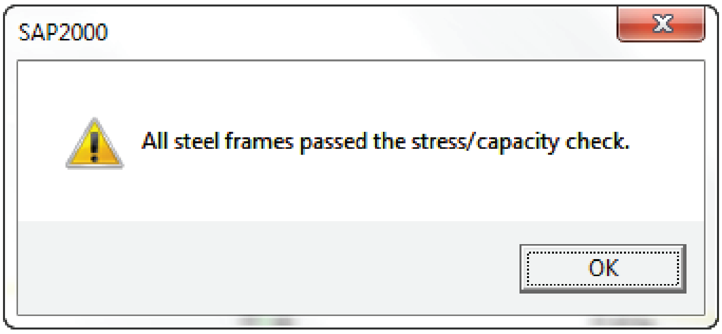


Figure 4 26 Checking capacity of all steel objects

4.1.3.6 Check Deflection Limitt

Refer Section 2.4 Serviceability Consideration for more details.

To optimize weight of building, unlock model, modify section and irritate above steps until total weight is smallest.

4.1.4. Export Joint Reaction

4.1.4.1 Transform from wind load with return period wind speed of 700 years to wind load with those of 50 years

In **Load Case Data – Linear Static** form, change **Scale Factor** for all Windload Case from **1** to **0.625** as Figure 4 27.

$$\frac{V_T}{V_{50}} = 0.36 + 0.1\ln(12T)$$

$$\Rightarrow \mathrm{V}_{50} = \frac{V_{700}}{0.36 + 0.1\ln(12 \times 700)} = 0.791 V_{700} \Rightarrow WL_{50} = 0.791^2 \times WL_{700}^2 = 0.626 \times WL_{700}^2$$

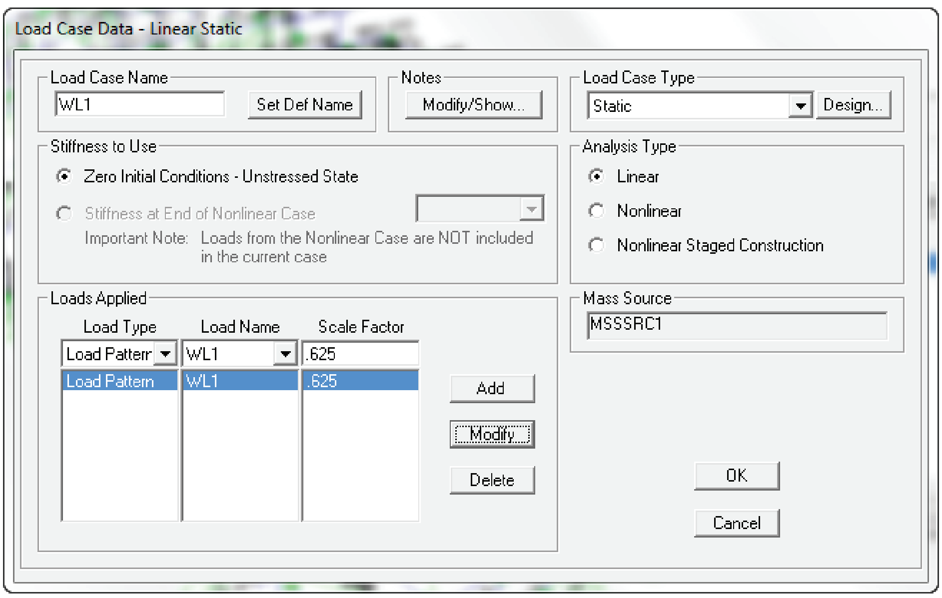
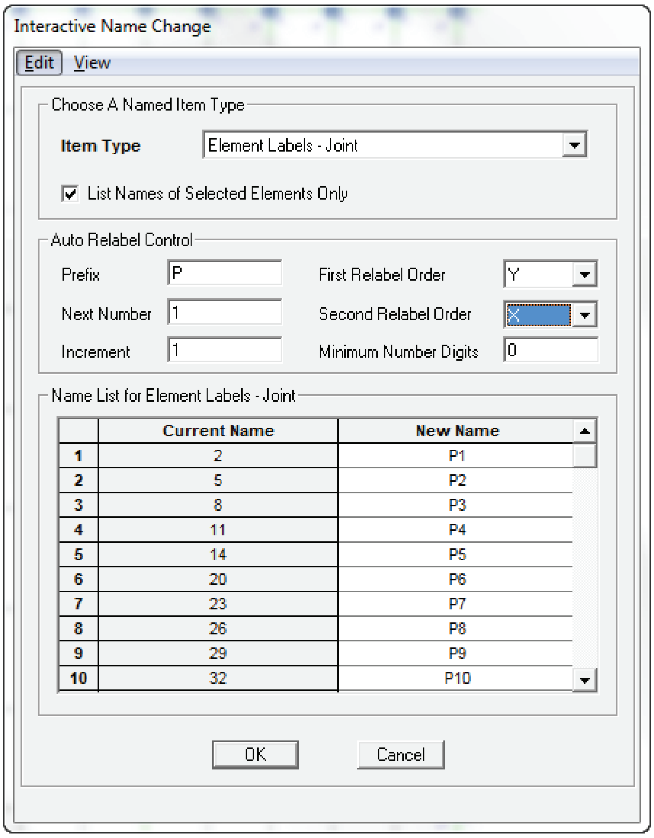


Figure 4 27 Load Case Data – Linear Static form

4.1.4.2 Changing label name of joints

Select the **Edit > Change Labels** command to access **Interactive Name Change** form. After enter all the information as figure below, select the **Edit > Auto Relabel > All In List** command to change label name.



	Current Name	New Name
1	2	P1
2	5	P2
3	8	P3
4	11	P4
5	14	P5
6	20	P6
7	23	P7
8	26	P8
9	29	P9
10	32	P10

Figure 4 28 Change Label Form

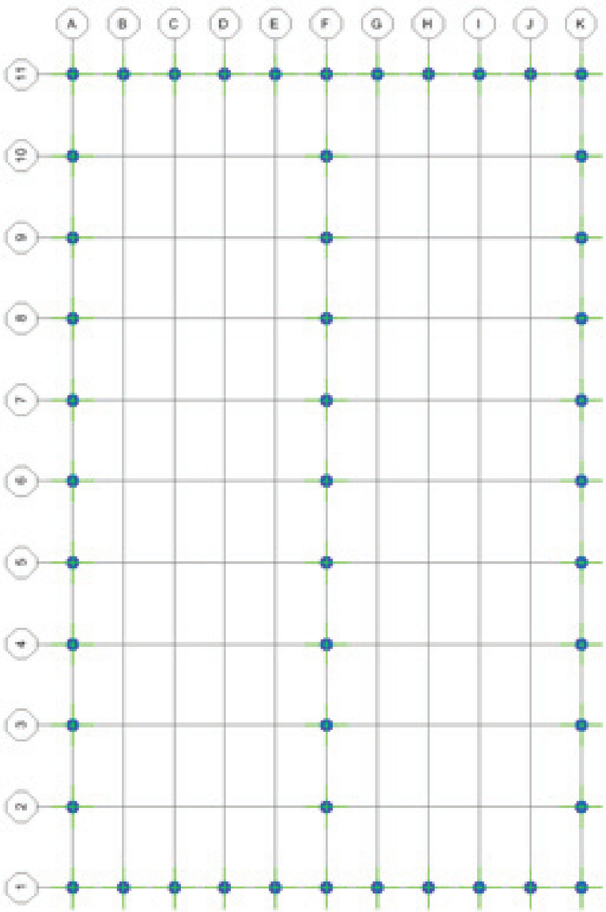


Figure 4 29 Joints in X-Y Plan @Z=0

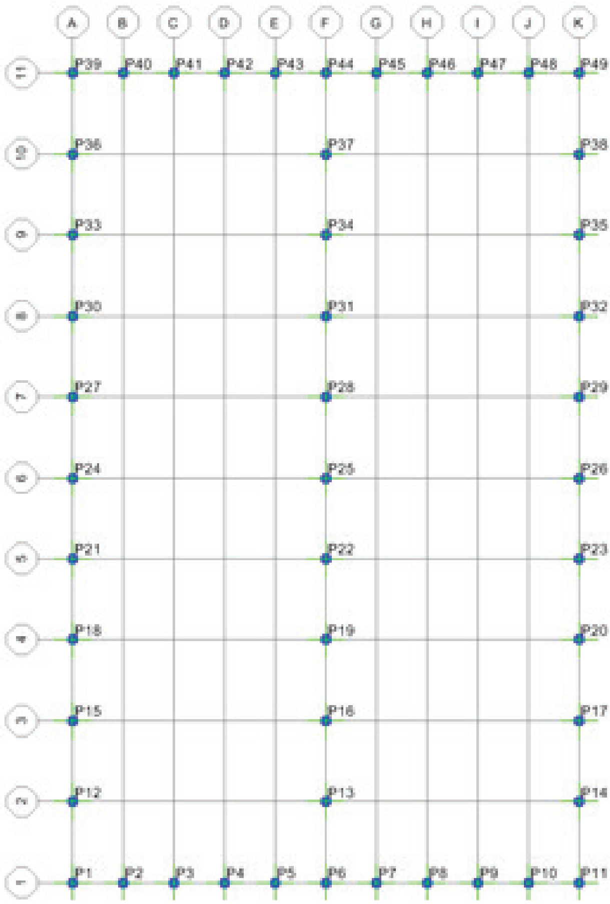


Figure 4 30 Label name after changing

4.1.4.3 Displaying the Joint Forces/Joint Reactions

Export Joint Reactions to Excel

• When steel columns locate on concrete columns, specify all steel columns and show table **Element Joint Forces – Frames**.

• When steel columns locate on foundations, specify all base joints and show table **Joint Reactions**.

Create JOINT REACTIONS Results

Open file "**7 - JOINT REACTIONS**" and move to sheet "**SteelColumn**", and then enter all the information about the project and Joint Labels Plan.

In Excel file that has just appeared, filter all base joints. Then insert one column on the left of column "E". Copy all data from column "B" to column "K".

In sheet "**InputS**" in file "**7 – JOINT REACTIONS**":

- Delete all data in area from columns A to J and from rows 4 to the end of sheet.
- Select cell "A4" and paste data into this. Select cell "X6" and change content in this cell to "Wind load Y".
- Move to sheet "SteelColumn" and click the HIDE button.
- Set Print Area before print PDF file.
- Recheck all information; make sure that Vertical Reactions Vz for Dead load and Live load are always positive.

4.1.5. Design Example 4a: Low-Rise Building Sap Model

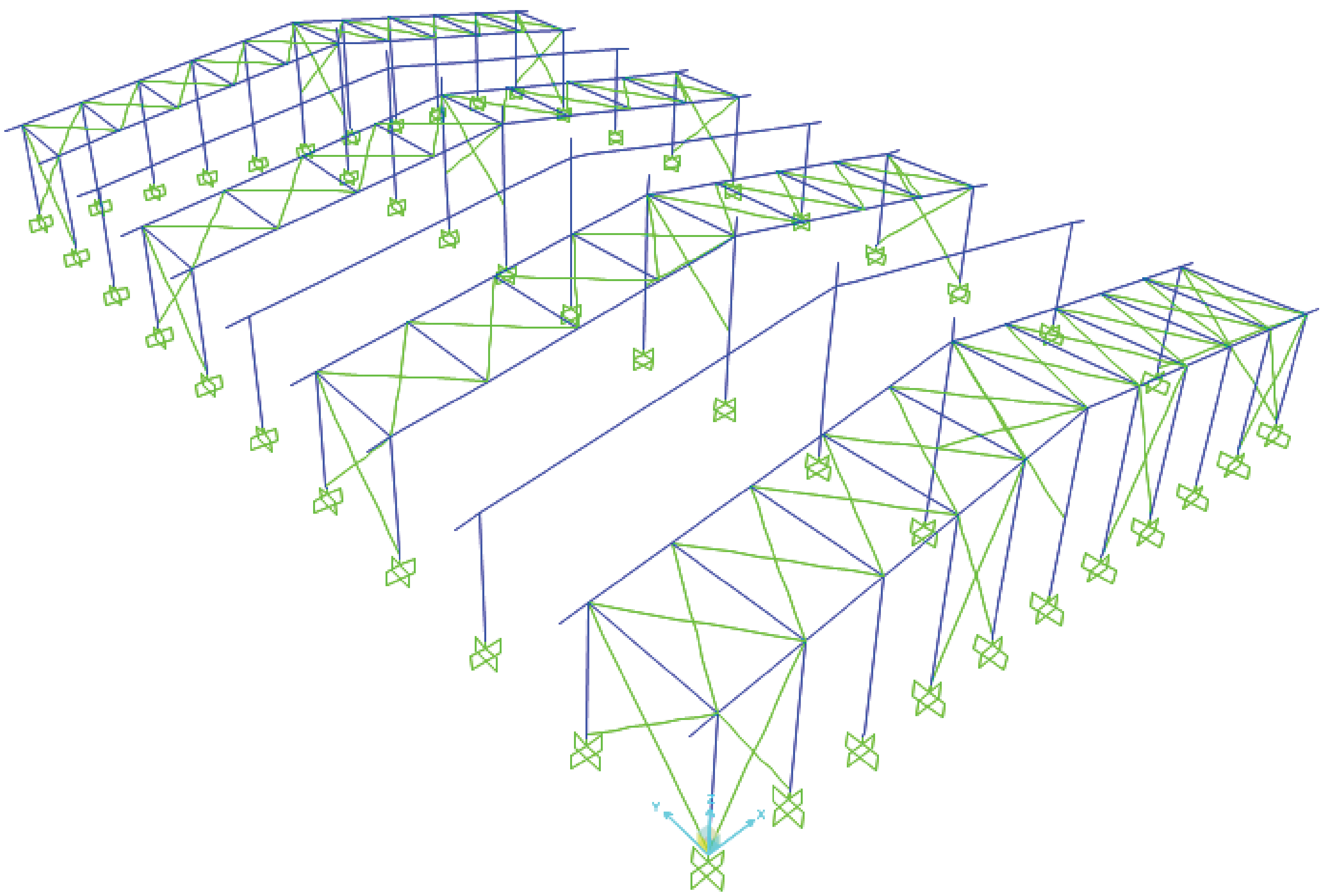


Figure 4 31 Low-rise building model

BMB. Best Metal Buildings. BMB & A J/SC	**PROJECT INFORMATION FORM (PIF)** (THÔNG TIN TRIỂN KHAI DỰ ÁN)	Code of doc.	BM-7.2.1-02
		Date of issue	24/10/2016
		Times of issue	04
		Project status	Priority □ / Normal ■
		Breakdown form	BMB&A ■ / Customer □

PROJECT INFORMATION

Project name:	Zhong Yi Warehouse				
Location	*Phnom Penh, Camobodia*				
Consultancy/ Design Company	Royal Haskoning □	Architype □	Meinhardt □	B.I.S.T □	Others □ ________________
Quotation no.: 148-17	Start date: 11/03/2017	Finish date:16/03/2017	Building no.:01	Area no.:4000	

Requirements: 1.New Est. ■ / 2.Arch.drwgs ■ / 3.Apprl.drwgs □ / 4.Design calculation □ / 5.Shop drwgs □

BUILDING DESCRIPTION

Material Specifications	1.BMB Standard ■ / 2.Tender's Standard □	
	Built-up member: fy= 34.5 kN/ cm^2	Purlin: Painted □ / Galvanized ■ fy= 45 kN/cm^2
	Anchor bolt: 5.6 ■ / 8.8 □	4. Connection bolt: 10.9
	Paint/ Finishing: Alkyd ■ / Galvanized □ / Epoxy □ / Thickness: 80 micron	

Exposure	B ■	C □	D □
Qty. of identical Bldings	01	Usage	Warehouse
Type of building	Multi span 1	Roof slope (%)	15%
Width (M)	50.0	Width Module (M)	10@5.0m O-O of brick wall
Length (M)	80.0	Bay Spacing (M)	10@8.0m O-O of Brick wall

Height (M)	8.0	Eave Height ■	Clear Height □
Type of End Frames	Post & Beam ■	One, Axis.................... □	Both ■
	Main Rigid Frame □	One, Axis.................... □	Both □
	Concrete Columns and rafters (by others) □	One, Axis.................... □	Both □
Type of Wall Bracing	Diagonal Rod ■ / Cable □ / Portal frame □ / None □		
Eave Condition	Gutter & Downspout ■ / RC Gutter (by other) □ / Gutter & Downspout (by other) □		

Notes: **Use Chinese steel.**
All steel columns are installed at Level +200mm on stump column.
At GL. F, all columns are installed on 5m-height RC column.
At GL.1, sliding doors 5mx4m are between GL.B&C and GL.I&H.
Gutter is local product 0.45mm; Downspout is PVC and put 1 column pass 1 column. (See DWG)

DESIGN LOADS

Follow BMB's design ■ / Follow Customer's design □

Live Load on Roof (kN/m^2)	0.57 (58.1 KG/m^2) ■ / Others		
Live Load on Frame (kN/m^2)	0.30 (30.6 KG/m^2) ■ / Others		
Wind Speed (KM/h)	110 (30.6 M/s) ■ / 130 (36.1 M/s) □ / 140 (38.9 M/s) □ / Other (……. M/s) □		
Collateral Load (KG/m^2)		Concentrated load (KG/m)	
Collateral Load hang on purlin □		Collateral Load don't hang on purlin □	

Notes: *(if collateral load is plaster ceiling → describe here also)*

ROOF, WALL SHEETINGS

Roof panel: Yes ■ None □	Bare Zinc-Alum AZ… □	Pre-Painted Zinc-Alum AZ 50 ■	7 ribs □ / 5 ribs ■
	Pre-Painted Zinc-Alum AZ… □	Thickness: 0.45mm	Seam □ / Clip-lock □
Wall panel: Yes ■ None □	Bare Zinc-Alum □	Pre-Painted Zinc-Alum AZ150 ■	7 ribs panel ■
	Pre-Painted Zinc-Alum AZ… □	Thickness: 0.40mm	5 ribs panel □

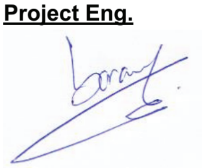
Notes: **Use Local panel; all trims and gutters are 0.45mm thk.**

OPEN WALL CONDITIONS	
Both Side wall	Open 4.0 M for block wall, sheeted up to roof.
Both End wall	Open 4.0 M for block wall, sheeted up to roof.
Notes:	

ROOF MONITOR			None ☐
Type	Jack roof ■	Ridge vent ☐	Other ☐
Overall width (M)	2.0	Length (M)	64.0
Eave Condition	Curved Eave ■	Open Eave ☐	Other ☐
Roof panel	Same as roof panel ■ / Other ☐ .... thick .... mm (nominal)		
Wall panel	Same as wall panel ■ / Other ☐ .... thick.............. mm (nominal)		
Notes:			

ROOF EXTENSIONS		None ☐
Panel	Same roof panel ■/ Other ☐ .... thick............... mm (nominal)	
Location	Side wall	End wall
Soffit panel	Yes ☐ / None ■	Yes ☐ / None ■
Quantity	02	02
Width (M)	1.5	1.5
Length (M)	80.0	50.0
Notes:		

CANOPY			None ☐
Panel	Same as roof panel ■ / Other ☐ .... thick ....mm (nominal)		
Eave Condition	Curved Eave ■	Open Eave ☐	Other ☐
Location	Both Side wall	End wall (GL.11)	End wall (GL.1)
Soffit panel	Yes ☐ / None ■	Yes ☐ / None ■	Yes ☐ / None ■
Quantity	04	01	02
Width (M)	2.5	2.5	2.5
Length (M)	4.0	4.0	5.0
Gutter, Downspout	None ■	None ■	None ■
Notes:			

Project Eng.  ros	*Notes for using this PIF* *1. Must fill in Exact Country of Project & Name of Consultancy/ Design Company at top of PIF* *2. Must make sketch drawing or provide architectural drawing.*

4.1.5.1 Defining

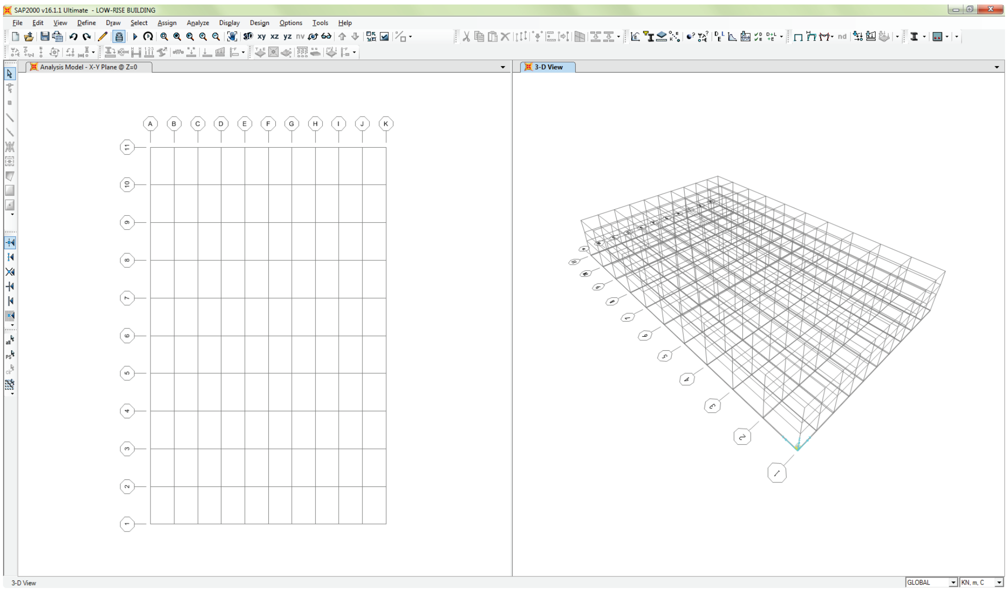


Figure 4 33 Setting up geometry and default units

Define Section Properties as table below:

Table 4 2 Frame Section Properties

TABLE: Frame Section Properties 01 - General								
SectionName	Material	Shape	d	bf1	tf1	tw	bf2	tf2
Text	Text	Text	m	m	m	m	m	m
1C		Nonprismatic						
1C-250212	STEEL3450	I/Wide Flange	0.25	0.212	0.008	0.005	0.212	0.008
1C-600212	STEEL3450	I/Wide Flange	0.6	0.212	0.008	0.005	0.212	0.008
1K		Nonprismatic						
1K-384164	STEEL3450	I/Wide Flange	0.384	0.164	0.006	0.005	0.164	0.008
1K-600164	STEEL3450	I/Wide Flange	0.6	0.164	0.006	0.005	0.164	0.008
2C264184	STEEL3450	I/Wide Flange	0.264	0.184	0.008	0.005	0.184	0.008
2K-384148	STEEL3450	I/Wide Flange	0.384	0.148	0.006	0.004	0.148	0.006
3K		Nonprismatic						
3K-384164	STEEL3450	I/Wide Flange	0.384	0.164	0.006	0.005	0.164	0.008
3K-600164	STEEL3450	I/Wide Flange	0.6	0.164	0.006	0.005	0.164	0.008
4K		Nonprismatic						
4K-600184	STEEL3450	I/Wide Flange	0.6	0.184	0.008	0.005	0.184	0.008
4K-825184	STEEL3450	I/Wide Flange	0.8	0.184	0.008	0.005	0.184	0.008
CDH	STEEL3450	I/Wide Flange	0.26	0.164	0.006	0.004	0.164	0.006
CON45x35	CON25	Rectangular	0.45	0.35				
EB	STEEL2350	Box/Tube	0.15	0.15	0.0028	0.0028		
IPE-200	STEEL2350	I/Wide Flange	0.2	0.1	0.008	0.0055	0.1	0.008
KDH	STEEL3450	I/Wide Flange	0.222	0.148	0.005	0.004	0.148	0.005

4.1.5.2 Modeling

Create general model

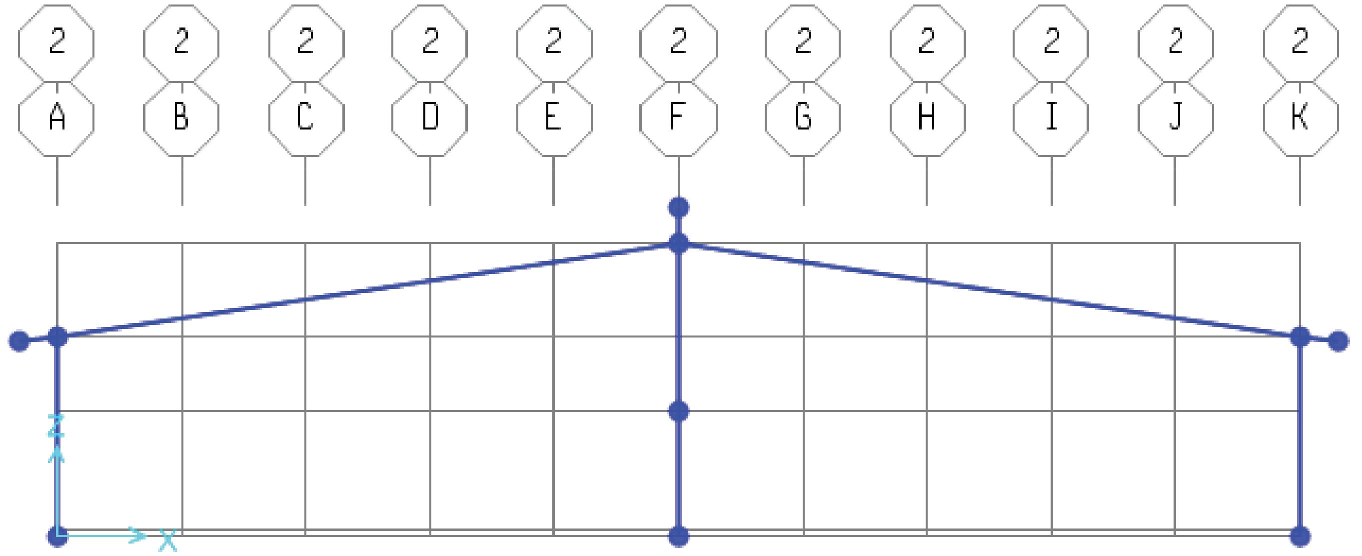


Figure 4 34 Main frame

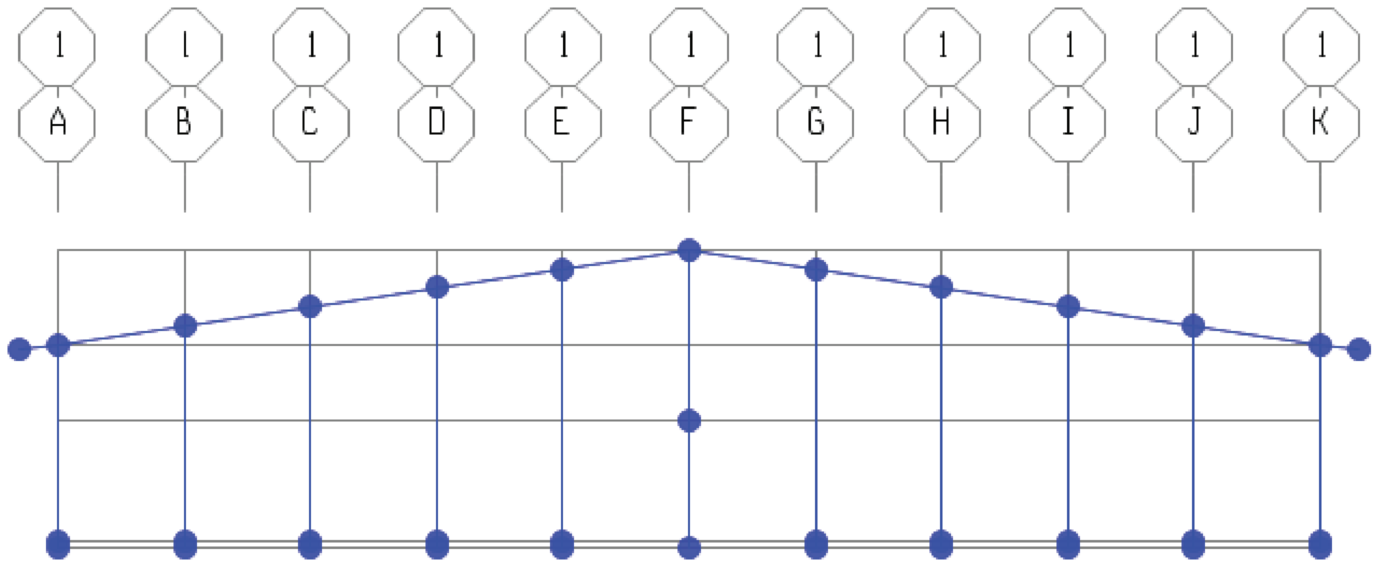


Figure 4 35 End wall frame

Define load patterns and load combinations

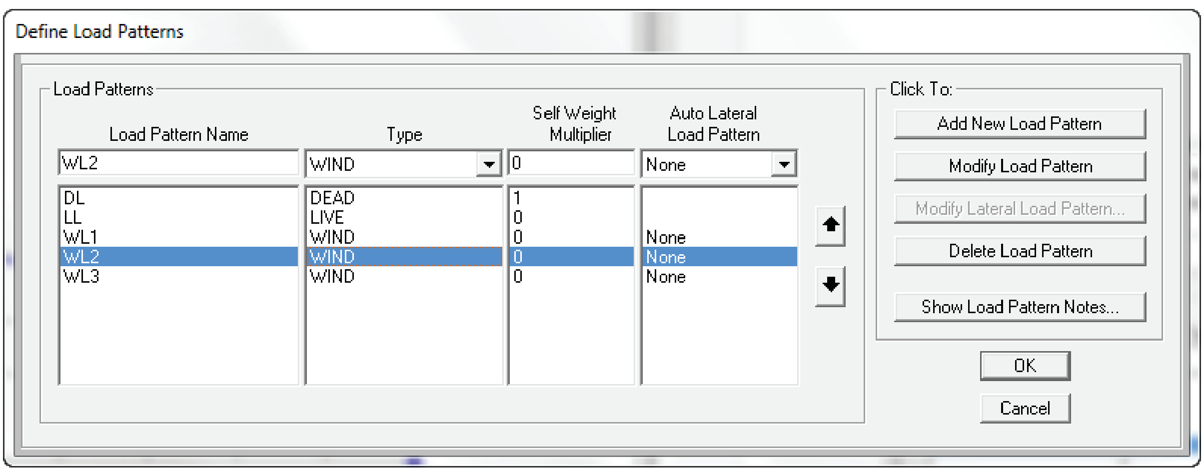
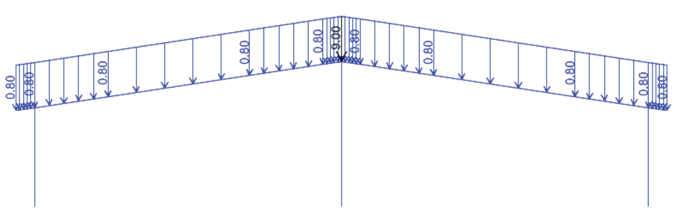
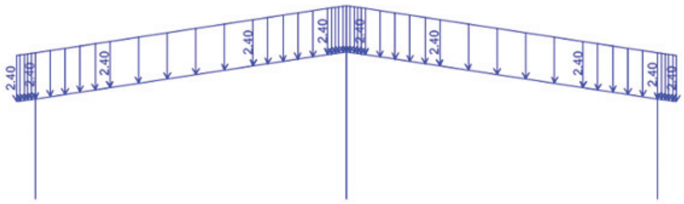


Figure 4 36 Define Load Patterns form

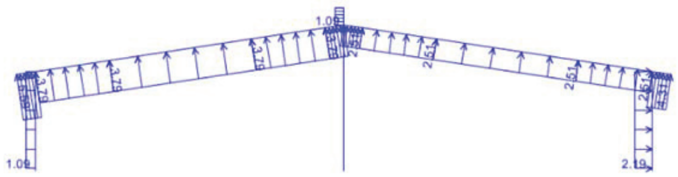
Assign load



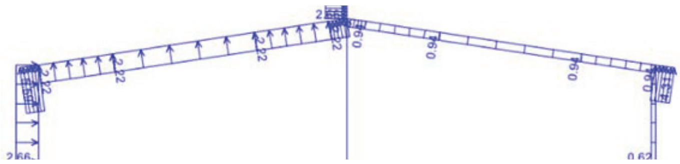
Dead load (DL)



Live load (LL)

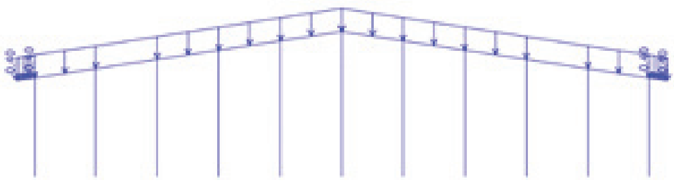


Wind load 1 (WL1)

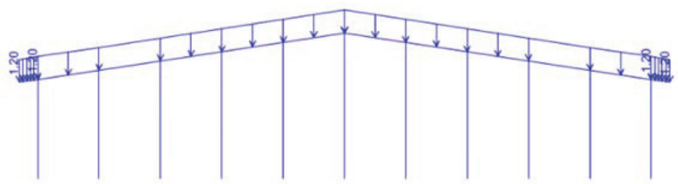


Wind load 2 (WL2)

Figure 4 37 Load applied on main frame

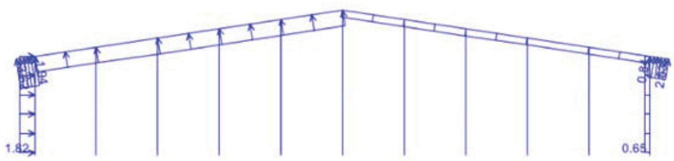


Dead load (DL)

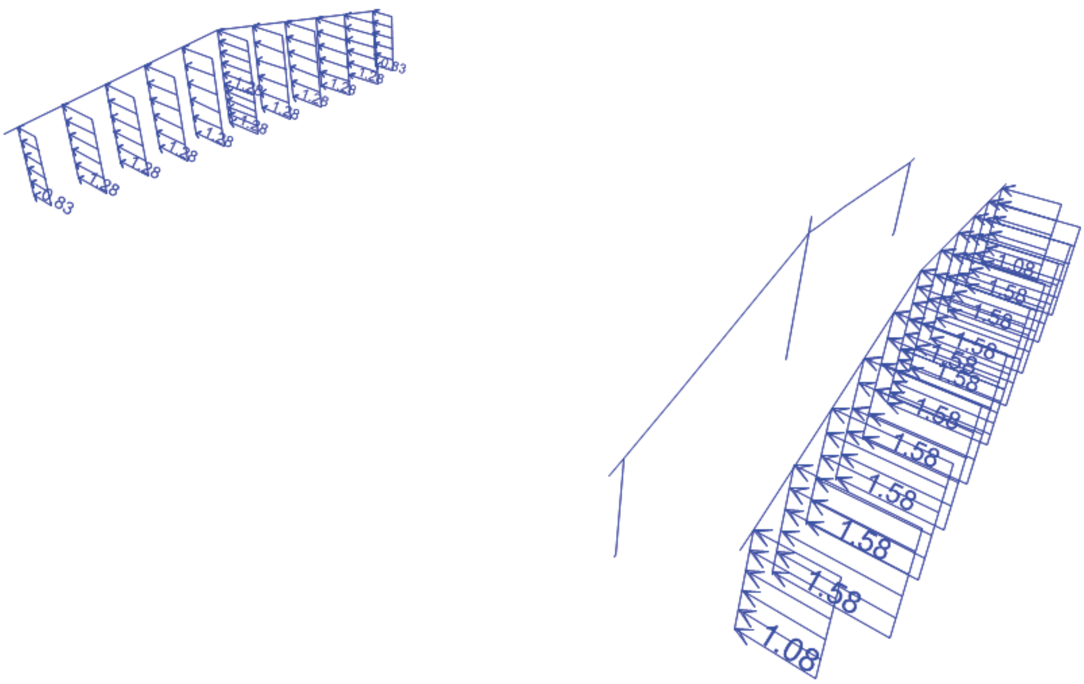


Live load (LL)

Wind load 1 (WL1)



Wind load 2 (WL2)



Wind load 3 (WL3)

Figure 4 38 Load applied on end-wall frame

Divide frame objects

Draw vertical lines that are located at x=6m, x=17.5m, x=23.5m, x=26.5m, x=32.5m and x=44m as shown in figure below.

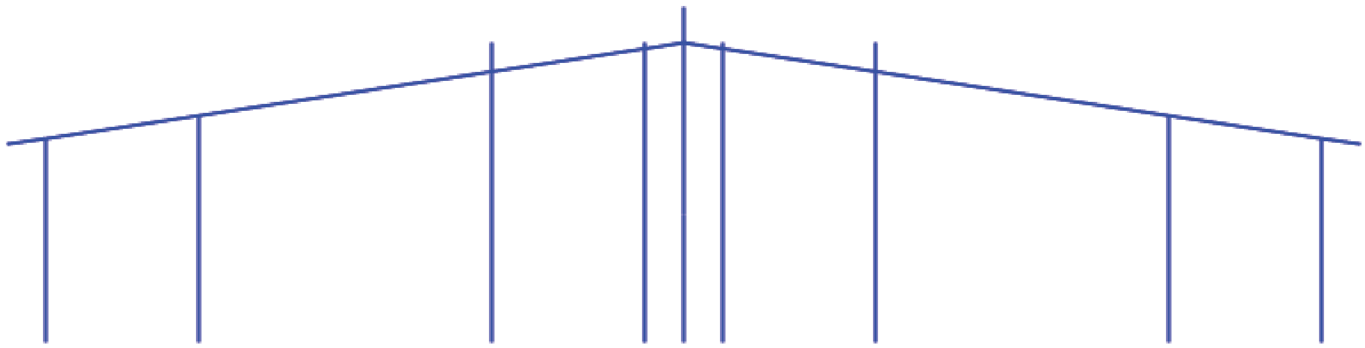


Figure 4 39 Vertical lines to divide rafter

Select rafters and vertical lines, then click the **Edit > Edit Lines > Divide Frames** command to show Divide Selected Frames form. In this form, make sure that **Break at intersection with selected Joints, Frames, Area Edges and Solid Edges** is selected before click the OK button.

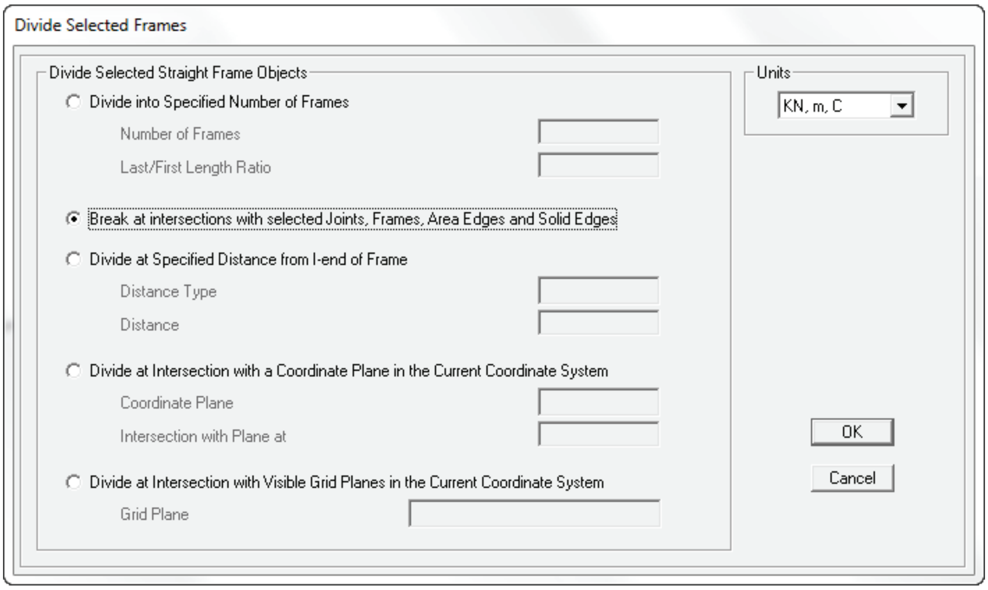


Figure 4 40 Divide Selected Frames form

Delete additional vertical lines.

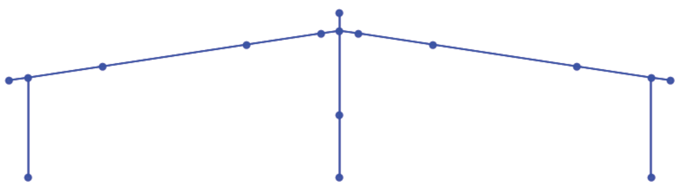


Figure 4 41 Main frame after dividing

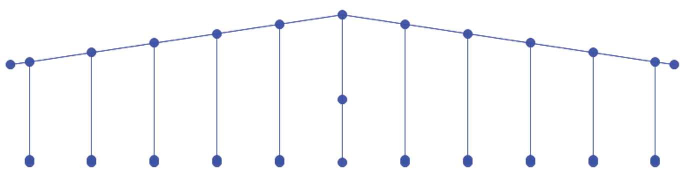


Figure 4 42 End wall frame

Assign member section, end releases and restraint

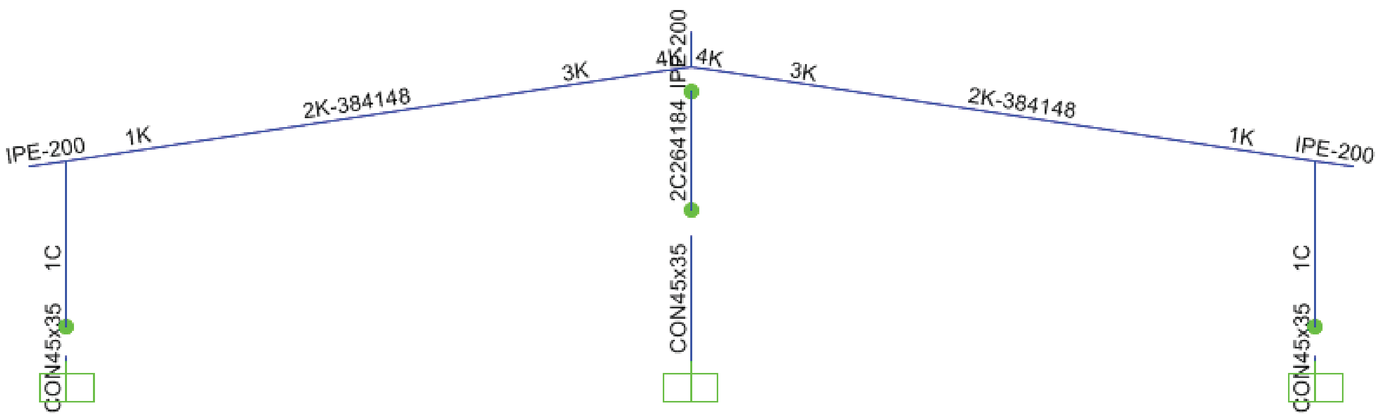


Figure 4 43 Main frame sections

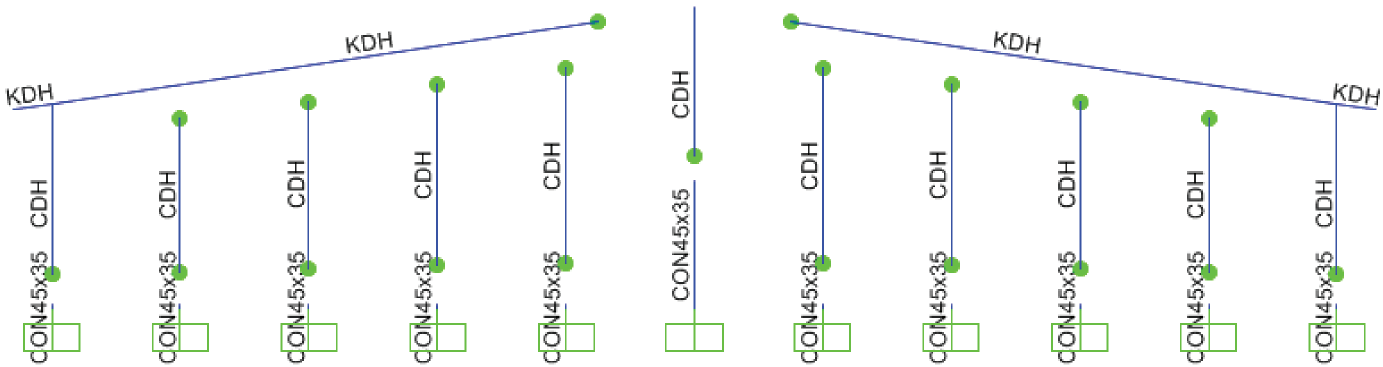


Figure 4 44 End wall frame sections

Calculate ULR for each member in frame

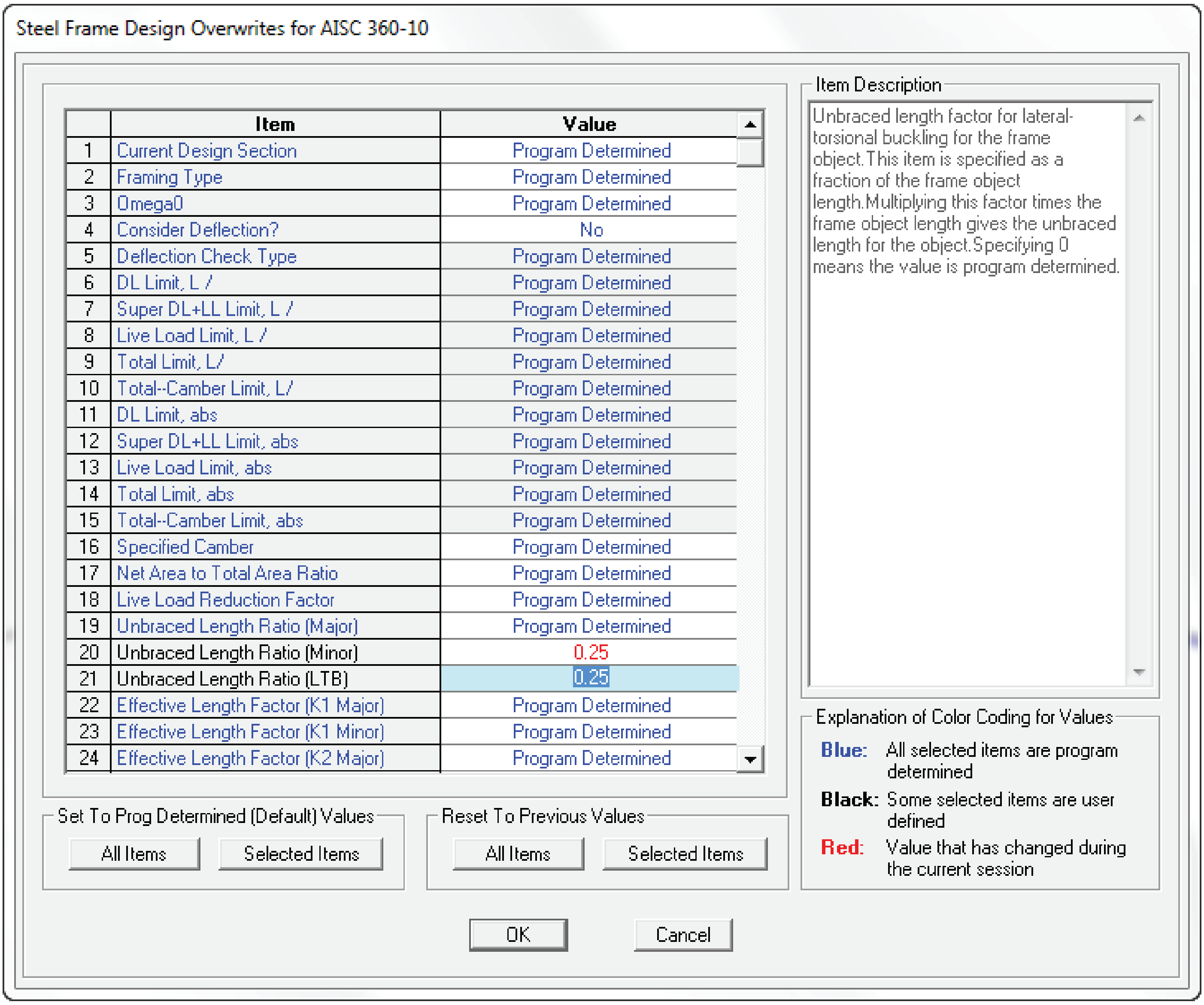
- Assume purlin spacing and girt spacing are 1.5m.
- **With rafters**: unbraced length for 1K, 3K and 4K are 1.5m, for 2K-384148 are 3m and for KDH, IPE-200 are their length (no bracing)
- **With outer columns**: unbraced length for 1C and CDH are 5m (base on architectural drawing).
- **With inner columns** (2C264184): there are not any ST or portal frame, so unbraced length for this column is taken to be its length or ULR is 1.

Table 4 3 Unbraced length ratio

Member	Length (m)	Unbraced Length (m)	ULR
1K	6.00	1.50	0.25
2K-384148	11.50	3.00	0.26
3K	6.00	1.50	0.25
4K	1.50	1.30	0.87
1C	7.80	5.00	0.64
2C264184	6.75	6.75	1.00
IPE-200	1.50	1.50	1.00
CDH	7.80	5.00	0.64
KDH	25.28	25.28	1.00

Select 1K section members and click the **Design > Steel Frame Design > View/Revise Overwrites** command to show **the Steel Frame Design Overwrites** for AISC 360-10 form.

At the Value column, type 0.25 for both **Unbraced Length Ratio (Minor) and Unbraced Length Ratio (LTB).**


Steel Frame Design Overwrites for AISC 360-10

	Item	Value
1	Current Design Section	Program Determined
2	Framing Type	Program Determined
3	Omega0	Program Determined
4	Consider Deflection?	No
5	Deflection Check Type	Program Determined
6	DL Limit, L /	Program Determined
7	Super DL+LL Limit, L /	Program Determined
8	Live Load Limit, L /	Program Determined
9	Total Limit, L/	Program Determined
10	Total--Camber Limit, L/	Program Determined
11	DL Limit, abs	Program Determined
12	Super DL+LL Limit, abs	Program Determined
13	Live Load Limit, abs	Program Determined
14	Total Limit, abs	Program Determined
15	Total--Camber Limit, abs	Program Determined
16	Specified Camber	Program Determined
17	Net Area to Total Area Ratio	Program Determined
18	Live Load Reduction Factor	Program Determined
19	Unbraced Length Ratio (Major)	Program Determined
20	Unbraced Length Ratio (Minor)	0.25
21	Unbraced Length Ratio (LTB)	0.25
22	Effective Length Factor (K1 Major)	Program Determined
23	Effective Length Factor (K1 Minor)	Program Determined
24	Effective Length Factor (K2 Major)	Program Determined

Item Description
Unbraced length factor for lateral-torsional buckling for the frame object. This item is specified as a fraction of the frame object length. Multiplying this factor times the frame object length gives the unbraced length for the object. Specifying 0 means the value is program determined.

Set To Prog Determined (Default) Values: All Items | Selected Items
Reset To Previous Values: All Items | Selected Items

Explanation of Color Coding for Values
Blue: All selected items are program determined
Black: Some selected items are user defined
Red: Value that has changed during the current session

OK | Cancel

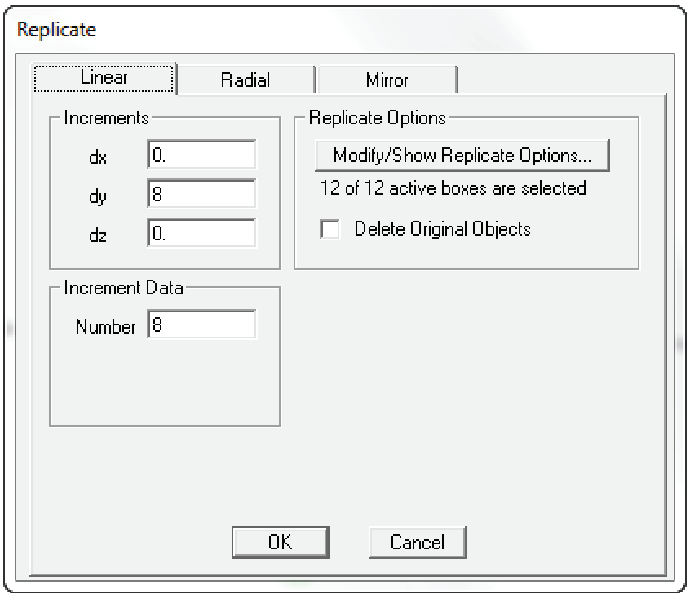
Figure 4 45 Steel frame design overwrites form

Repeat these steps to assign ULR for the other member.

Replicate Frame

Select frame at X-Z Plan @ Y=8 and then select the Edit > Replicate.

To generate 8 additional frames with 8m bay spacing, enter like below:


Replicate

Linear | Radial | Mirror

Increments
dx 0.
dy 8
dz 0.

Replicate Options
Modify/Show Replicate Options...
12 of 12 active boxes are selected
Delete Original Objects

Increment Data
Number 8

OK | Cancel

Figure 4 46 Replicate form

Draw Bracing System

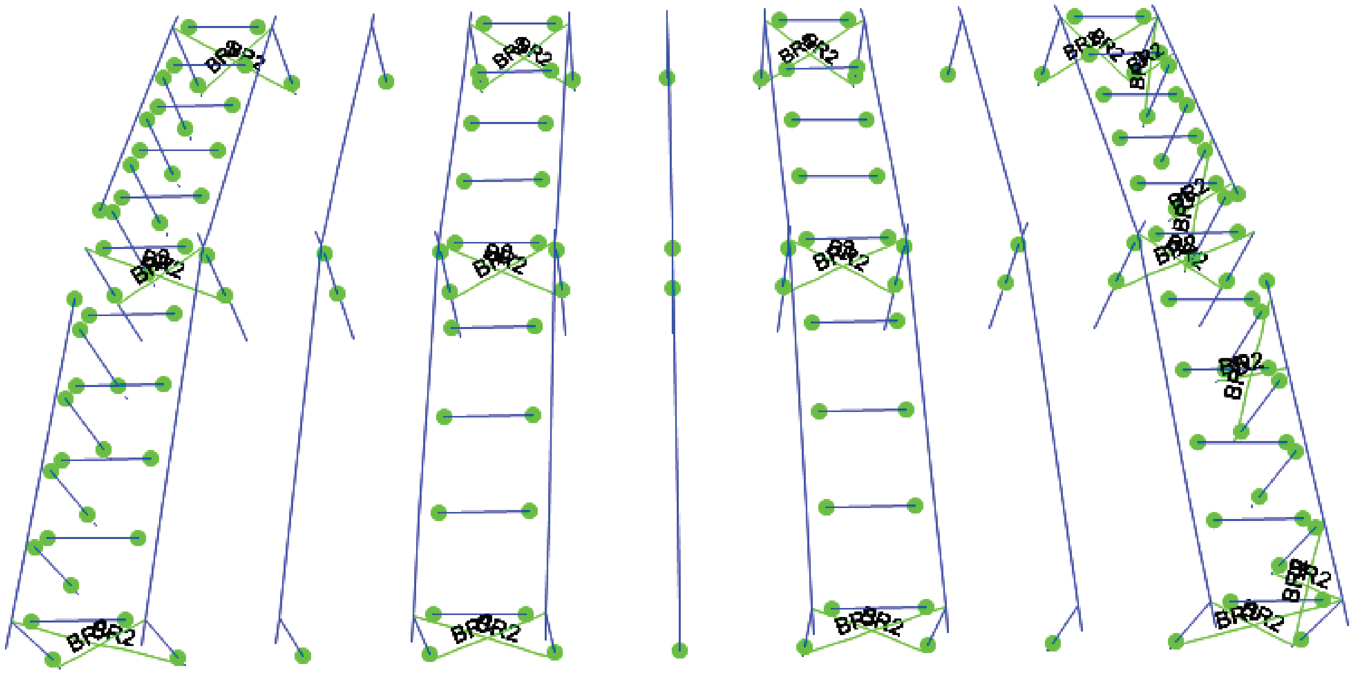


Figure 4 47 Column brace system

4.1.5.3 Analyzing

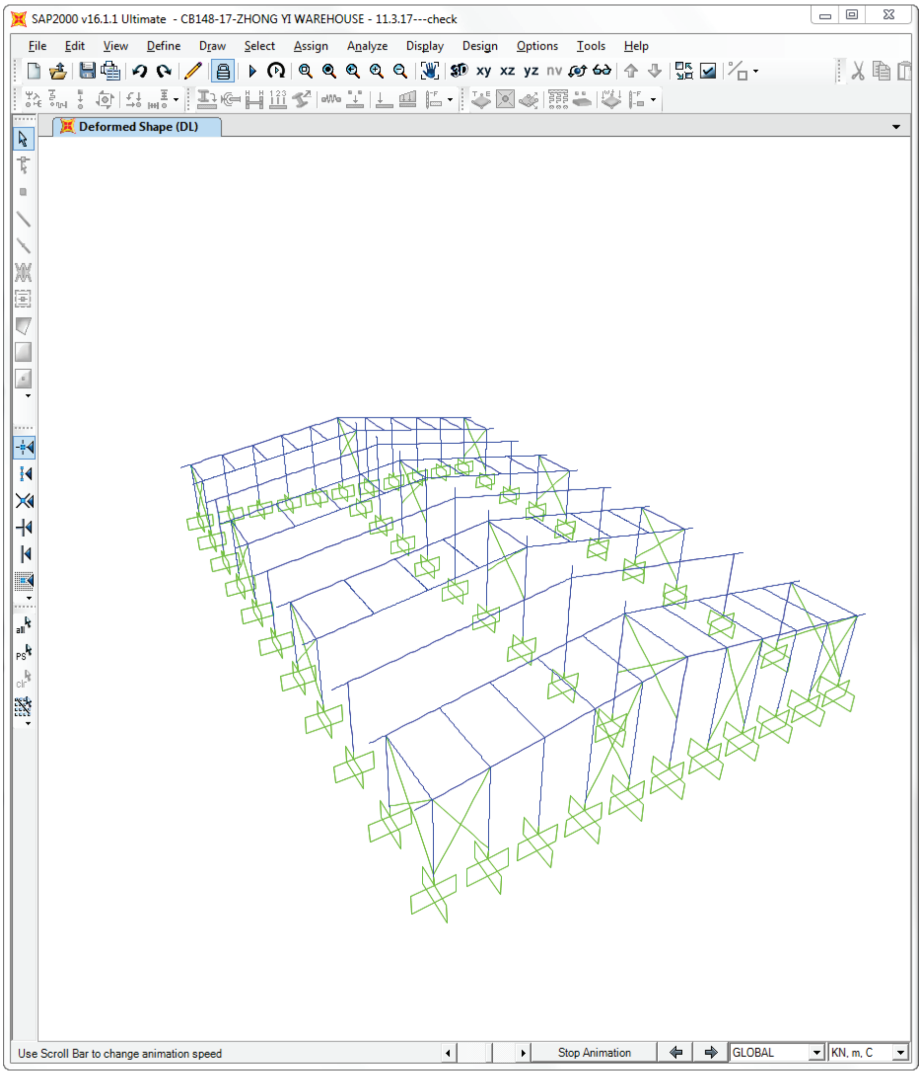


Figure 4 48 Model after running analysis

4.1.5.4 Design Steel Frame

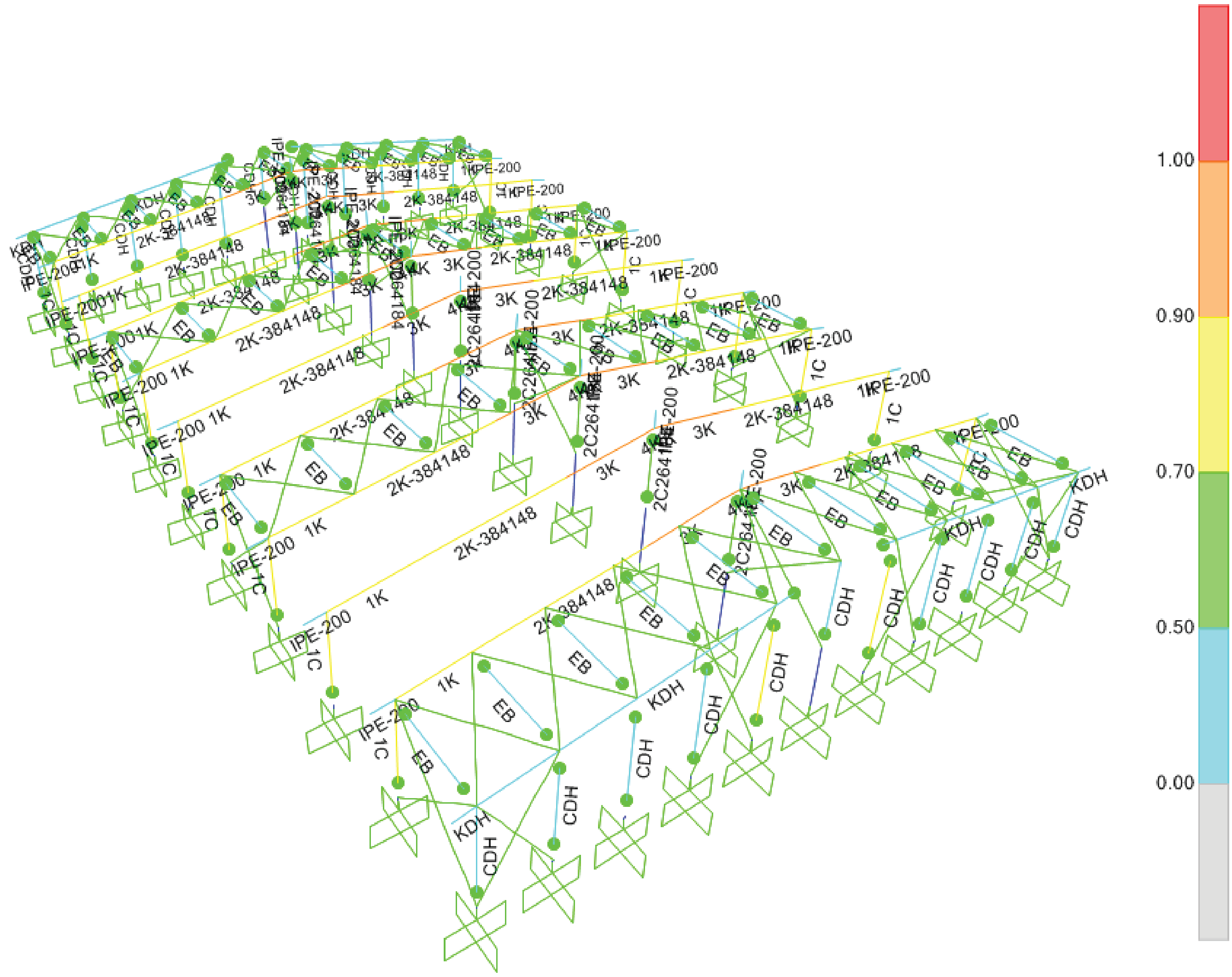


Figure 4 49 Model after checking

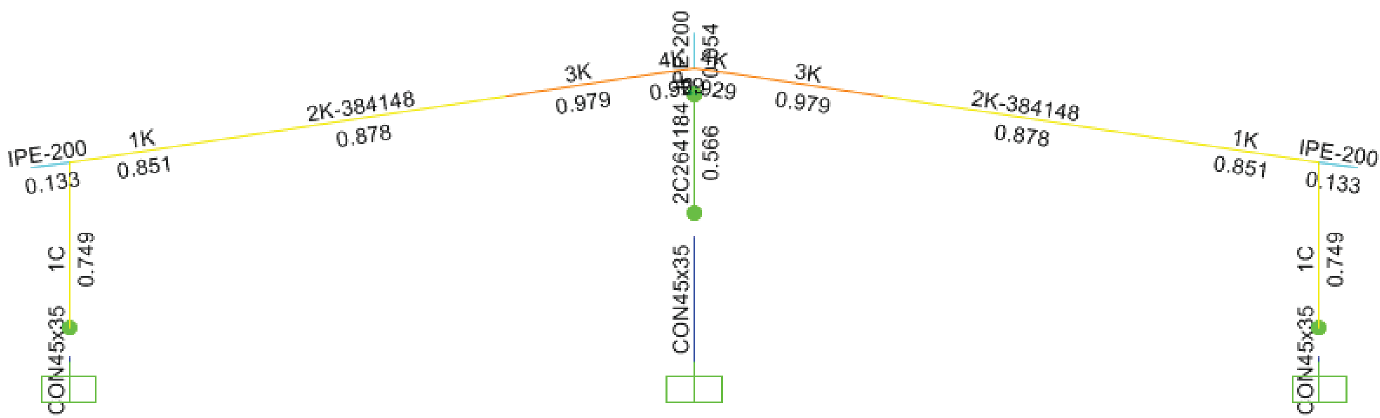


Figure 4 50 Main frame at plane Y=8

To check whether all steel frames passed the stress check, click the **Design menu > Steel Frame Design > Verify All Members Passed** command to display as figure below.

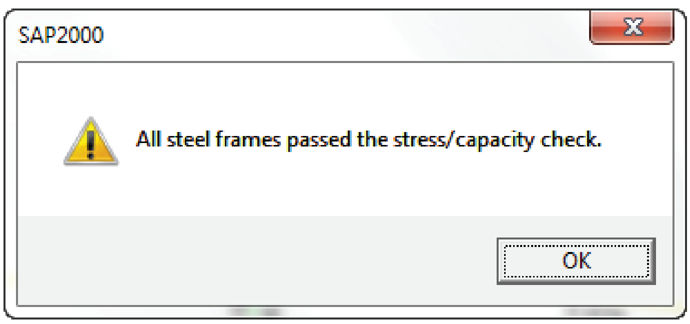


Figure 4 51 Checking capacity of all steel objects

4.1.5.5 Check Deflection Limitation

Create combos for deflection checking:

Name	LCombination
CV1	DL + 0.4224WL1
CV2	DL + 0.4224WL2
CV3	DL + 0.4224WL3
CV	CV1 + CV2 + CV3

Vertical Deflection

Click the **Display > Show Deformed Shape** command which will show a Deformed Shape form. Select **COMB1 (DL + LL)** from **Case/Combo Name** drop-down list and click OK to accept.

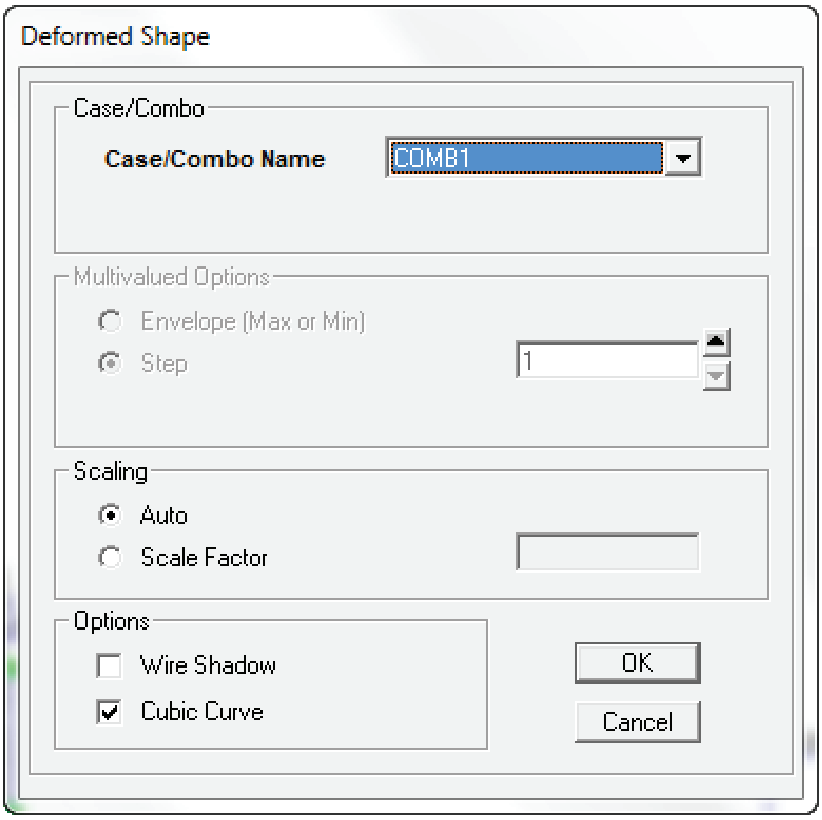
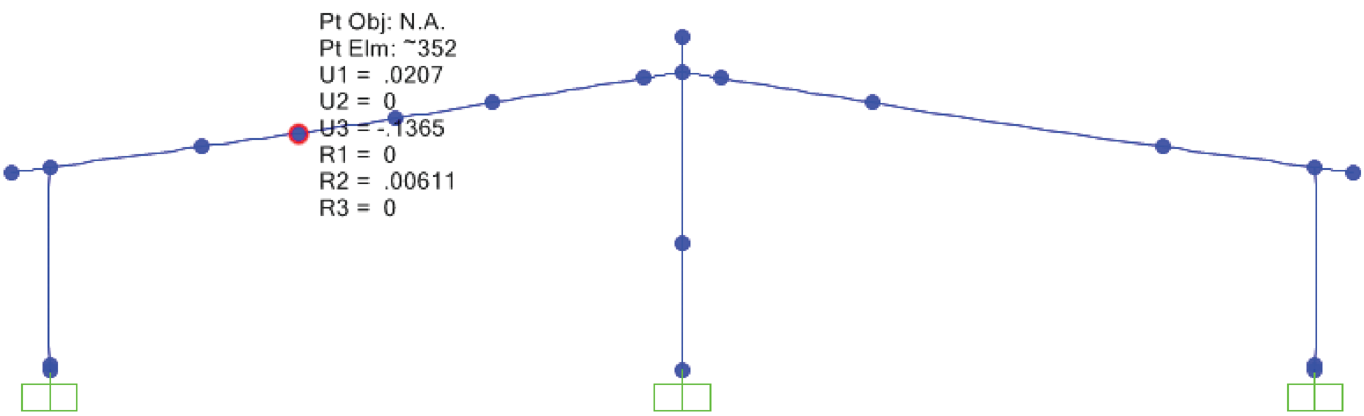


Figure 4 52 Deformed Shape form

Checking if vertical deflection (U3) of mid points of rafter are excess deflection limit. Put the cursor at mid points of rafter to see if deflection of that point is excess the limit or not.

Compute deflection limit: $\Delta = \dfrac{L}{120} = \dfrac{25}{120} = 0.208\ m$



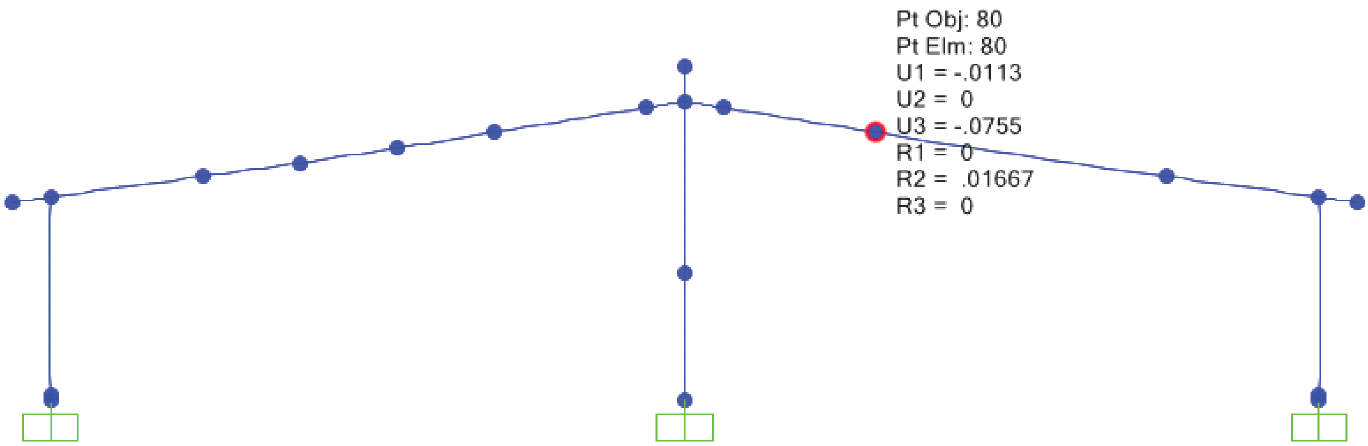


Figure 4 54 Deflection of point 113 – 0.0755m

Horizontal Deflection

Checking if horizontal deflection of combination **CV** of top of columns are excess deflection limit. Put the cursor at points located at the top of column to see if the horizontal deflection (U1 or U2) of that point is excess the limit or not.

Compute horizontal deflection limit:

$$\Delta = \frac{h}{60} = \frac{8}{60} = 0.133\,m$$

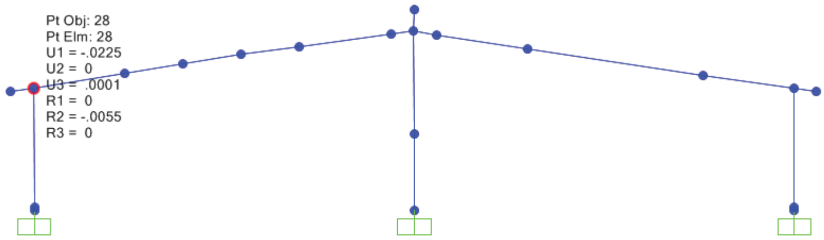


Figure 4 55 Deflection of point 28 – 0.0225m

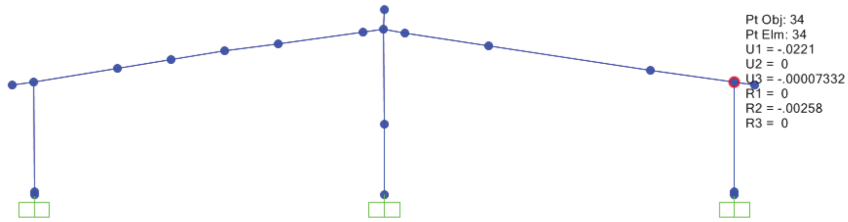


Figure 4 56 Deflection of point 34 – 0.0221m

4.1.5.6 Export joint reaction

Transform from wind load with return period wind speed of 700 years to wind load with those of 50 years

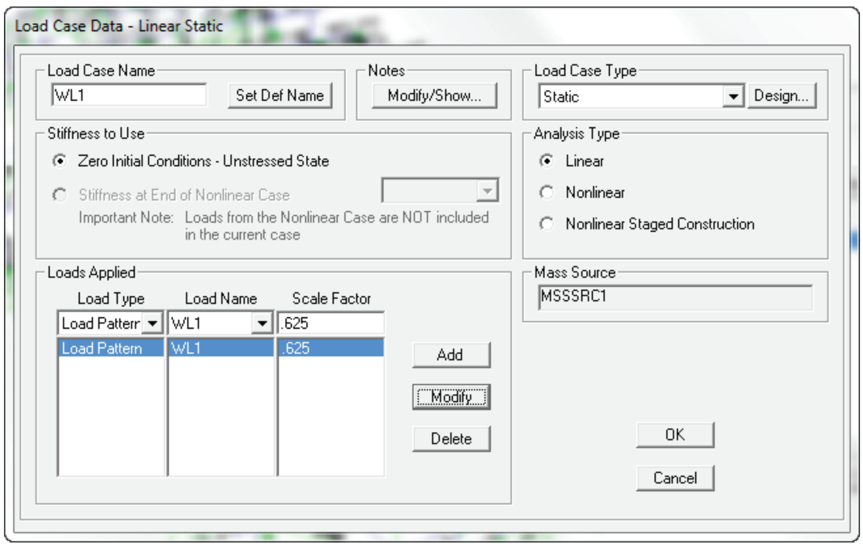


Figure 4 57 Load Case Data – Linear Static form

Changing label name of joints

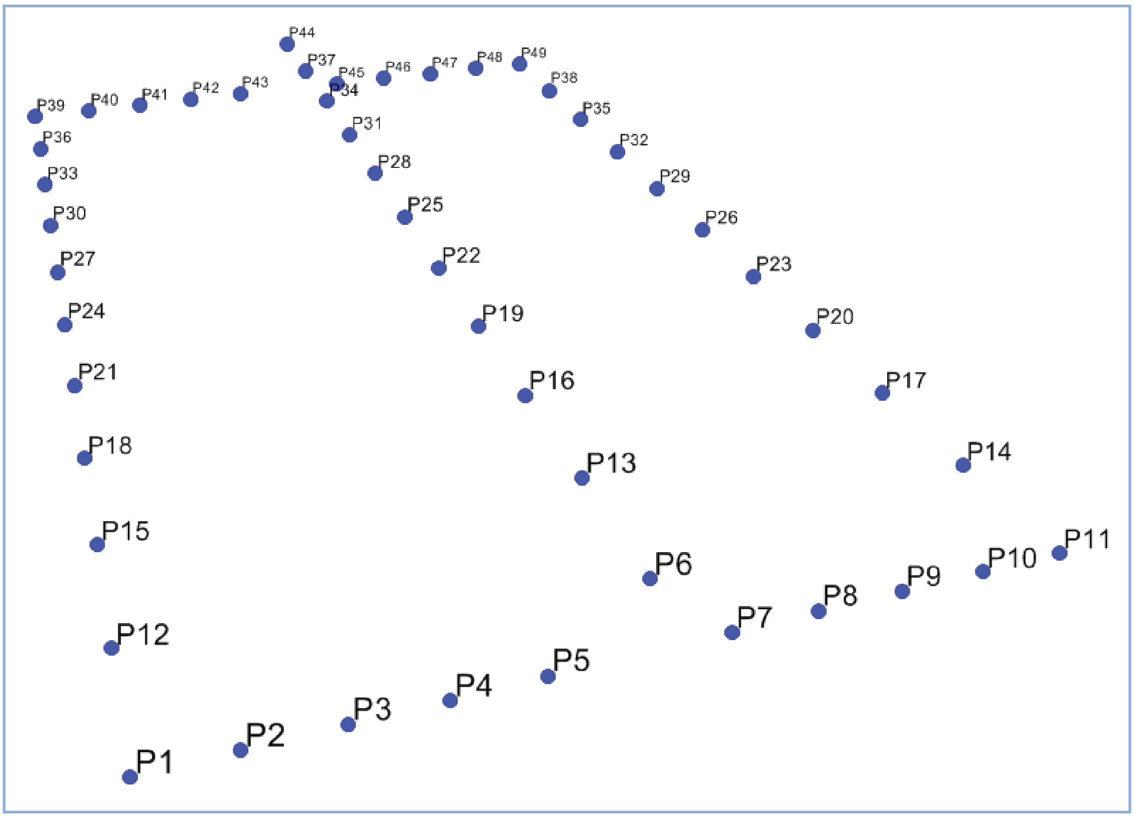


Figure 4 58 Labels after change

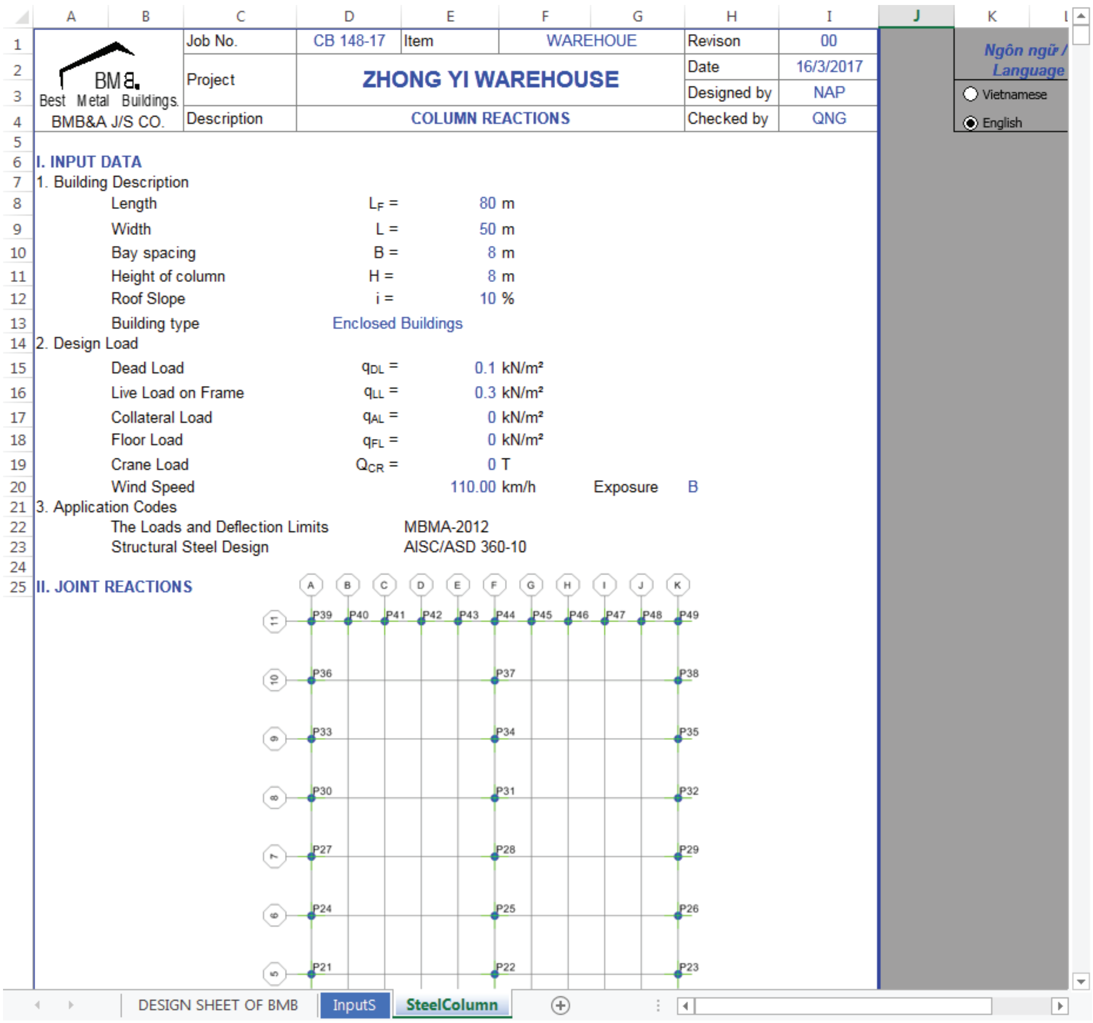


Figure 4 59 Column reactions file

Element Joint Forces - Frames

File View Format-Filter-Sort Select Options

Units: As Noted Element Joint Forces - Frames

Frame Text	Joint Text	OutputCase Text	CaseType Text	F1 KN	F2 KN	F3 KN	M1 KN-m	M2 KN-m
24	P15	DL	LinStatic	6.049	0	18.24	0	0
24	65	DL	LinStatic	-6.049	0	-14.976	0	47.1819
24	P15	LL	LinStatic	13.418	0	32.929	0	0
24	65	LL	LinStatic	-13.418	0	-32.929	0	104.6626
24	P15	WL1	LinStatic	-16.6	0	-35.755	0	0
24	65	WL1	LinStatic	11.281	0	35.755	0	-108.7342
24	P15	WL2	LinStatic	-15.227	0	-23.277	0	0
24	65	WL2	LinStatic	2.269	0	23.277	0	-68.237
24	P15	WL3	LinStatic	0	0	0	0	0
24	65	WL3	LinStatic	0	0	0	0	0
28	P17	DL	LinStatic	-6.049	0	18.24	0	0
28	78	DL	LinStatic	6.049	0	-14.976	0	-47.1819
28	P17	LL	LinStatic	-13.418	0	32.929	0	0
28	78	LL	LinStatic	13.418	0	-32.929	0	-104.6626
28	P17	WL1	LinStatic	2.835	0	-22.773	0	0
28	78	WL1	LinStatic	-13.506	0	22.773	0	63.7311
28	P17	WL2	LinStatic	0.091	0	-9.624	0	0
28	78	WL2	LinStatic	-3.124	0	9.624	0	12.5392
28	P17	WL3	LinStatic	0	0	0	0	0
28	78	WL3	LinStatic	0	0	0	0	0

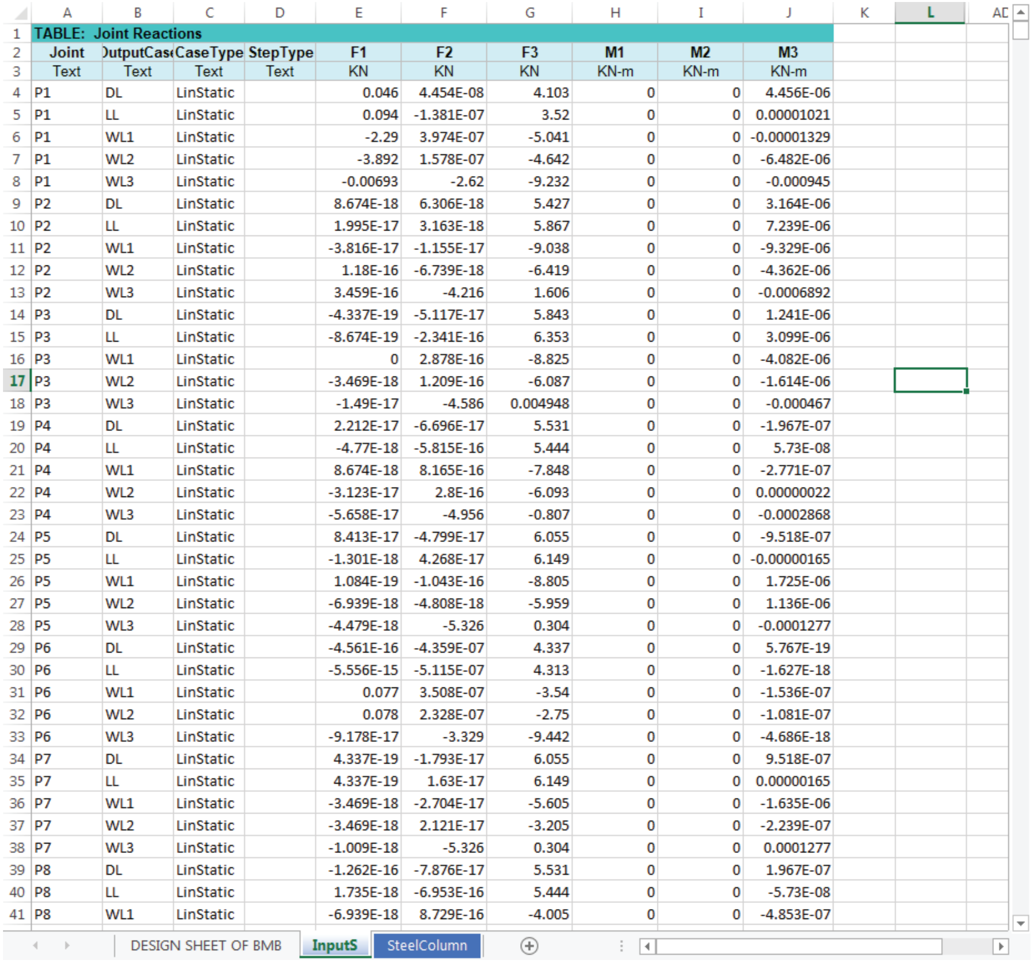
Record: 1 of 2270 Add Tables... Done

Figure 4 60 Element Joints Forces – Frame table

TABLE: Element Joint Forces - Frames

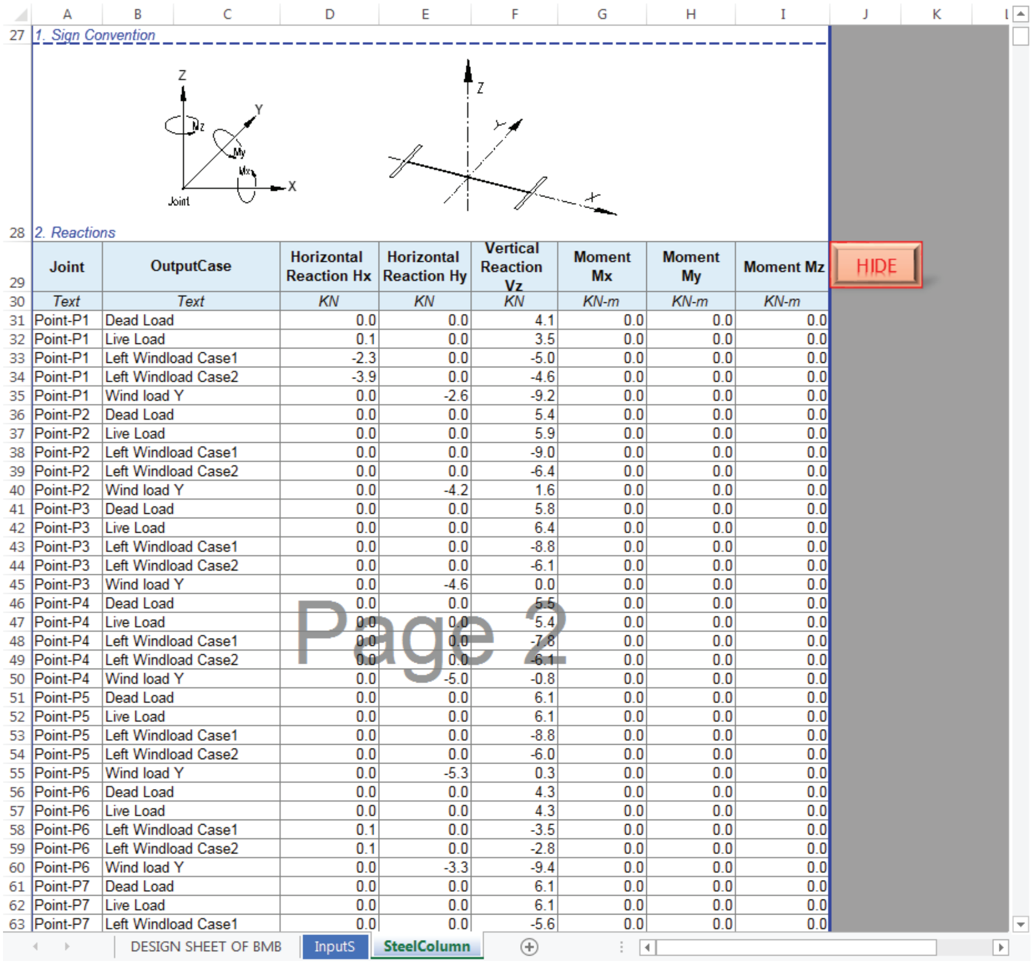
Frame	Joint	OutputCa	CaseTy		F1	F2	F3	M1	M2	M3	FrameEle
39	P1	DL	LinStatic		0.046	4.454E-08	4.103	0	0	0.000004456	39-1
39	P1	LL	LinStatic		0.094	-1.381E-07	3.52	0	0	0.00001021	39-1
39	P1	WL1	LinStatic		-2.29	3.974E-07	-5.041	0	0	-0.00001329	39-1
39	P1	WL2	LinStatic		-3.892	1.578E-07	-4.642	0	0	-0.000006482	39-1
39	P1	WL3	LinStatic		-0.00693	-2.62	-9.232	0	0	-0.000945	39-1
45	P2	DL	LinStatic		8.674E-18	6.306E-18	5.427	0	0	0.000003164	45-1
45	P2	LL	LinStatic		1.995E-17	3.163E-18	5.867	0	0	0.000007239	45-1
45	P2	WL1	LinStatic		-3.816E-17	-1.155E-17	-9.038	0	0	-0.000009329	45-1
45	P2	WL2	LinStatic		1.18E-16	-6.739E-18	-6.419	0	0	-0.000004362	45-1
45	P2	WL3	LinStatic		3.459E-16	-4.216	1.606	0	0	-0.0006892	45-1
47	P3	DL	LinStatic		-4.337E-19	-5.117E-17	5.843	0	0	0.000001241	47-1
47	P3	LL	LinStatic		-8.674E-19	-2.341E-16	6.353	0	0	0.000003099	47-1
47	P3	WL1	LinStatic		0	2.878E-16	-8.825	0	0	-0.000004082	47-1
47	P3	WL2	LinStatic		-3.469E-18	1.209E-16	-6.087	0	0	-0.000001614	47-1
47	P3	WL3	LinStatic		-1.49E-17	-4.586	0.004948	0	0	-0.000467	47-1
49	P4	DL	LinStatic		2.212E-17	-6.696E-17	5.531	0	0	-1.967E-07	49-1
49	P4	LL	LinStatic		-4.77E-18	-5.815E-16	5.444	0	0	5.73E-08	49-1
49	P4	WL1	LinStatic		8.674E-18	8.165E-16	-7.848	0	0	-2.771E-07	49-1
49	P4	WL2	LinStatic		-3.123E-17	2.8E-16	-6.093	0	0	0.00000022	49-1
49	P4	WL3	LinStatic		-5.658E-17	-4.956	-0.807	0	0	-0.0002868	49-1
51	P5	DL	LinStatic		8.413E-17	-4.799E-17	6.055	0	0	-9.518E-07	51-1
51	P5	LL	LinStatic		-1.301E-18	4.268E-17	6.149	0	0	-0.00000165	51-1
51	P5	WL1	LinStatic		1.084E-19	-1.043E-16	-8.805	0	0	0.000001725	51-1
51	P5	WL2	LinStatic		-6.939E-18	-4.808E-18	-5.959	0	0	0.000001136	51-1
51	P5	WL3	LinStatic		-4.479E-18	-5.326	0.304	0	0	-0.0001277	51-1
155	P6	DL	LinStatic		-4.561E-16	-4.359E-07	4.337	0	0	5.767E-19	155-1
155	P6	LL	LinStatic		-5.556E-15	-5.115E-07	4.313	0	0	-1.627E-18	155-1
155	P6	WL1	LinStatic		0.077	3.508E-07	-3.54	0	0	-1.536E-07	155-1
155	P6	WL2	LinStatic		0.078	2.328E-07	-2.75	0	0	-1.081E-07	155-1
155	P6	WL3	LinStatic		-9.178E-17	-3.329	-9.442	0	0	-4.686E-18	155-1
55	P7	DL	LinStatic		4.337E-19	-1.793E-17	6.055	0	0	9.518E-07	55-1
55	P7	LL	LinStatic		4.337E-19	1.63E-17	6.149	0	0	0.00000165	55-1
55	P7	WL1	LinStatic		-3.469E-18	-2.704E-17	-5.605	0	0	-0.000001635	55-1
55	P7	WL2	LinStatic		-3.469E-18	2.121E-17	-3.205	0	0	-2.239E-07	55-1
55	P7	WL3	LinStatic		-1.009E-18	-5.326	0.304	0	0	0.0001277	55-1
57	P8	DL	LinStatic		-1.262E-16	-7.876E-17	5.531	0	0	1.967E-07	57-1
57	P8	LL	LinStatic		1.735E-18	-6.953E-16	5.444	0	0	-5.73E-08	57-1
57	P8	WL1	LinStatic		-6.939E-18	8.729E-16	-4.005	0	0	-4.853E-07	57-1

Figure 4 61 Data of element joint forces after filter and sort



Joint	OutputCase	CaseType	StepType	F1	F2	F3	M1	M2	M3
Text	Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
P1	DL	LinStatic		0.046	4.454E-08	4.103	0	0	4.456E-06
P1	LL	LinStatic		0.094	-1.381E-07	3.52	0	0	0.00001021
P1	WL1	LinStatic		-2.29	3.974E-07	-5.041	0	0	-0.00001329
P1	WL2	LinStatic		-3.892	1.578E-07	-4.642	0	0	-6.482E-06
P1	WL3	LinStatic		-0.00693	-2.62	-9.232	0	0	-0.000945
P2	DL	LinStatic		8.674E-18	6.306E-18	5.427	0	0	3.164E-06
P2	LL	LinStatic		1.995E-17	3.163E-18	5.867	0	0	7.239E-06
P2	WL1	LinStatic		-3.816E-17	-1.155E-17	-9.038	0	0	-9.329E-06
P2	WL2	LinStatic		1.18E-16	-6.739E-18	-6.419	0	0	-4.362E-06
P2	WL3	LinStatic		3.459E-16	-4.216	1.606	0	0	-0.0006892
P3	DL	LinStatic		-4.337E-19	-5.117E-17	5.843	0	0	1.241E-06
P3	LL	LinStatic		-8.674E-19	-2.341E-16	6.353	0	0	3.099E-06
P3	WL1	LinStatic		0	2.878E-16	-8.825	0	0	-4.082E-06
P3	WL2	LinStatic		-3.469E-18	1.209E-16	-6.087	0	0	-1.614E-06
P3	WL3	LinStatic		-1.49E-17	-4.586	0.004948	0	0	-0.000467
P4	DL	LinStatic		2.212E-17	-6.696E-17	5.531	0	0	-1.967E-07
P4	LL	LinStatic		-4.77E-18	-5.815E-16	5.444	0	0	5.73E-08
P4	WL1	LinStatic		8.674E-18	8.165E-16	-7.848	0	0	-2.771E-07
P4	WL2	LinStatic		-3.123E-17	2.8E-16	-6.093	0	0	0.00000022
P4	WL3	LinStatic		-5.658E-17	-4.956	-0.807	0	0	-0.0002868
P5	DL	LinStatic		8.413E-17	-4.799E-17	6.055	0	0	-9.518E-07
P5	LL	LinStatic		-1.301E-18	4.268E-17	6.149	0	0	-0.00000165
P5	WL1	LinStatic		1.084E-19	-1.043E-16	-8.805	0	0	1.725E-06
P5	WL2	LinStatic		-6.939E-18	-4.808E-18	-5.959	0	0	1.136E-06
P5	WL3	LinStatic		-4.479E-18	-5.326	0.304	0	0	-0.0001277
P6	DL	LinStatic		-4.561E-16	-4.359E-07	4.337	0	0	5.767E-19
P6	LL	LinStatic		-5.556E-15	-5.115E-07	4.313	0	0	-1.627E-18
P6	WL1	LinStatic		0.077	3.508E-07	-3.54	0	0	-1.536E-07
P6	WL2	LinStatic		0.078	2.328E-07	-2.75	0	0	-1.081E-07
P6	WL3	LinStatic		-9.178E-17	-3.329	-9.442	0	0	-4.686E-18
P7	DL	LinStatic		4.337E-19	-1.793E-17	6.055	0	0	9.518E-07
P7	LL	LinStatic		4.337E-19	1.63E-17	6.149	0	0	0.00000165
P7	WL1	LinStatic		-3.469E-18	-2.704E-17	-5.605	0	0	-1.635E-06
P7	WL2	LinStatic		-3.469E-18	2.121E-17	-3.205	0	0	-2.239E-07
P7	WL3	LinStatic		-1.009E-18	-5.326	0.304	0	0	0.0001277
P8	DL	LinStatic		-1.262E-16	-7.876E-17	5.531	0	0	1.967E-07
P8	LL	LinStatic		1.735E-18	-6.953E-16	5.444	0	0	-5.73E-08
P8	WL1	LinStatic		-6.939E-18	8.729E-16	-4.005	0	0	-4.853E-07

Figure 4 62 Data after paste



Joint	OutputCase	Horizontal Reaction Hx	Horizontal Reaction Hy	Vertical Reaction Vz	Moment Mx	Moment My	Moment Mz
Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
Point-P1	Dead Load	0.0	0.0	4.1	0.0	0.0	0.0
Point-P1	Live Load	0.1	0.0	3.5	0.0	0.0	0.0
Point-P1	Left Windload Case1	-2.3	0.0	-5.0	0.0	0.0	0.0
Point-P1	Left Windload Case2	-3.9	0.0	-4.6	0.0	0.0	0.0
Point-P1	Wind load Y	0.0	-2.6	-9.2	0.0	0.0	0.0
Point-P2	Dead Load	0.0	0.0	5.4	0.0	0.0	0.0
Point-P2	Live Load	0.0	0.0	5.9	0.0	0.0	0.0
Point-P2	Left Windload Case1	0.0	0.0	-9.0	0.0	0.0	0.0
Point-P2	Left Windload Case2	0.0	0.0	-6.4	0.0	0.0	0.0
Point-P2	Wind load Y	0.0	-4.2	1.6	0.0	0.0	0.0
Point-P3	Dead Load	0.0	0.0	5.8	0.0	0.0	0.0
Point-P3	Live Load	0.0	0.0	6.4	0.0	0.0	0.0
Point-P3	Left Windload Case1	0.0	0.0	-8.8	0.0	0.0	0.0
Point-P3	Left Windload Case2	0.0	0.0	-6.1	0.0	0.0	0.0
Point-P3	Wind load Y	0.0	-4.6	0.0	0.0	0.0	0.0
Point-P4	Dead Load	0.0	0.0	5.5	0.0	0.0	0.0
Point-P4	Live Load	0.0	0.0	5.4	0.0	0.0	0.0
Point-P4	Left Windload Case1	0.0	0.0	-7.8	0.0	0.0	0.0
Point-P4	Left Windload Case2	0.0	0.0	-6.1	0.0	0.0	0.0
Point-P4	Wind load Y	0.0	-5.0	-0.8	0.0	0.0	0.0
Point-P5	Dead Load	0.0	0.0	6.1	0.0	0.0	0.0
Point-P5	Live Load	0.0	0.0	6.1	0.0	0.0	0.0
Point-P5	Left Windload Case1	0.0	0.0	-8.8	0.0	0.0	0.0
Point-P5	Left Windload Case2	0.0	0.0	-6.0	0.0	0.0	0.0
Point-P5	Wind load Y	0.0	-5.3	0.3	0.0	0.0	0.0
Point-P6	Dead Load	0.0	0.0	4.3	0.0	0.0	0.0
Point-P6	Live Load	0.0	0.0	4.3	0.0	0.0	0.0
Point-P6	Left Windload Case1	0.1	0.0	-3.5	0.0	0.0	0.0
Point-P6	Left Windload Case2	0.1	0.0	-2.8	0.0	0.0	0.0
Point-P6	Wind load Y	0.0	-3.3	-9.4	0.0	0.0	0.0
Point-P7	Dead Load	0.0	0.0	6.1	0.0	0.0	0.0
Point-P7	Live Load	0.0	0.0	6.1	0.0	0.0	0.0
Point-P7	Left Windload Case1	0.0	0.0	-5.6	0.0	0.0	0.0

Figure 4 63 Joint reactions after hide rows

4.1.6. Design Example 4b: High-Rise Building Sap Model

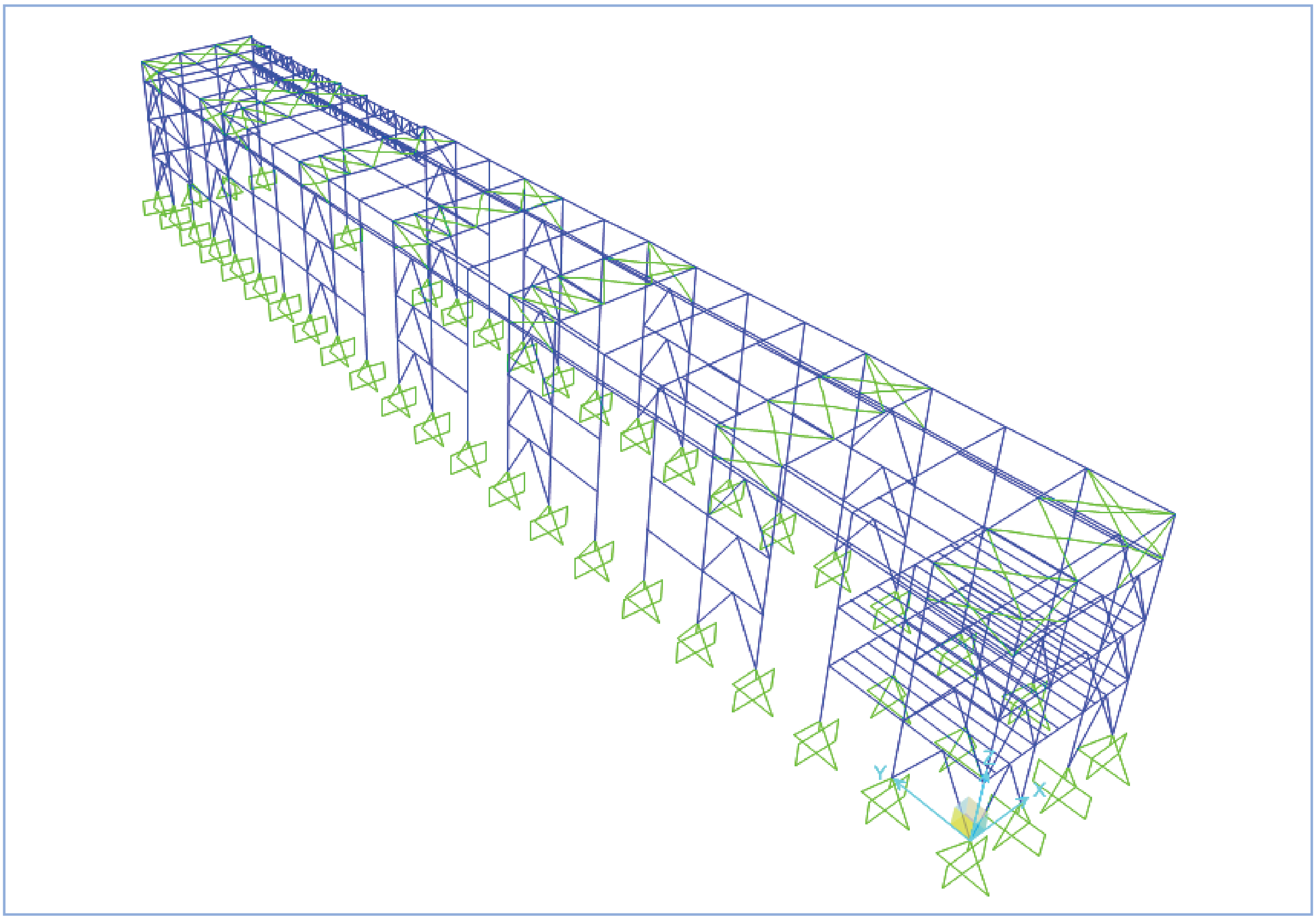


Figure 4 64 High-rise building model

4.1.6.1 Defining

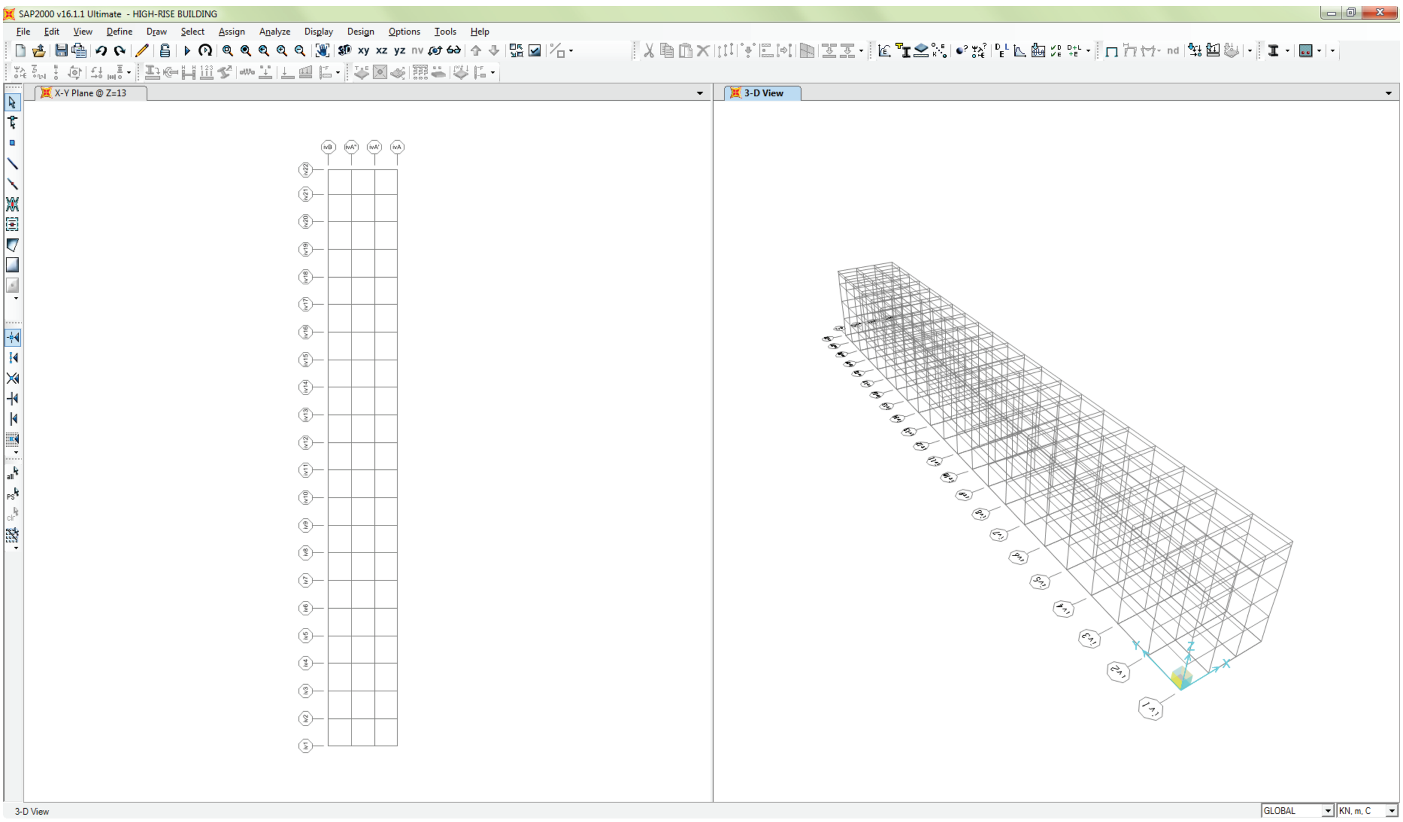


Figure 4 65 Setting up geometry and default units

BMB. Best Metal Buildings. BMB & A J/SC	PROJECT INFORMATION FORM (PIF) (THÔNG TIN TRIỂN KHAI DỰ ÁN)	Code of doc.	BM-7.2.1-02
		Date of issue	24/10/2016
		Times of issue	04
		Project status	Priority □ / Normal ■
		Breakdown form	BMB&A ■ / Customer □

PROJECT INFORMATION

Project name:	Ruby Project				
Location	*Thilawa Zone B*				
Consultancy/ Design Company	Royal Haskoning □	Architype □	Meinhardt □	B.I.S.T □	Others □ ____________
Quotation no.:MM727/17	Start date:01/12/2017	Finish date:03/12/2017	Building no.:1	Area no.:1	
Requirements: 1.New Est. ■ / 2.Arch.drwgs ■ / 3.Apprl.drwgs □ / 4.Design calculation □ / 5.Shop drwgs □					

BUILDING DESCRIPTION

Material Specifications	1.BMB Standard ■/ 2.Tender's Standard □		
	Built-up member: fy= 24.5 kN/ cm²	Purlin: Painted □ / Galvanized ■ fy=45 kN/cm²	
	Anchor bolt: 5.6 ■ / 8.8 □	4. Connection bolt: 10.9	
	Paint/ Finishing: Alkyd ■ / Galvanized □ / Epoxy □ / Thickness: 140 micron		
Exposure	B ■	C □	D □
Qty. of identical Bldings	1	Usage	Paint Coating Process (CPL Building) **GRIDLINE iv1 to iv22**
Type of building	Single slope	Roof slope (%)	5 %
Width (M)	20 (GL iii21 to middle column)	Width Module (M)	1@20 (center to center steel column)
Length (M)	167 (GL iv1 to iv22)	Bay Spacing (M)	20@8 + 1@7 (center to center steel column)

Height (M)	23 – at GL ivB	Eave Height ■	Clear Height □
Type of End Frames	Post & Beam ■	One, Axis.................. □	Both ■
	Main Rigid Frame □	One, Axis.................. □	Both □
	Concrete Columns and rafters (by others) □	One, Axis.................. □	Both □
Type of Wall Bracing	Diagonal Rod ■ / Cable □ / Portal frame □ / None □		
Eave Condition	Gutter & Downspout ■/ RC Gutter (by other) □ / Gutter & Downspout (by other) □		

Notes: **DO NOT put any column in GL ivA : iv16 ; iv17 ; iv19 ; iv20 ; iv21**

DESIGN LOADS	Follow BMB's design ■ / Follow Customer's design □		
Live Load on Roof (kN/m²)	0.57 (58.1 KG/m²) ■/ Others		
Live Load on Frame (kN/m²)	0.30 (30.6 KG/m²) ■/ Others		
Wind Speed (KM/h)	110 (30.6 M/s) □ / 130 (36.1 M/s) □ / 160 (44 M/s) ■ / Other (……. M/s) □		
Collateral Load (KG/m²)		Concentrated load (KG/m)	
Collateral Load hang on purlin □		Collateral Load don't hang on purlin □	

Notes: Earthquake Zone Global Seismic Hazard Zone IIB

ROOF, WALL SHEETINGS

Roof panel: Yes ■ None □	Bare Zinc-Alum AZ… □	Pre-Painted Zinc-Alum AZ… □	7 ribs □ / 5 ribs □
	Pre-Painted Zinc-Alum AZ… ■	Thickness: 0.50mm	Seam ■ / Clip-lock □
Wall panel: Yes ■ None □	Bare Zinc-Alum □	Pre-Painted Zinc-Alum AZ… □	7 ribs panel ■
	Pre-Painted Zinc-Alum AZ… ■	Thickness: 0.50mm	5 ribs panel □

Notes:				
INSULATION Roof ☐ / Wall ■ / None ☐				
Fiberglass thick 50mm ■	Air Bubble P2 ☐	Rockwall ☐	Sandwich ☐	Other ☐
Fiberglass thick 100mm ☐	Air Bubble A2 ☐	PU ☐	Panel ☐	
Fiberglass density: 10-12 KG/m^3 ■ / Or other kind ☐ Density: .... KG/m^3				
Other insulation:				
Notes:				

OPEN WALL CONDITIONS	
Both Side wall	1m for brick wall - sheet to roof ■
End wall GL ivB	1m for brick wall - sheet to roof ■
End wall GL ivA – iv1 to iv15	1m for brick wall - sheet to roof ■
End wall GL ivA – iv15 to iv22	Open 20.5 M - sheet to roof ■
Notes:	

ROOF MONITOR None ☐			
Type	Jack roof ■	Ridge vent ☐	Other ☐
Overall width (M)	3.5	Length (M)	= ~~6~~ **7 (pcs) x 16.880 = 118.16**
Eave Condition	Curved Eave ■	Open Eave ☐	Other ☐
Roof panel	Same as roof panel ■ / Other ☐ .... thick .... mm (nominal)		
Wall panel	Same as wall panel ■/ Other ☐ .... thick.............. mm (nominal)		
Notes:			

MEZZANINE SLAB None ■		
Type	Concrete slab ☐ thick............. mm / Decking panel ☐ thick .... mm	
	Mezzanine with Checker Plate ☐ thick .... mm	
Quantity	Mezzanine slab 1	Mezzanine slab 2
Location		
Area (m^2)		
Live load (KG/m^2)		
Dead load (KG/m^2)		
Others load (KG/m^2)		
Height up to top of Concrete Slab (M)		
Type of support shear load	Shear stud D20 ☐ Angle V ☐	
Notes:		

ROOF EXTENSIONS None ■		
Panel	Same roof panel ☐/ Other ☐ .... thick............... mm (nominal)	
Location	Side wall	End wall
Soffit panel	Yes ☐ / None ☐	Yes ☐ / None ☐
Quantity		
Width (M)		
Length (M)		
Notes:		

CANOPY		None ■	
Panel	Same as roof panel ☐ / Other ☐ .... thick............... mm (nominal)		
Eave Condition	Curved Eave ☐	Open Eave ☐	Other ☐
Location	Side wall	End wall	
Soffit panel	Yes ☐ / None ☐	Yes ☐ / None ☐	
Quantity			
Width (M)			
Length (M)			
Gutter, Downspout	Yes ☐	None ☐	
Notes:			

FASCIA		None ■
Panel	Same as wall panel ☐ / Other ☐ .... thick............... mm (nominal)	
Location	Side wall	End wall
Soffit panel	Yes ☐ / None ☐	Yes ☐ / None ☐
Type		
High (M)		
Length (M)		
Fascia projection (M)		
Notes:		

PARTITIONS		None ☐
Quantity	Partition 1	Partition 2
Location		
Length (M)		
High (M)		
Type	Vertical of house ☐ / Horizontal of house ☐	Vertical of house ☐ / Horizontal of house ☐
Wall Condition	One side ☐ / Double side ☐	One side ☐ / Double side ☐
	Fully Sheeted ☐ / Block wall ☐	Fully Sheeted ☐ / Blockwall ☐
	Open up to .... M ☐ / Open up to access ☐	Open up to .... M ☐ / Open up to access ☐
Wall Panel	Same as wall panel ☐ / Other ☐ .... thick............... mm (nominal)	
Notes:		

CRANE SYSTEMS (The number of effective cranes on one column: 1)				None ☐
Quantity	**Crane No.1**	Crane No.2	Crane No.3	Crane No.4
Location	Full length of building			
Crane capacity (TON)	**10T**			
Crane Span (M)	20			
Length of Crane Run (M)	167			
Bracket Height	+19.5m			

<u>Notes:</u> Only supply, install running beam ■ / Our supply does not include the Crane System and Crane Rails ☐	
Entire supply of Crane System include Crane, running beam, for crane rails and electrical systems ...☐	
Notes:	

ACCESSORIES - UTILITY ITEMS OR OTHER REQUIREMENTS (If Applicable)

Accessories	Length	Width	Height	Qty.	Accessories	Length	Width	Height	Qty.
Rolling door					Rlg.door (motor)				
Door frame	0.85		2	10	Steel stair				
Double sliding dr					Stair handrail				
Window (Steel)					Opening frame				
Window (Almn)					Wall light	1		20.2	10
Steel Louver (+ mesh)	5		1.5	10	Ladder				

Notes: Only supply frame for doors,sliding doors,rolling doors,windows

Project Eng.
(Sign & write down full name)

Nguyen Minh Phuc Nguyen

Notes for using this PIF

1. Must fill in Exact Country of Project & Name of Consultancy/ Design Company at top of PIF

2. Must make sketch drawing or provide architectural drawing.

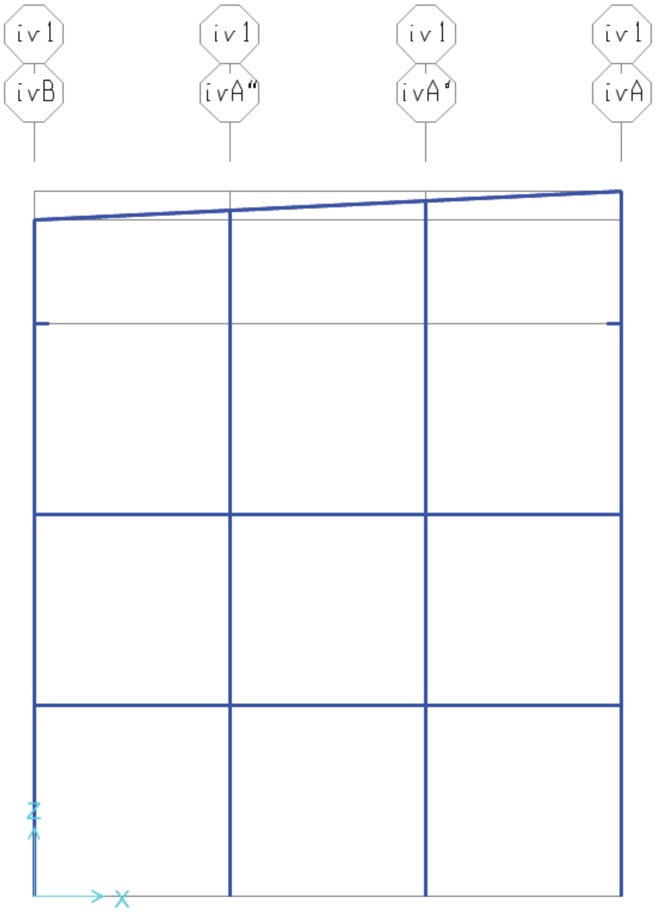
Define Section Properties as table below:

Table 4 4 Frame Section Properties

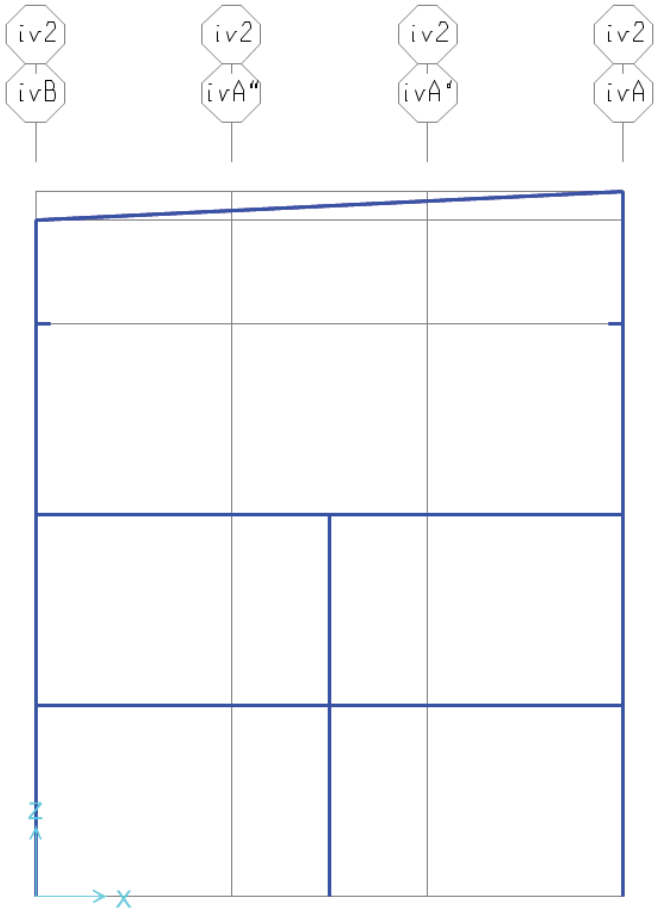
SectionName	Material	Shape	d	bf1	tf1	tw	bf2	tf2
Text	Text	Text	m	m	m	m	m	m
1K		Nonprismatic						
1K516248	STEEL3450	I/Wide Flange	0.516	0.248	0.012	0.006	0.248	0.01
1K900248	STEEL3450	I/Wide Flange	0.9	0.248	0.012	0.006	0.248	0.01
1KA		Nonprismatic						
2K	STEEL3450	I/Wide Flange	0.516	0.184	0.01	0.005	0.184	0.01
3K		Nonprismatic						
3K350184	STEEL3450	I/Wide Flange	0.35	0.184	0.01	0.005	0.184	0.01
3K766184	STEEL3450	I/Wide Flange	0.766	0.184	0.01	0.005	0.184	0.01
4K766184	STEEL3450	I/Wide Flange	0.766	0.184	0.01	0.005	0.184	0.01
5K		Nonprismatic						
5K-766	STEEL3450	I/Wide Flange	0.766	0.212	0.01	0.006	0.212	0.01
5K-900	STEEL3450	I/Wide Flange	0.9	0.212	0.01	0.006	0.212	0.01
B1	STEEL3450	I/Wide Flange	0.31	0.148	0.006	0.005	0.148	0.006
B2	STEEL3450	I/Wide Flange	0.512	0.164	0.008	0.005	0.164	0.008
BR	STEEL3450	I/Wide Flange	0.516	0.212	0.01	0.008	0.212	0.01
C1	STEEL3450	I/Wide Flange	0.8	0.496	0.018	0.012	0.496	0.018
C2		Nonprismatic						
C2-D	STEEL3450	I/Wide Flange	0.8	0.372	0.016	0.008	0.372	0.016
C2-T	STEEL3450	I/Wide Flange	0.8	0.372	0.014	0.008	0.372	0.014
C3	STEEL3450	I/Wide Flange	0.8	0.372	0.016	0.01	0.372	0.016
CD	STEEL3450	I/Wide Flange	0.524	0.248	0.014	0.01	0.248	0.014
CDH	STEEL3450	I/Wide Flange	0.52	0.372	0.012	0.006	0.372	0.012
CR (500~800~500)		Nonprismatic						
CR(800~800~500)		Nonprismatic						
CR2(800~1500~800)		Nonprismatic						
CR2-1500	STEEL3450	I/Wide Flange	1.5	0.372	0.014	0.01	0.372	0.014
CR2-800	STEEL3450	I/Wide Flange	0.8	0.372	0.014	0.01	0.372	0.014
CR3(800~1800~800)		Nonprismatic						
CR3-1800	STEEL3450	I/Wide Flange	1.8	0.372	0.016	0.012	0.372	0.016
CR3-800	STEEL3450	I/Wide Flange	0.8	0.372	0.016	0.012	0.372	0.016
CR500184	STEEL3450	I/Wide Flange	0.5	0.184	0.008	0.006	0.184	0.008
CR800184	STEEL3450	I/Wide Flange	0.8	0.184	0.008	0.006	0.184	0.008
D310164	STEEL3450	I/Wide Flange	0.31	0.164	0.006	0.005	0.164	0.006
EB	STEEL2350	Box/Tube	0.15	0.15	0.0028	0.0028		
G1	STEEL3450	I/Wide Flange	0.77	0.184	0.012	0.008	0.184	0.012
I-200	STEEL3450	I/Wide Flange	0.2	0.164	0.008	0.005	0.164	0.008
I-314	STEEL3450	I/Wide Flange	0.314	0.184	0.008	0.005	0.184	0.008
I-392212	STEEL3450	I/Wide Flange	0.392	0.248	0.01	0.006	0.248	0.01
IPE200	STEEL2350	I/Wide Flange	0.2	0.1	0.008	0.0055	0.1	0.008
JB1	STEEL3450	I/Wide Flange	0.392	0.212	0.01	0.006	0.212	0.01
KDH	STEEL3450	I/Wide Flange	0.314	0.164	0.008	0.005	0.164	0.008
P141	STEEL2350	Pipe	0.1413			0.00396		
P168	STEEL2350	Pipe	0.1683			0.00396		
ST264164	STEEL3450	I/Wide Flange	0.264	0.164	0.008	0.005	0.164	0.008
V100x10	STEEL2350	Angle	0.1	0.1	0.01	0.01		

4.1.6.2 Modeling

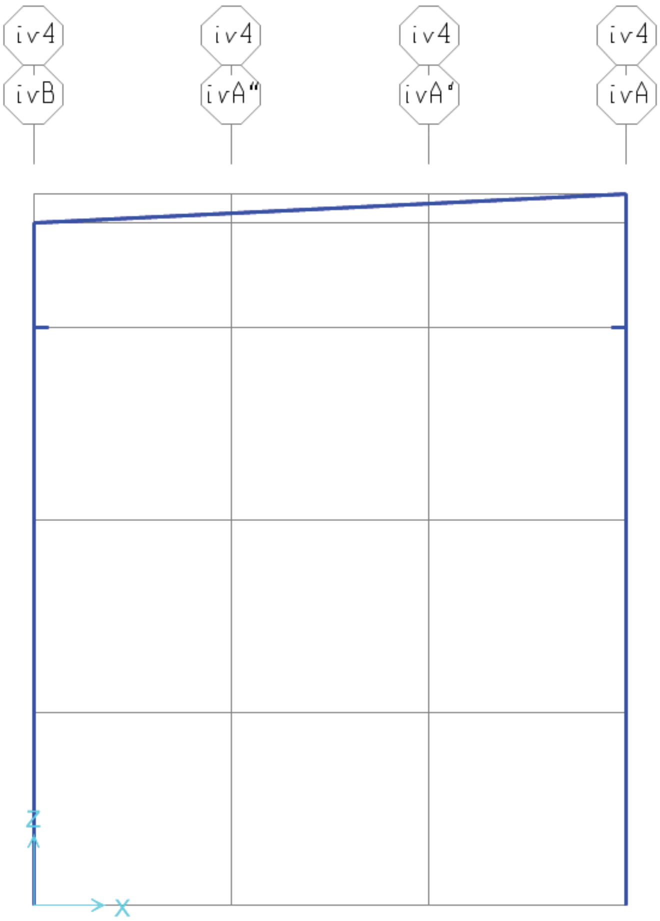
Create general model



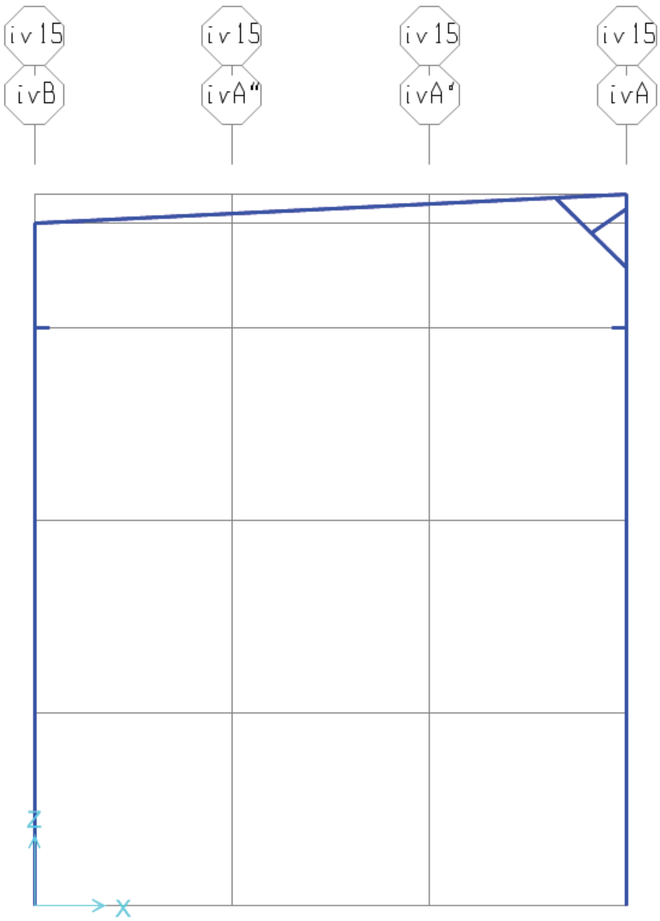
End wall frame – GL iv1



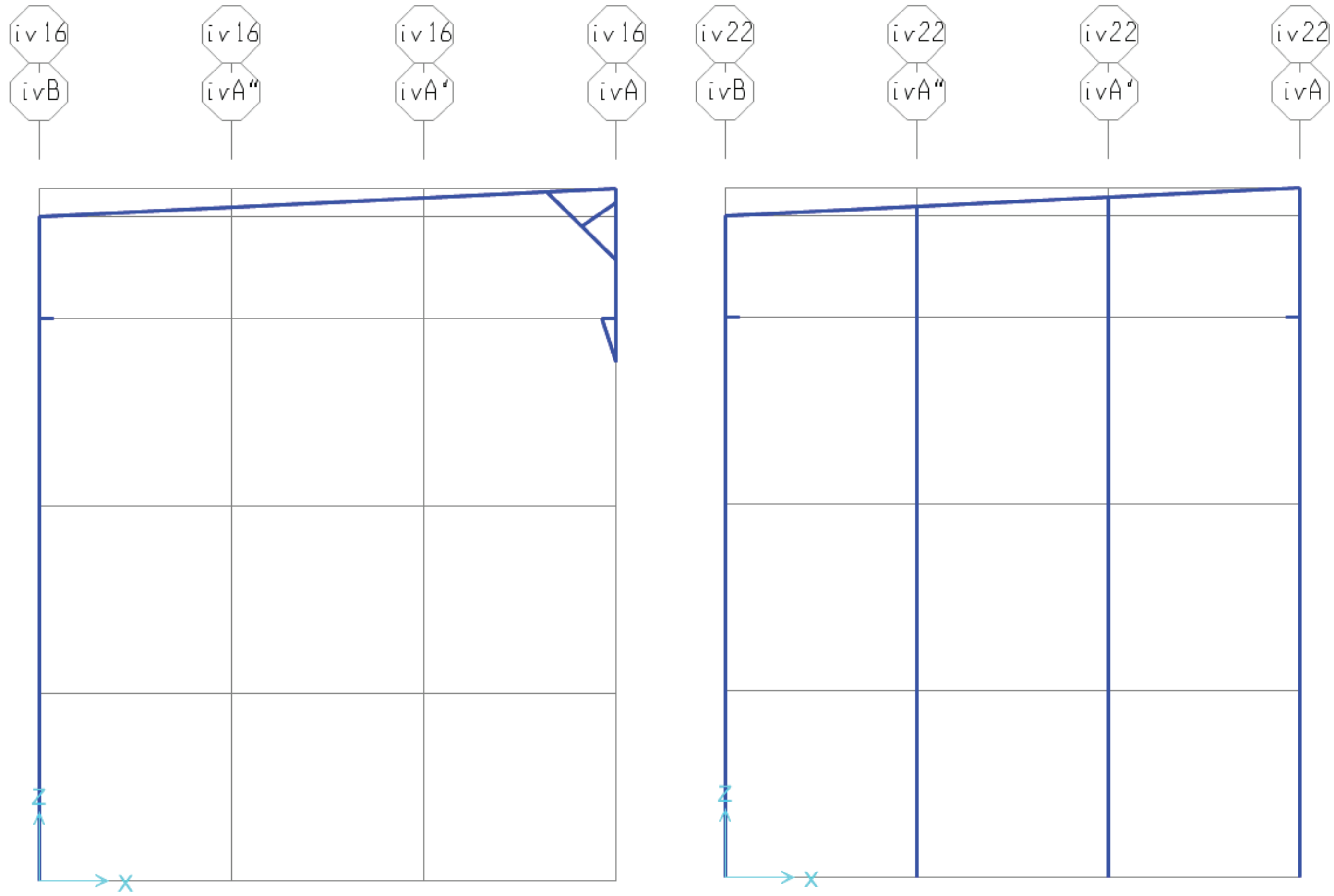
Main frame – GL iv2 to GL iv3



Main frame – GL iv4 to GL iv14



Main frame – GL iv15 & GL iv18



Main frame – GL iv16 to GL iv21 except GL iv18 *Main frame – GL iv22*

Figure 4 66 Main Frame

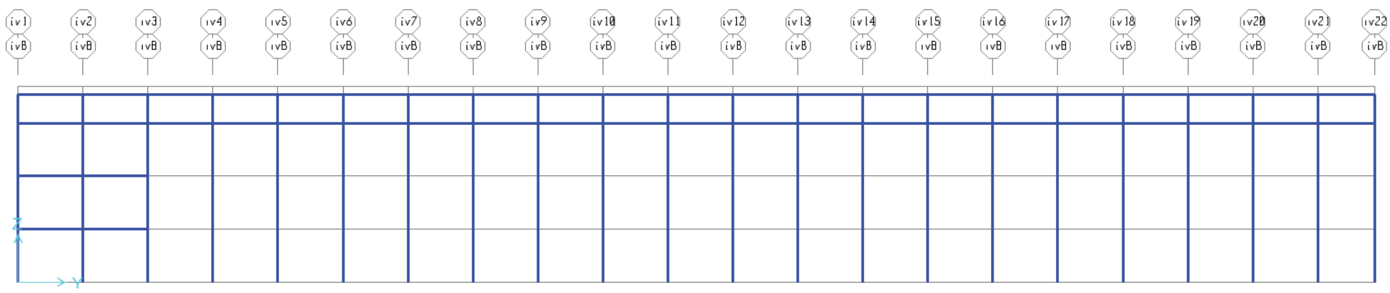


Figure 4 67 Elevation – GL ivB

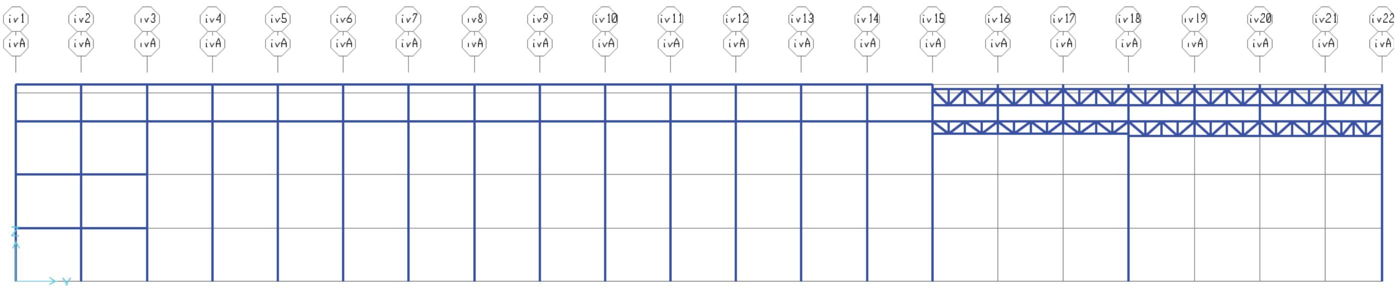


Figure 4 68 Elevation – GL ivA

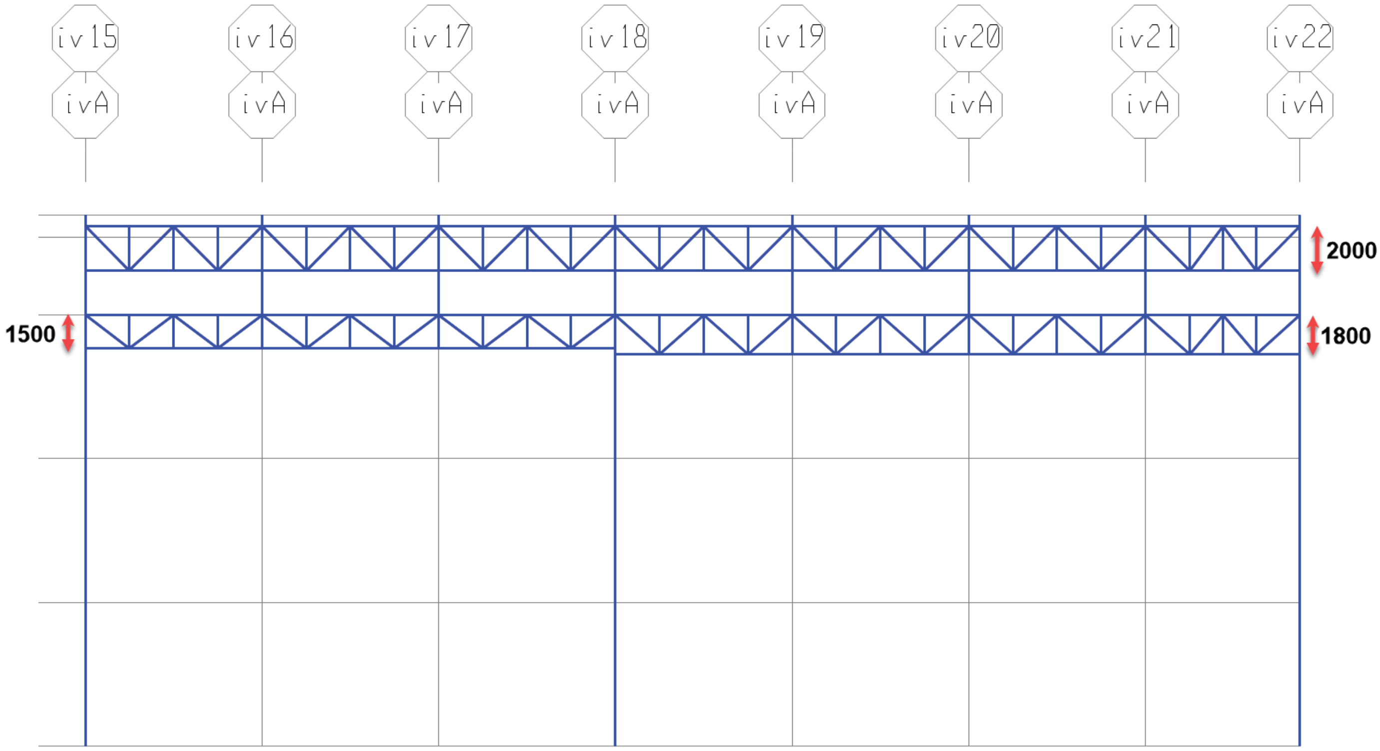


Figure 4 69 Jack beam and crane bracing

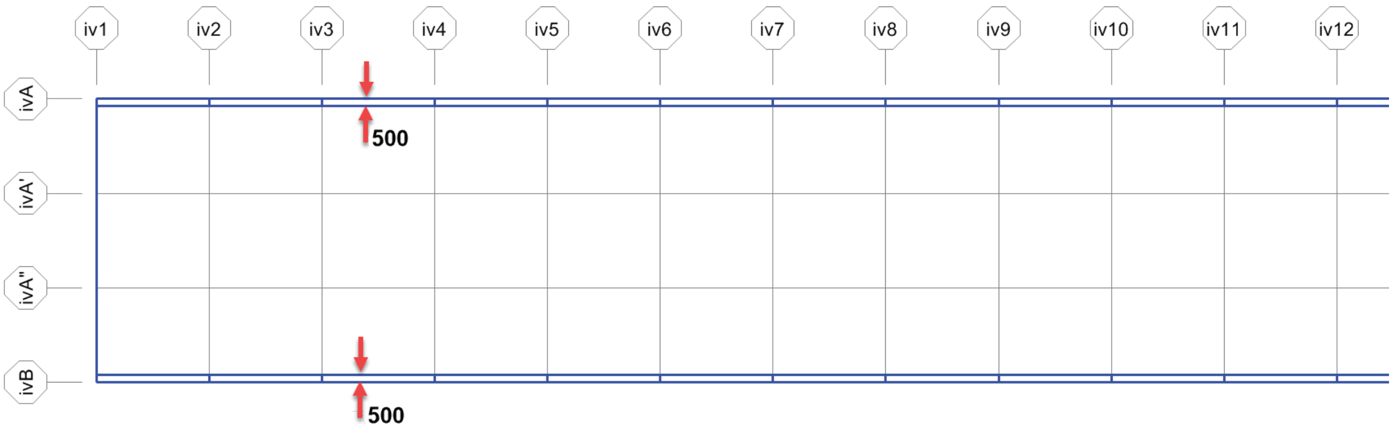


Figure 4 70 Crane runway beam & bracing – level +19.5m

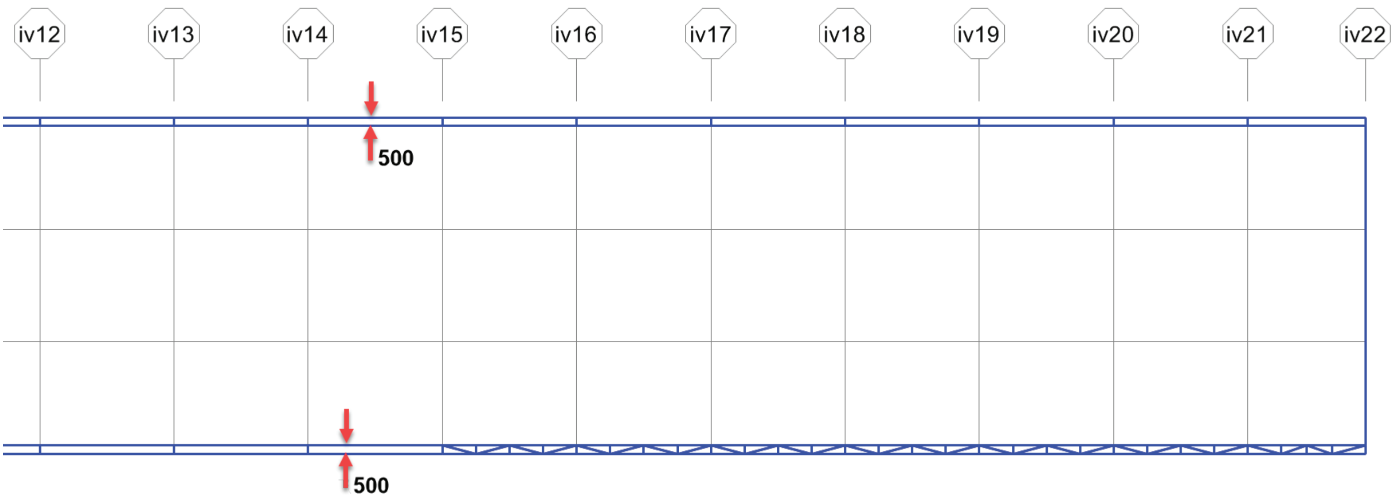


Figure 4 71 Crane runway beam & bracing – level +19.5m

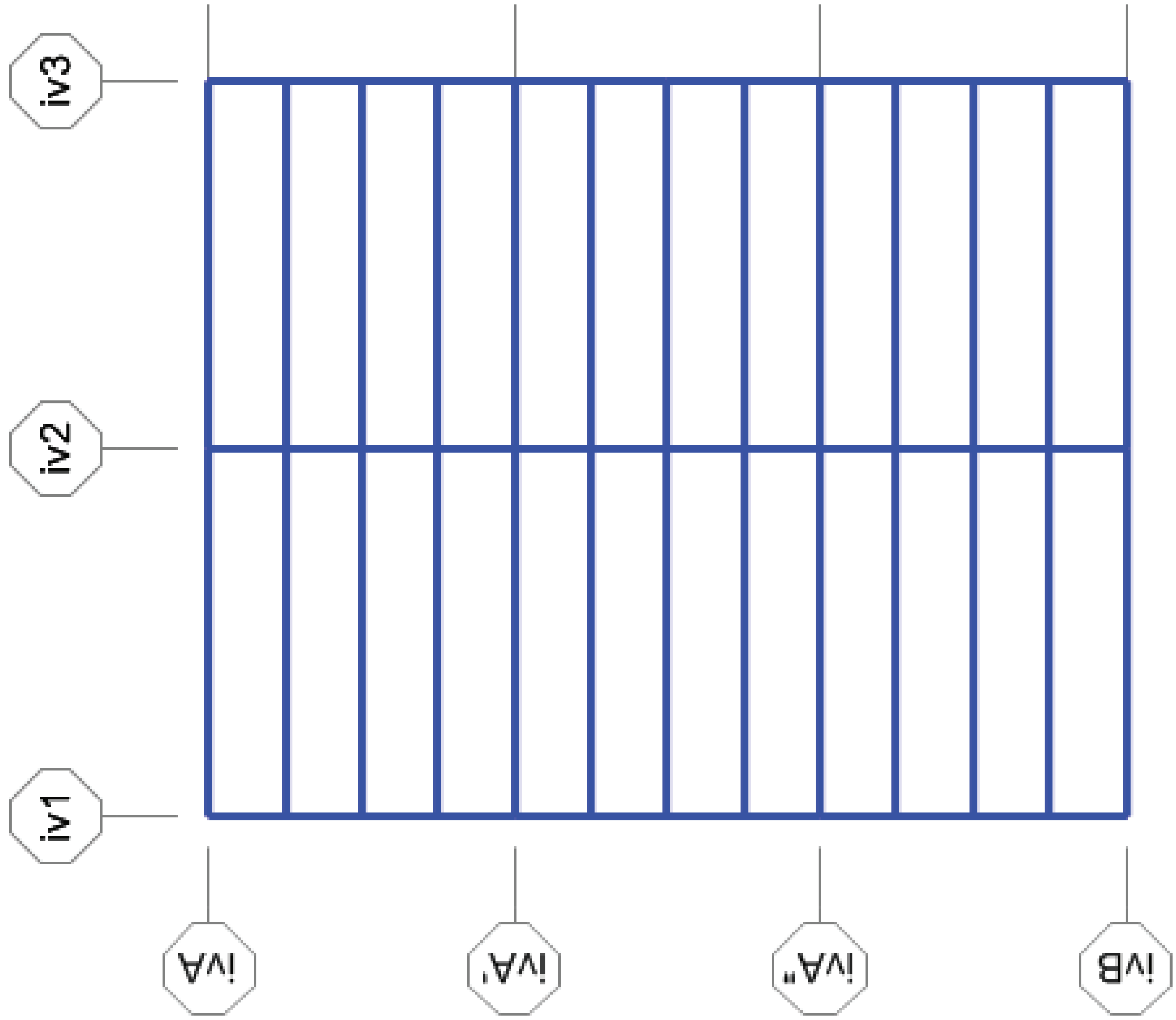


Figure 4 72 Mezzanine floor – level +6.5m, +13m

Define load patterns and load combinations

Click the **Define > Load Patterns** command to access the **Define Load Patterns** form. In **Load Patterns** area, create load patterns including DL, LL, FL1, FL2, FL3, WL1, WL2, WL3, WL4, WL5, WL6, WL7, WL8, EL1, EL2, CR1, CR2 as shown in figure below.

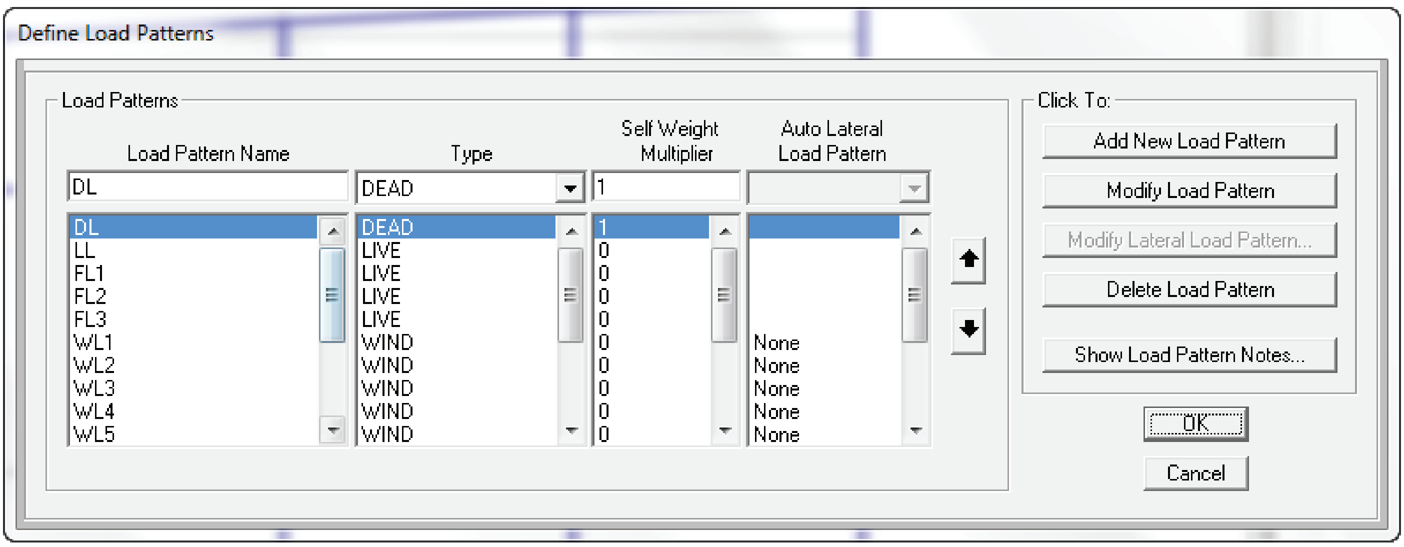


Figure 4 73 Define Load Patterns form

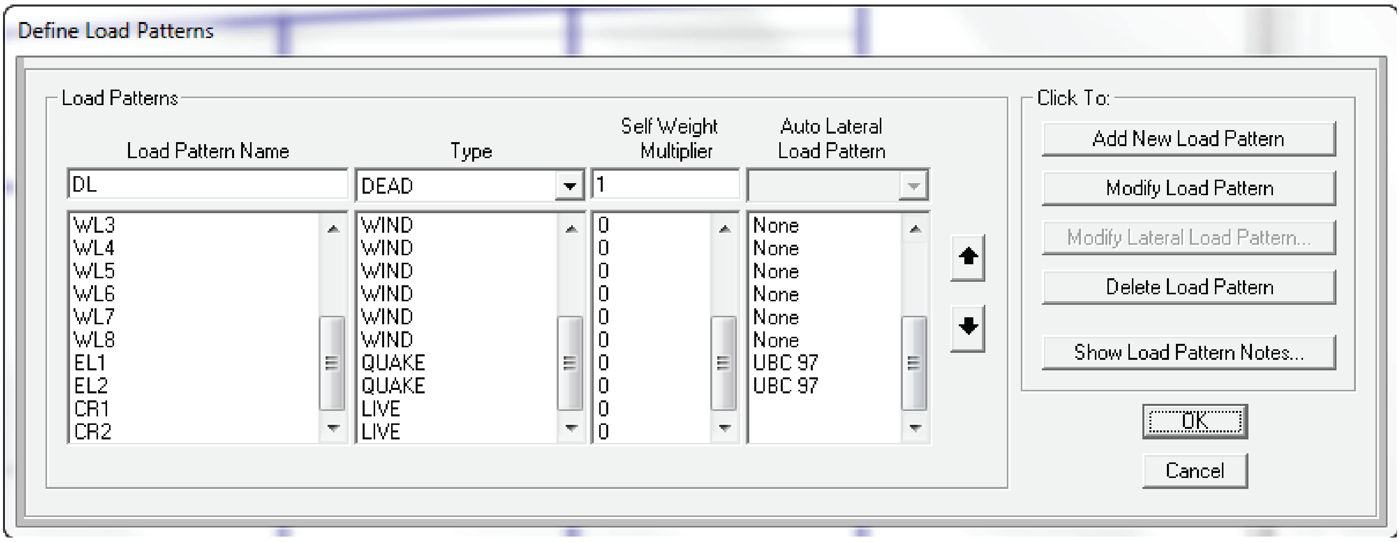


Figure 4 74 Define Load Patterns form

Select EL1 in Load Patterns area and then click the **Modify Lateral Load Pattern** button in Click to area to display the **1997 UBC Seismic Load Pattern** form. Enter all information as figure below.

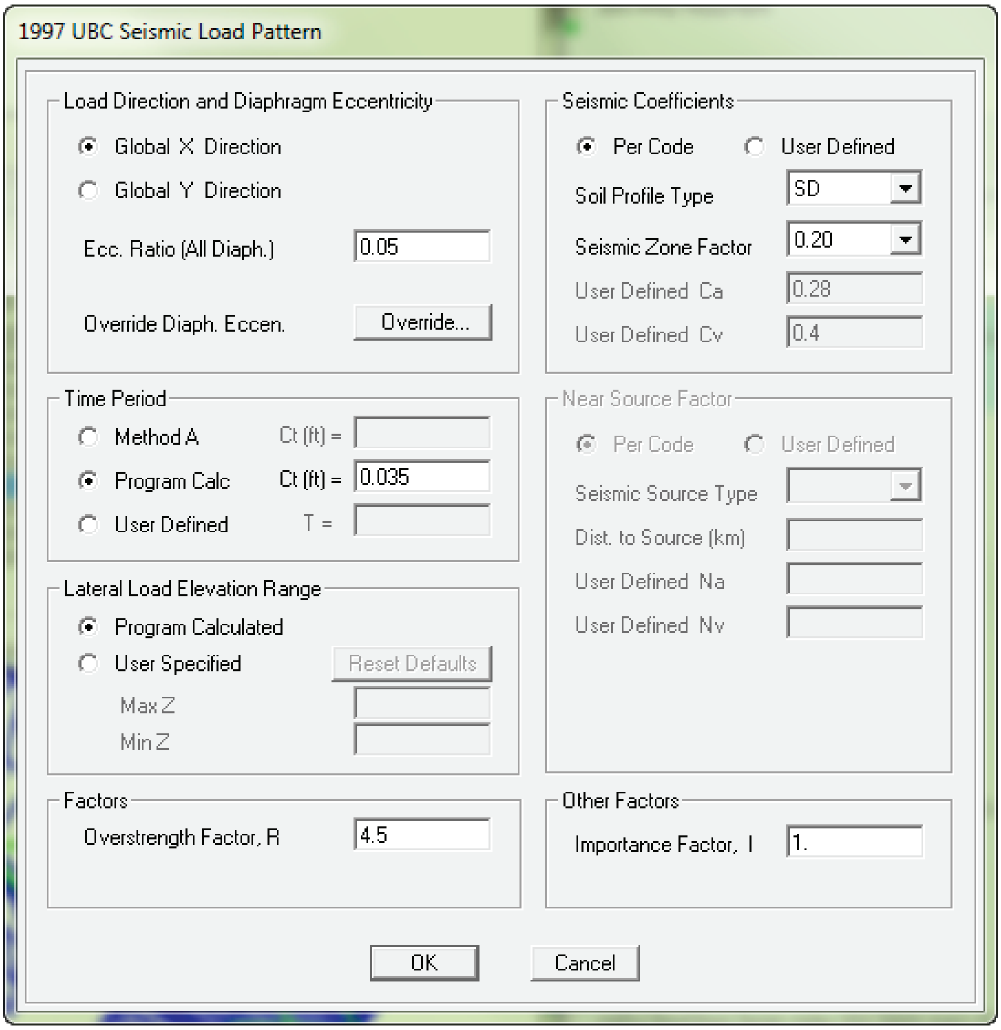


Figure 4 75 EL1 definition

Select EL2 in **Load Patterns** area and then click the **Modify Lateral Load Pattern** button in Click to area to display the **1997 UBC Seismic Load Pattern** form. Enter all information as figure below.

Note: Based on **UBC 97** standard, we have all the parameters for seismic zone 2B as above.

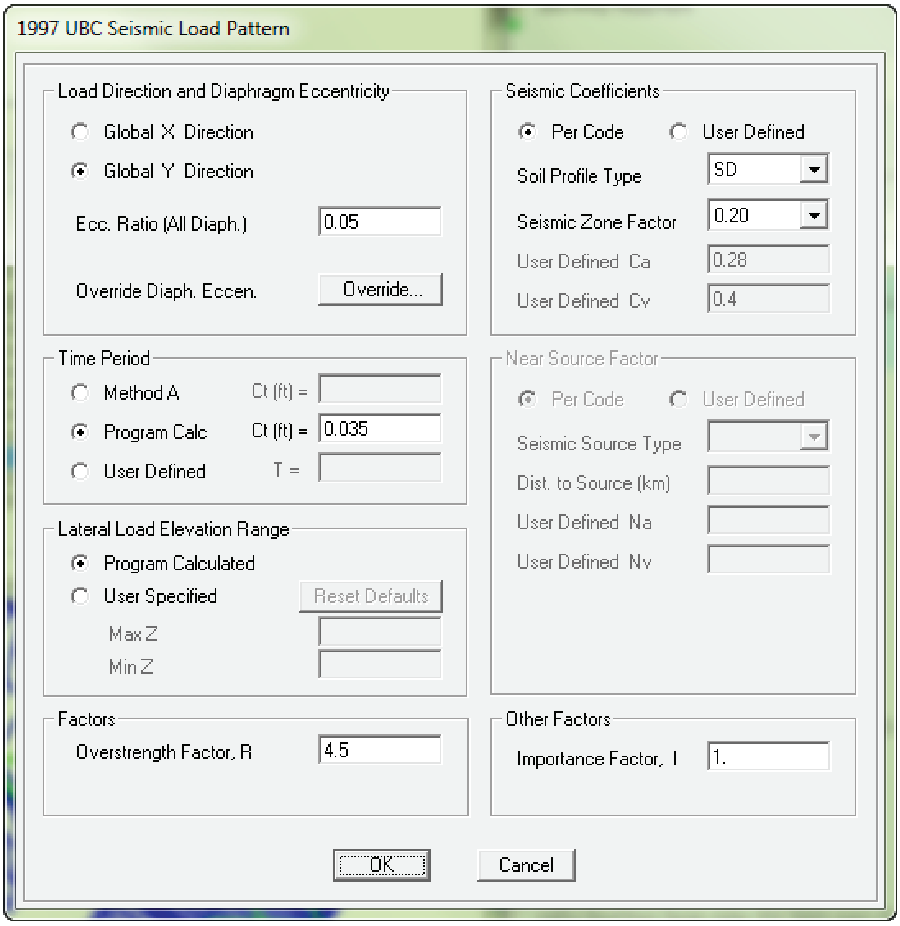


Figure 4 76 EL2 definition

Define mass source: click the **Define > Mass** Source command to show the Mass Source form.

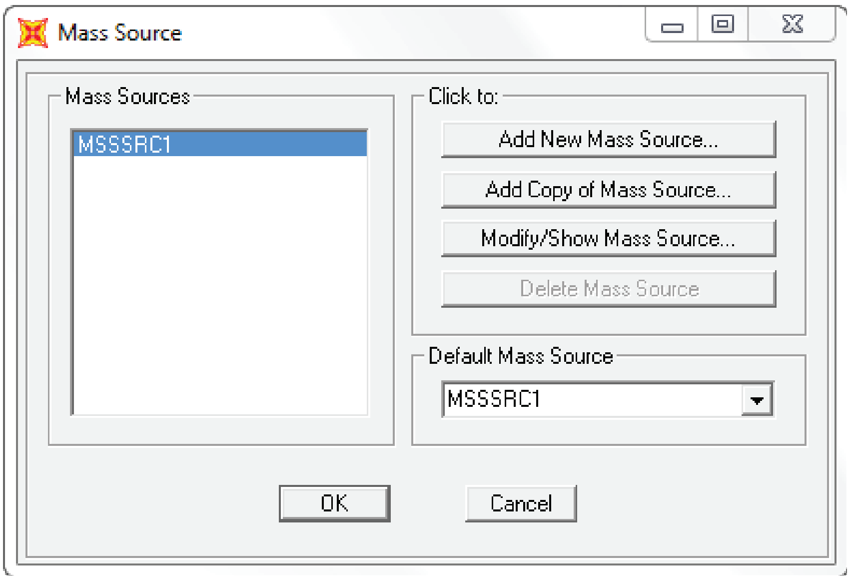


Figure 4 77 Mass Source form

Click the **Modify/Show Mass source** button to access the **Mass Source Data** form. Enter all information like figure below:

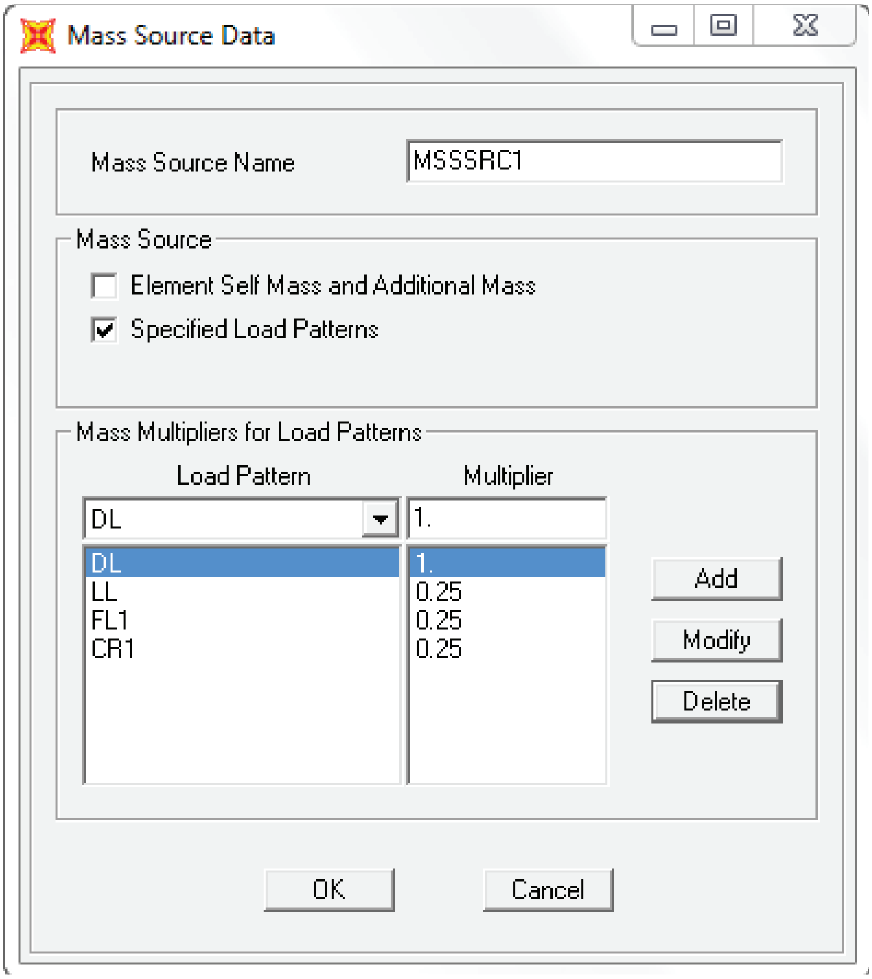


Figure 4 78 Mass Source Data form

Define load combinations follow ASCE 7-10 standard using .s2k text file.

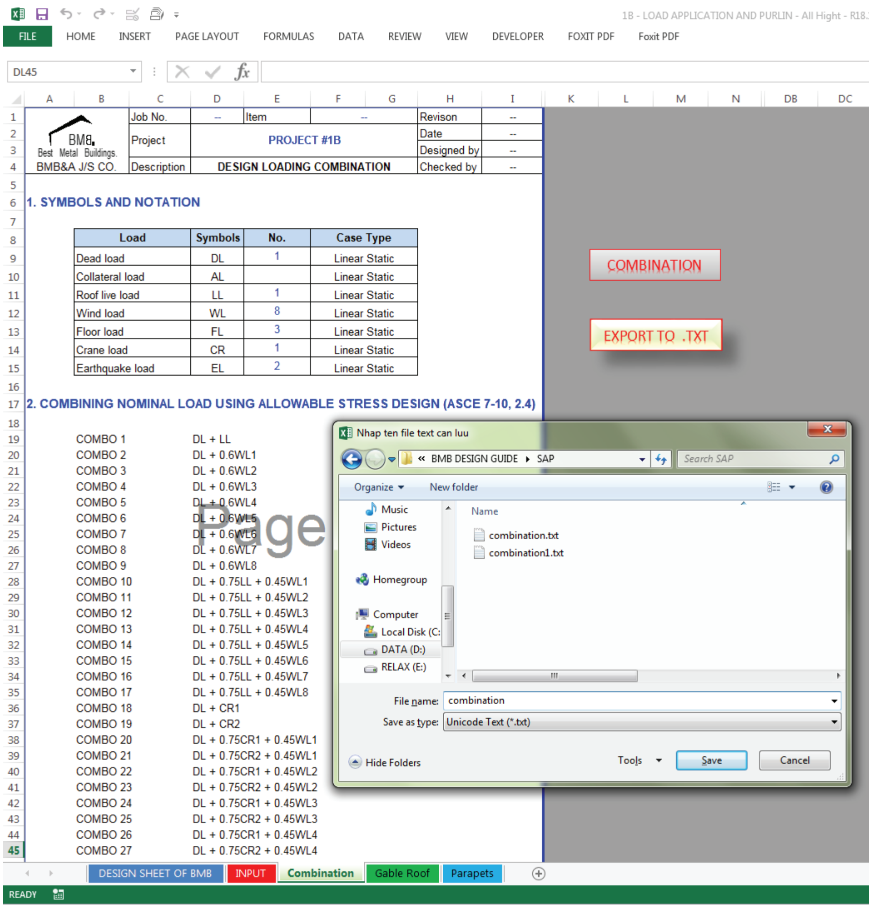


Figure 4 79 Design loading combination sheet

- In SAP2000, you have to define at least one load combination before export .s2k text file. Click the **Define > Load Combinations** command, in the **Define Load Combinations** form which has just appeared, click the **Add New Combo** button and create an combination as shown in figure below.

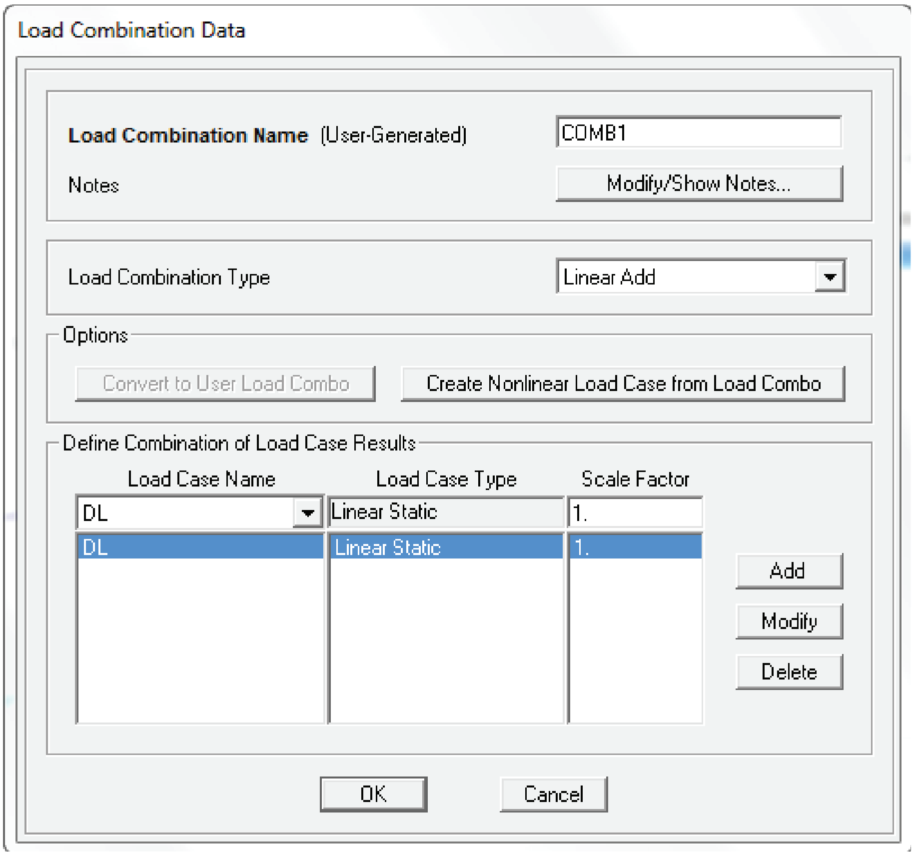


Figure 4 80 Create load combination COMB1

- After create any combination, click the **File > Export > Sap2000 .s2k Text File** command to display the **Choose Tables for Export to Text File** form shown in figure below.

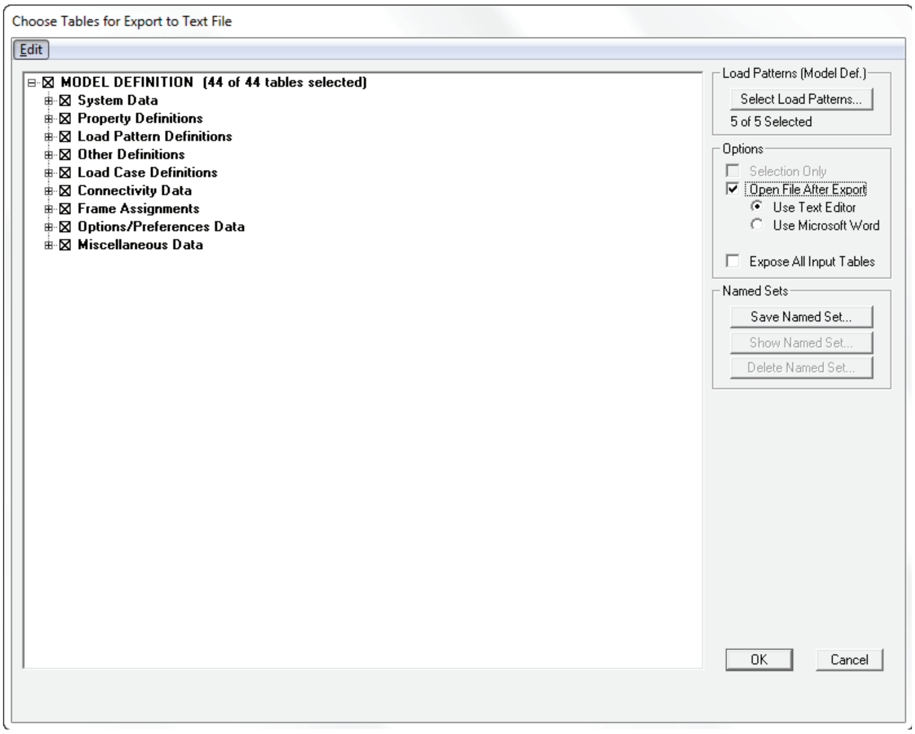


Figure 4 81 Choose Tables for Export to Text File form

- Make sure that all tables, load patterns, and Open File After Export option are selected before click the **OK** button.
- Copy all data in combination.txt text file and paste to .s2k text file at highlight text shown in figure below.

```
TABLE:  "LOAD PATTERN DEFINITIONS"
   LoadPat=DL    DesignType=DEAD    SelfWtMult=1
   LoadPat=LL    DesignType=LIVE    SelfWtMult=0
   LoadPat=WL1    DesignType=WIND    SelfWtMult=0    AutoLoad=None
   LoadPat=WL2    DesignType=WIND    SelfWtMult=0    AutoLoad=None
   LoadPat=WL3    DesignType=WIND    SelfWtMult=0    AutoLoad=None

TABLE:  "AUTO WAVE 3 - WAVE CHARACTERISTICS - GENERAL"
   WaveChar=Default    WaveType="From Theory"    KinFactor=1
SWaterDepth=45    WaveHeight=18    WavePeriod=12
WaveTheory=Linear

TABLE:  "COMBINATION DEFINITIONS"
   ComboName=COMB1    ComboType="Linear Add"    AutoDesign=No
CaseType="Linear Static"    CaseName=DL    ScaleFactor=1
SteelDesign=None    ConcDesign=None    AlumDesign=None
ColdDesign=None

TABLE:  "FUNCTION - RESPONSE SPECTRUM - USER"
   Name=UNIFRS    Period=0    Accel=1    FuncDamp=0.05
   Name=UNIFRS    Period=1    Accel=1

TABLE:  "FUNCTION - TIME HISTORY - USER"
   Name=RAMPTH    Time=0    Value=0
   Name=RAMPTH    Time=1    Value=1
   Name=RAMPTH    Time=4    Value=1
   Name=UNIFTH    Time=0    Value=1
   Name=UNIFTH    Time=1    Value=1

TABLE:  "FUNCTION - POWER SPECTRAL DENSITY - USER"
   Name=UNIFPSD    Frequency=0    Value=1
   Name=UNIFPSD    Frequency=1    Value=1

TABLE:  "FUNCTION - STEADY STATE - USER"
   Name=UNIFSS    Frequency=0    Value=1
   Name=UNIFSS    Frequency=1    Value=1

TABLE:  "GROUPS 1 - DEFINITIONS"
   GroupName=ALL    Selection=Yes    SectionCut=Yes    Steel=Yes
Concrete=Yes    Aluminum=Yes    ColdFormed=Yes    Stage=Yes
Bridge=Yes    AutoSeismic=No    AutoWind=No    SelDesSteel=No
SelDesAlum=No    SelDesCold=No    MassWeight=Yes    Color=Red

TABLE:  "JOINT PATTERN DEFINITIONS"
   Pattern=Default

TABLE:  "MASS SOURCE"
   MassSource=MSSSRC1    Elements=Yes    Masses=Yes    Loads=No
IsDefault=Yes
```

Figure 4 82 Substitute combination defined by SAP with combination exported from excel.

- Save and close file. Return to SAP and import this file. Click the **File > Import > SAP2000 .s2k Text File** command to display **Import Tabular Database** shown in figure below. Click the OK button, specify created .s2k text file in appeared window and click the Done button to import file.

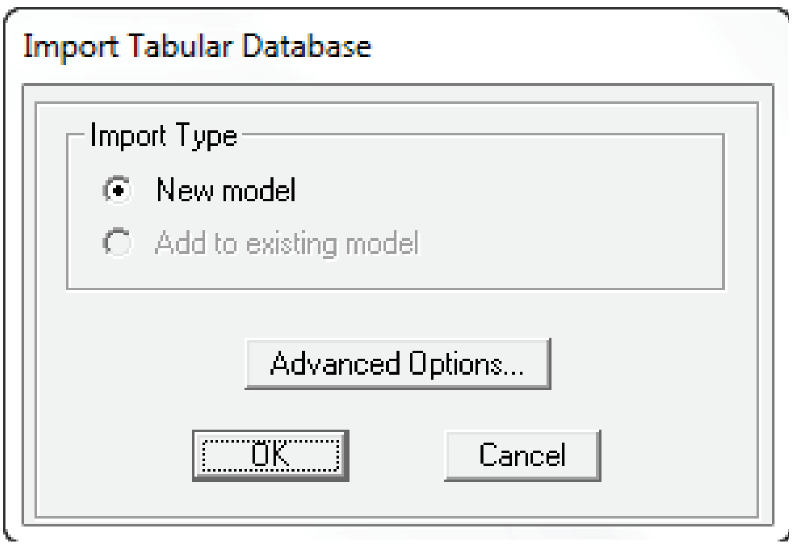


Figure 4 83 Import Tabular Database form

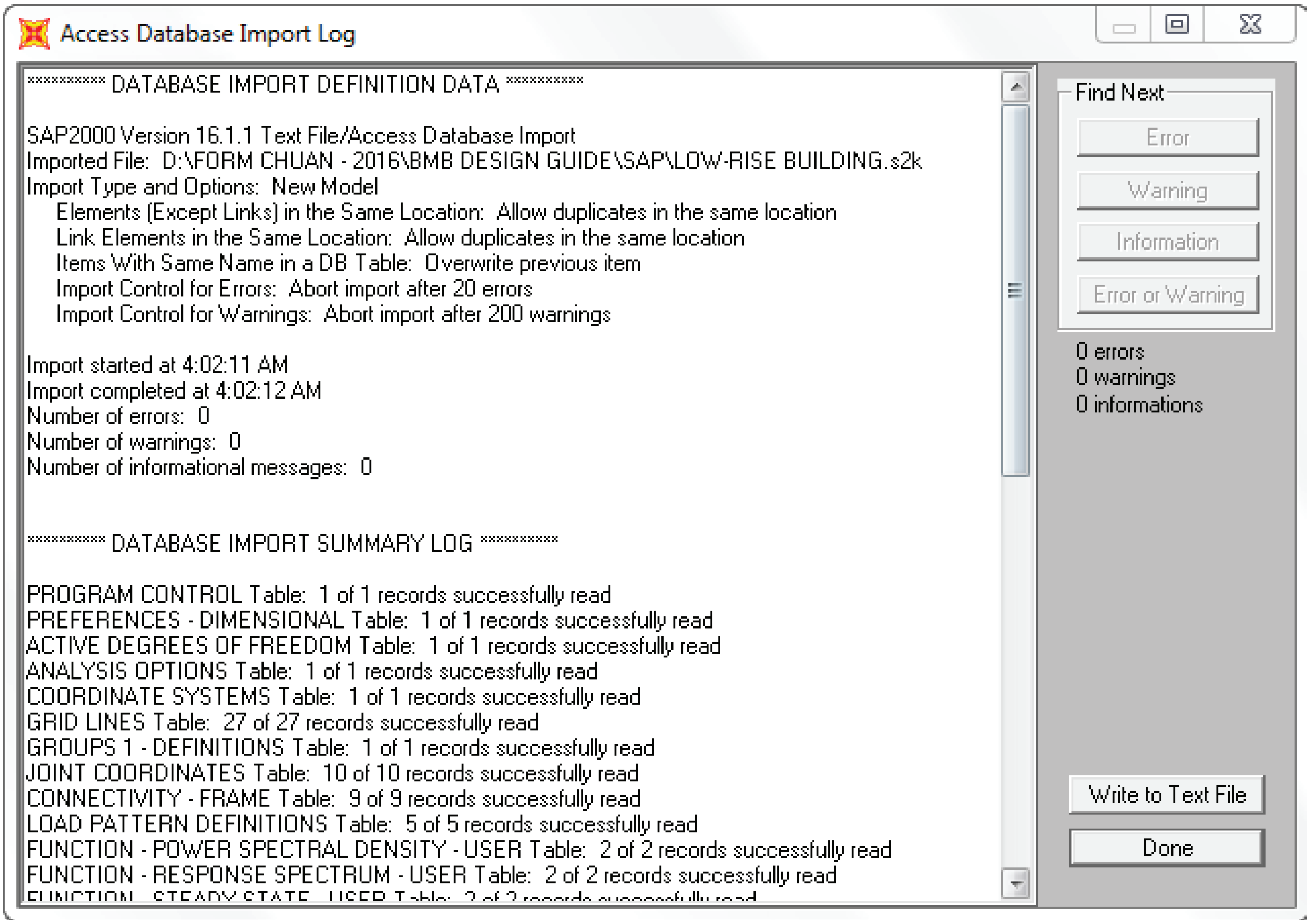


Figure 4 84 Click the Done button to finish importing .s2k text file

Assign loads

In this step, the dead, live and wind loads will be applied to the model.

Calculate wind loads which will be assigned to frame using "**1A - LOAD APPLICATION AND PURLIN – All Height**" file.

- In sheet "**Input**", enter all the information as below:

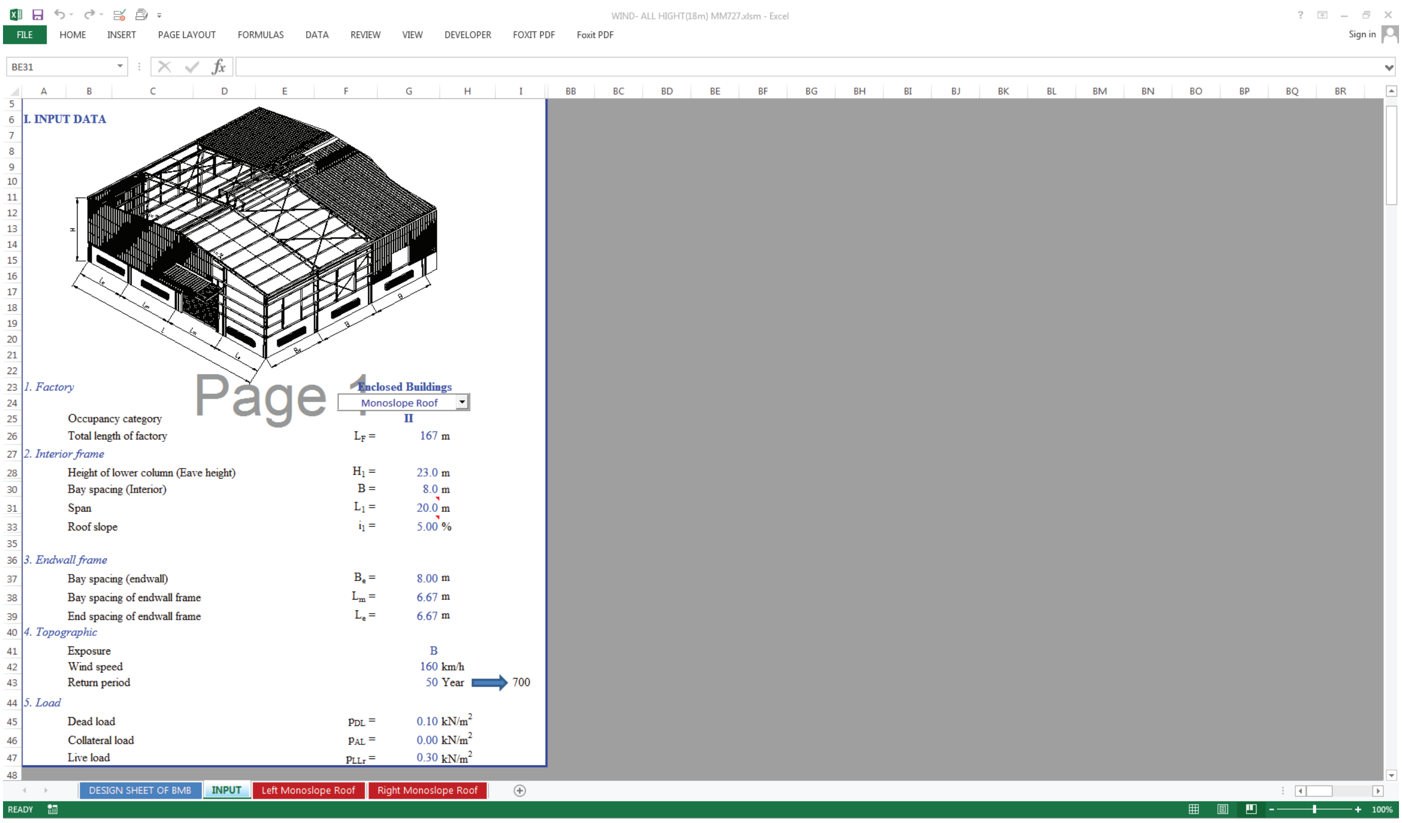


Figure 4 85 Input sheet

- Go to sheet "**Left Monoslope Roof**" and then click the Calculation button to compute wind load blowing from left to right.

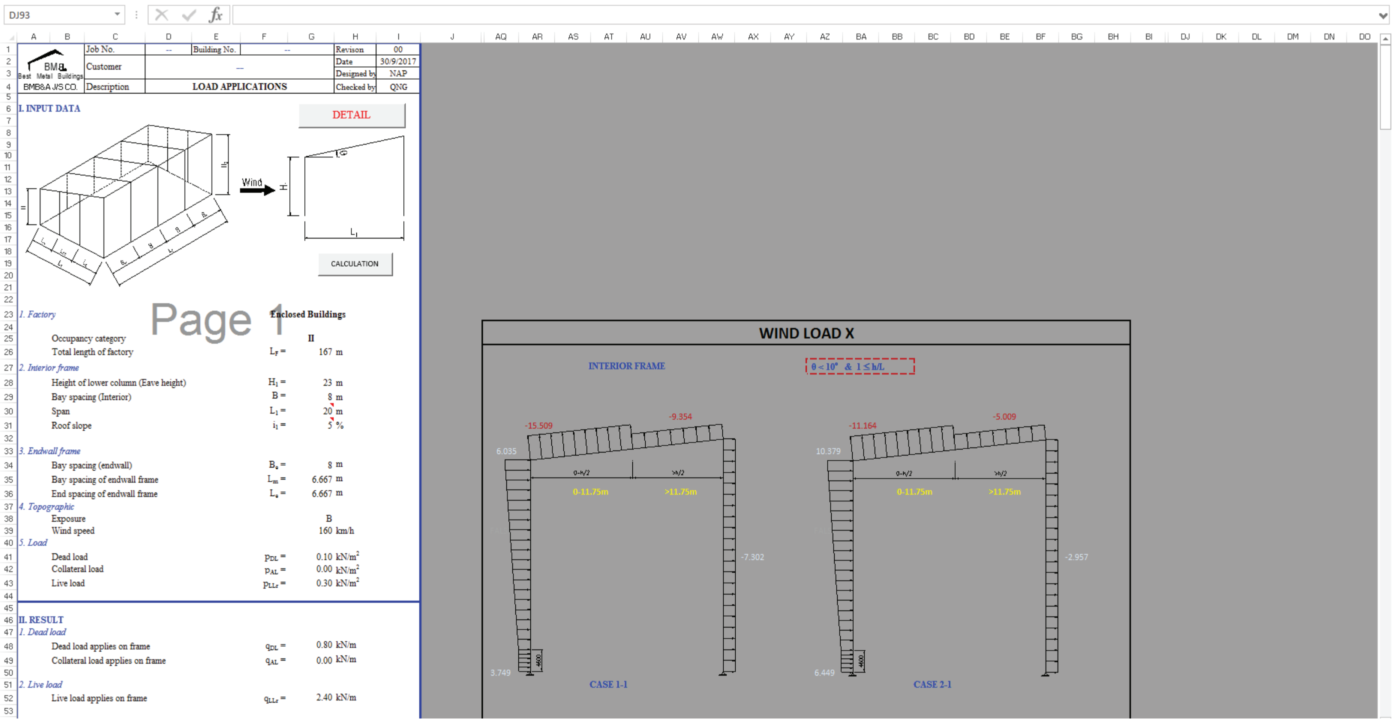


Figure 4 86 Left Monoslope Roof sheet

- Go to sheet "**Right Monoslope Roof**" and then click the Calculation button to compute wind load blowing from right to left.

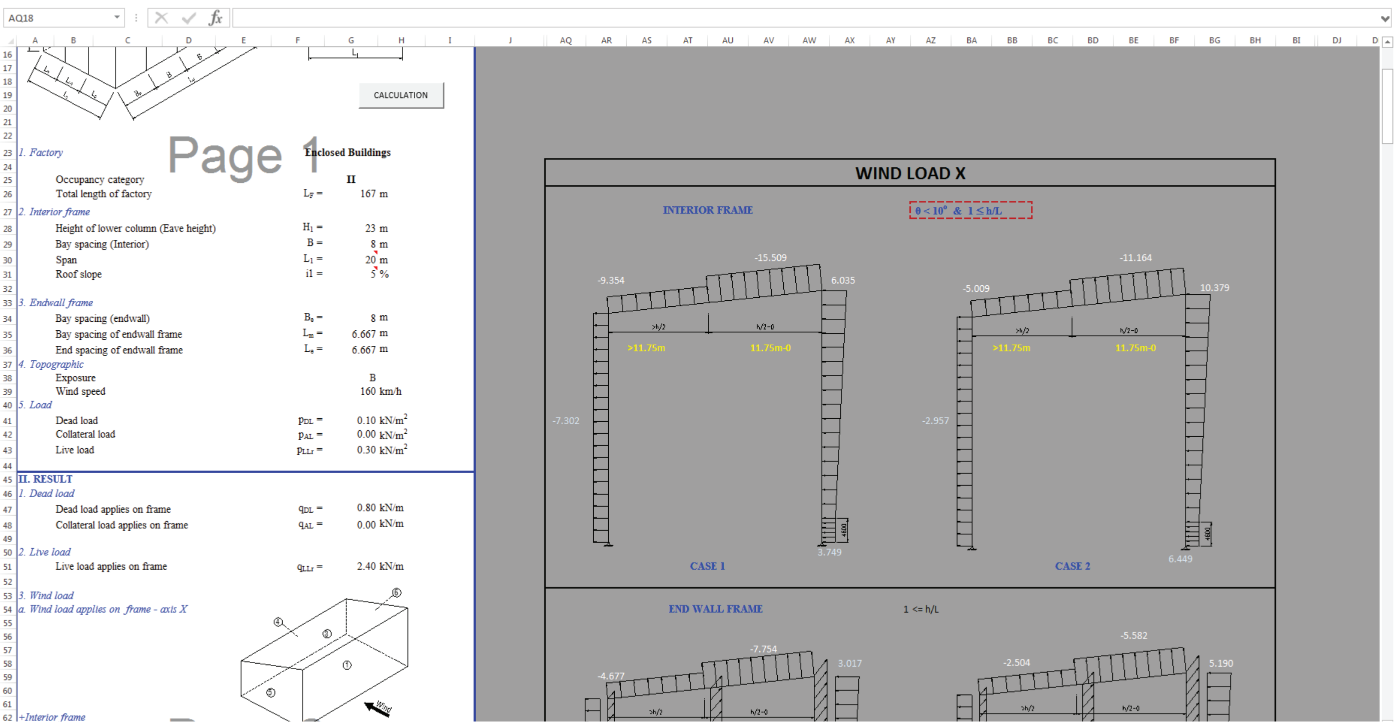
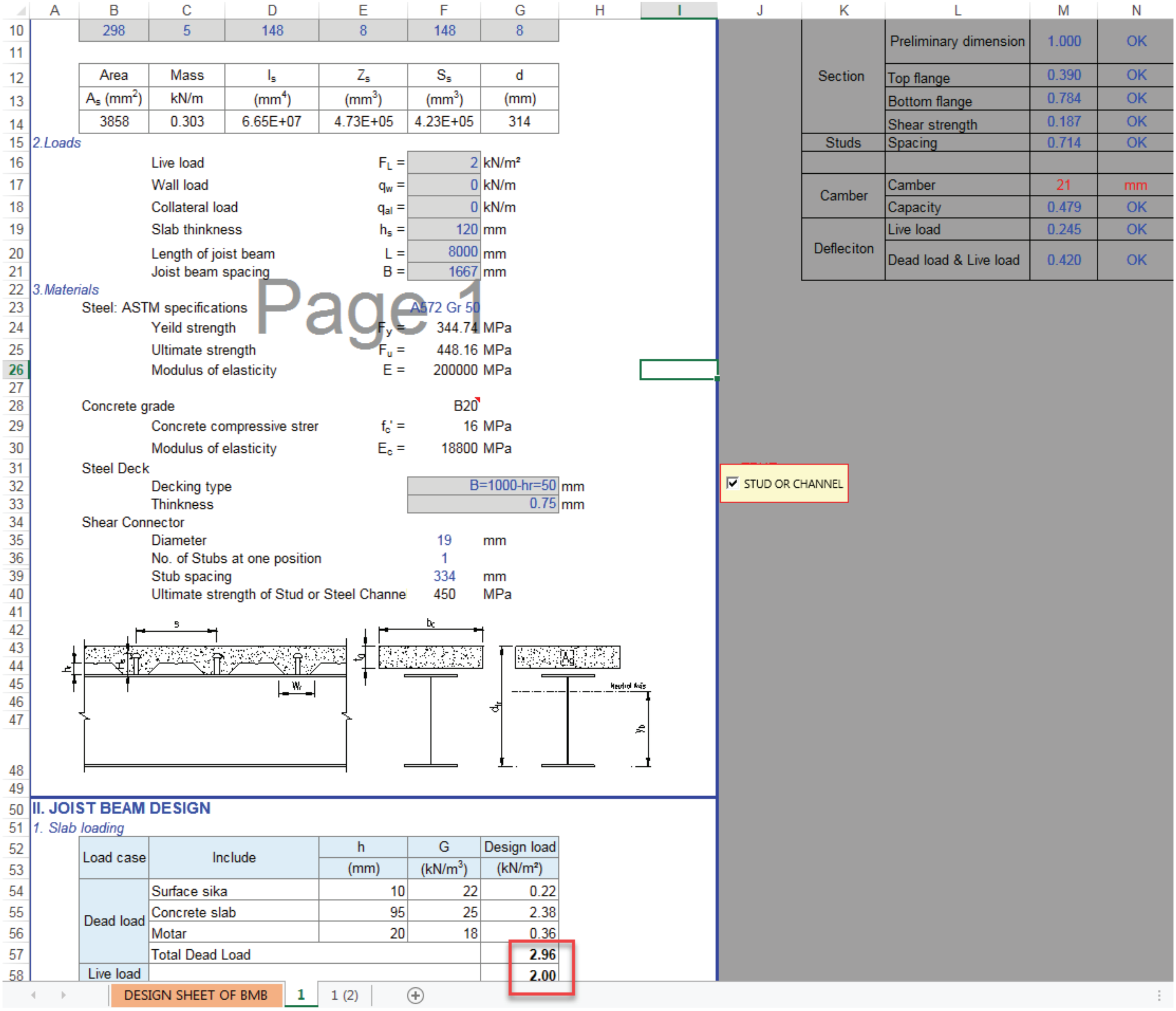


Figure 4 87 Right Monoslope Roof sheet

Calculate dead load and floor load using "**2 - COMPOSITE BEAM & DECKING**" file.

In sheet "**1**", enter all the information as figure below:



B	C	D	E	F	G
298	5	148	8	148	8
Area	Mass	I_s	Z_s	S_s	d
A_s (mm²)	kN/m	(mm⁴)	(mm³)	(mm³)	(mm)
3858	0.303	6.65E+07	4.73E+05	4.23E+05	314

2.Loads

Live load	F_L =	2	kN/m²
Wall load	q_w =	0	kN/m
Collateral load	q_{al} =	0	kN/m
Slab thinkness	h_s =	120	mm
Length of joist beam	L =	8000	mm
Joist beam spacing	B =	1667	mm

3.Materials

Steel: ASTM specifications		A572 Gr 50	
Yeild strength	F_y =	344.74	MPa
Ultimate strength	F_u =	448.16	MPa
Modulus of elasticity	E =	200000	MPa
Concrete grade		B20	
Concrete compressive strer	f_c' =	16	MPa
Modulus of elasticity	E_c =	18800	MPa
Steel Deck			
Decking type		B=1000-hr=50	mm
Thinkness		0.75	mm
Shear Connector			
Diameter		19	mm
No. of Stubs at one position		1	
Stub spacing		334	mm
Ultimate strength of Stud or Steel Channe		450	MPa

K	L	M	N
Section	Preliminary dimension	1.000	OK
	Top flange	0.390	OK
	Bottom flange	0.784	OK
	Shear strength	0.187	OK
Studs	Spacing	0.714	OK
Camber	Camber	21	mm
	Capacity	0.479	OK
Defleciton	Live load	0.245	OK
	Dead load & Live load	0.420	OK

STUD OR CHANNEL

II. JOIST BEAM DESIGN

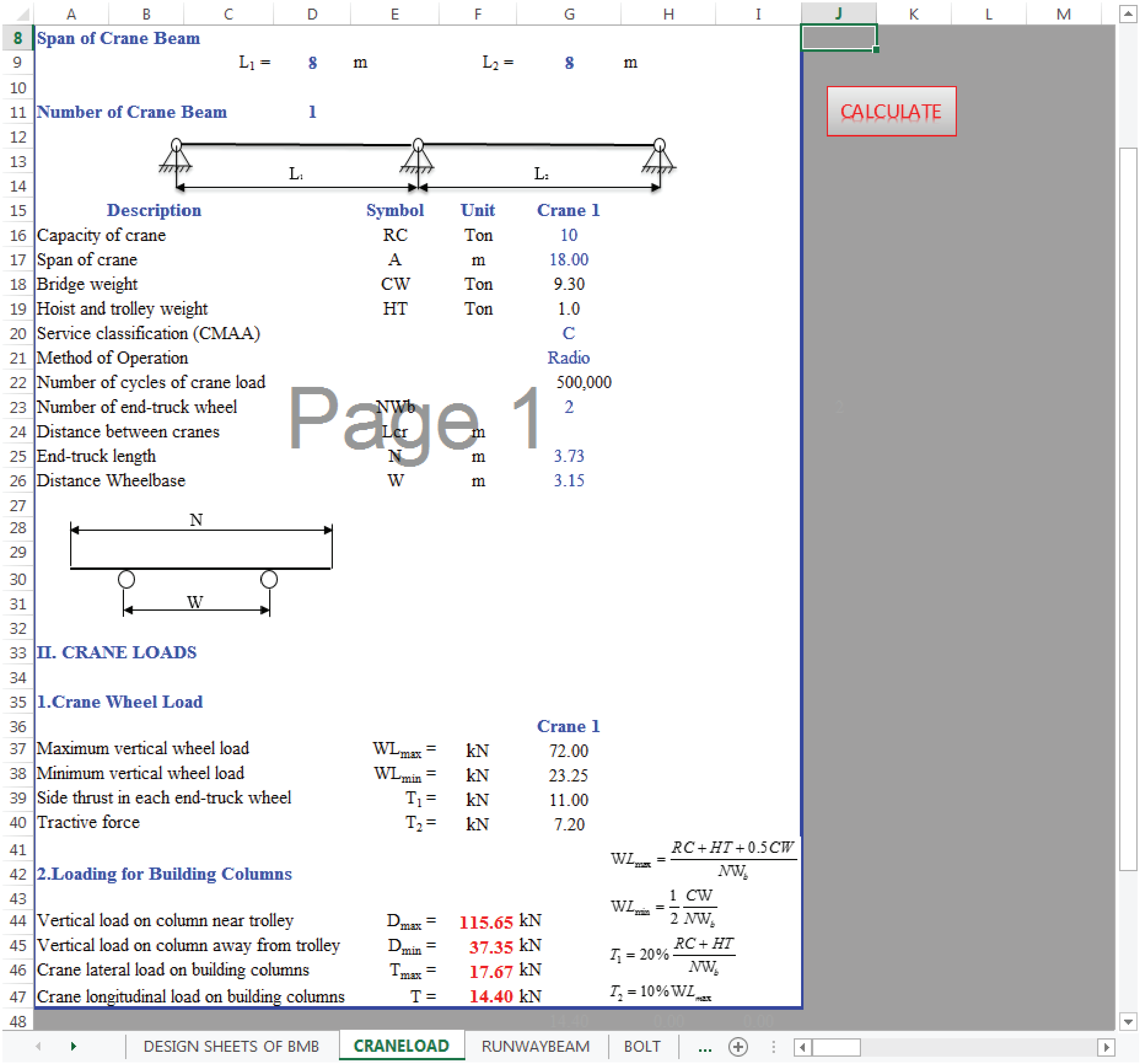
1. Slab loading

Load case	Include	h (mm)	G (kN/m³)	Design load (kN/m²)
Dead load	Surface sika	10	22	0.22
	Concrete slab	95	25	2.38
	Motar	20	18	0.36
	Total Dead Load			**2.96**
Live load				**2.00**

DESIGN SHEET OF BMB | 1 | 1 (2)

Figure 4 88 Calculate load for mezzanine floor

Use 2.96 kN/m2 for dead load and 2 kN/m2 for live load to apply on mezzanine floor.
Calculate crane load using "**3 - CRANE BEAM DESIGN**" file. Enter all the information as figure below:



Span of Crane Beam

$L_1 =$ 8 m $L_2 =$ 8 m

Number of Crane Beam 1

Description	Symbol	Unit	Crane 1
Capacity of crane	RC	Ton	10
Span of crane	A	m	18.00
Bridge weight	CW	Ton	9.30
Hoist and trolley weight	HT	Ton	1.0
Service classification (CMAA)			C
Method of Operation			Radio
Number of cycles of crane load			500,000
Number of end-truck wheel	NWb		2
Distance between cranes	Lcr	m	
End-truck length	N	m	3.73
Distance Wheelbase	W	m	3.15

II. CRANE LOADS

1.Crane Wheel Load

		Unit	Crane 1
Maximum vertical wheel load	$WL_{max} =$	kN	72.00
Minimum vertical wheel load	$WL_{min} =$	kN	23.25
Side thrust in each end-truck wheel	$T_1 =$	kN	11.00
Tractive force	$T_2 =$	kN	7.20

$$WL_{max} = \frac{RC + HT + 0.5CW}{NW_b}$$

$$WL_{min} = \frac{1}{2}\frac{CW}{NW_b}$$

$$T_1 = 20\% \frac{RC + HT}{NW_b}$$

$$T_2 = 10\% WL_{max}$$

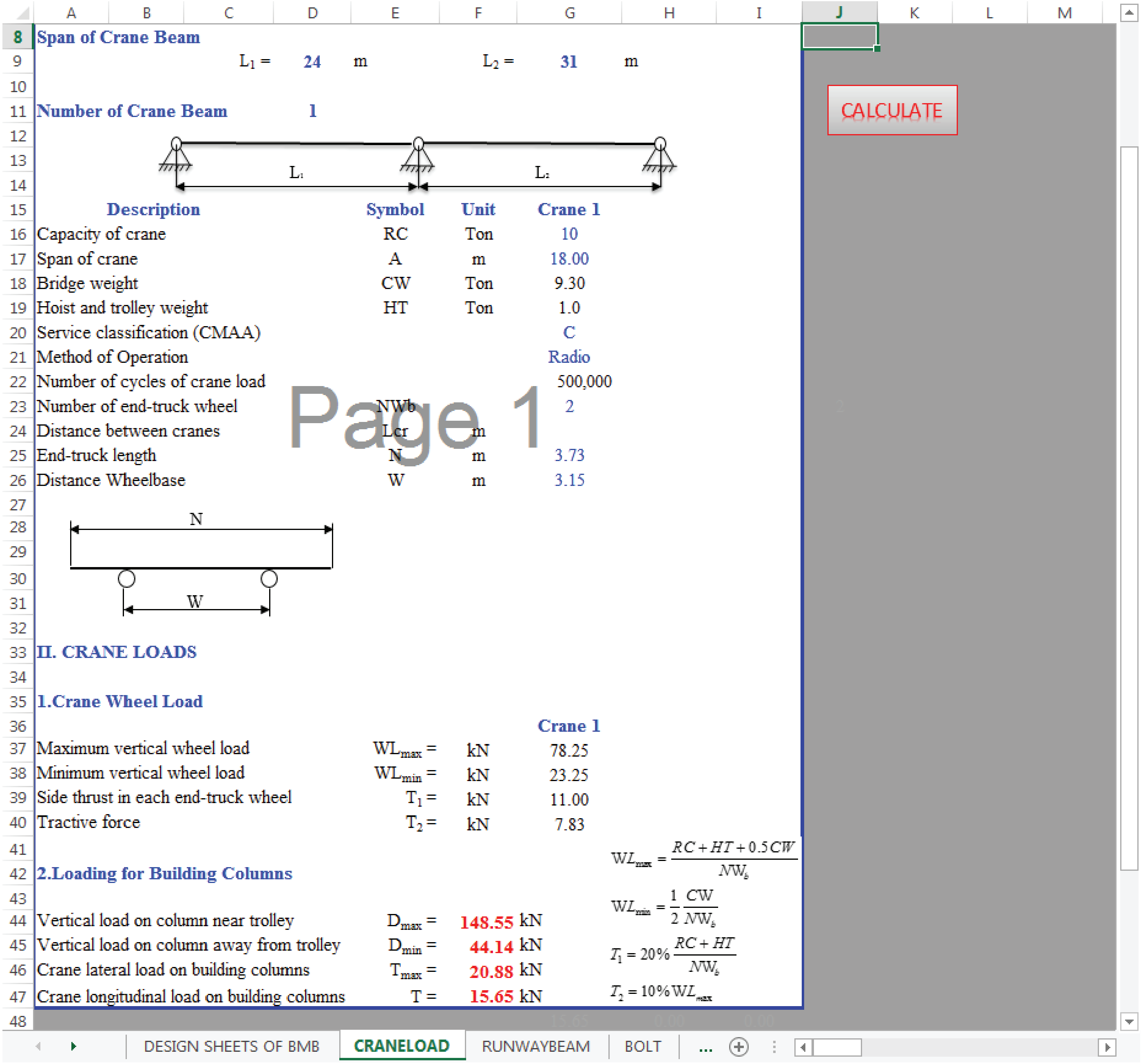
2.Loading for Building Columns

Vertical load on column near trolley	$D_{max} =$	115.65	kN
Vertical load on column away from trolley	$D_{min} =$	37.35	kN
Crane lateral load on building columns	$T_{max} =$	17.67	kN
Crane longitudinal load on building columns	T =	14.40	kN

CALCULATE

DESIGN SHEETS OF BMB | CRANELOAD | RUNWAYBEAM | BOLT | ...

Figure 4 89 Crane load for bay spacing 8m



CALCULATE

Span of Crane Beam

$L_1 =$ 24 m $L_2 =$ 31 m

Number of Crane Beam 1

L₁ L₂

Description	Symbol	Unit	Crane 1
Capacity of crane	RC	Ton	10
Span of crane	A	m	18.00
Bridge weight	CW	Ton	9.30
Hoist and trolley weight	HT	Ton	1.0
Service classification (CMAA)			C
Method of Operation			Radio
Number of cycles of crane load			500,000
Number of end-truck wheel	NWb		2
Distance between cranes	Lcr	m	
End-truck length	N	m	3.73
Distance Wheelbase	W	m	3.15

Page 1

N

W

II. CRANE LOADS

1.Crane Wheel Load

	Symbol	Unit	Crane 1
Maximum vertical wheel load	$WL_{max} =$	kN	78.25
Minimum vertical wheel load	$WL_{min} =$	kN	23.25
Side thrust in each end-truck wheel	$T_1 =$	kN	11.00
Tractive force	$T_2 =$	kN	7.83

$$WL_{max} = \frac{RC + HT + 0.5CW}{NW_b}$$

$$WL_{min} = \frac{1}{2}\frac{CW}{NW_b}$$

$$T_1 = 20\%\frac{RC + HT}{NW_b}$$

$$T_2 = 10\%\,WL_{max}$$

2.Loading for Building Columns

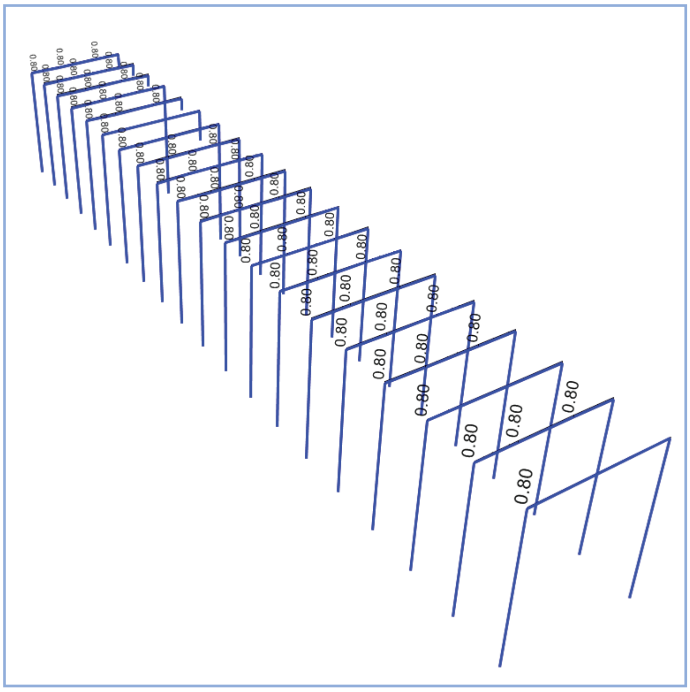
Vertical load on column near trolley	$D_{max} =$	148.55	kN
Vertical load on column away from trolley	$D_{min} =$	44.14	kN
Crane lateral load on building columns	$T_{max} =$	20.88	kN
Crane longitudinal load on building columns	$T =$	15.65	kN

DESIGN SHEETS OF BMB | CRANELOAD | RUNWAYBEAM | BOLT | ...

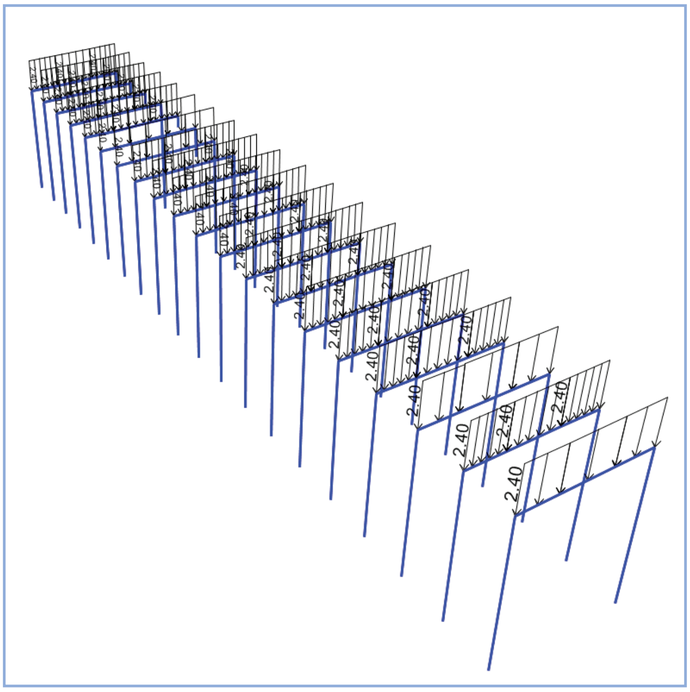
Figure 4 90 Crane load for bay spacing 24m and 31m

Assign Load

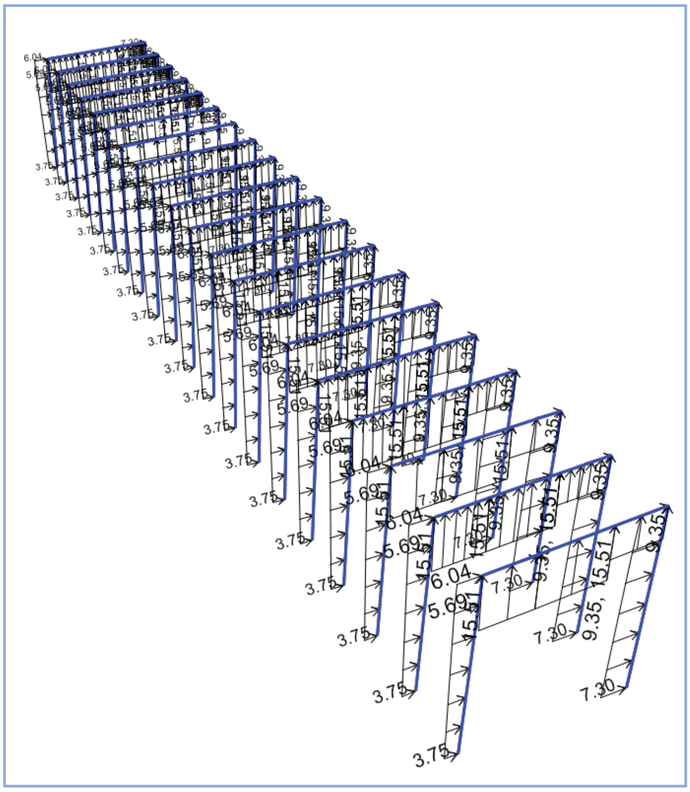
After calculate load applied on frame, we assign them to frame as shown in these figure below.



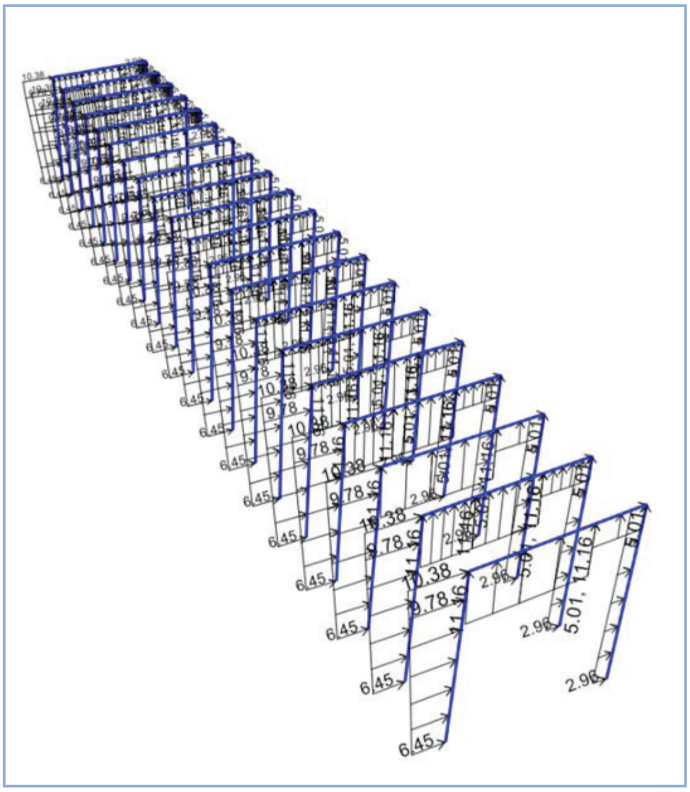
Dead load (DL)



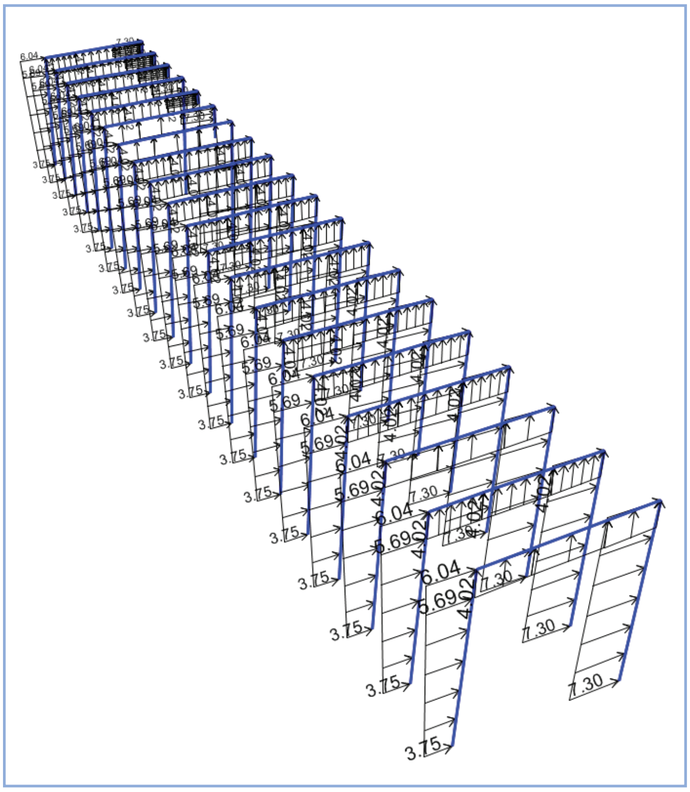
Live load (LL)



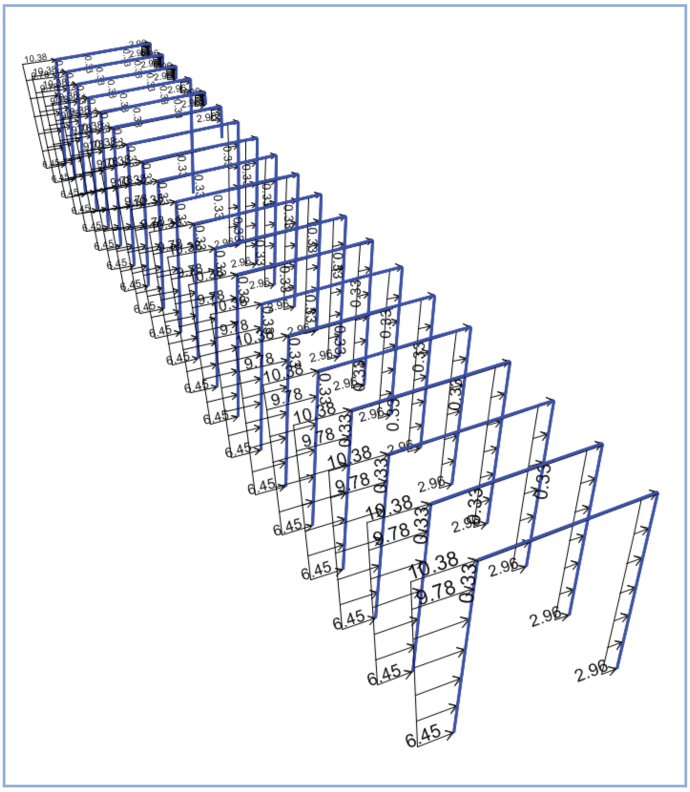
Wind load 1 (WL1)



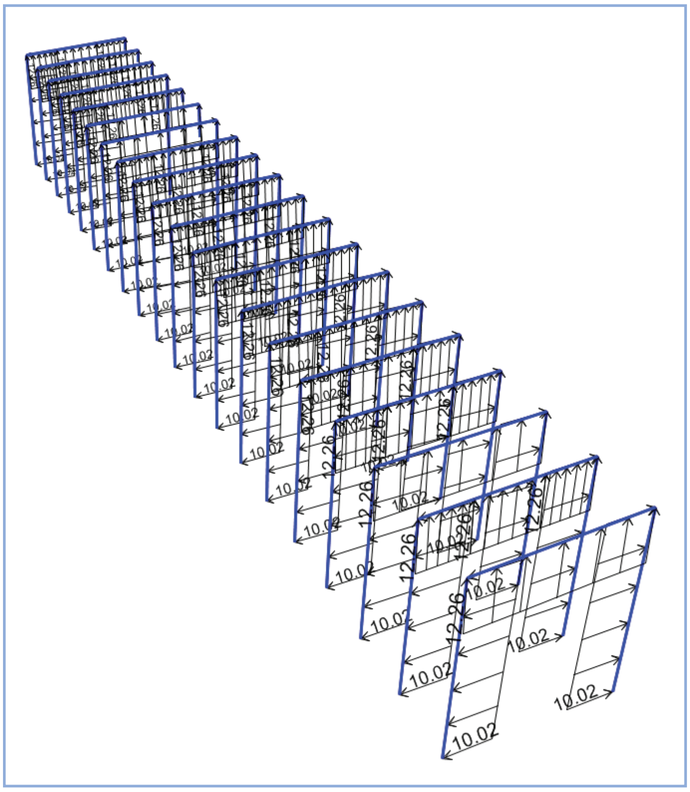
Wind load 2 (WL2)



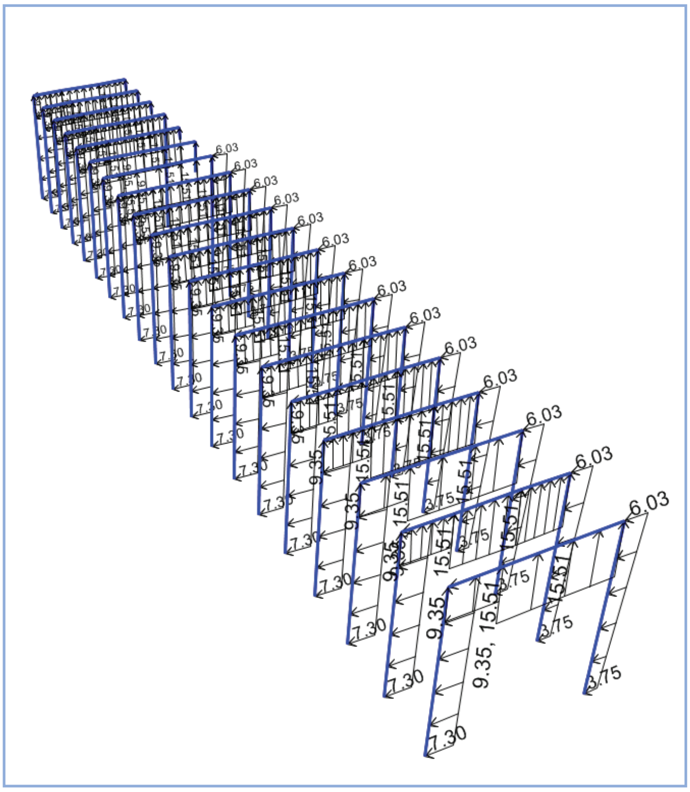
Wind load 3 (WL3)



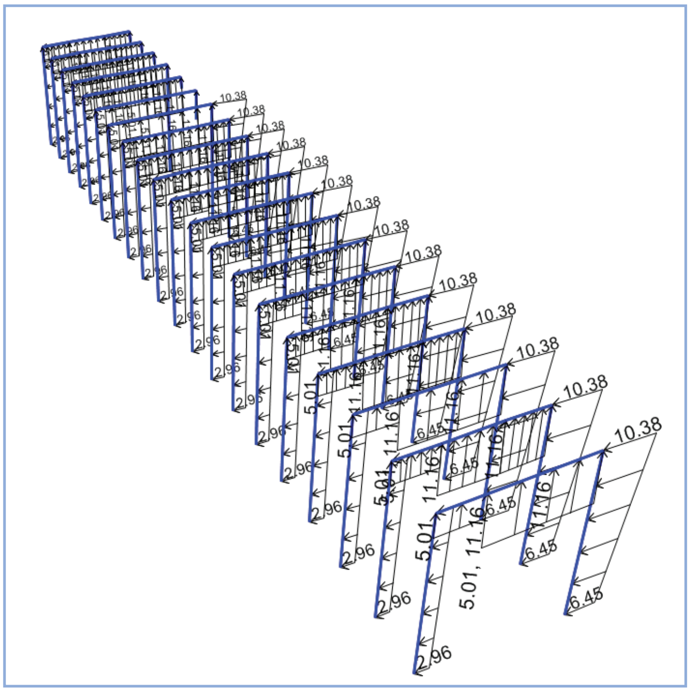
Wind load 4 (WL4)



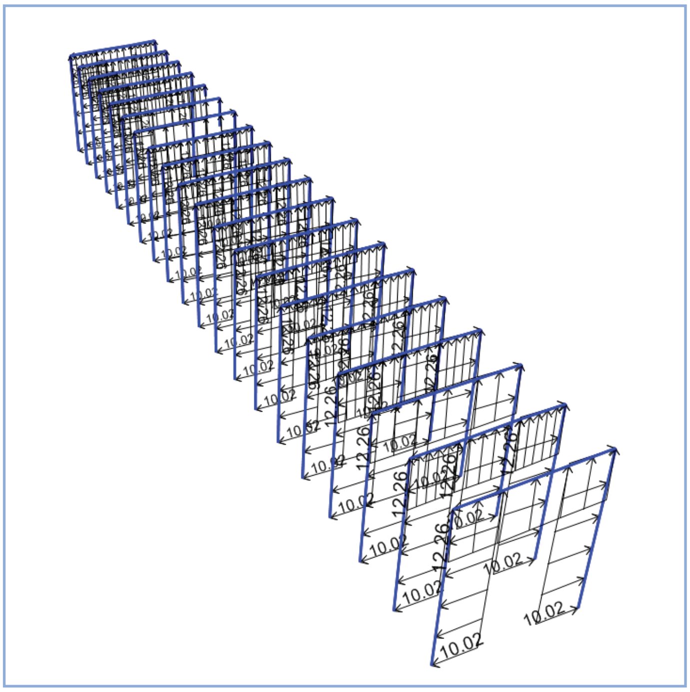
Wind load 5 (WL5)



Wind load 6 (WL6)



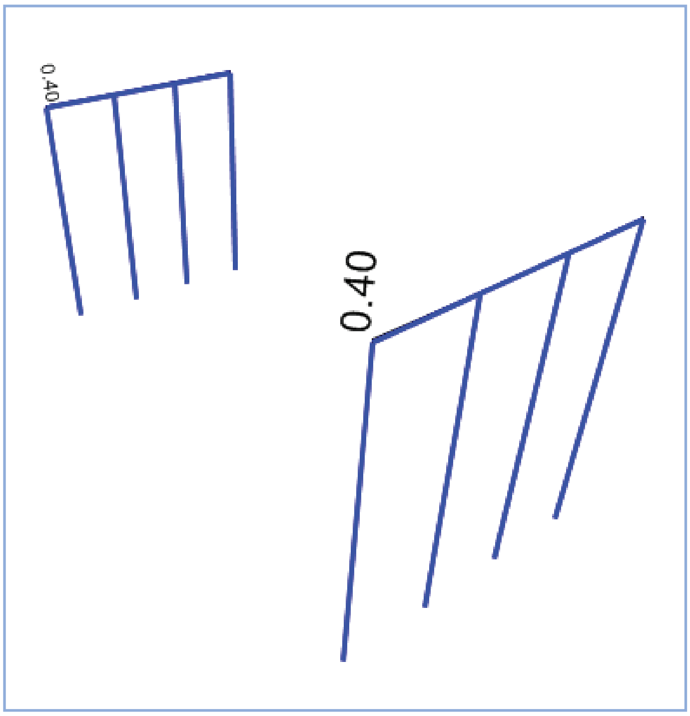
Wind load 7 (WL7)



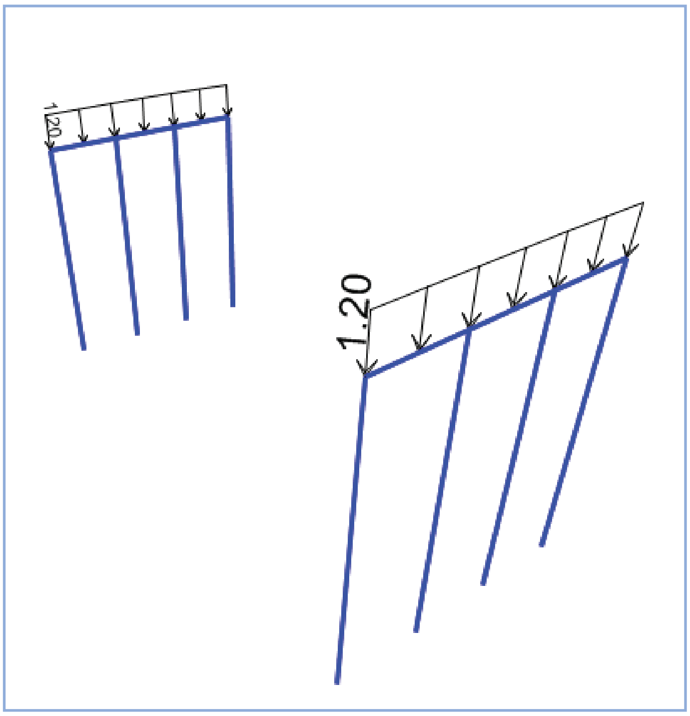
Wind load 8 (WL8)

Figure 4 91 Load applied Frame with 8m bay spacing

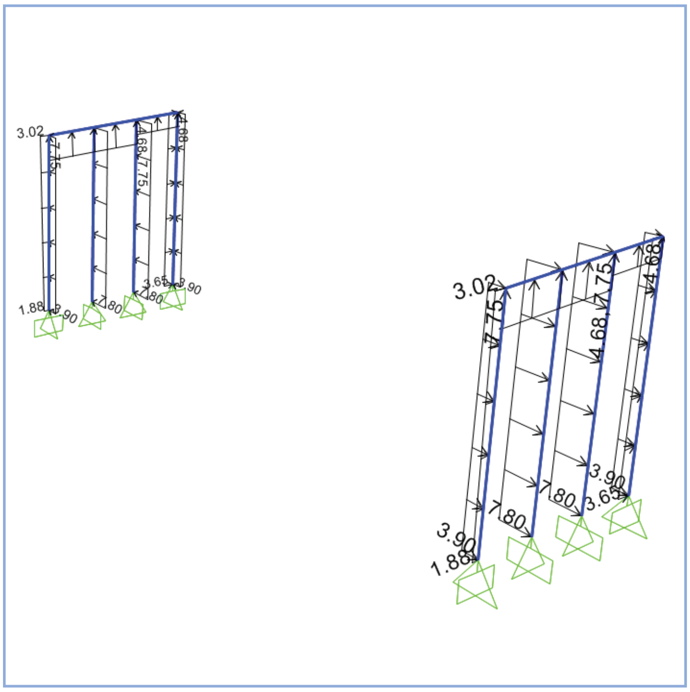
- End wall frame



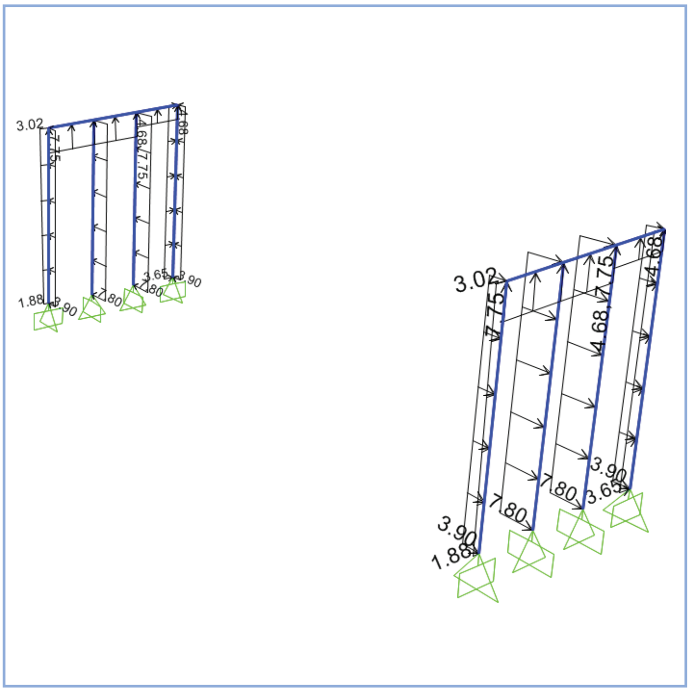
Dead load (DL)



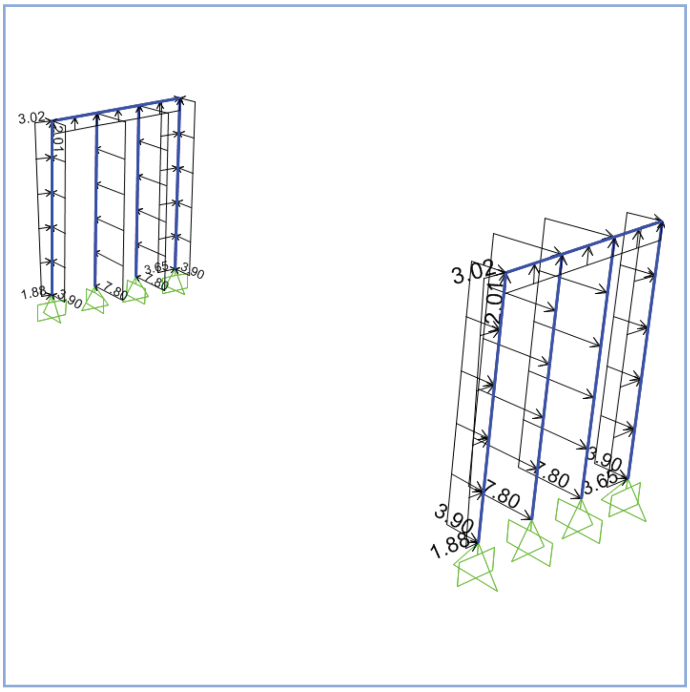
Live load (LL)



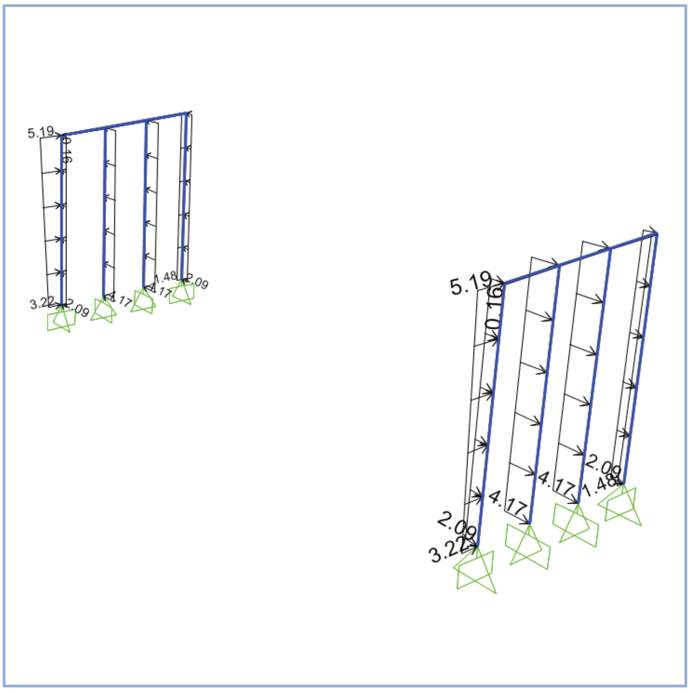
Wind load 1 (WL1)



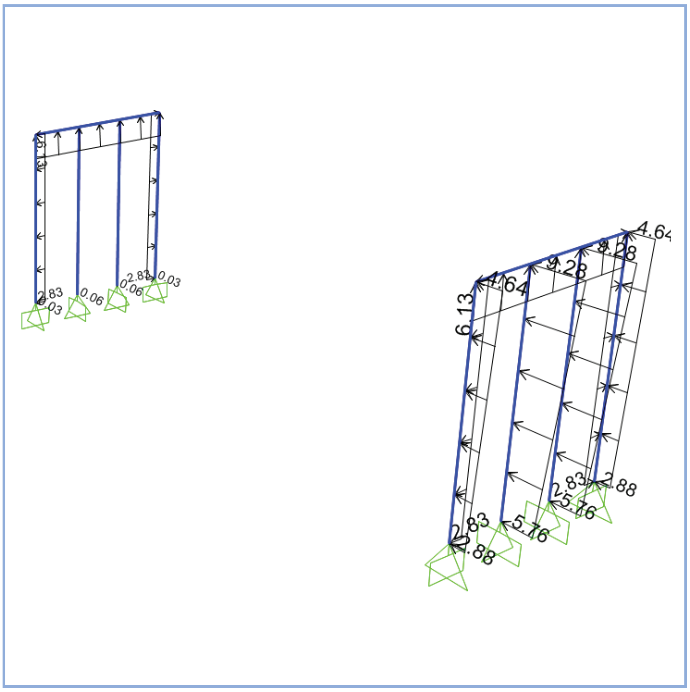
Wind load 2 (WL2)



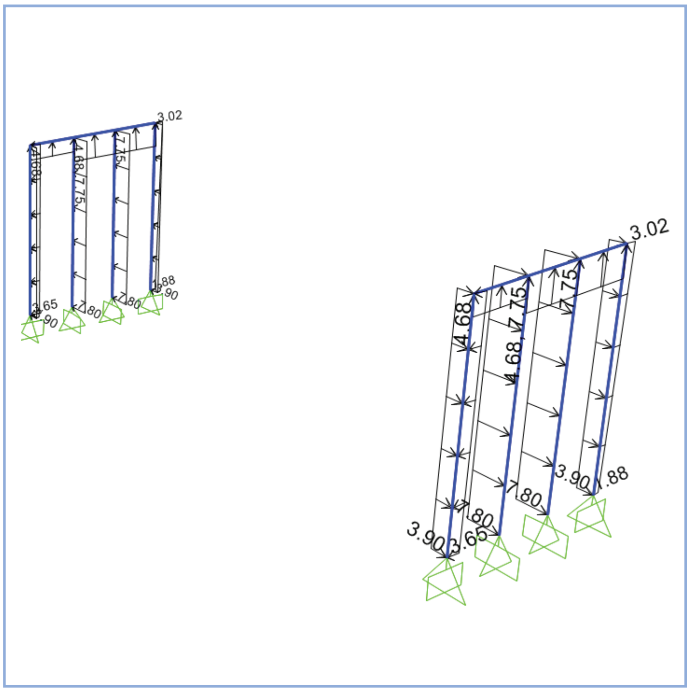
Wind load 3 (WL3)



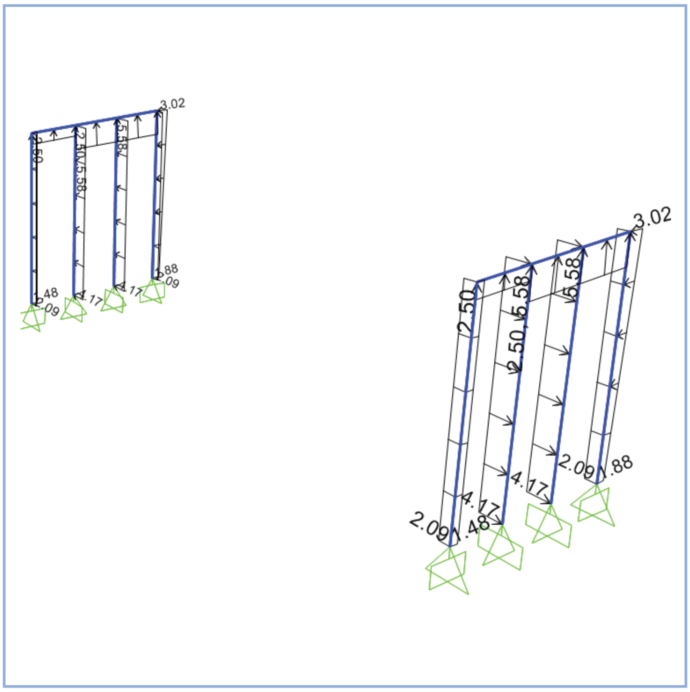
Wind load 4 (WL4)



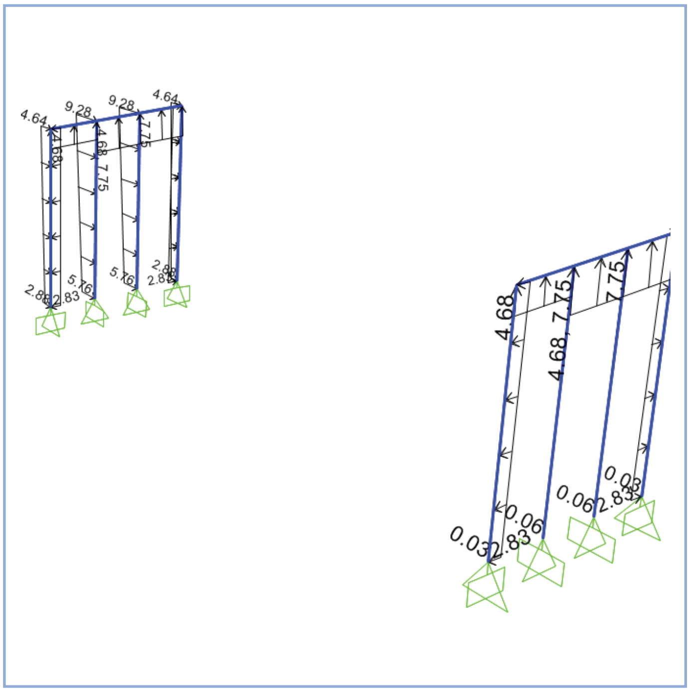
Wind load 5 (WL5)



Wind load 6 (WL6)

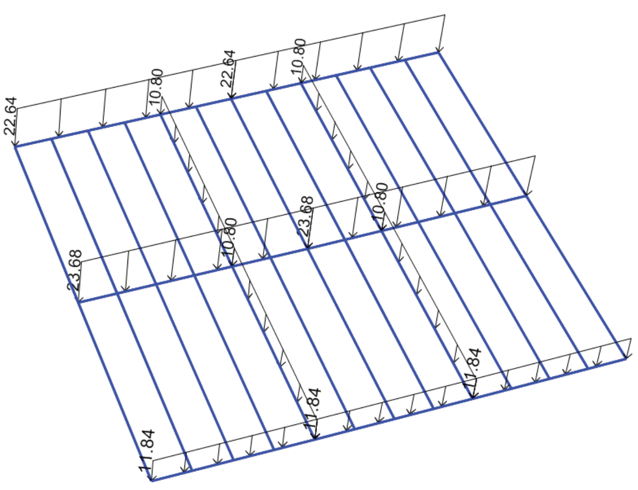


Wind load 7 (WL7

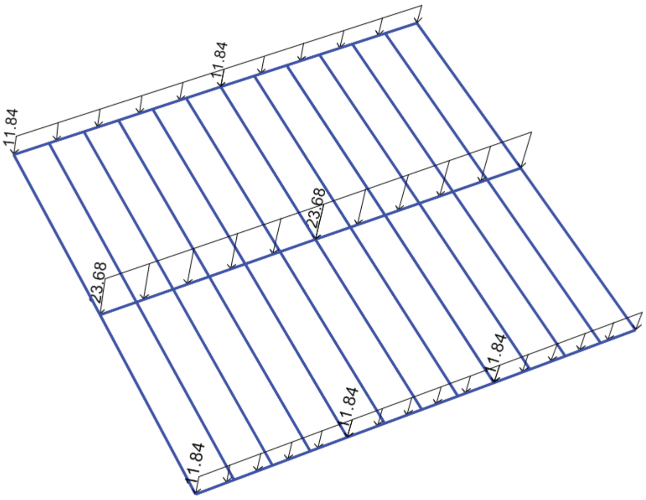


Wind load 8 (WL8)

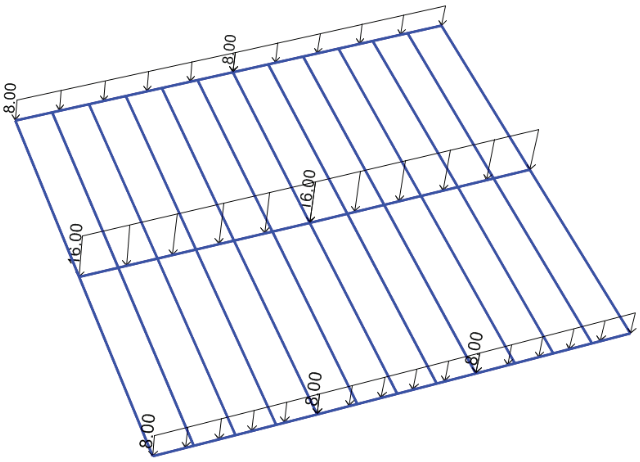
Figure 4 92 Load applied End-wall Frame



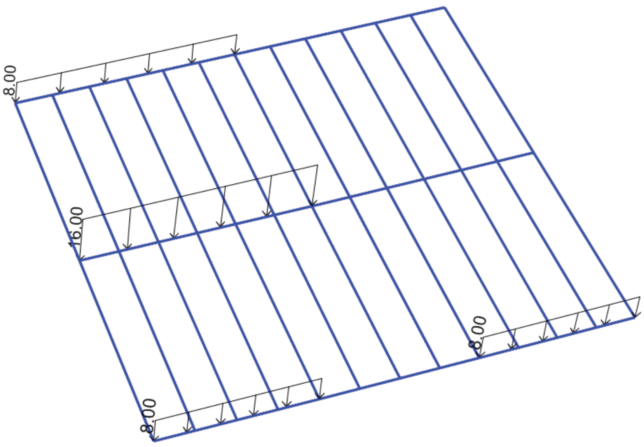
Deal load (DL) – level +6.5m



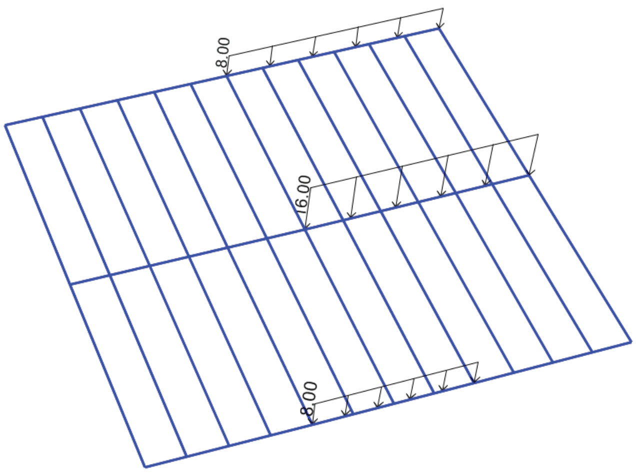
Deal load (DL) – level +13m



Floor load 1 (FL1)



Floor load 2 (FL2)



Floor load 3 (FL3)

Figure 4 93 Mezzanine floor

- Crane bracket

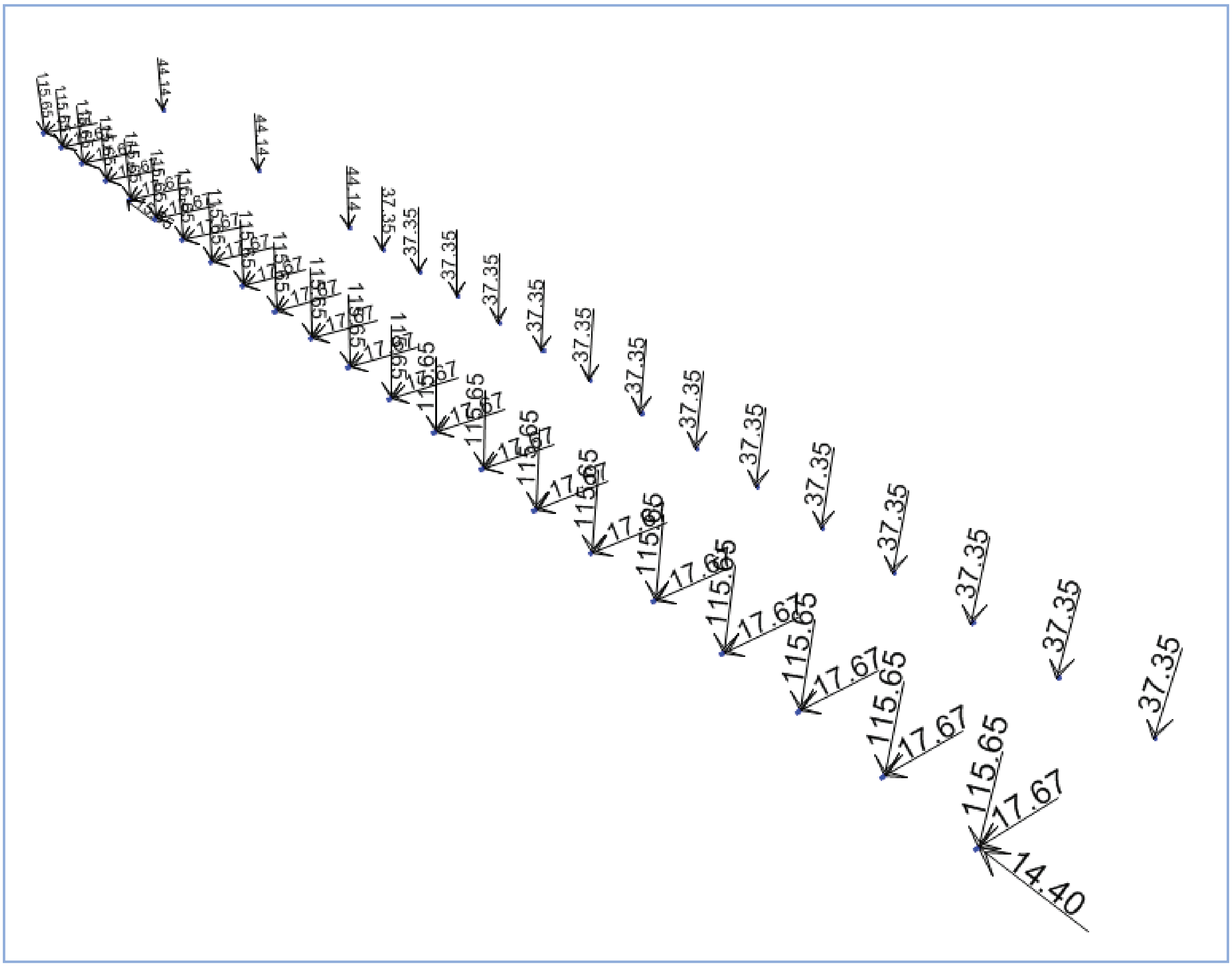


Figure 4 94 Crane load 1 (CR1)

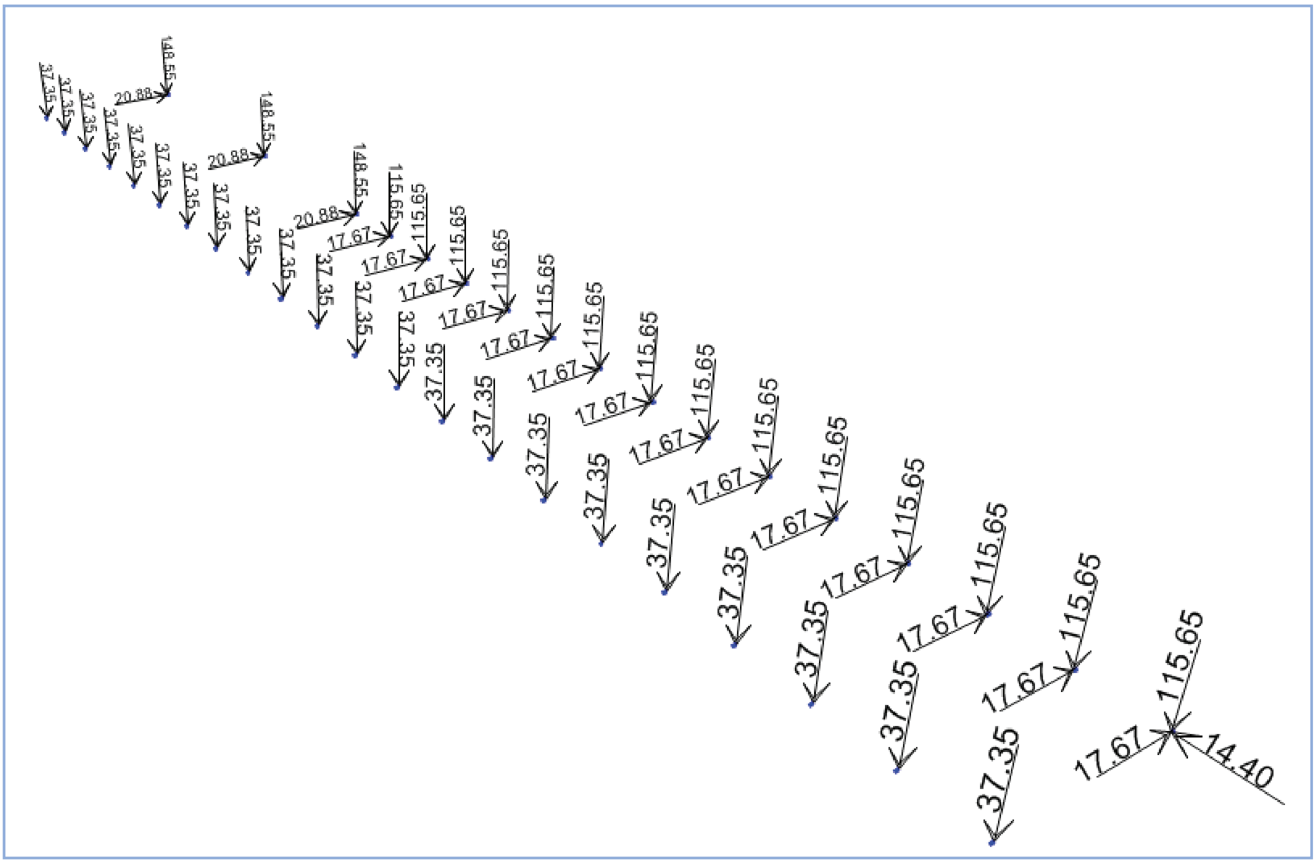
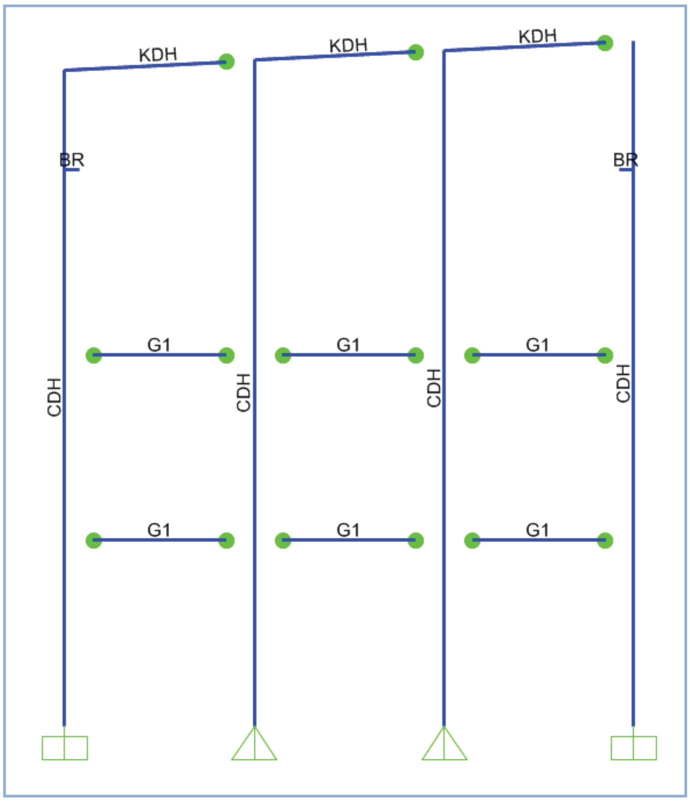


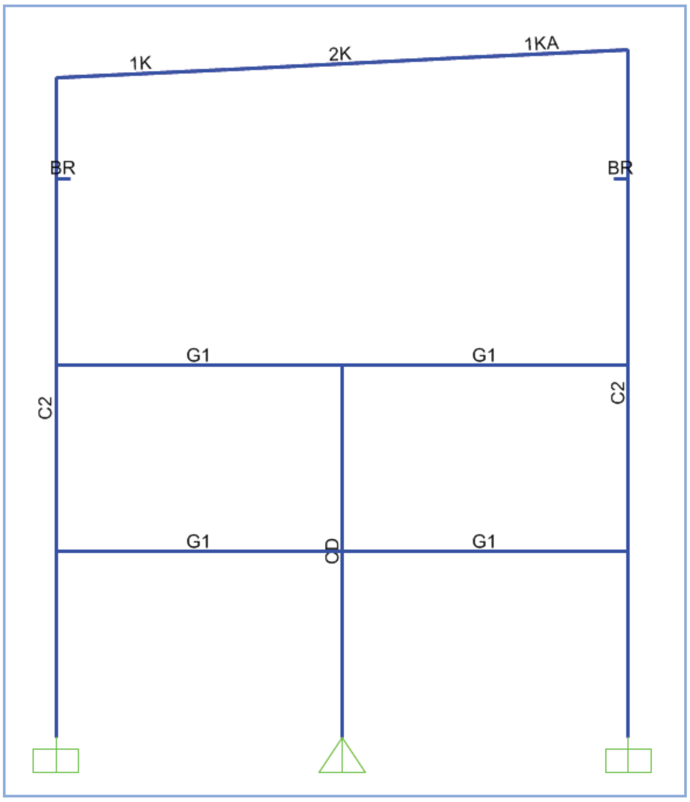
Figure 4 95 Crane load 2 (CR2)

Divide frame objects and assign member section, end releases and restraint

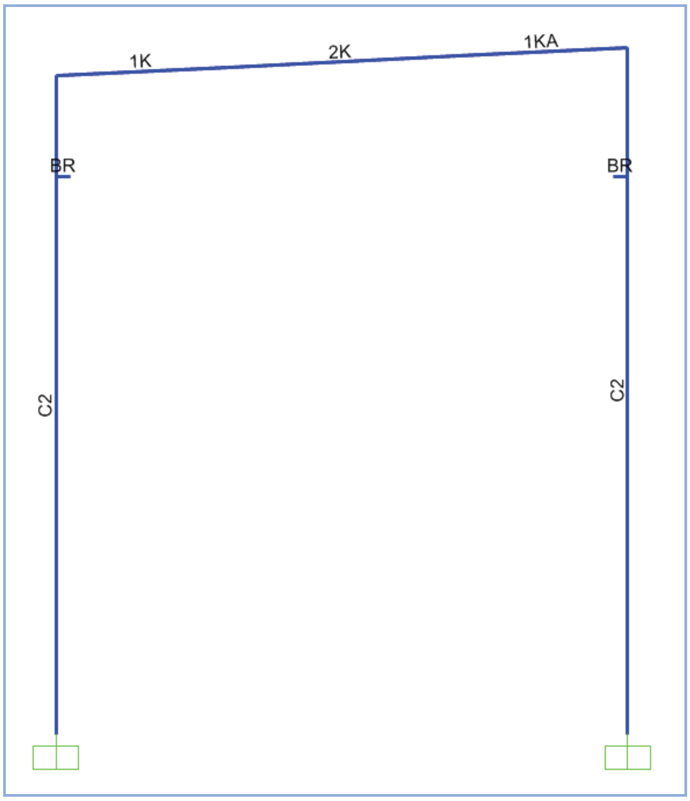
By the same manner in modeling low-rise building, we have model as shown in figure below:



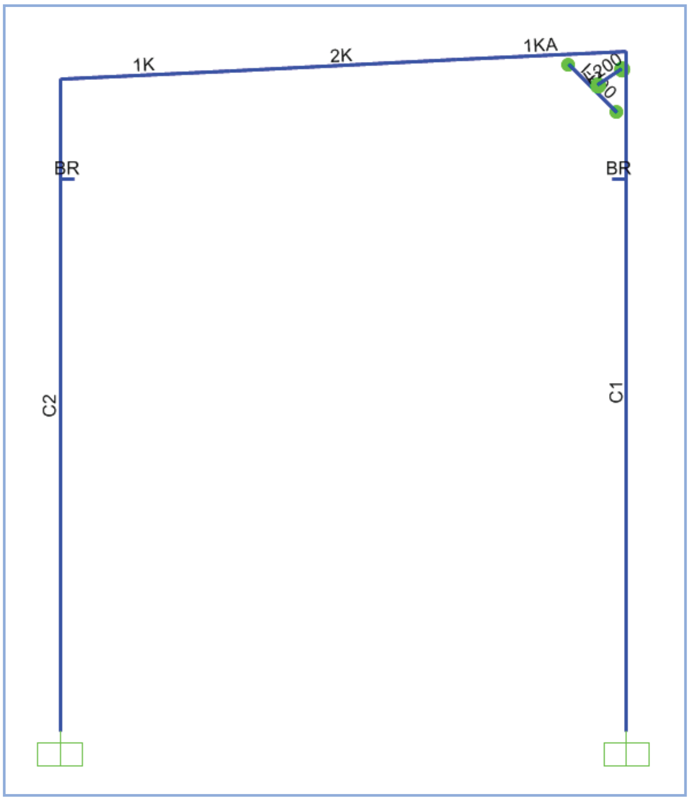
End wall frame – GL iv1



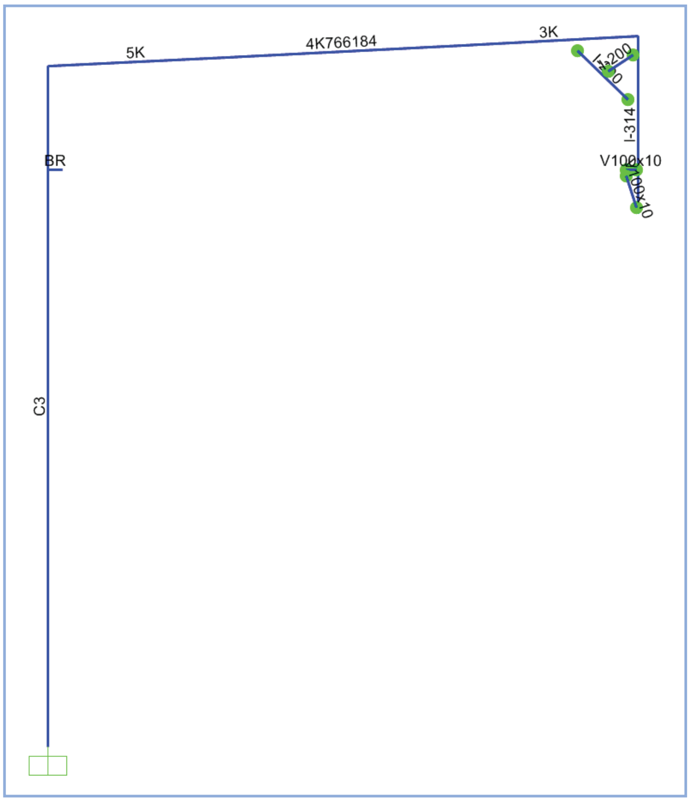
Main frame – GL iv2 to GL iv3



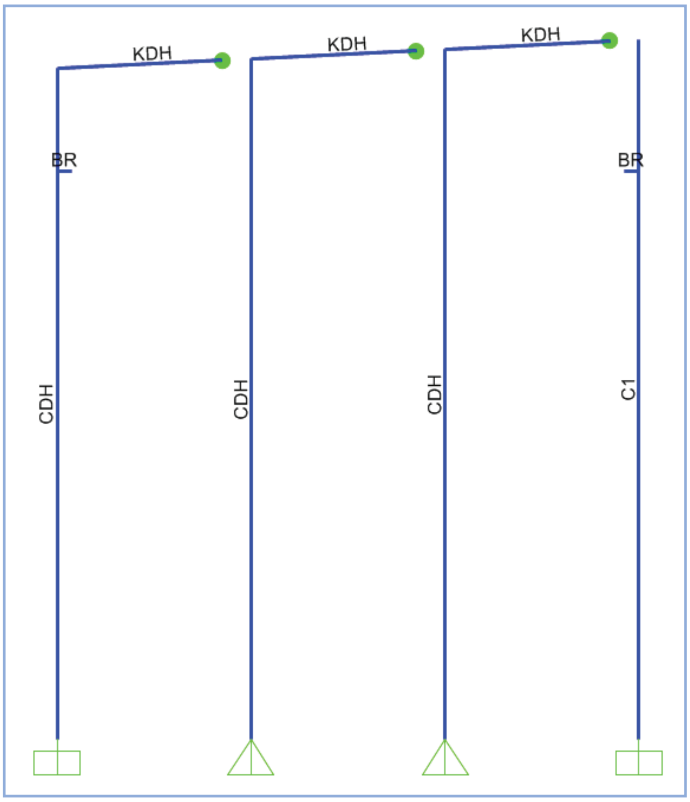
Main frame – GL iv4 to GL iv14



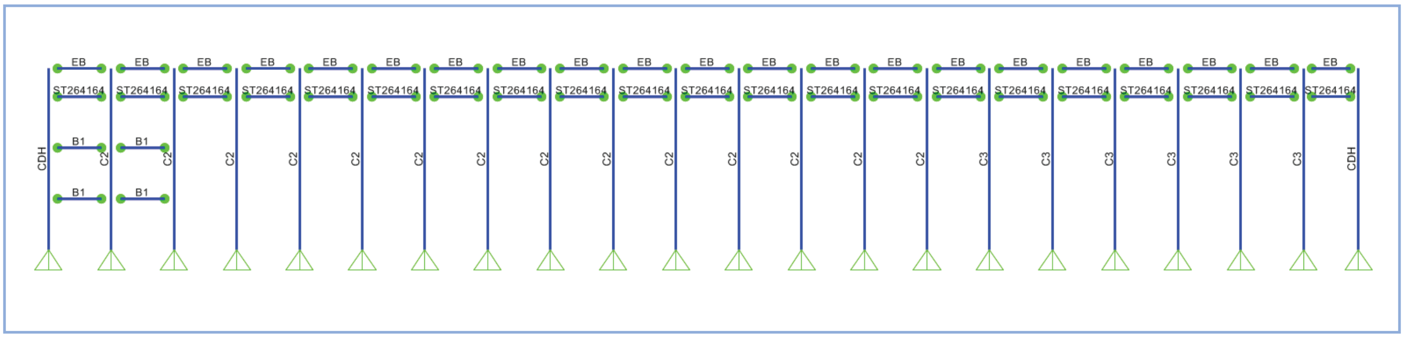
Main frame – GL iv15 & GL iv18



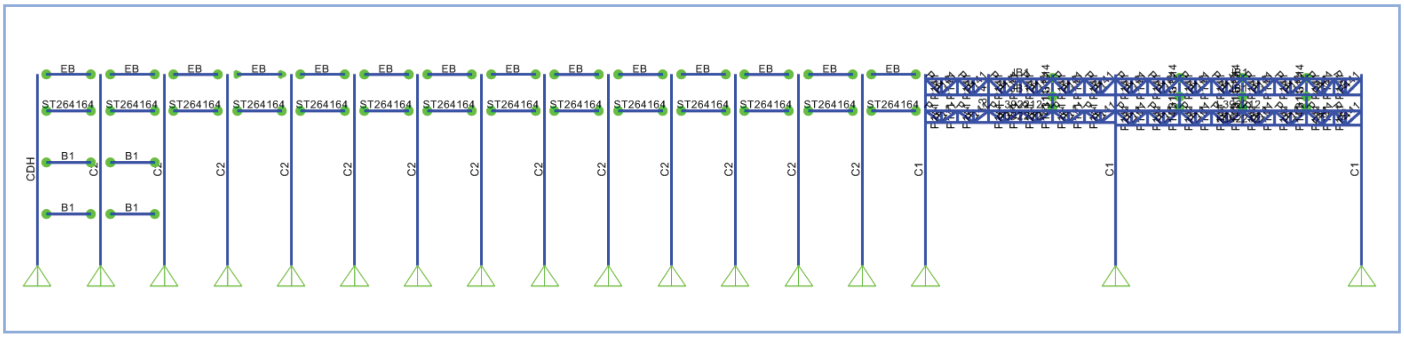
Main frame – GL iv16 to GL iv21 except GL iv18



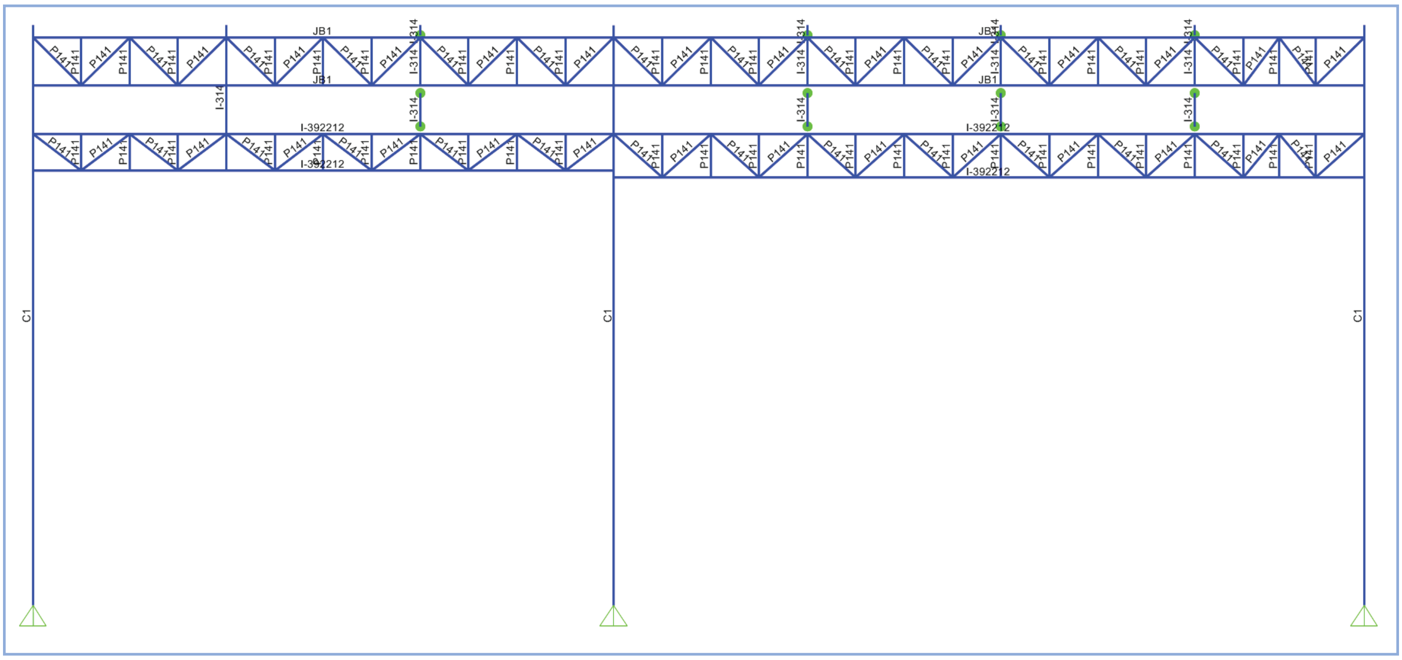
Main frame – GL iv22



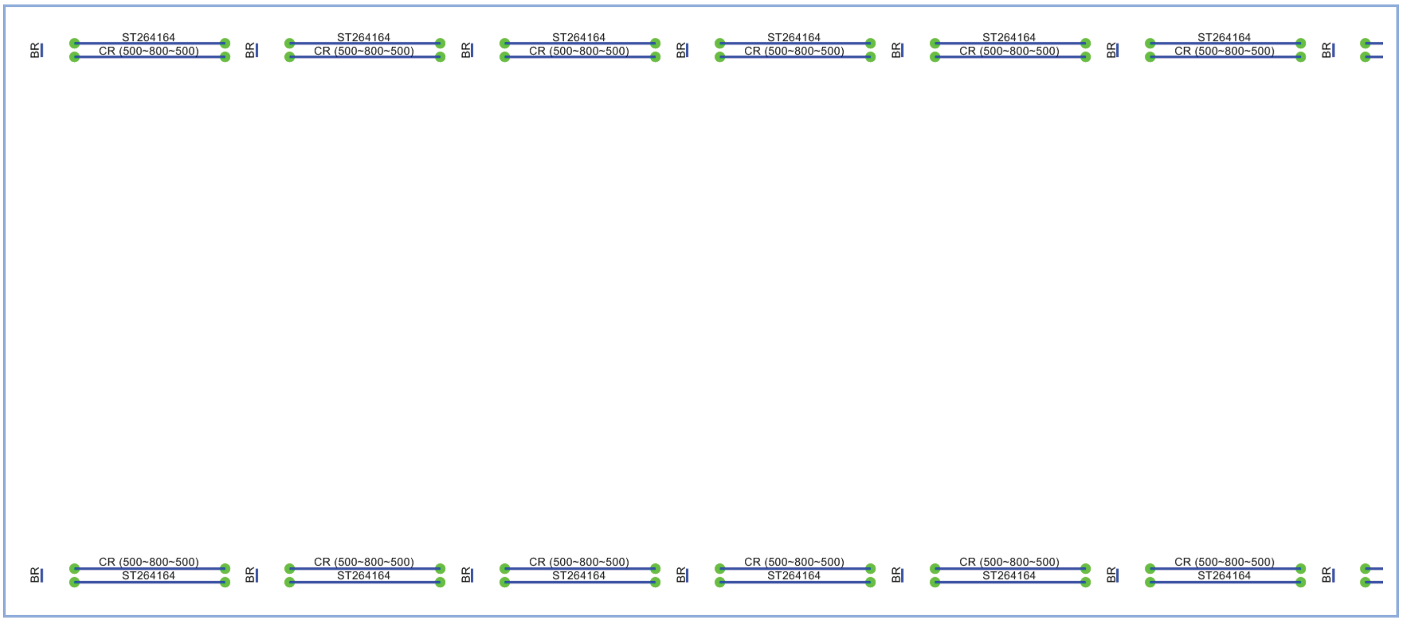
Elevation – GL ivB



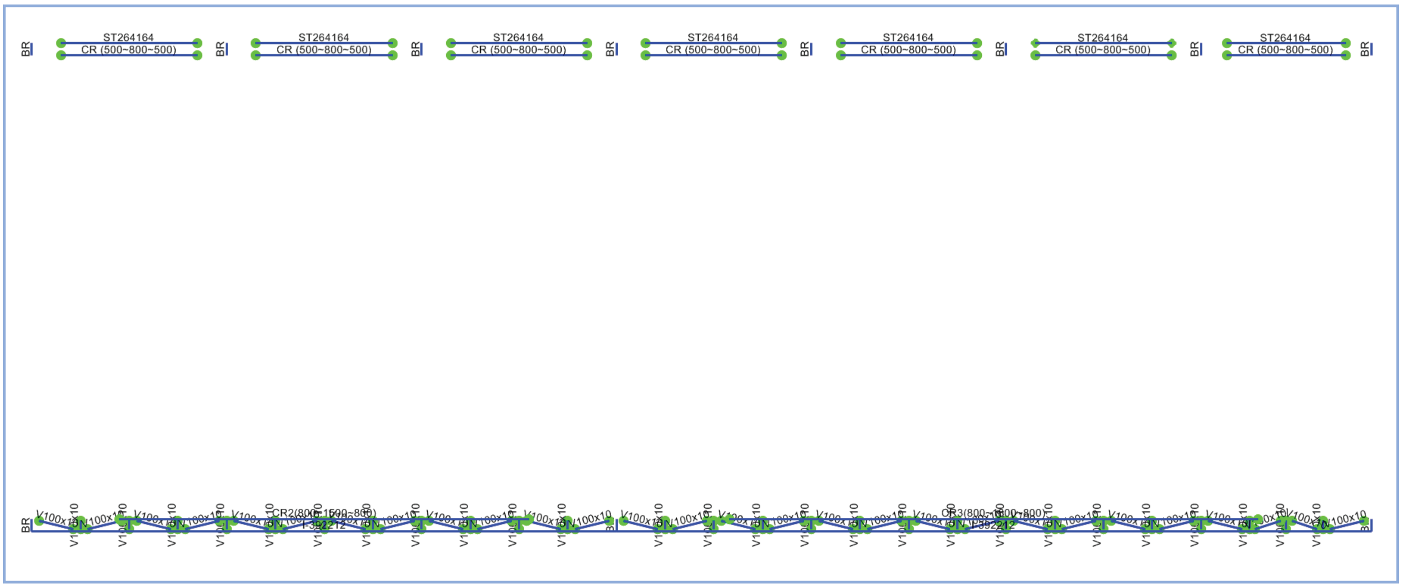
Elevation – GL ivA



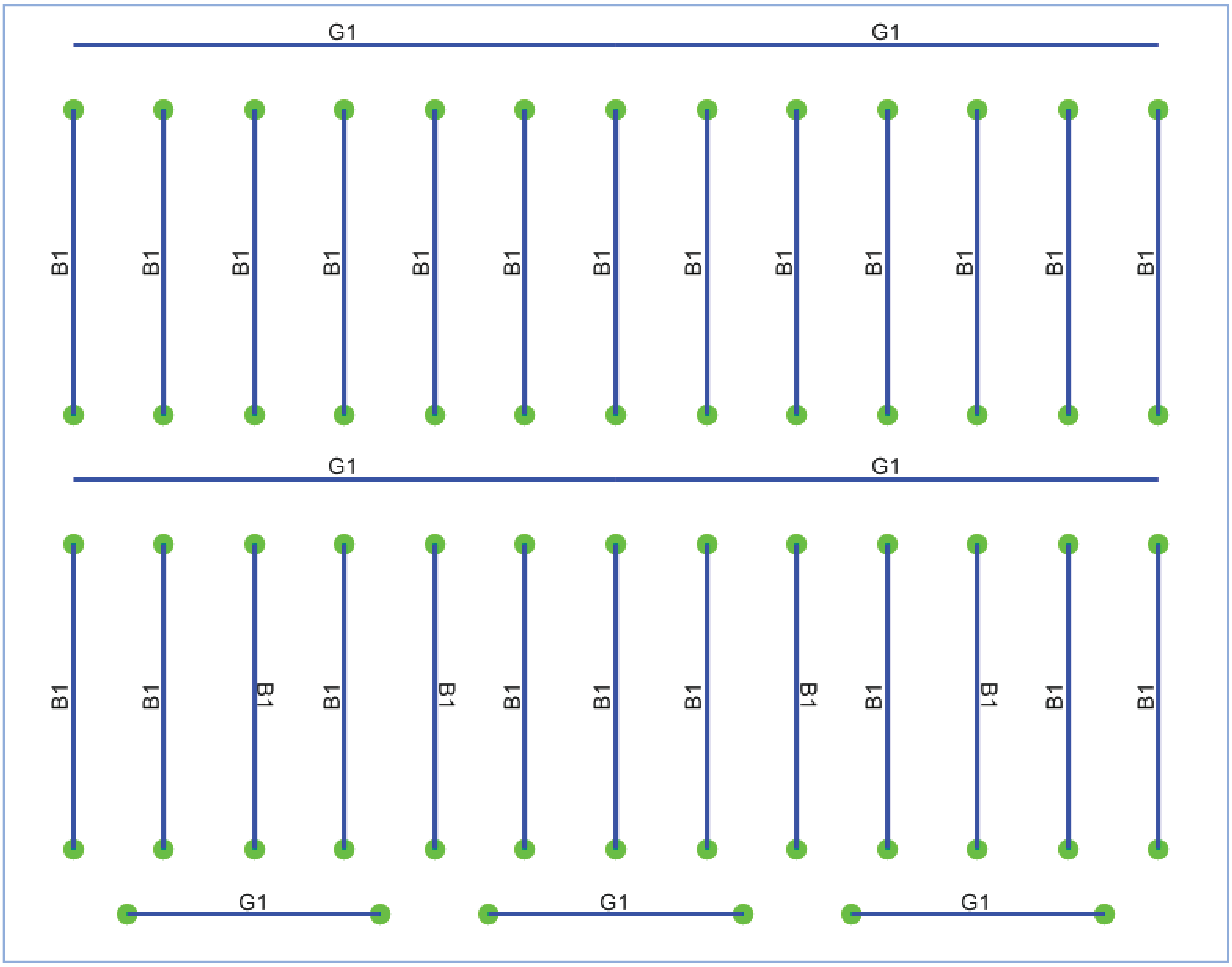
Jack beam and crane bracing



Crane runway beam & bracing – level +19.5m



Crane runway beam & bracing – level +19.5m



Mezzanine floor – level +6.5m, +13m

Calculate ULR for each member in frame

- Assume purlin spacing and girt spacing are 1.5m.
- **With rafters**: unbraced length for 1K, 1KA, 5K, 3K are 1.5m, for 2K and 4k766184 are 3m and for KDH is their length (no bracing).
- **With outer columns**: unbraced length for C1, C2, C3 and CDH are 2.5m (based on architectural drawing).
- With the other members, set ULR as Program Determined.

Table 4 5 Unbraced length ratio

Member	Length (m)	Unbraced Length (m)	ULR
1K	6	1.5	0.25
2K	8	3	0.38
1KA	6	1.5	0.25
5K	6	1.5	0.25
4K766184	8	3	0.38
3K	6	1.5	0.25
KDH	6.667	6.667	1.00
C1	23	2.5	0.11
C2	23	2.5	0.11
C3	23	2.5	0.11
CDH	23	2.5	0.11

Select 1K section members and click the **Design > Steel Frame Design > View/Revise Overwrites** command to show the **Steel Frame Design Overwrites** for AISC 360-10 form.

At the Value column, type **0.25** for both **Unbraced Length Ratio (Minor)** and **Unbraced Length Ratio (LTB)**.

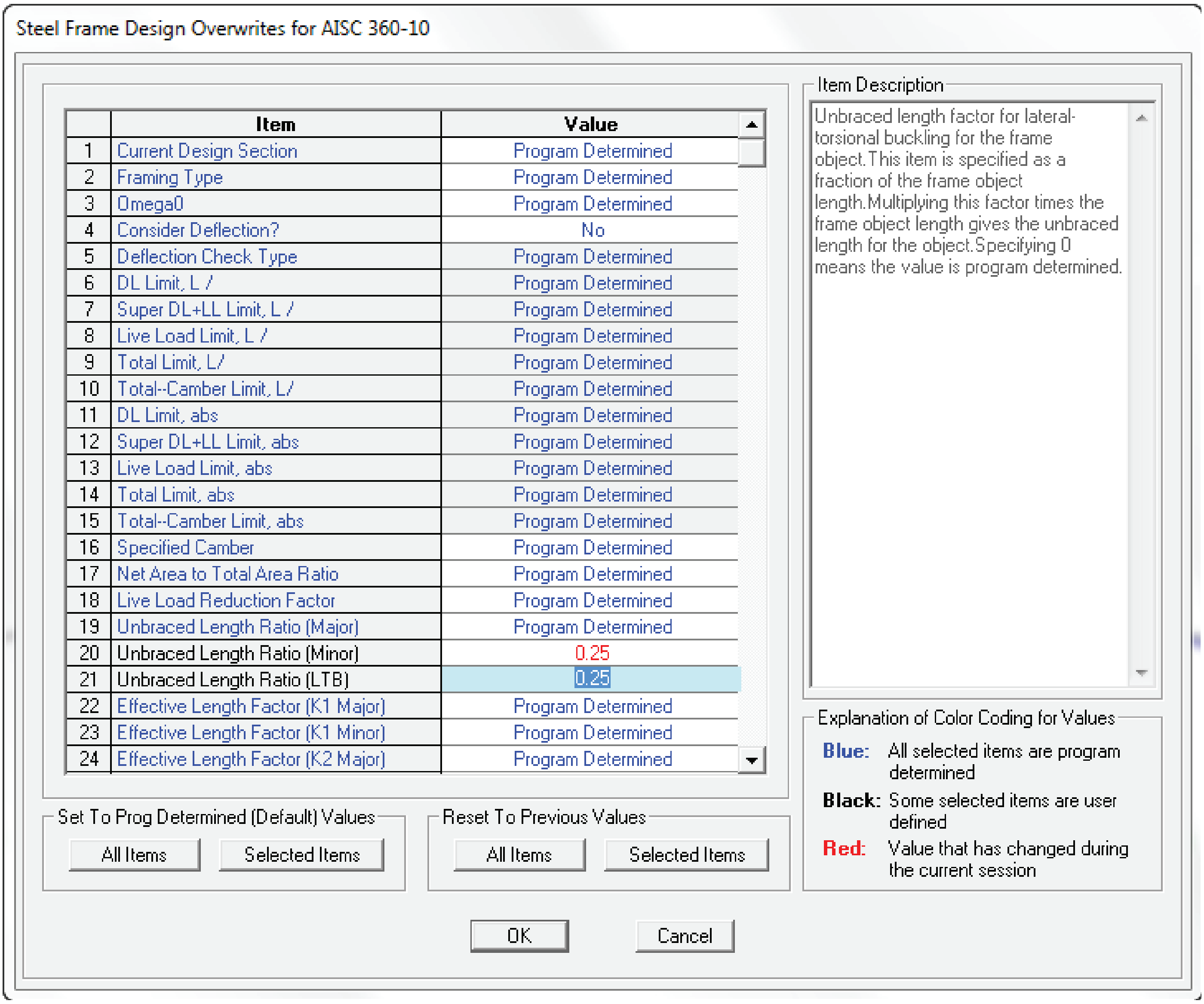
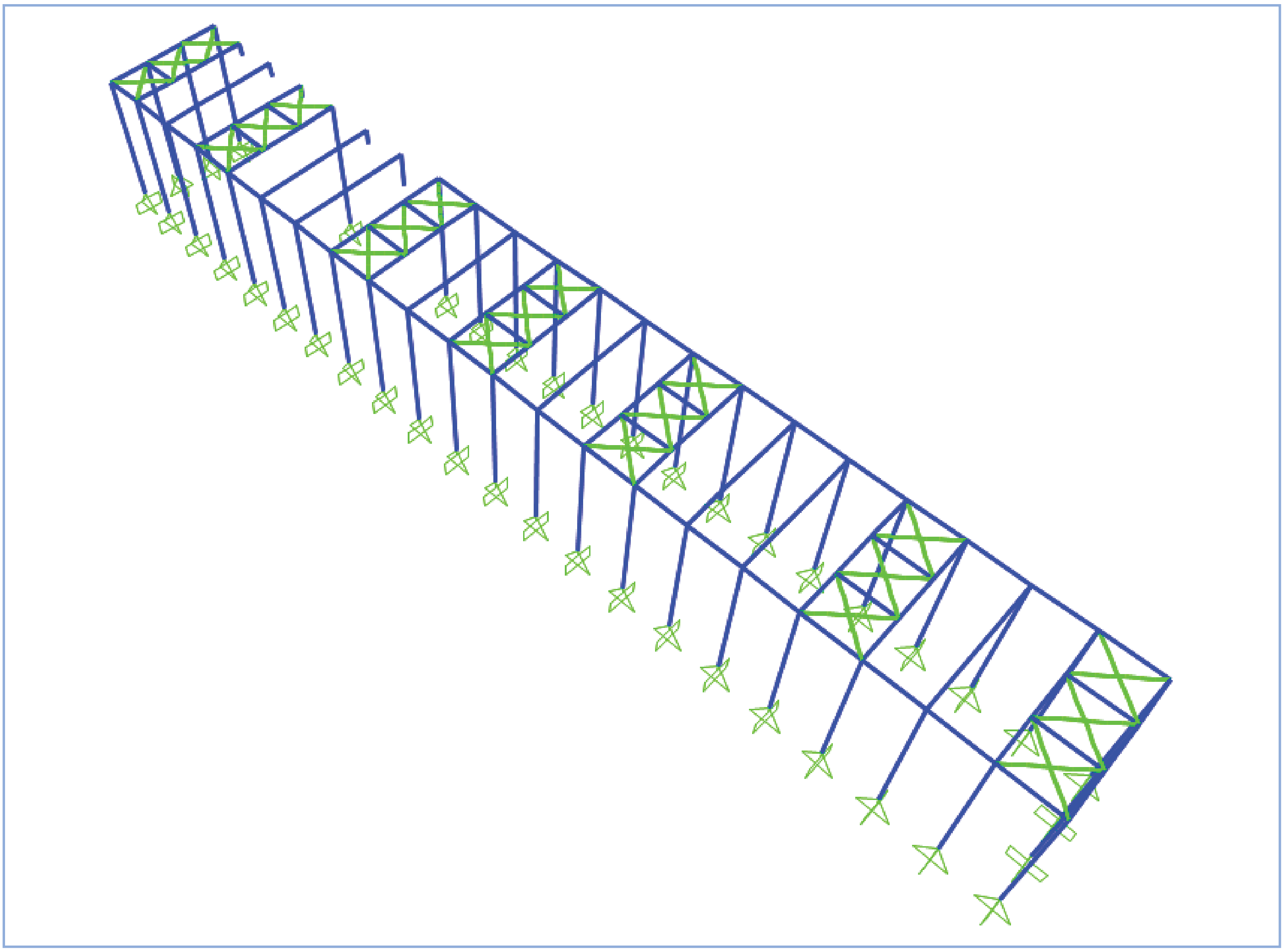
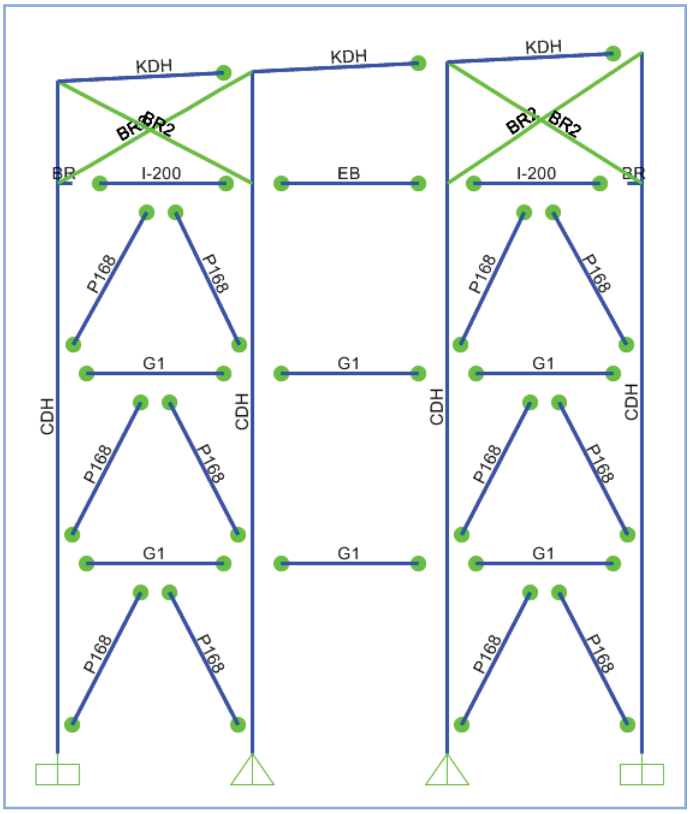


Figure 4 96 Steel frame design overwrites form

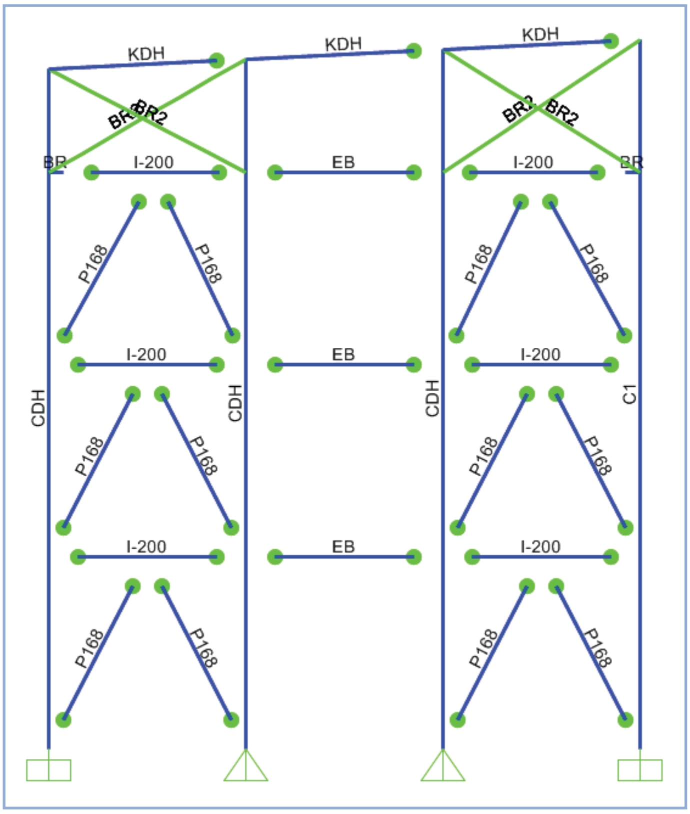
Repeat these steps to assign ULR for the other member.

Draw Bracing System

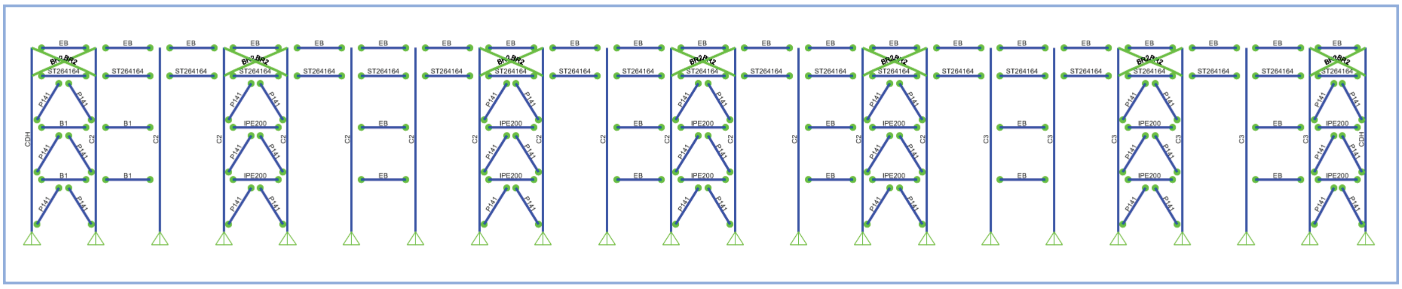




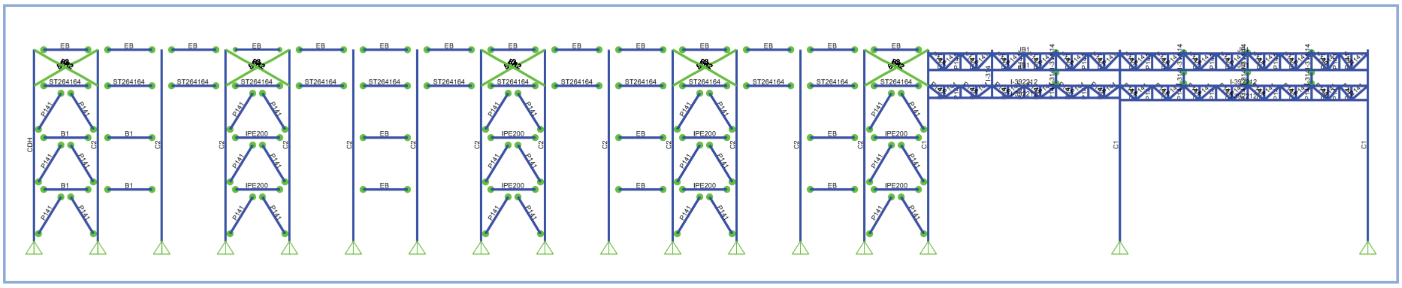
End wall GL iv1



End wall GL iv22



Elevation GL ivB



Elevation GL ivA

4.1.6.3 Analyzing

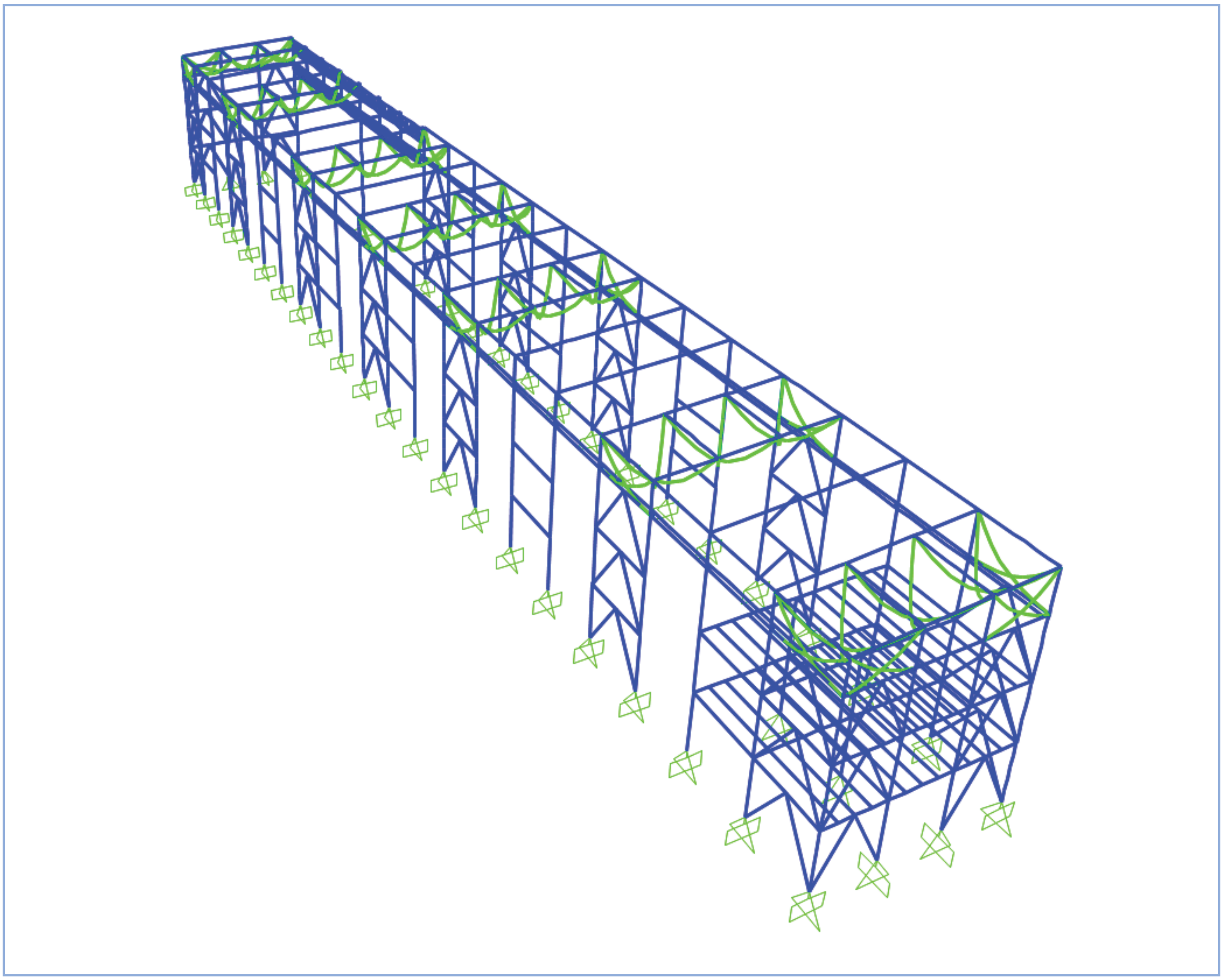


Figure 4 98 Model after running analysis

4.1.6.4 Design Steel Frame

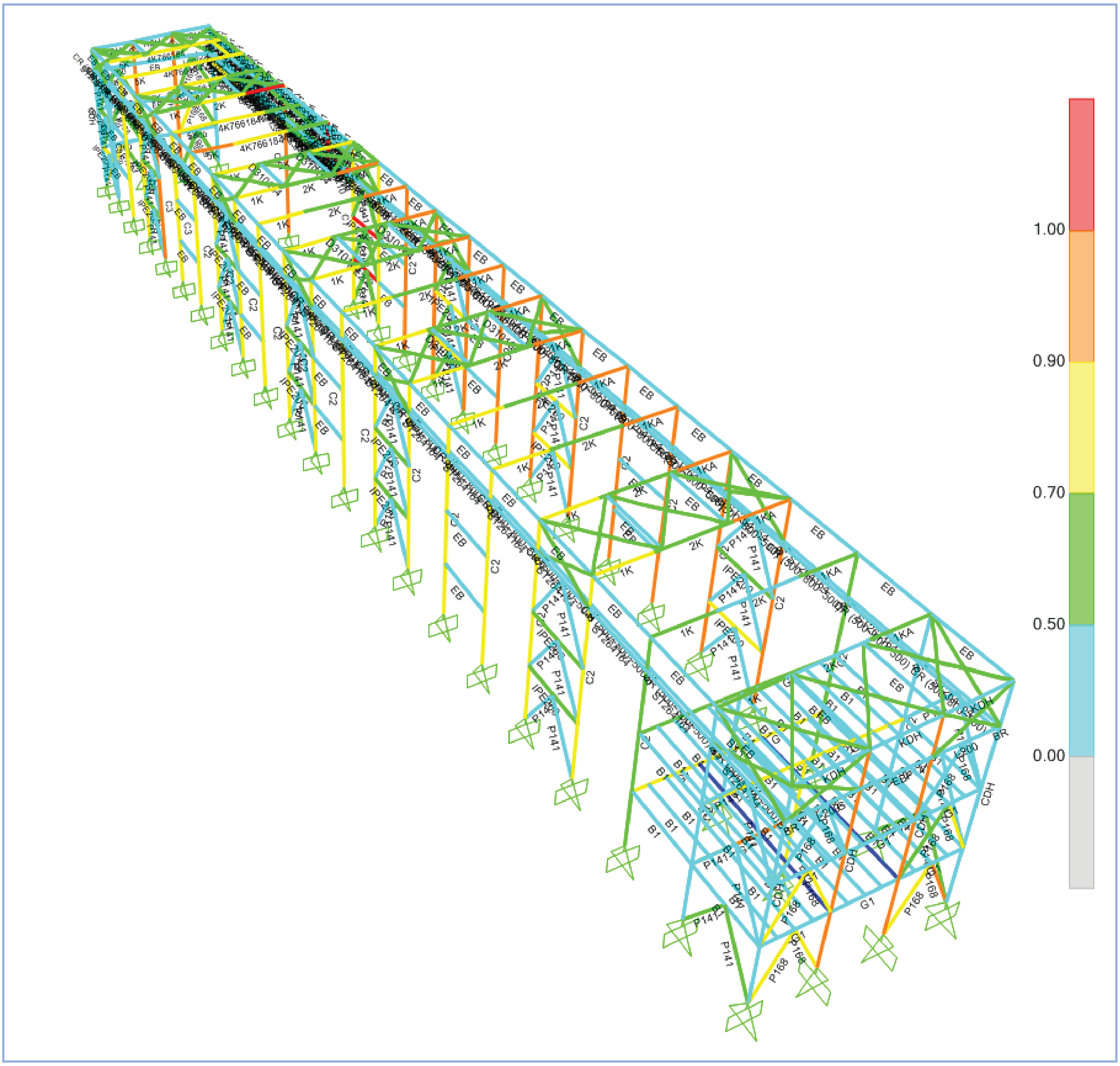


Figure 4 99 Model after checking

To see detail information of checking strength, right click on the **1K** member to show the **Steel Stress Check Information** form.

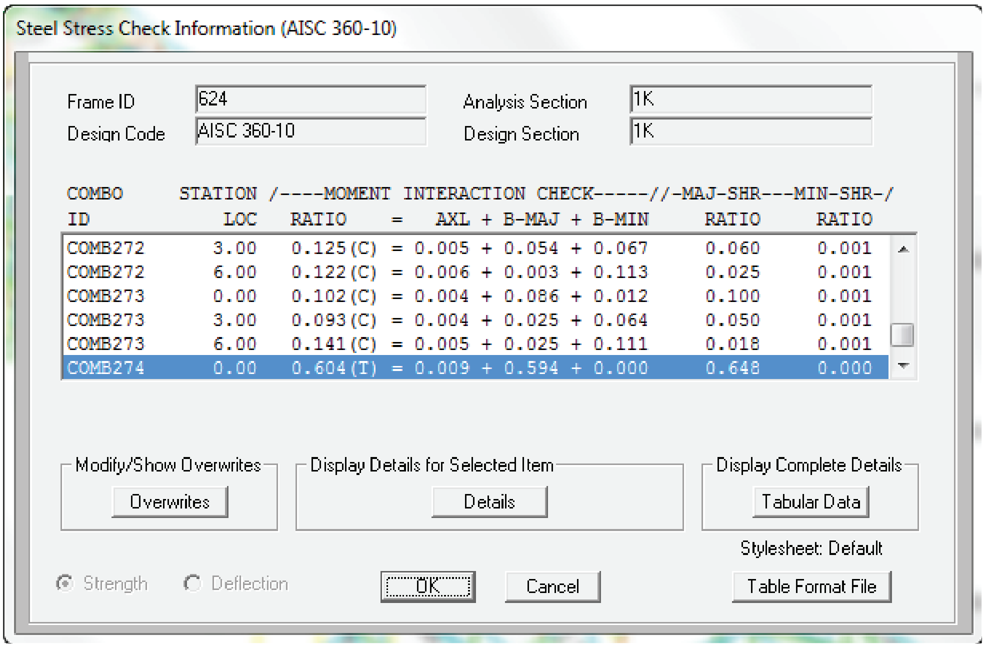


Figure 4 100 Steel Stress Check Information form

Click the **Details** button to show more detail information.

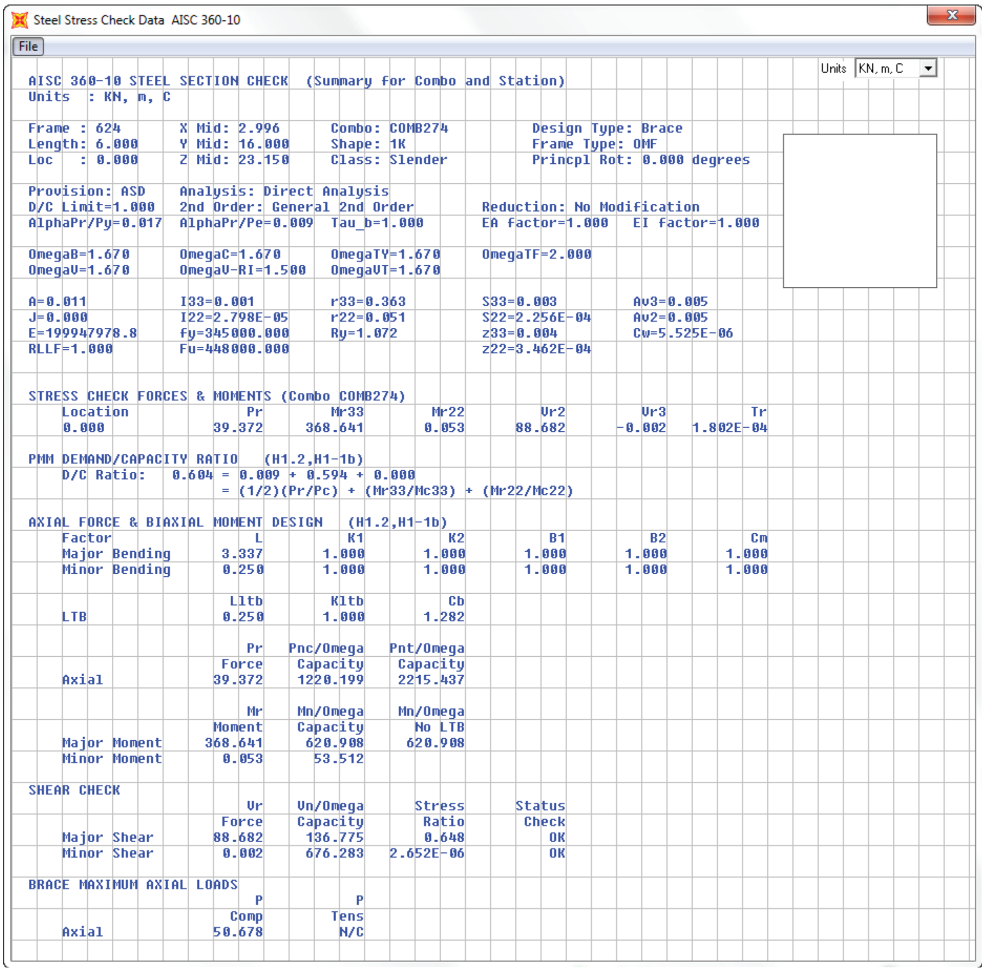


Figure 4 101 Steel Stress Check Data form

NOTE

Unbrace length ratio for major bending is 3.337 which is taken to be distance between two bracing point in major axis (20m) divided by actual length of member (6m). You should carefully review this ratio to ensure that the design process is consistent with your expectations.

To check whether all steel frames passed the stress check, click the **Design menu > Steel Frame Design > Verify All Members Passed** command to display as figure below.

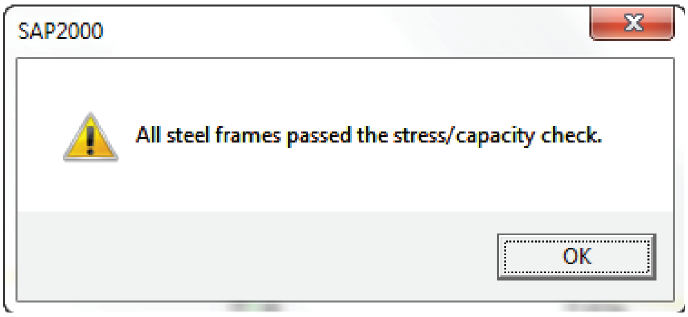


Figure 4 102 Checking capacity of all steel objects

4.1.6.5 Check Deflection Limitation

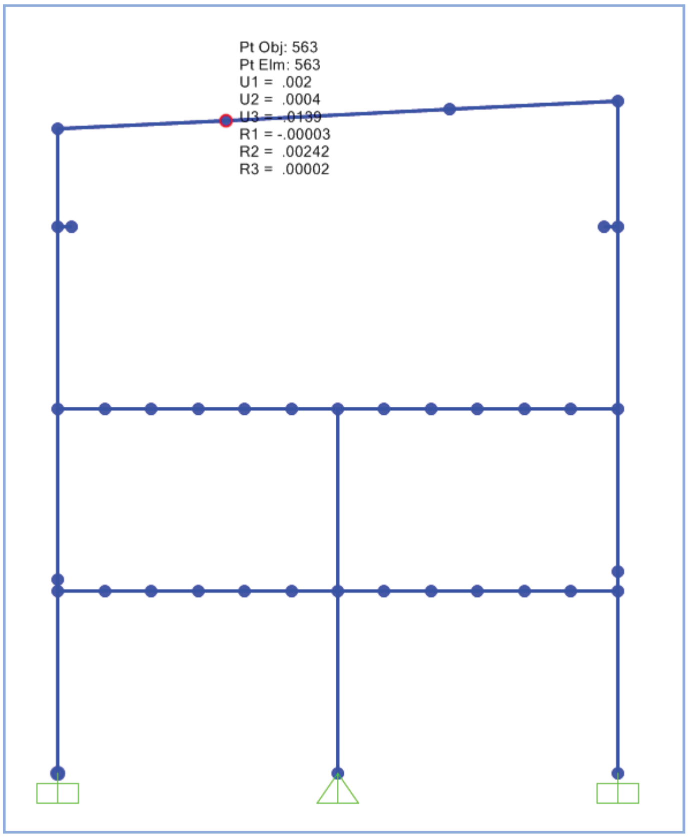
Create combos for deflection checking:

Name	Combination
CV1	DL + 0.4224WL1
CV2	DL + 0.4224WL2
CV3	DL + 0.4224WL3
CV4	DL + 0.4224WL4
CV5	DL + 0.4224WL5
CV6	DL + 0.4224WL6
CV7	DL + 0.4224WL7
CV8	DL + 0.4224WL8
CVN1	DL + 0.75CR1 + 0.3168WL1
CVN2	DL + 0.75CR1 + 0.3168WL2
CVN3	DL + 0.75CR1 + 0.3168WL3
CVN4	DL + 0.75CR1 + 0.3168WL4
CVN5	DL + 0.75CR2 + 0.3168WL1
CVN6	DL + 0.75CR2 + 0.3168WL2
CVN7	DL + 0.75CR2 + 0.3168WL3
CVN8	DL + 0.75CR2 + 0.3168WL4
CVN9	DL + 0.75CR1 + 0.3168WL6
CVN10	DL + 0.75CR1 + 0.3168WL7
CVN11	DL + 0.75CR2 + 0.3168WL6
CVN12	DL + 0.75CR2 + 0.3168WL7
CV	CV1 + CV2 + ... + CVN12

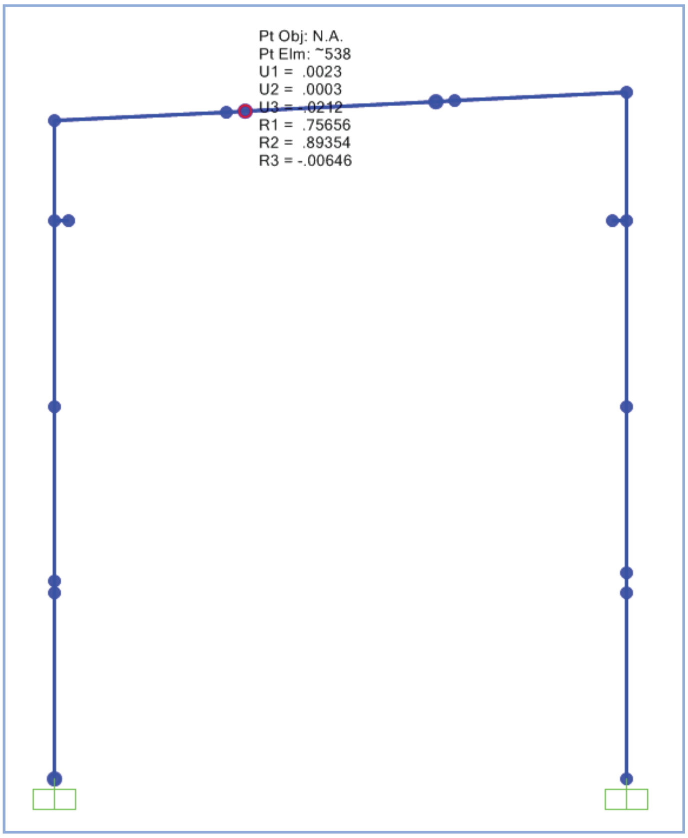
Checking if vertical deflection (U3) of mid points of rafter are excess deflection limit.

Compute deflection limit: $\Delta = \frac{L}{120} = \frac{20}{120} = 0.167\,m$

Put the cursor at mid points of rafter to see if deflection of that point is excess the limit or not.



Deflection of point 563 – 0.0109m



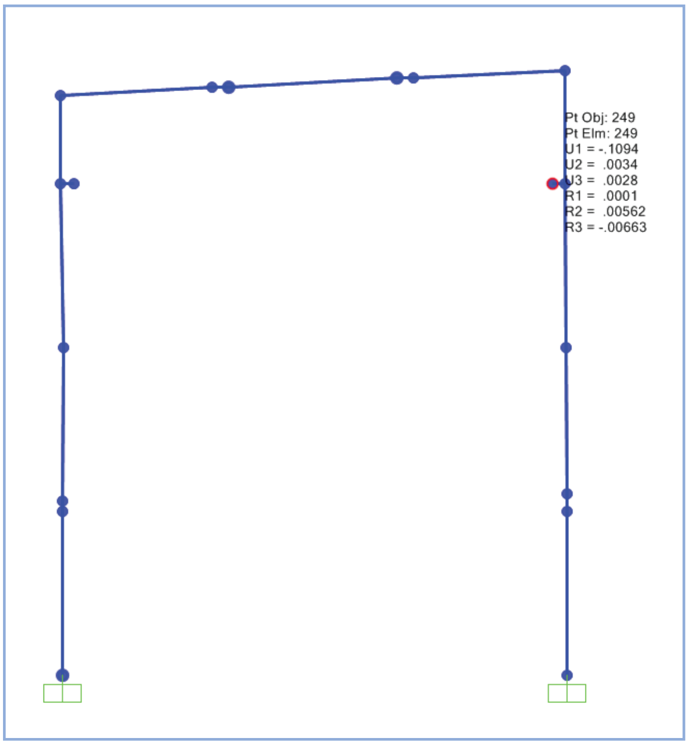
Deflection of point 538 – 0.0212m

Click the Display > Show Deformed Shape command which will show the **Deformed Shape** form. Select **CV** from **Case/Combo Name** drop-down list and click OK to accept.

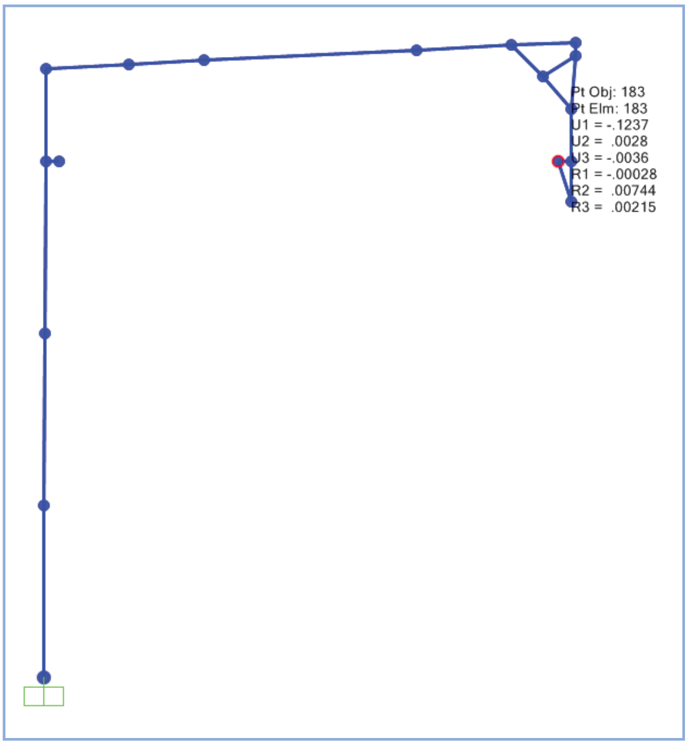
Checking if horizontal deflection of crane bracket are excess deflection limit.

Compute horizontal deflection limit: $\Delta = \frac{h}{100} = \frac{19.5}{100} = 0.195m$

Put the cursor at points located at the crane bracket to see if the horizontal deflection (U1 or U2) of that point is excess the limit or not.



Deflection of point 249 – 0.1094m

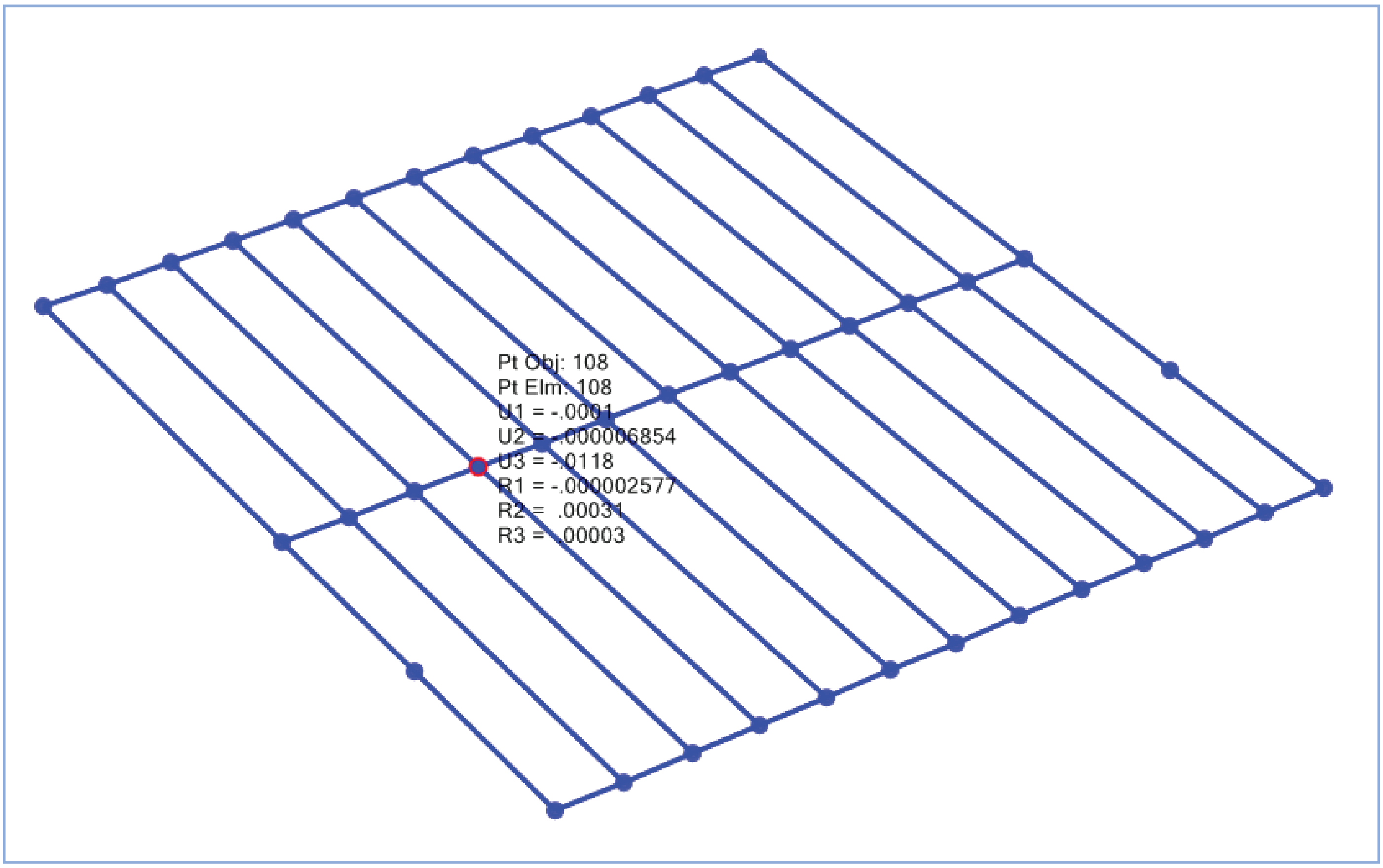


Deflection of point 183 – 0.1237m

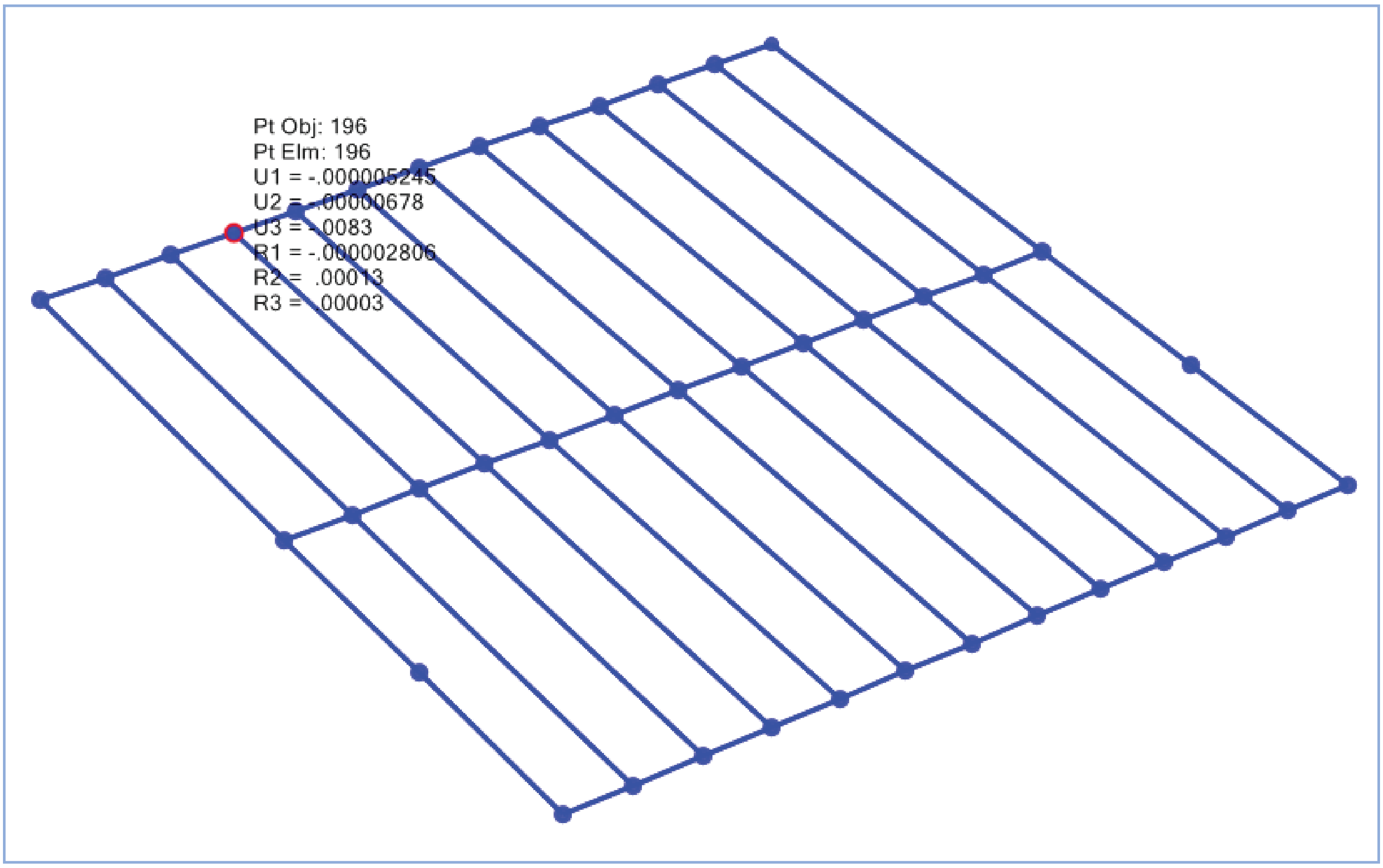
Checking if vertical deflection of girder are excess deflection limit.

Compute vertical deflection limit: $\Delta = \dfrac{L}{240} = \dfrac{10}{240} = 0.042\,m$

Put the cursor at points located at the middle of girder to see if the vertical deflection (U1 or U2) of that point is excess the limit or not.



Deflection of point 249 – 0.1094m



Deflection of point 196 – 0.0083m

4.1.6.6 Export joint reaction

Transform from wind load with return period wind speed of 700 years to wind load with those of 50 years

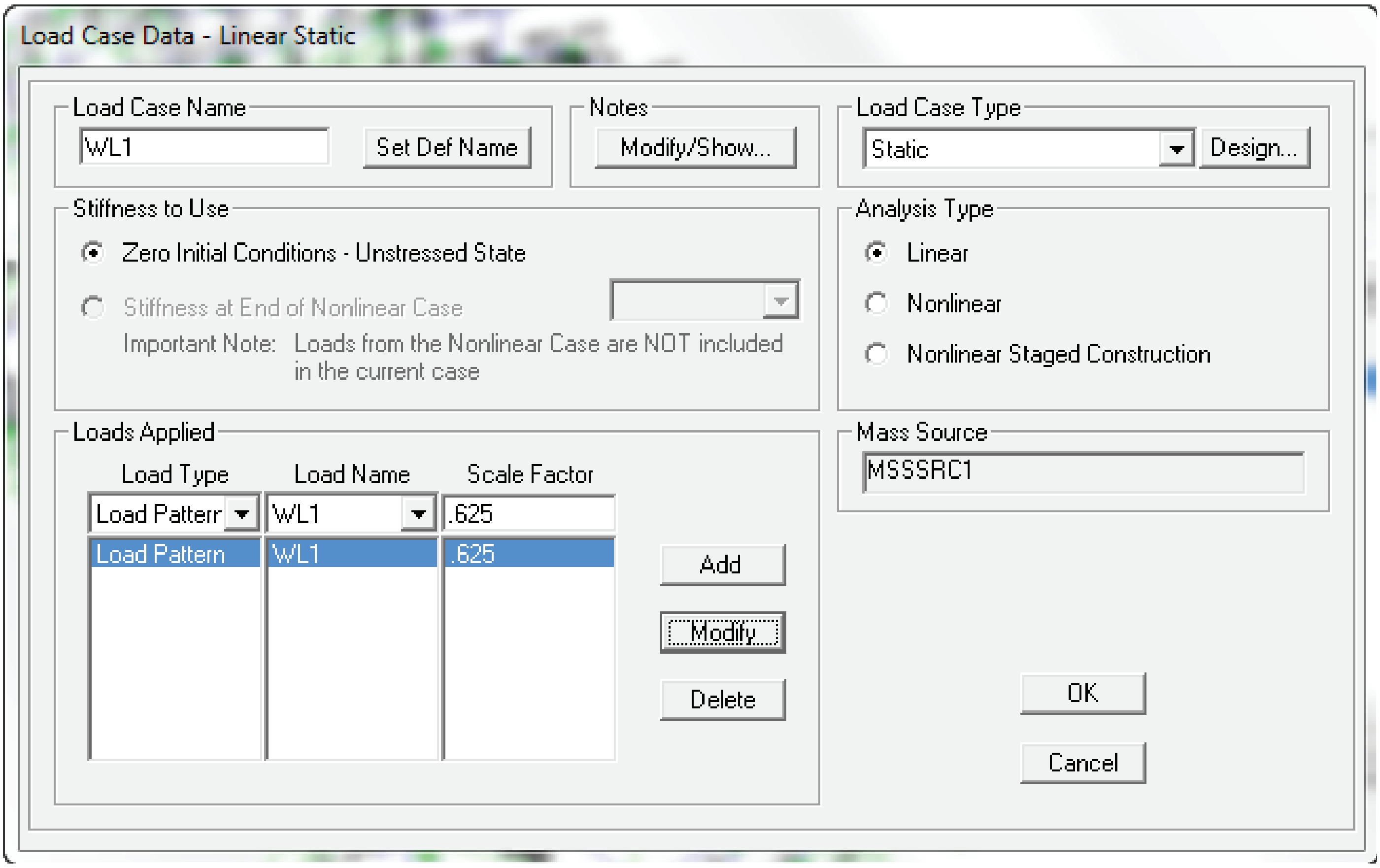


Figure 4 103 Load Case Data – Linear Static form

Changing label name of joints

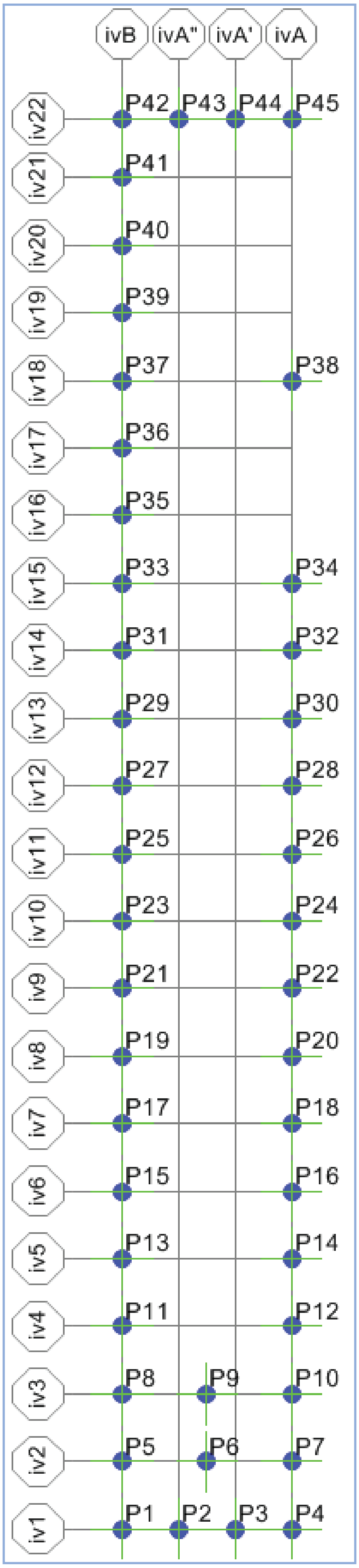
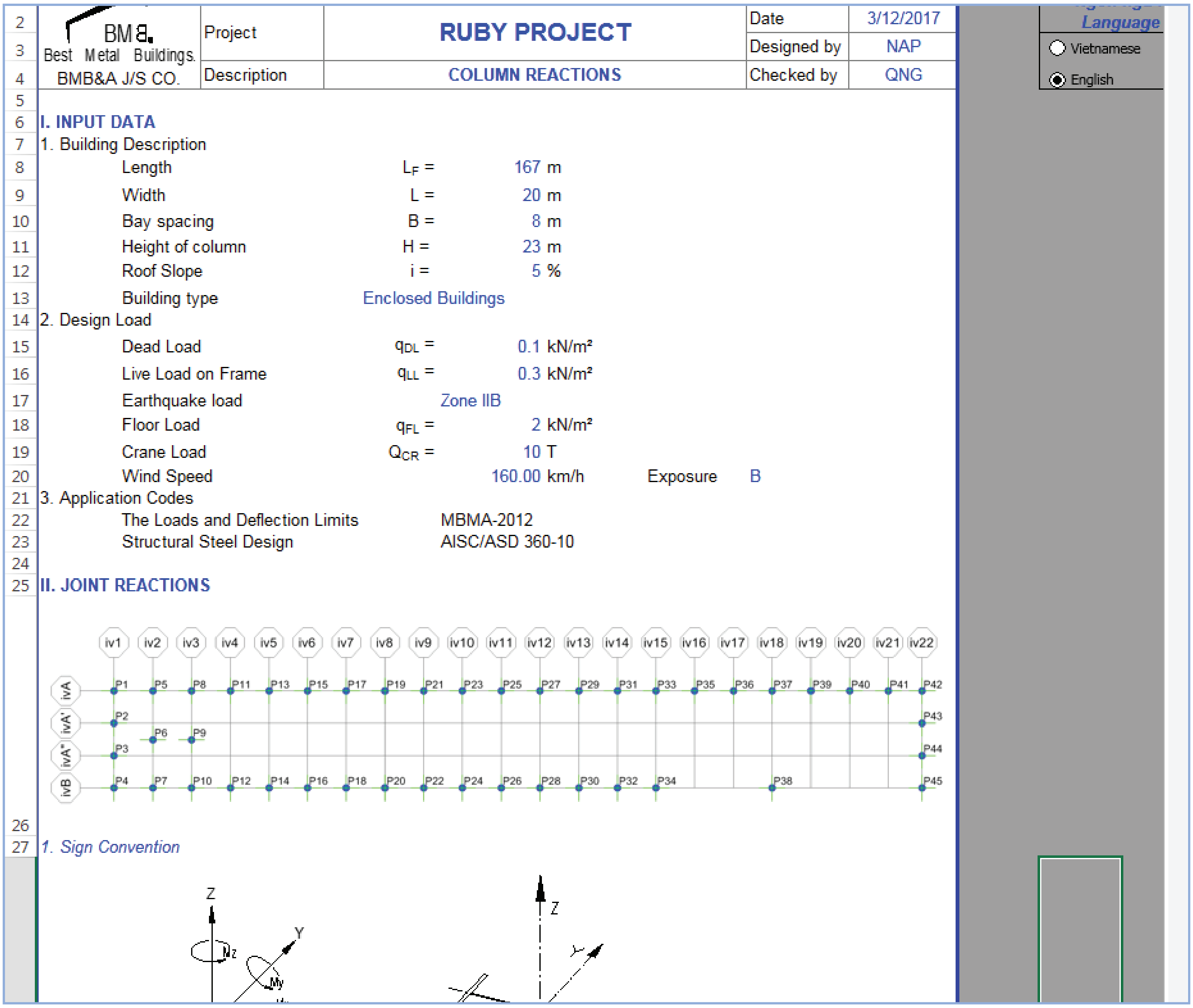


Figure 4 104 Labels after change



BMB Best Metal Buildings	Project	RUBY PROJECT		Date	3/12/2017
				Designed by	NAP
BMB&A J/S CO.	Description	COLUMN REACTIONS		Checked by	QNG

Language: Vietnamese / English

I. INPUT DATA

1. Building Description

Length	L_F =	167	m
Width	L =	20	m
Bay spacing	B =	8	m
Height of column	H =	23	m
Roof Slope	i =	5	%
Building type	Enclosed Buildings		

2. Design Load

Dead Load	q_{DL} =	0.1	kN/m²		
Live Load on Frame	q_{LL} =	0.3	kN/m²		
Earthquake load	Zone IIB				
Floor Load	q_{FL} =	2	kN/m²		
Crane Load	Q_{CR} =	10	T		
Wind Speed		160.00	km/h	Exposure	B

3. Application Codes

The Loads and Deflection Limits	MBMA-2012
Structural Steel Design	AISC/ASD 360-10

II. JOINT REACTIONS

1. Sign Convention

Figure 4 105 Column reactions file

NOTE

Change Collateral load to Earthquake load in Design load area. Do not insert any more rows or column in this sheet.

Run the model and specify all joints in active plan. Select the **Display > Show Tables** command to show Choose Tables for Display form. Make sure that the Table: **Joint Reactions** item and **DL, LL, FL1, FL2, FL3, CR1, CR2, WL1, WL2, WL3, WL4, WL5, WL6, WL7, WL8, EL1, EL2** in the Select Load Cases form are selected before click the OK button.

Joint Reactions

File View Format-Filter-Sort Select Options

Units: As Noted — Joint Reactions

Joint Text	OutputCase Text	CaseType Text	F1 KN	F2 KN	F3 KN	M1 KN-m	M2 KN-m	M3 KN-m
P1	DL	LinStatic	13.56	2.351	145.032	0	0.615	0.0004886
P1	LL	LinStatic	-0.016	0.308	5.022	0	-0.0543	0.000203
P1	WL1	LinStatic	-34.426	21.371	-115.853	0	-26.1868	-0.0077
P1	WL2	LinStatic	-36.311	11.487	-128.174	0	-28.7126	-0.0025
P1	WL3	LinStatic	-48.534	22.579	-156.256	0	-36.5107	-0.0067
P1	WL4	LinStatic	-38.564	12.268	-128.847	0	-30.3766	-0.0016
P1	WL5	LinStatic	11.909	-25.65	-77.189	0	13.1126	0.0035
P1	WL6	LinStatic	43.611	21.896	217.576	0	33.9604	-0.0141
P1	WL7	LinStatic	36.851	12.104	193.237	0	27.2765	-0.0086
P1	WL8	LinStatic	12.394	0.399	2.517	0	13.1879	0.0017
P1	EL1	LinStatic	-21.793	-0.258	-99.633	0	-15.5845	0.0015
P1	EL2	LinStatic	-0.553	-23.909	-89.724	0	-0.0973	-0.1776
P1	CR1	LinStatic	7.419	-1.959	130.355	0	4.925	-0.0291
P1	CR2	LinStatic	-6.089	-0.002696	-7.305	0	-4.1668	0.0003708
P1	FL1	LinStatic	7.622	0.864	63.127	0	0.3299	0.0002197
P1	FL2	LinStatic	5.923	0.837	48.585	0	-0.6089	0.0004404
P1	FL3	LinStatic	1.698	0.027	14.541	0	0.9387	-0.0002208
P2	DL	LinStatic	-13.02	-0.002652	240.528	0.1646	0	-0.000006594
P2	LL	LinStatic	-0.217	0.001389	9.284	0.0007146	0	-0.000006527
P2	WL1	LinStatic	-25.872	71.896	104.692	-340.8842	0	0.0008783
P2	WL2	LinStatic	-25.105	38.505	111.136	-182.5022	0	0.0005207

Record: 1 of 765 — Add Tables... — Done

Figure 4 106 Element Joints Forces – Frame table

	Joint	OutputCase	CaseType	F1	F2	F3	M1	M2	M3
1	TABLE: Joint Reactions								
2	Joint	OutputCase	CaseType	F1	F2	F3	M1	M2	M3
3	Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
4	P1	DL	LinStatic	13.56	2.351	145.032	0	0.615	0.0004886
5	P1	LL	LinStatic	-0.016	0.308	5.022	0	-0.0543	0.000203
6	P1	WL1	LinStatic	-34.426	21.371	-115.853	0	-26.1868	-0.0077
7	P1	WL2	LinStatic	-36.311	11.487	-128.174	0	-28.7126	-0.0025
8	P1	WL3	LinStatic	-48.534	22.579	-156.256	0	-36.5107	-0.0067
9	P1	WL4	LinStatic	-38.564	12.268	-128.847	0	-30.3766	-0.0016
10	P1	WL5	LinStatic	11.909	-25.65	-77.189	0	13.1126	0.0035
11	P1	WL6	LinStatic	43.611	21.896	217.576	0	33.9604	-0.0141
12	P1	WL7	LinStatic	36.851	12.104	193.237	0	27.2765	-0.0086
13	P1	WL8	LinStatic	12.394	0.399	2.517	0	13.1879	0.0017
14	P1	EL1	LinStatic	-21.793	-0.258	-99.633	0	-15.5845	0.0015
15	P1	EL2	LinStatic	-0.553	-23.909	-89.724	0	-0.0973	-0.1776
16	P1	CR1	LinStatic	7.419	-1.959	130.355	0	4.925	-0.0291
17	P1	CR2	LinStatic	-6.089	-0.002696	-7.305	0	-4.1668	0.0003708
18	P1	FL1	LinStatic	7.622	0.864	63.127	0	0.3299	0.0002197
19	P1	FL2	LinStatic	5.923	0.837	48.585	0	-0.6089	0.0004404
20	P1	FL3	LinStatic	1.698	0.027	14.541	0	0.9387	-0.0002208
21	P2	DL	LinStatic	-13.02	-0.002652	240.528	0.1646	0	-0.000006594
22	P2	LL	LinStatic	-0.217	0.001389	9.284	0.0007146	0	-0.000006527
23	P2	WL1	LinStatic	-25.872	71.896	104.692	-340.8842	0	0.0008783
24	P2	WL2	LinStatic	-25.105	38.505	111.136	-182.5022	0	0.0005207
25	P2	WL3	LinStatic	-38.691	71.907	188.878	-340.9069	0	0.0008838
26	P2	WL4	LinStatic	-27.666	38.511	151.135	-182.4868	0	0.0005309
27	P2	WL5	LinStatic	2.47	-65.006	-41.431	326.4516	0	-0.0006454
28	P2	WL6	LinStatic	31.745	71.869	-203.171	-341.1807	0	0.0004952
29	P2	WL7	LinStatic	29.694	38.479	-184.574	-182.7928	0	0.0001412
30	P2	WL8	LinStatic	2.31	0.571	-39.448	-3.3364	0	0.0001707
31	P2	EL1	LinStatic	-18.885	0.0062	95.678	0.0953	0	0.0001161
32	P2	EL2	LinStatic	0.251	-45.597	-5.425	259.1744	0	-0.0005969
33	P2	CR1	LinStatic	4.892	-0.054	-24.413	0.4486	0	0.00000749
34	P2	CR2	LinStatic	-6.019	0.003564	43.775	0.045	0	0.00001829
35	P2	FL1	LinStatic	-7.292	-0.001562	101.217	0.0874	0	0.000000921
36	P2	FL2	LinStatic	-8.073	0.009305	61.738	0.017	0	0.00004952
37	P2	FL3	LinStatic	0.782	-0.011	39.479	0.0705	0	-0.0000486
38	P3	DL	LinStatic	12.685	-0.007224	236.806	0.206	0	-0.00001481
39	P3	LL	LinStatic	-0.051	-0.003801	5.423	0.0475	0	-0.00001405
40	P3	WL1	LinStatic	-26.321	72.072	-155.079	-345.5814	0	-0.0013
41	P3	WL2	LinStatic	-25.027	38.573	-144.142	-184.9689	0	-0.0006286

Joint Reactions — Program Control

Figure 4 107 Data of element joint forces after filter and sort

	A	B	C	D	E	F	G	H	I	J
1	TABLE: Joint Reactions									
2	Joint	)utputCas(	CaseType	StepType	F1	F2	F3	M1	M2	M3
3	Text	Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
4	P1	DL	LinStatic		13.56	2.351	145.032	0	0.615	0.0004886
5	P1	LL	LinStatic		-0.016	0.308	5.022	0	-0.0543	0.000203
6	P1	WL1	LinStatic		-34.426	21.371	-115.853	0	-26.1868	-0.0077
7	P1	WL2	LinStatic		-36.311	11.487	-128.174	0	-28.7126	-0.0025
8	P1	WL3	LinStatic		-48.534	22.579	-156.256	0	-36.5107	-0.0067
9	P1	WL4	LinStatic		-38.564	12.268	-128.847	0	-30.3766	-0.0016
10	P1	WL5	LinStatic		11.909	-25.65	-77.189	0	13.1126	0.0035
11	P1	WL6	LinStatic		43.611	21.896	217.576	0	33.9604	-0.0141
12	P1	WL7	LinStatic		36.851	12.104	193.237	0	27.2765	-0.0086
13	P1	WL8	LinStatic		12.394	0.399	2.517	0	13.1879	0.0017
14	P1	EL1	LinStatic		-21.793	-0.258	-99.633	0	-15.5845	0.0015
15	P1	EL2	LinStatic		-0.553	-23.909	-89.724	0	-0.0973	-0.1776
16	P1	CR1	LinStatic		7.419	-1.959	130.355	0	4.925	-0.0291
17	P1	CR2	LinStatic		-6.089	-0.002696	-7.305	0	-4.1668	0.0003708
18	P1	FL1	LinStatic		7.622	0.864	63.127	0	0.3299	0.0002197
19	P1	FL2	LinStatic		5.923	0.837	48.585	0	-0.6089	0.0004404
20	P1	FL3	LinStatic		1.698	0.027	14.541	0	0.9387	-0.0002208
21	P2	DL	LinStatic		-13.02	-0.002652	240.528	0.1646	0	-6.594E-06
22	P2	LL	LinStatic		-0.217	0.001389	9.284	0.0007146	0	-6.527E-06
23	P2	WL1	LinStatic		-25.872	71.896	104.692	-340.8842	0	0.0008783
24	P2	WL2	LinStatic		-25.105	38.505	111.136	-182.5022	0	0.0005207
25	P2	WL3	LinStatic		-38.691	71.907	188.878	-340.9069	0	0.0008838
26	P2	WL4	LinStatic		-27.666	38.511	151.135	-182.4868	0	0.0005309
27	P2	WL5	LinStatic		2.47	-65.006	-41.431	326.4516	0	-0.0006454
28	P2	WL6	LinStatic		31.745	71.869	-203.171	-341.1807	0	0.0004952
29	P2	WL7	LinStatic		29.694	38.479	-184.574	-182.7928	0	0.0001412
30	P2	WL8	LinStatic		2.31	0.571	-39.448	-3.3364	0	0.0001707
31	P2	EL1	LinStatic		-18.885	0.0062	95.678	0.0953	0	0.0001161
32	P2	EL2	LinStatic		0.251	-45.597	-5.425	259.1744	0	-0.0005969
33	P2	CR1	LinStatic		4.892	-0.054	-24.413	0.4486	0	0.00000749
34	P2	CR2	LinStatic		-6.019	0.003564	43.775	0.045	0	0.00001829
35	P2	FL1	LinStatic		-7.292	-0.001562	101.217	0.0874	0	9.21E-07
36	P2	FL2	LinStatic		-8.073	0.009305	61.738	0.017	0	0.00004952
37	P2	FL3	LinStatic		0.782	-0.011	39.479	0.0705	0	-0.0000486
38	P3	DL	LinStatic		12.685	-0.007224	236.806	0.206	0	-0.00001481
39	P3	LL	LinStatic		-0.051	-0.003801	5.423	0.0475	0	-0.00001405
40	P3	WL1	LinStatic		-26.321	72.072	-155.079	-345.5814	0	-0.0013
41	P3	WL2	LinStatic		-25.027	38.573	-144.142	-184.9689	0	-0.0006286

DESIGN SHEET OF BMB | InputS | SteelColumn

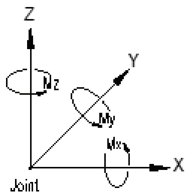
Figure 4 108 Data after paste

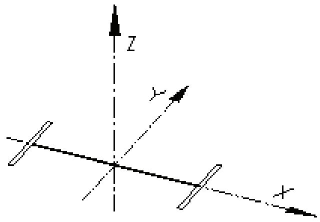
Select area from cell "W11" to cell "Z11" and click the Home > Insert command to insert more cells above active area. Revise load case name as shown in figure below.

W	X	Y	Z
DL	Dead Load		
AL	Collateral Load		
LL	Live Load		
WL1	Left Windload Case1		
WL2	Left Windload Case2		
WL3	Left Windload Case3		
WL4	Left Windload Case4		
WL5	Left Windload-Y		
WL6	Right Windload Case 1		
WL7	Right Windload Case 2		
WL8	Right Windload-Y		

Move to sheet "**SteelColumn**" and click the HIDE button.

1. Sign Convention





2. Reactions

Joint	OutputCase	Horizontal Reaction Hx	Horizontal Reaction Hy	Vertical Reaction Vz	Moment Mx	Moment My	Moment Mz
Text	*Text*	*KN*	*KN*	*KN*	*KN-m*	*KN-m*	*KN-m*
Point-P1	Dead Load	0.0	0.0	4.1	0.0	0.0	0.0
Point-P1	Live Load	0.1	0.0	3.5	0.0	0.0	0.0
Point-P1	Left Windload Case1	-2.3	0.0	-5.0	0.0	0.0	0.0
Point-P1	Left Windload Case2	-3.9	0.0	-4.6	0.0	0.0	0.0
Point-P1	Wind load Y	0.0	-2.6	-9.2	0.0	0.0	0.0
Point-P2	Dead Load	0.0	0.0	5.4	0.0	0.0	0.0
Point-P2	Live Load	0.0	0.0	5.9	0.0	0.0	0.0
Point-P2	Left Windload Case1	0.0	0.0	-9.0	0.0	0.0	0.0
Point-P2	Left Windload Case2	0.0	0.0	-6.4	0.0	0.0	0.0
Point-P2	Wind load Y	0.0	-4.2	1.6	0.0	0.0	0.0
Point-P3	Dead Load	0.0	0.0	5.8	0.0	0.0	0.0
Point-P3	Live Load	0.0	0.0	6.4	0.0	0.0	0.0
Point-P3	Left Windload Case1	0.0	0.0	-8.8	0.0	0.0	0.0
Point-P3	Left Windload Case2	0.0	0.0	-6.1	0.0	0.0	0.0
Point-P3	Wind load Y	0.0	-4.6	0.0	0.0	0.0	0.0
Point-P4	Dead Load	0.0	0.0	5.5	0.0	0.0	0.0
Point-P4	Live Load	0.0	0.0	5.4	0.0	0.0	0.0
Point-P4	Left Windload Case1	0.0	0.0	-7.8	0.0	0.0	0.0
Point-P4	Left Windload Case2	0.0	0.0	-6.1	0.0	0.0	0.0
Point-P4	Wind load Y	0.0	-5.0	-0.8	0.0	0.0	0.0
Point-P5	Dead Load	0.0	0.0	6.1	0.0	0.0	0.0
Point-P5	Live Load	0.0	0.0	6.1	0.0	0.0	0.0
Point-P5	Left Windload Case1	0.0	0.0	-8.8	0.0	0.0	0.0
Point-P5	Left Windload Case2	0.0	0.0	-6.0	0.0	0.0	0.0
Point-P5	Wind load Y	0.0	-5.3	0.3	0.0	0.0	0.0
Point-P6	Dead Load	0.0	0.0	4.3	0.0	0.0	0.0
Point-P6	Live Load	0.0	0.0	4.3	0.0	0.0	0.0
Point-P6	Left Windload Case1	0.1	0.0	-3.5	0.0	0.0	0.0
Point-P6	Left Windload Case2	0.1	0.0	-2.8	0.0	0.0	0.0
Point-P6	Wind load Y	0.0	-3.3	-9.4	0.0	0.0	0.0
Point-P7	Dead Load	0.0	0.0	6.1	0.0	0.0	0.0
Point-P7	Live Load	0.0	0.0	6.1	0.0	0.0	0.0
Point-P7	Left Windload Case1	0.0	0.0	-5.6	0.0	0.0	0.0

HIDE

DESIGN SHEET OF BMB | InputS | SteelColumn

Figure 4 109 Joint reactions after hide rows

Select area from cell "A1" to cell "I795" and select the Page Layout > Print Area > Set Print Area command. Recheck all information; make sure that Vertical Reactions Vz for Dead load and Live load are always positive.

4.2. Pinned Base Plate Design

4.2.1. Input

4.2.1.1 Loading

Run the model and specify all joints that connecting as pin to foundation and have the same column section. Select the **Display > Show Tables** command to show **Choose Tables for Display** form. Make sure that the **Table: Joint Reactions** item and **all combinations** in the **Select Load Cases** form are selected before click the OK button.

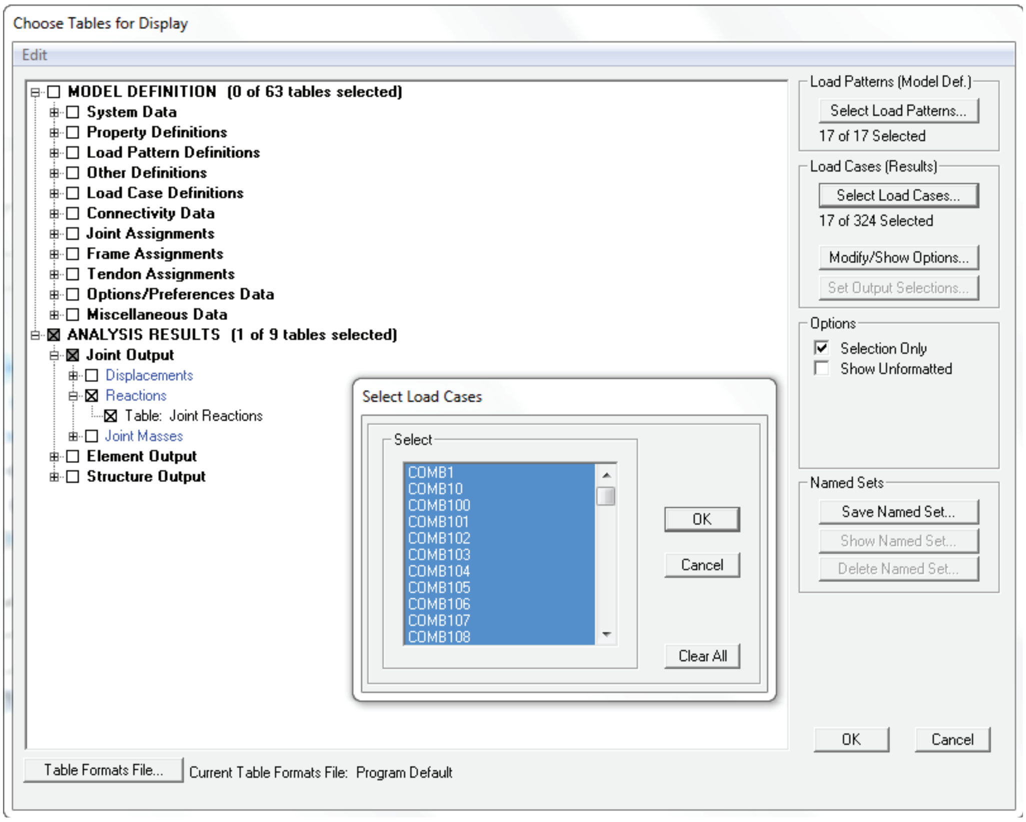
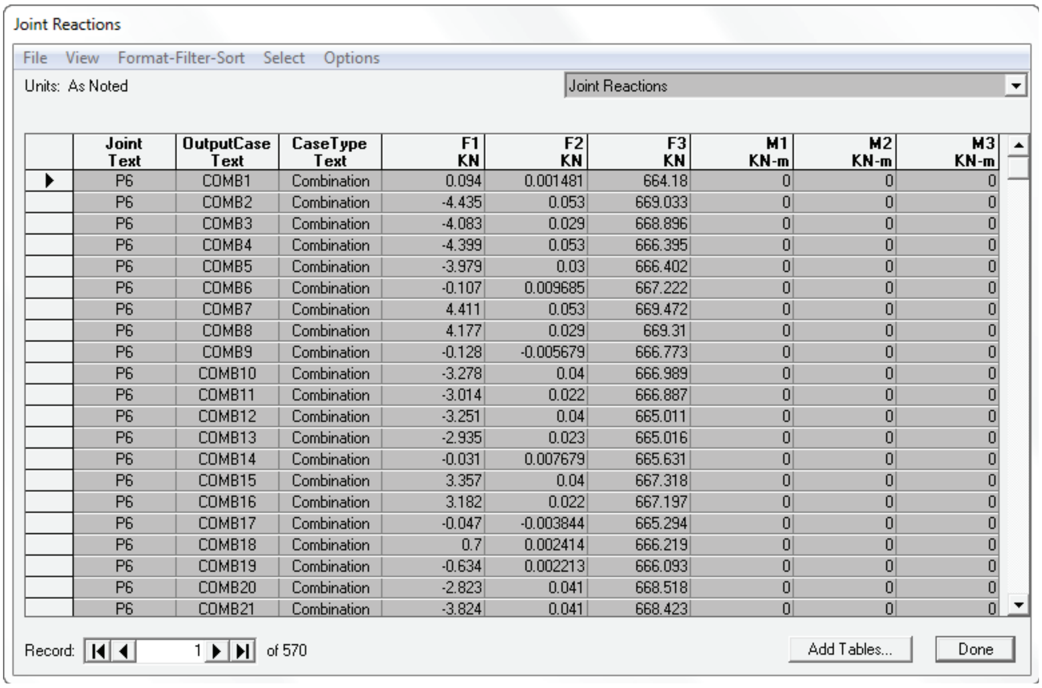


Figure 4 110 Choose Tables for Display form

In Joint Reactions tables, select **File > Export Current Table > To Excel.**



Joint Text	OutputCase Text	CaseType Text	F1 KN	F2 KN	F3 KN	M1 KN-m	M2 KN-m	M3 KN-m
P6	COMB1	Combination	0.094	0.001481	664.18	0	0	0
P6	COMB2	Combination	-4.435	0.053	669.033	0	0	0
P6	COMB3	Combination	-4.083	0.029	668.896	0	0	0
P6	COMB4	Combination	-4.399	0.053	666.395	0	0	0
P6	COMB5	Combination	-3.979	0.03	666.402	0	0	0
P6	COMB6	Combination	-0.107	0.009685	667.222	0	0	0
P6	COMB7	Combination	4.411	0.053	669.472	0	0	0
P6	COMB8	Combination	4.177	0.029	669.31	0	0	0
P6	COMB9	Combination	-0.128	-0.005679	666.773	0	0	0
P6	COMB10	Combination	-3.278	0.04	666.989	0	0	0
P6	COMB11	Combination	-3.014	0.022	666.887	0	0	0
P6	COMB12	Combination	-3.251	0.04	665.011	0	0	0
P6	COMB13	Combination	-2.935	0.023	665.016	0	0	0
P6	COMB14	Combination	-0.031	0.007679	665.631	0	0	0
P6	COMB15	Combination	3.357	0.04	667.318	0	0	0
P6	COMB16	Combination	3.182	0.022	667.197	0	0	0
P6	COMB17	Combination	-0.047	-0.003844	665.294	0	0	0
P6	COMB18	Combination	0.7	0.002414	666.219	0	0	0
P6	COMB19	Combination	-0.634	0.002213	666.093	0	0	0
P6	COMB20	Combination	-2.823	0.041	668.518	0	0	0
P6	COMB21	Combination	-3.824	0.041	668.423	0	0	0

Figure 4 111 Joint Reactions table

In excel file just exported, filter largest value in “F3” column and enter this value and correspondence shear to Pmax and Vp in “Base Plate Calculation” file.

NOTE

All value in Joint Reactions file are counter sign with value in "Base Plate Calculation" file, so we have to change the sign when enter axial force into calculation file. Shear forces will be taken as absolute values in all cases.

Do as above manner, filter the smallest value in "F3" column and enter this value and correspondence shear to Tmax and VT, filter largest absolute value in "F1" and "F2" column and take them to V_{max} and T_V.

1. LOADS

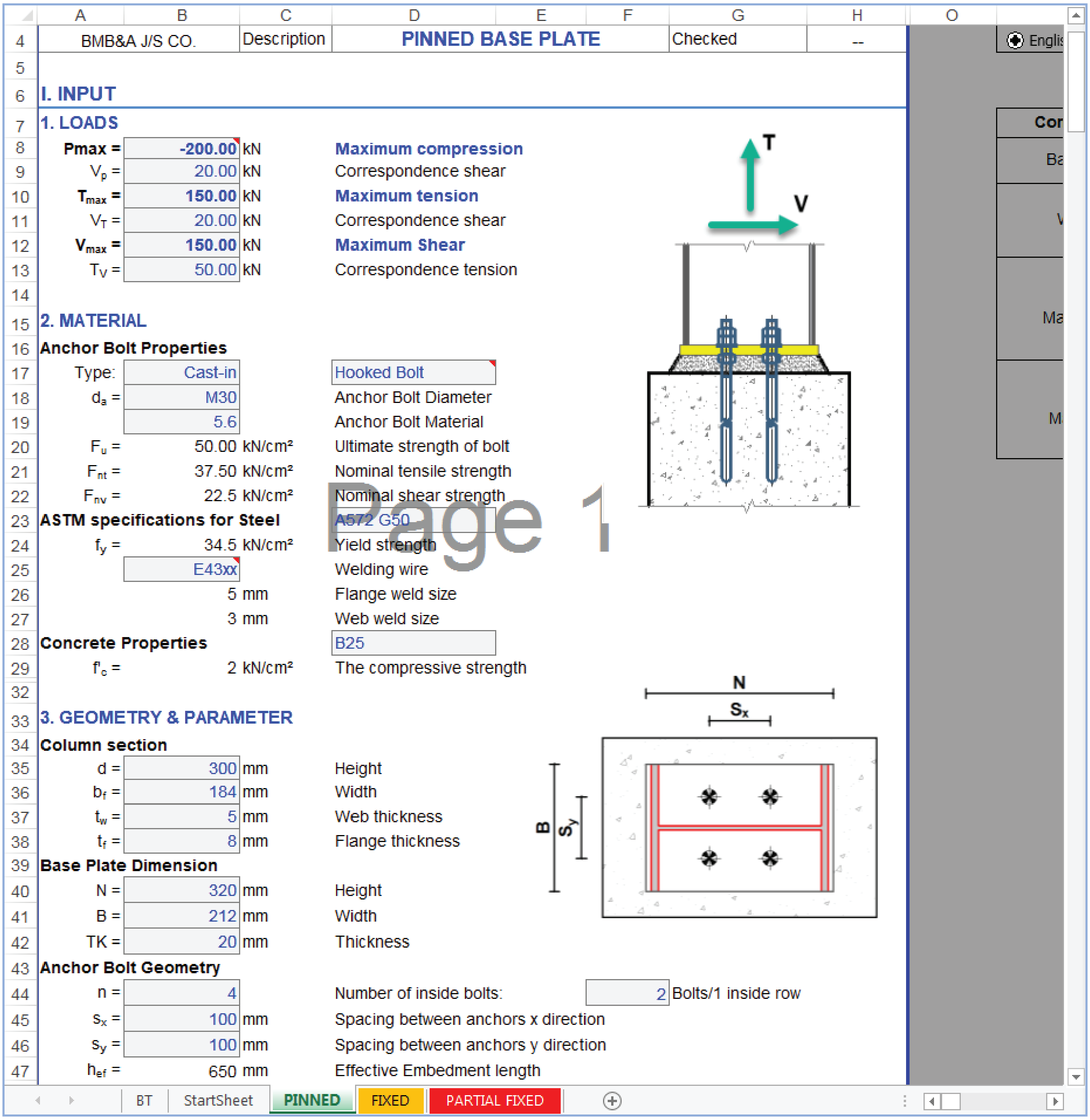
Pmax =	-992.10	kN	**Maximum compression**
V_p =	0.05	kN	Correspondence shear
T_{max} =	1.00	kN	**Maximum tension**
V_T =	0.16	kN	Correspondence shear
V_{max} =	7.10	kN	**Maximum Shear**
T_V =	1.00	kN	Correspondence tension

NOTE

Compressive force will have negative sign "-", and tensile force will have positive sign "+" or no sign. If there is not tensile force at base, value for it will be taken as 1.

4.2.1.2 Material, geometry parameter

In order to illustrate a pinned base plate connection with full of dangerous cases, we assume information as figure below.



4	BMB&A J/S CO.	Description		PINNED BASE PLATE — Checked — --
5				
6	I. INPUT			
7	1. LOADS			
8	Pmax =	-200.00	kN	Maximum compression
9	V_p =	20.00	kN	Correspondence shear
10	T_{max} =	150.00	kN	Maximum tension
11	V_T =	20.00	kN	Correspondence shear
12	V_{max} =	150.00	kN	Maximum Shear
13	T_V =	50.00	kN	Correspondence tension
14				
15	2. MATERIAL			
16	Anchor Bolt Properties			
17	Type:	Cast-in		Hooked Bolt
18	d_a =	M30		Anchor Bolt Diameter
19		5.6		Anchor Bolt Material
20	F_u =	50.00	kN/cm²	Ultimate strength of bolt
21	F_{nt} =	37.50	kN/cm²	Nominal tensile strength
22	F_{nv} =	22.5	kN/cm²	Nominal shear strength
23	ASTM specifications for Steel			A572 G50
24	f_y =	34.5	kN/cm²	Yield strength
25		E43xx		Welding wire
26		5	mm	Flange weld size
27		3	mm	Web weld size
28	Concrete Properties			B25
29	f'_c =	2	kN/cm²	The compressive strength
32				
33	3. GEOMETRY & PARAMETER			
34	Column section			
35	d =	300	mm	Height
36	b_f =	184	mm	Width
37	t_w =	5	mm	Web thickness
38	t_f =	8	mm	Flange thickness
39	Base Plate Dimension			
40	N =	320	mm	Height
41	B =	212	mm	Width
42	TK =	20	mm	Thickness
43	Anchor Bolt Geometry			
44	n =	4		Number of inside bolts: 2 Bolts/1 inside row
45	s_x =	100	mm	Spacing between anchors x direction
46	s_y =	100	mm	Spacing between anchors y direction
47	h_{ef} =	650	mm	Effective Embedment length

4.2.2. Checking base plate

4.2.2.1 Base plate for concentric axial compressive load

Calculate base plate area	$A_1 = NB = 320x212 = 67840mm^2$	678.4 cm²
Assume spacing between outermost anchor bolt and concrete edge is 200 mm		
Length of concrete foundation	$N_c = \left(\frac{n}{2}-1\right)s_x + 2*200 = \left(\frac{4}{2}-1\right)100 + 2*200$	500 mm
Width of concrete foundation	$B_c = s_y + 2x200 = 100 + 2x200$	500 mm

Calculate effective concrete area	$$A_2 = \min\left(N_c \frac{B}{N} N_c, N_c B_c\right)$$ $$A_2 = \min\left(500x\frac{212}{320}x500, 500x500\right)$$	1656.25 cm²
Maxlimum bearing stress between plate and concrete	$$f_{p(\max)} = \frac{1}{\Omega} 0.85 f_c' \sqrt{\frac{A_2}{A_1}}$$ $$f_{p(\max)} = \frac{1}{2.31} 0.85x20000\sqrt{\frac{1656.25}{678.4}}$$	11498.92 kN/m²
Allowable bearing force	$$\frac{P_p}{\Omega} = f_{p(\max)} A_1 = 11498.92x678.4x10^{-4}$$	780.1 kN
Critical base plate cantilever dimension, l, is the larger of m, n and λn':	$$m = \frac{N - 0.95d}{2} = \frac{320 - 0.95x300}{2}$$	17.5 mm
	$$n = \frac{B - 0.8b_f}{2} = \frac{212 - 0.8x184}{2}$$	32.4 mm
	$$X = \frac{4db_f}{\left(d + b_f\right)^2} \frac{\Omega P_{\max}}{P_p} = \frac{4x300x184}{(300+184)^2} \frac{200}{780.1}$$	0.24
	$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} = \frac{2x\sqrt{0.24}}{1 + \sqrt{1 - 0.24}}$$	0.53
	$$\lambda n' = \frac{\lambda\sqrt{db_f}}{4} = 0.53\frac{\sqrt{300x184}}{4}$$	30.87 mm
	$$l = \max(m, n, \lambda n') = \max(17.5, 32.4, 30.87)$$	32.4 mm
For the yielding limit state, the required minimum thickness of the base plate	$$\frac{P_{\max}}{BN}\frac{l^2}{2} = \frac{1}{\Omega} F_y \frac{t_p^2}{4} \Rightarrow t_p = l\sqrt{\frac{2\Omega P_{\max}}{F_y BN}}$$ $$t_p = 32.4x\sqrt{\frac{2x1.67x200}{34.5x212x320}}$$	5.47 mm

4.2.2.2 Base plate for uplift load

Tension for each anchor bolt	$$\frac{T_{\max}}{n} = \frac{150}{4}$$	37.5 mm
Distance from bolt rows to column web	$$a = \frac{s_y - t_w}{2} = \frac{100 - 5}{2}$$	47.5 mm
Moment caused by tension on bolt	$$M_a = \frac{T_{\max}}{n} a = 37.5x47.5$$	1781.25 kNmm
Width of effective area to assist	$$b_{eff} = 2a = 2x47.5$$	95 mm

bending moment		
Required minimum thickness of base plate	$\frac{1}{\Omega}F_y\frac{b_{eff}t_p^2}{4}=M_a \Rightarrow t_p=\sqrt{\frac{4\Omega M_a}{F_y b_{eff}}}=\sqrt{\frac{4\times1.67\times1781.25}{0.345\times95}}$	19.05 mm
Demand/capacity thickness of base plate	$\frac{\max(5.47,19.05)}{20}=0.953<1$	OK

4.2.2.3 Design welding

Weld capacity due to anchor tension

Load angle factor when force perpendicular with welding	$\chi=1+0.5\sin^{1.5}(\theta)=1+0.5\sin^{1.5}(90)$	1.5
Width of effective area resisting tension force of one bolt	$b_{eff}=2a=2x47.5$	95 mm
Allowable bearing capacity of column web weld	$\frac{R_w}{\Omega}=\frac{1}{\Omega}\chi0.6F_{EXX}\frac{\sqrt{2}}{2}b_{f\_web}=\frac{1}{2}\times1.5\times0.6\times43\times\frac{\sqrt{2}}{2}\times0.3$	4.10 kN/cm
Force apply on column web weld	$\frac{T_{\max}}{nb_{eff}}=\frac{150}{4x9.5}$	3.95 kN/cm
Demand/capacity welding	$\frac{3.96}{4.1}=0.962<1$	OK

Elastic method weld shear capacity

Load angle factor when force parallel with welding	$\chi=1+0.5\sin^{1.5}(\theta)=1+0.5\sin^{1.5}(0)$	1.0
Total web weld length	$L_{shear}=2d=2x300=600$	600 mm
Allowable bearing capacity of column web weld	$\frac{R_w}{\Omega}=\frac{1}{\Omega}\chi0.6F_{EXX}\frac{\sqrt{2}}{2}b_{f\_web}=\frac{1}{2}\times1.0\times0.6\times43\times\frac{\sqrt{2}}{2}\times0.3$	2.74 kN/cm
Force apply on column web weld	$\frac{V_{\max}}{L_{shear}}=\frac{150}{60}$	2.5 kN/cm
Demand/capacity welding	$\frac{2.5}{2.74}=0.914<1$	OK

Elastic method weld axial capacity

Load angle factor when force perpendicular with welding	$\chi=1+0.5\sin^{1.5}(\theta)=1+0.5\sin^{1.5}(90)$	1.5
Total flange weld length	$L=4b_f=4x184$	736 mm
Allowable bearing capacity of column web weld	$\frac{R_w}{\Omega}=\frac{1}{\Omega}\chi0.6F_{EXX}\frac{\sqrt{2}}{2}b_{f\_flange}=\frac{1}{2}\times1.5\times0.6\times43\times\frac{\sqrt{2}}{2}\times0.5$	6.84 kN/cm

Force apply on column flange weld	$\frac{T_{max}}{L} = \frac{150}{73.6}$	2.04 kN/cm
Demand/capacity welding	$\frac{2.04}{6.84} = 0.298 < 1$	OK

4.2.3. Anchor bolt checking

4.2.3.1 Checking for combination maximum tension

Steel strength of a single anchor in tension

Allowable tensile capacity of each bolt	$\frac{R_n}{\Omega} = \frac{1}{\Omega} F_{nt} \frac{\pi d_b^2}{4} = \frac{1}{2} 37.5 \times \frac{\pi 3^2}{4}$	132.54 kN
Required tension force for each bolt	$\frac{T}{n} = \frac{150}{4}$	37.5 kN
Demand/Capacity	$\frac{37.5}{132.54} = 0.283 < 1$	OK

Pullout of anchor in tension

The pullout strength in tension of a single hooked bolt	$\frac{N_{pn}}{\Omega} = \psi_{c,P} \frac{0.9 f'_c e_b d_a}{\Omega} = 1.4 \frac{0.9x2x4.5x3x3}{2.14}$	47.69 kN
The pullout strength in tension of a single hooked bolt: Assume there is no cracking in concrete so modification factor for pullout shall be taken as 1.4 Effective dimension to compute pullout strength of hooked bolt eh shall be taken as $4.5d_a$		
Demand/Capacity	$\frac{37.5}{47.69} = 0.786 < 1$	OK

Steel strength of a single anchor in tension

Allowable shear capacity of each bolt	$\frac{V_{sa}}{\Omega} = \frac{0.8F_{nv}\pi d_b^2 / 4}{\Omega} = \frac{0.8 \times 22.5\pi \times 3^2 / 4}{2}$	63.62 kN
Demand/Capacity	$\frac{20}{4x63.62} = 0.079 < 1$	OK

Interaction of tensile and shear forces of anchor

Demand/Capacity	$\frac{0.786 + 0.079}{1.2} = 0.721 < 1$	OK

4.2.3.2 Checking for combination maximum shear

Steel strength of a single anchor in tension

Allowable tensile capacity of each bolt	$\frac{R_n}{\Omega} = \frac{1}{\Omega} F_{nt} \frac{\pi d_b^2}{4} = \frac{1}{2} 37.5 \times \frac{\pi 3^2}{4}$	132.54 kN
Required tension force for each bolt	$\frac{T}{n} = \frac{50}{4}$	12.5 kN
Demand/Capacity	$\frac{12.5}{132.54} = 0.094 < 1$	OK

Pullout of anchor in tension

The pullout strength in tension of a single hooked bolt	$\frac{N_{pn}}{\Omega} = \psi_{c,P} \frac{0.9 f_c^{'} e_b d_a}{\Omega} = 1.4 \frac{0.9 \times 2 \times 4.5 \times 3 \times 3}{2.14}$	47.69 kN
Assume there is no cracking in concrete so modification factor for pullout shall be taken as 1.4 Effective dimension to compute pullout strength of hooked bolt eh shall be taken as $4.5d_a$		
Demand/Capacity	$\frac{12.5}{47.69} = 0.262 < 1$	OK

Steel strength of anchors in shear

Allowable shear capacity of each bolt	$\frac{V_{sa}}{\Omega} = \frac{0.8 F_{nv} \pi d_b^2 / 4}{\Omega} = \frac{0.8 x 22.5 x \pi x 3^2 / 4}{2}$	63.62 kN
Demand/Capacity	$\frac{150}{4x63.62} = 0.589 < 1$	OK

Interaction of tensile and shear forces of anchor

Demand/Capacity	$\frac{0.262 + 0.589}{1.2} = 0.710 < 1$	OK

4.3. Fixed Base Plate Design

4.3.1. Loading

Run the model and specify all joints that connecting as fixed to foundation and have the same column section. Select the **Display > Show Tables** command to show **Choose Tables for Display** form. Make sure that the Table: Joint Reactions item and **all combinations** in the **Select Load Cases** form are selected before click the OK button.

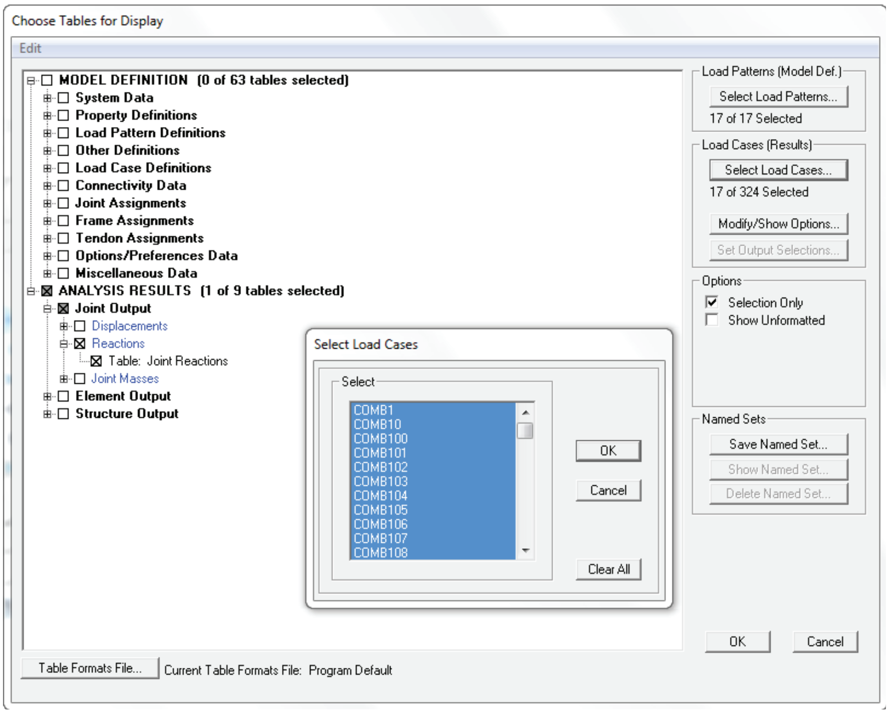
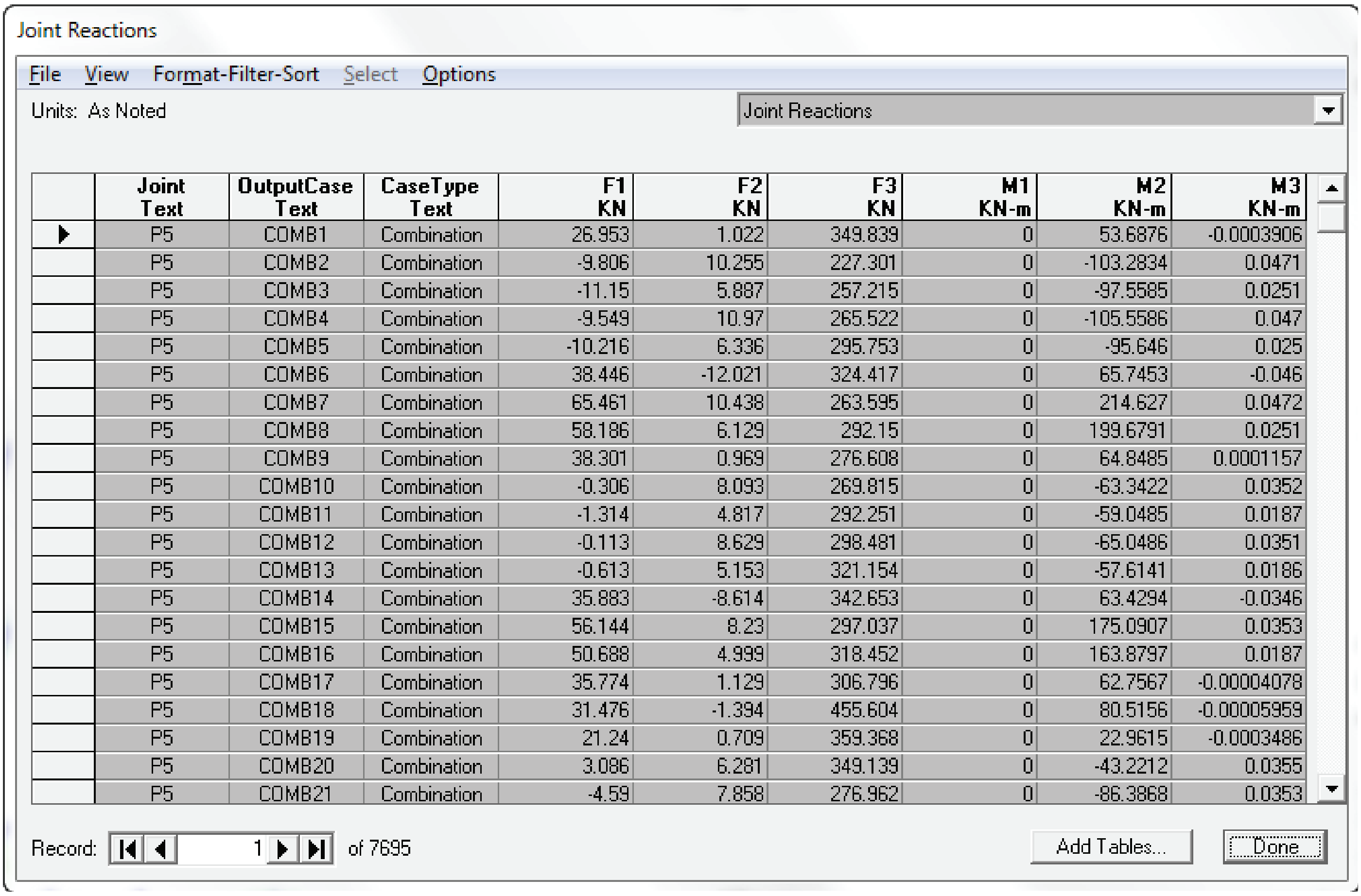


Figure 4 113 Choose Tables for Display form

In Joint Reactions tables, select **File > Export Current Table > To Excel**.



Joint Reactions

File View Format-Filter-Sort Select Options

Units: As Noted — Joint Reactions

Joint Text	OutputCase Text	CaseType Text	F1 KN	F2 KN	F3 KN	M1 KN-m	M2 KN-m	M3 KN-m
P5	COMB1	Combination	26.953	1.022	349.839	0	53.6876	-0.0003906
P5	COMB2	Combination	-9.806	10.255	227.301	0	-103.2834	0.0471
P5	COMB3	Combination	-11.15	5.887	257.215	0	-97.5585	0.0251
P5	COMB4	Combination	-9.549	10.97	265.522	0	-105.5586	0.047
P5	COMB5	Combination	-10.216	6.336	295.753	0	-95.646	0.025
P5	COMB6	Combination	38.446	-12.021	324.417	0	65.7453	-0.046
P5	COMB7	Combination	65.461	10.438	263.595	0	214.627	0.0472
P5	COMB8	Combination	58.186	6.129	292.15	0	199.6791	0.0251
P5	COMB9	Combination	38.301	0.969	276.608	0	64.8485	0.0001157
P5	COMB10	Combination	-0.306	8.093	269.815	0	-63.3422	0.0352
P5	COMB11	Combination	-1.314	4.817	292.251	0	-59.0485	0.0187
P5	COMB12	Combination	-0.113	8.629	298.481	0	-65.0486	0.0351
P5	COMB13	Combination	-0.613	5.153	321.154	0	-57.6141	0.0186
P5	COMB14	Combination	35.883	-8.614	342.653	0	63.4294	-0.0346
P5	COMB15	Combination	56.144	8.23	297.037	0	175.0907	0.0353
P5	COMB16	Combination	50.688	4.999	318.452	0	163.8797	0.0187
P5	COMB17	Combination	35.774	1.129	306.796	0	62.7567	-0.00004078
P5	COMB18	Combination	31.476	-1.394	455.604	0	80.5156	-0.00005959
P5	COMB19	Combination	21.24	0.709	359.368	0	22.9615	-0.0003486
P5	COMB20	Combination	3.086	6.281	349.139	0	-43.2212	0.0355
P5	COMB21	Combination	-4.59	7.858	276.962	0	-86.3868	0.0353

Record: 1 of 7695 — Add Tables... — Done

Figure 4 114 Joint Reactions table

In excel file just exported, filter largest value in “F3” column and enter this value, correspondence shear and moment M2 to P_{max}, V_P and M_P in “Base Plate Calculation” file.

NOTE

All values in Joint Reactions file are countered sign with value in "Base Plate Calculation" file, so we have to change the sign when enter axial force into calculation file. Shear forces and moment will be taken as absolute values in all cases.

Do as above manner, filter the smallest value in "F3" column and enter this value and correspondence shear and moment M2 to T_{max}, V_T and M_T

Filter all positive values of F3, and then filter the largest absolute value of M2. Enter this value, correspondence compressive force and shear to $M1_{max}$, P_{M1} and V_{M1}.

Filter all negative values of F3, and then filter the largest absolute value of M2. Enter this value, correspondence tensile force and shear to $M2_{max}$, P_{M2} and V_{M2}.

Filter all negative values of F3, and then filter the largest absolute value of F1. Enter this value, correspondence tensile force and shear to V_{max}, T_V and M_V.

1. LOADS

Pmax =	**-601.60**	kN	**Maximum compression**
V_p =	37.20	kN	Correspondence shear
M_P =	79.70	kN.m	Correspondence moment
T_{max} =	**71.10**	kN	**Maximum tension**
V_T =	21.90	kN	Correspondence shear
M_T =	19.30	kN.m	Correspondence moment
$M1_{max}$ =	**604.90**	kN.m	**Maximum Moment**
P_{M1} =	-2.60	kN	Correspondence compression
V_{M1} =	70.20	kN	Correspondence shear
$M2_{max}$ =	**616.90**	kN.m	**Maximum Moment**
T_{M2} =	24.40	kN	Correspondence tension
V_{M2} =	71.70	kN	Correspondence shear
V_{max} =	**71.70**	kN	**Maximum Shear**
T_V =	24.40	kN	Correspondence tension
M_V =	616.90	kN.m	Correspondence moment

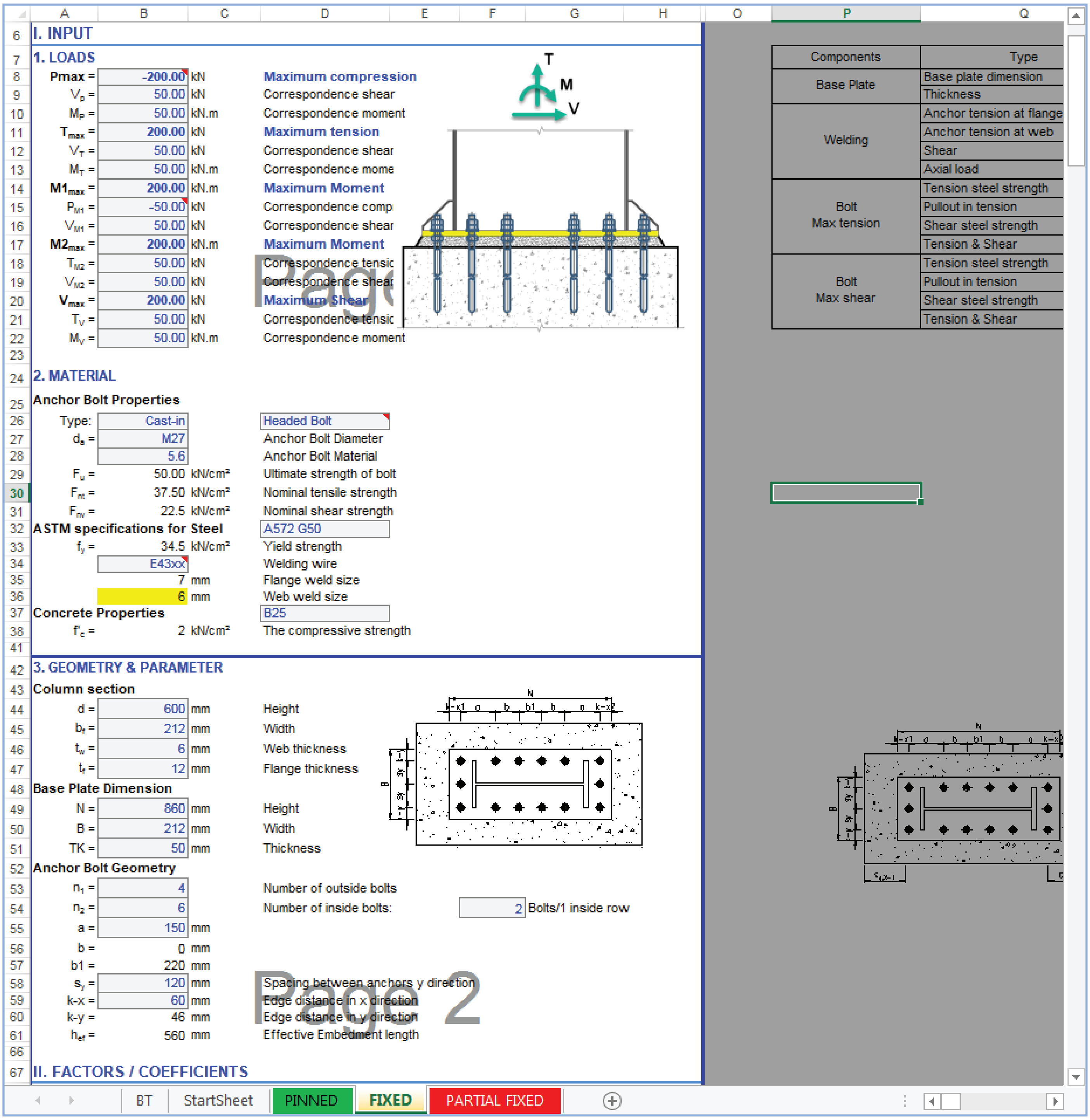
Figure 4 115 Internal force for calculation

NOTE

Compressive force will have negative sign "-", and tensile force will have positive sign "+" or no sign. If there is not tensile force at base, value for it will be taken as 1.

4.3.2. Material, geometry parameter

In order to illustrate a fixed base plate connection with full of dangerous cases, we assume information as figure below.



4.3.3. Checking base plate

Calculate base plate area	$A_1 = NB = 860x212$	1823.2 cm^2
Calculate concrete area: Assume spacing between outermost anchor bolt and concrete edge is 200 mm		
Length of concrete foundation	$N_c = N + (c_{a,x-1} - k_{x1}) + (c_{a,x-2} - k_{x2})$ $N_c = 860 + (200 - 60) + (200 - 60)$	1140 mm

Width of concrete foundation	$B_c = B + (c_{a,y-1} - k_{y1}) + (c_{a,y-2} - k_{y2})$ $B_c = 212 + (200-46) + (200-46)$	520 mm
Calculate effective concrete area	$A_2 = \min\left(N_c \dfrac{B}{N} N_c , N_c B_c \right)$ $A_2 = \min\left(1140 \times \dfrac{212}{860} \times 1140, 1140 \times 520 \right)$	320366.5 mm²
Maximum bearing stress between plate and concrete	$f_{p(\max)} = \dfrac{1}{\Omega} 0.85 f_c' \sqrt{\dfrac{A_2}{A_1}} = \dfrac{1}{2.31} 0.85x20000x\sqrt{\dfrac{3203.7}{1823.2}}$	9755.36 kN/m²
Maximum resultant bearing force	$q_{\max} = f_{p(\max)} B = 9755.36x0.212$	2068.14 kN/m
Critical base plate cantilever dimension is the larger of m, n:	$m = \dfrac{N - 0.95d}{2} = \dfrac{860 - 0.95x600}{2}$	145 mm
	$n = \dfrac{B - 0.8b_f}{2} = \dfrac{212 - 0.8x212}{2}$	21.2 mm

4.3.3.1 Checking base plate with combo Mp + Pmax

The equivalent eccentricity of base plate	$e = \dfrac{M_P}{P_{\max}} = \dfrac{50}{200} = 0.25\,\text{m}$	250 mm
The critical eccentricity	$e_{crit} = \dfrac{N}{2} - \dfrac{P}{2q_{\max}} = \dfrac{860}{2} - \dfrac{200}{2x2068.14}$	381.65 mm
Conclusion	$e < e_{crit}$ => Case of a base plate with small moment ⇨ No anchor rod forces exists.	
The concrete bearing length	$Y = N - 2e = 860 - 2x250$	360 mm
Bearing stress between base plate and concrete	$f_p = \dfrac{P}{BY} = \dfrac{200}{0.212x0.360}$	2620.55 kN/m²
Determine minimum plate thickness with m (Y>m)	$\dfrac{1}{\Omega} F_y \dfrac{Bt_p^2}{4} = f_p B \dfrac{m^2}{2}$ $\rightarrow t_p = m\sqrt{\dfrac{2\Omega f_p}{F_y}} = 145\sqrt{\dfrac{2x1.67x2620.55}{34.5x10^4}}$	23.13 mm

4.3.3.2 Checking base plate with combo T_{max} + M_T

It's always the case of base plate with large moment when considering combo momnent with tension.

$$f_p = f_{p,\max} = 9755.36\frac{kN}{m^2}$$

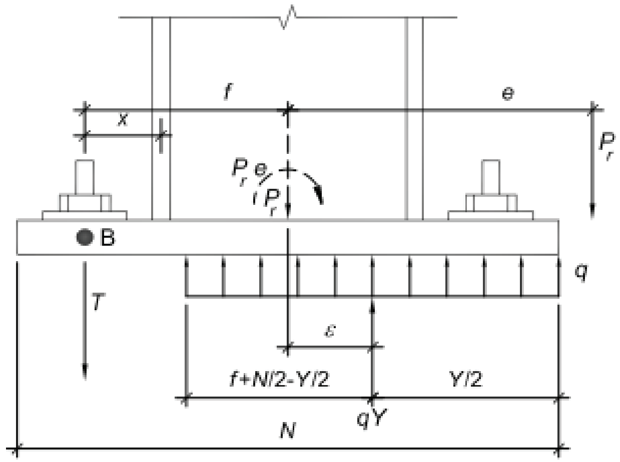


Figure 3.4.1. Base plate with large moment.

Determine bearing length Y:

- Equilibrium momentum with rotate center at the edge of compresion concrete zone:

$$\frac{P_1}{h_1 - Y}\left[(h_1-Y)^2+(h_2-Y)^2+(h_3-Y)^2+(h_4-Y)^2+(h_5-Y)^2\right]+q_{\max}\frac{Y^2}{2}=M+P\left(\frac{N}{2}-Y\right)$$

$$\Leftrightarrow \mathrm{R}\left(\sum_{i=1}^{5}h_i^2-2Y\sum_{i=1}^{5}h_i+5Y^2\right)=M+P\left(\frac{N}{2}-Y\right)-q_{\max}\frac{Y^2}{2}$$

$$\Leftrightarrow \mathrm{R}=\frac{M+P\left(\frac{N}{2}-Y\right)-q_{\max}\frac{Y^2}{2}}{\left(\sum_{i=1}^{5}h_i^2-2Y\sum_{i=1}^{5}h_i+5Y^2\right)}=\frac{50+200\left(\frac{0.86}{2}-Y\right)-2068.14\frac{Y^2}{2}}{12951x10^{-4}-2Yx2.15+5Y^2}=\frac{136-200Y-1034.07Y^2}{12951x10^{-4}-4.3Y+5Y^2}$$

With: $\sum_{i=1}^{5}h_i^2=800^2+650^2+430^2+210^2+60^2=1295100mm^2$

$$\sum_{i=1}^{5}h_i=800+650+430+210+60=2150mm$$

$$\mathrm{R}=\frac{P_1}{h_1-Y}$$

- Equilibrium force equation: $\frac{P_1}{h_1-Y}\left[(h_1-Y)+(h_2-Y)+(h_3-Y)+(h_4-Y)+(h_5-Y)\right]=P+q_{\max}Y$

$$\Leftrightarrow \mathrm{R}\left(\sum_{i=1}^{5}h_i-5Y\right)=P+q_{\max}Y$$

$$\Leftrightarrow \left(q_{\max}+5\mathrm{R}\right)Y=\mathrm{R}\sum_{i=1}^{5}h_i-P$$

$$\Leftrightarrow Y=\frac{\mathrm{R}\sum_{i=1}^{5}h_i-P}{\left(q_{\max}+5\mathrm{R}\right)}=\frac{2.15\mathrm{R}-200}{2068.14+5\mathrm{R}}$$

- Assume Y=0.001, set this value into Equation R: $\mathrm{R}=\frac{136-200x0.001-1034.07x0.001^2}{1295x10^{-4}+4.3x0.001+5x0.001^2}=105.2$

- Set R into Equation Y: $Y=\frac{2.15x105.2-200}{2068.14+5x105.2}=0.01$

$\Delta Y=\frac{0.01-0.001}{0.001}=9>0.05$ => Recalculate until this change not excess 0.05

- Set Y=0.01 into Equation R: $R = \dfrac{136 - 200x0.01 - 1034.07x0.01^2}{1295x10^{-4} - 4.3x0.01 + 5x0.01^2} = 106.89$

$$Y = \frac{2.15x106.89 - 200}{2068.14 + 5x106.89} = 0.01146$$

$\Delta Y = \dfrac{0.01146 - 0.01}{0.01} = 0.146 > 0.05$ => Recalculate until this change not excess 0.05

- Set Y=0.01146 into Equation R: $R = \dfrac{136 - 200x0.01146 - 1034.07x0.01146^2}{1295x10^{-4} - 4.3x0.01146 + 5x0.01146^2} = 107.16$

$$Y = \frac{2.15x107.16 - 200}{2068.14 + 5x107.16} = 0.01167$$

$$\Delta Y = \frac{0.01167 - 0.01146}{0.01146} = 0.018 < 0.05 \text{ => OK}$$

Determine maximum anchor tension: $R = \dfrac{P_1}{h_1 - Y} \Rightarrow P_1 = R(h_1 - Y) = 107.16(0.8 - 0.01167) = 84.5$ kN

Tension in one outermost bolt: $T_{\max} = \dfrac{P_1}{n_1 / 2} = \dfrac{84.5}{4/2} = 42.25$ kN

Determine minimum plate thickness with m (Y<m):

$$\frac{1}{\Omega} F_y \frac{Bt_p^2}{4} = f_p BY(m - \frac{Y}{2})$$

$$\rightarrow t_p = \sqrt{\frac{4\Omega f_p Y(m - \frac{Y}{2})}{F_y}} = \sqrt{\frac{4x1.67x9755.36x11.67x(145 - \frac{11.67}{2})}{34.5x10^4}} = 17.51 \text{ mm}$$

Determine minimum plate thickness follow tension in bolt: $\dfrac{1}{\Omega} F_y \dfrac{Bt_p^2}{4} = P_1 x$

$$\rightarrow t_p = \sqrt{\frac{4\Omega P_1 x}{BF_y}} = \sqrt{\frac{4x1.67x84.5x76}{212x34.5}} = 24.22 \text{ mm}$$

➡ Calculate similarly for combos $M1_{\max} \& P_{M1}, M2_{\max} \& T_{M2}, V_{\max} \& M_V$ we find that required minimum base plate thickness is 44.62mm which is still smaller than currently thickness of 50mm.

➡ Maximun tension in anchor bolts governed by combination $M2_{max}$ & T_{M2}

➡ Use these force to checking bolts:

Total axial force P = 50 kN

Total shear force V_g = 50 kN

Total moment M = 200 kNm

Maximum anchor tension Tmax = 82.41 kN

Shear for one bolt V = 5 kN

4.3.3.3 Design welding

Weld capacity due to anchor tension.

Load angle factor when force perpendicular with welding:

$$\chi = 1 + 0.5\sin^{1.5}(\theta) = 1 + 0.5\sin^{1.5}(90) = 1.5$$

Width of effective area resisting tension force of one bolt:

$$b_{eff} = 2a = 2x70 = 140 \text{ mm}$$

Allowable bearing capacity of column flange weld:

$$\frac{R_w}{\Omega} = \frac{1}{\Omega}\chi 0.6F_{EXX}\frac{\sqrt{2}}{2}b_{f\_flange} = \frac{1}{2}x1.5x0.6x43x\frac{\sqrt{2}}{2}x0.7 = 9.58\frac{kN}{cm}$$

Force apply on column flange weld:

$$\frac{T_{max}}{b_{eff}} = \frac{82.41}{14} = 5.89\frac{kN}{cm}$$

➡ Demand/capacity welding: $\frac{5.89}{9.58} = 0.615 < 1$

➡ OK

Elastic method weld shear capacity

Load angle factor when force parallel with welding:

$$\chi = 1 + 0.5\sin^{1.5}(\theta) = 1 + 0.5\sin^{1.5}(0) = 1.0$$

Total web weld length:

$$L_{shear} = 2d = 2x600 = 1200 \text{ mm}$$

Allowable bearing capacity of column web weld:

$$\frac{R_w}{\Omega} = \frac{1}{\Omega}\chi 0.6F_{EXX}\frac{\sqrt{2}}{2}b_{f\_web} = \frac{1}{2}x1.0x0.6x43x\frac{\sqrt{2}}{2}x0.6 = 5.47\frac{kN}{cm}$$

Force apply on column web weld:

$$\frac{V_{max}}{L_{shear}} = \frac{200}{120} = 1.67\frac{kN}{cm}$$

➡ Demand/capacity welding: $\frac{1.67}{5.47} = 0.305 < 1$

➡ OK

Elastic method weld axial capacity

Load angle factor when force perpendicular with welding:

$$\chi = 1 + 0.5\sin^{1.5}(\theta) = 1 + 0.5\sin^{1.5}(90) = 1.5$$

Total flange weld length

$$L = 4b_f = 4x212 = 848 \text{ mm}$$

Inertia moment of weld:

$$I = 4b_f\left(d/2\right)^2 + 2(d-2t_f)^3/12 = 4x212x(600/2)^2 + 2x(600-2x12)^3/12 = 108170.5cm^3$$

Allowable bearing capacity of column web weld:

$$\frac{R_w}{\Omega} = \frac{1}{\Omega}\chi 0.6F_{EXX}\frac{\sqrt{2}}{2}b_{f\_flange} = \frac{1}{2}x1.5x0.6x43x\frac{\sqrt{2}}{2}x0.7 = 9.58\frac{kN}{cm}$$

Force apply on column flange weld:

$$\max\left(\frac{T_{\max}}{L}+\frac{M_T(d/2)}{I},\frac{T_{M2}}{L}+\frac{M2_{\max}(d/2)}{I}\right) = \max\left(\frac{200}{84.8}+\frac{5000(60/2)}{108170.5},\frac{50}{84.8}+\frac{20000(600/2)}{109170.5}\right) = 6.14$$

➡ Demand/capacity welding: $\frac{6.14}{9.58} = 0.64 < 1$

➡ OK

4.3.4. Anchor bolt checking

Checking for combination maximum tension

Total axial force P = 50 kN

Total shear force V_g = 50 kN

Total moment M = 200 kNm

Maximum anchor tension T_{max} = 82.41 kN

Shear for one bolt V = 5 kN

4.3.4.1 Steel strength of a single anchor in tension

Allowable tensile capacity of each bolt:

$$\frac{R_n}{\Omega} = \frac{1}{\Omega}F_{nt}\frac{\pi d_b^2}{4} = \frac{1}{2}37.5x\frac{\pi 2.7^2}{4} = 107.35\text{ kN}$$

Demand/Capacity: $\frac{82.41}{107.35} = 0.77 < 1$

➡ OK

4.3.4.2 Pullout of anchor in tension

The pullout strength in tension of a single headed bolt:

$$\frac{N_{pn}}{\Omega} = \psi_{c,P}\frac{A_{brg}8f_c'}{\Omega} = 1.4\frac{8.83x8x2}{2.14} = 92.43\text{ kN}$$

With: $A_{brg} = \frac{\sqrt{3}}{2}F^2 - \frac{\pi d_b^2}{4} = \frac{\sqrt{3}}{2}4.1^2 - \frac{\pi x 2.7^2}{4} = 8.83cm^2$ (F is nominal dimension of bolt's head, take $F \approx 1.5d_b$)

Assume there is no cracking in concrete so modification factor for pullout shall be taken as 1.4

➡ Demand/Capacity = $\frac{82.41}{92.43} = 0.89 < 1$

➡ OK

4.3.4.3 Steel strength of anchors in shear

Allowable shear capacity of each bolt:

$$\frac{V_{sa}}{\Omega} = \frac{0.8F_{nv}\pi d_b^2/4}{\Omega} = \frac{0.8x22.5x\pi x2.7^2/4}{2} = 51.53\ \text{kN}$$

➡ Demand/Capacity = $\frac{5}{51.53} = 0.097 < 1$

➡ OK

4.3.4.4 Interaction of tensile and shear forces of anchor:

$$\frac{0.89+0.097}{1.2} = 0.823 < 1$$

➡ OK

Checking for combination maximum shear

Total axial force P = 50 kN
Total shear force V_g = 200 kN
Total moment M = 50 kNm
Maximum anchor tension T_{max} = 22.74 kN
Shear for one bolt V = 20 kN

4.3.4.5 Steel strength of a single anchor in tension

Allowable tensile capacity of each bolt:

$$\frac{R_n}{\Omega} = \frac{1}{\Omega}F_{nt}\frac{\pi d_b^2}{4} = \frac{1}{2}37.5x\frac{\pi 2.7^2}{4} = 107.35\ \text{kN}$$

Demand/Capacity: $\frac{22.74}{107.35} = 0.21 < 1$

➡ OK

4.3.4.6 Pullout of anchor in tension

The pullout strength in tension of a single headed bolt:

$$\frac{N_{pn}}{\Omega} = \psi_{c,P}\frac{A_{brg}8f_c'}{\Omega} = 1.4\frac{8.83x8x2}{2.14} = 92.43\ \text{kN}$$

With: $A_{brg} = \frac{\sqrt{3}}{2}F^2 - \frac{\pi d_b^2}{4} = \frac{\sqrt{3}}{2}4.1^2 - \frac{\pi x2.7^2}{4} = 8.83cm^2$ (F is nominal dimension of bolt's head, take $F \approx 1.5d_b$)

Assume there is no cracking in concrete so modification factor for pullout shall be taken as 1.4

➡ Demand/Capacity = $\frac{22.74}{92.43} = 0.25 < 1$

➡ OK

4.3.4.7 Steel strength of anchors in shear

Allowable shear capacity of each bolt:

$$\frac{V_{sa}}{\Omega} = \frac{0.8F_{mv}\pi d_b^2 / 4}{\Omega} = \frac{0.8x22.5x\pi x2.7^2 / 4}{2} = 51.53 \text{ kN}$$

➡ Demand/Capacity = $\frac{20}{51.53} = 0.39 < 1$

➡ OK

4.3.4.8 Interaction of tensile and shear forces of anchor:

$$\frac{0.25 + 0.39}{1.2} = 0.53 < 1 \ : \text{OK}$$

4.4. Horizontal Knee Connection Design

4.4.1. Input

4.4.1.1 Loading

Run the model and specify all rafter having section 1K. Select the **Display > Show Tables** command to show **Choose Tables for Display** form. Make sure that the **Table: Element Forces - Frames** item and **all combinations** in the **Select Load Cases** form are selected before click the OK button.

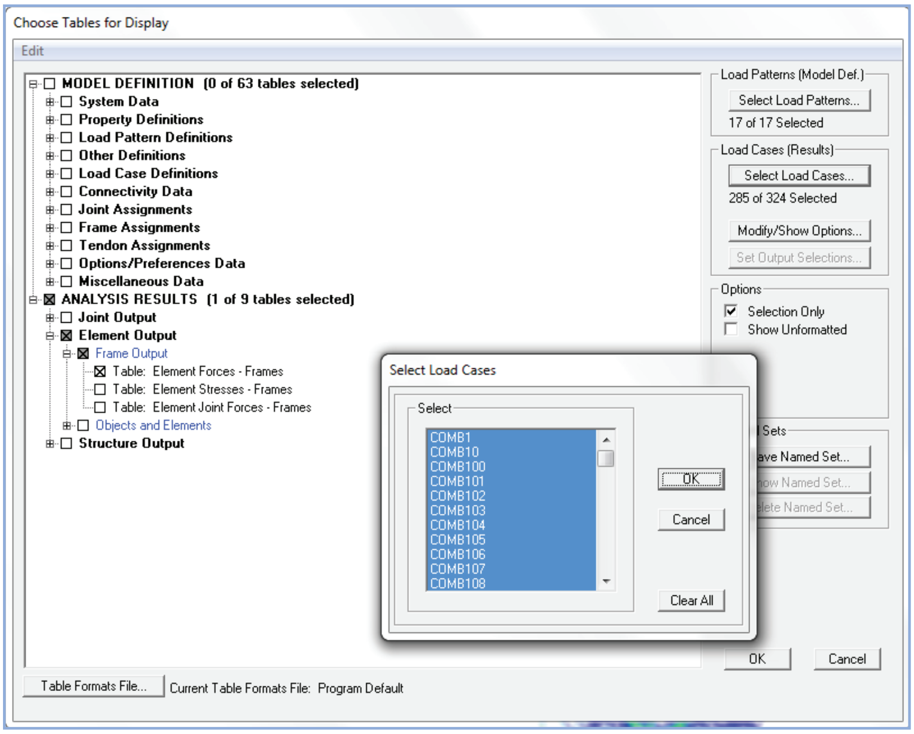
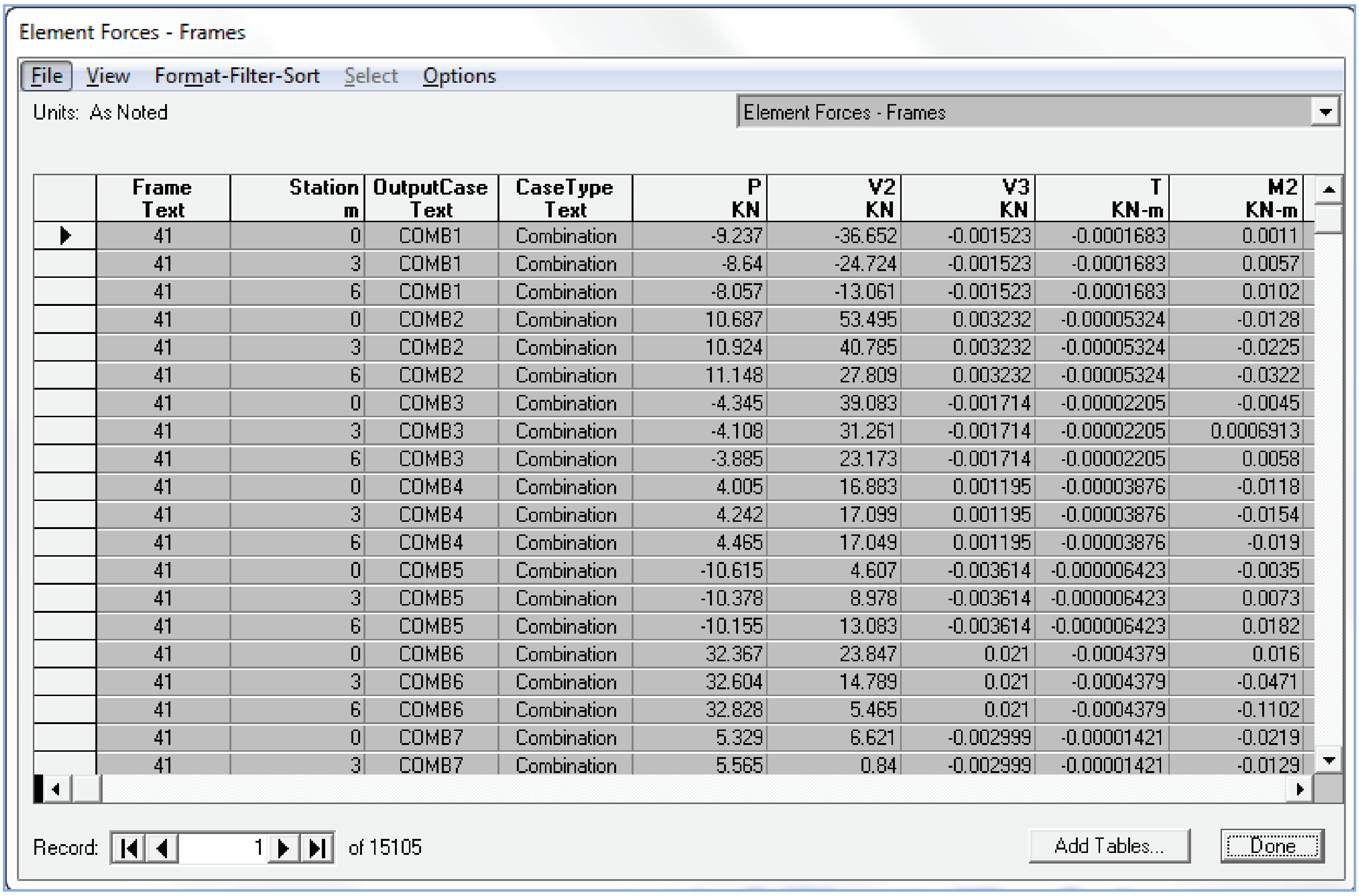


Figure 4 116 Choose Tables for Display form

In Element Forces tables, select **File > Export Current Table > To Excel**.



Frame Text	Station m	OutputCase Text	CaseType Text	P KN	V2 KN	V3 KN	T KN-m	M2 KN-m
41	0	COMB1	Combination	-9.237	-36.652	-0.001523	-0.0001683	0.0011
41	3	COMB1	Combination	-8.64	-24.724	-0.001523	-0.0001683	0.0057
41	6	COMB1	Combination	-8.057	-13.061	-0.001523	-0.0001683	0.0102
41	0	COMB2	Combination	10.687	53.495	0.003232	-0.00005324	-0.0128
41	3	COMB2	Combination	10.924	40.785	0.003232	-0.00005324	-0.0225
41	6	COMB2	Combination	11.148	27.809	0.003232	-0.00005324	-0.0322
41	0	COMB3	Combination	-4.345	39.083	-0.001714	-0.00002205	-0.0045
41	3	COMB3	Combination	-4.108	31.261	-0.001714	-0.00002205	0.0006913
41	6	COMB3	Combination	-3.885	23.173	-0.001714	-0.00002205	0.0058
41	0	COMB4	Combination	4.005	16.883	0.001195	-0.00003876	-0.0118
41	3	COMB4	Combination	4.242	17.099	0.001195	-0.00003876	-0.0154
41	6	COMB4	Combination	4.465	17.049	0.001195	-0.00003876	-0.019
41	0	COMB5	Combination	-10.615	4.607	-0.003614	-0.000006423	-0.0035
41	3	COMB5	Combination	-10.378	8.978	-0.003614	-0.000006423	0.0073
41	6	COMB5	Combination	-10.155	13.083	-0.003614	-0.000006423	0.0182
41	0	COMB6	Combination	32.367	23.847	0.021	-0.0004379	0.016
41	3	COMB6	Combination	32.604	14.789	0.021	-0.0004379	-0.0471
41	6	COMB6	Combination	32.828	5.465	0.021	-0.0004379	-0.1102
41	0	COMB7	Combination	5.329	6.621	-0.002999	-0.00001421	-0.0219
41	3	COMB7	Combination	5.565	0.84	-0.002999	-0.00001421	-0.0129

Figure 4 117 Joint Reactions table

In excel file just exported, in Station column, select station that use K-type connection, usually “0”. Filter the smallest value in “M3” column and enter this value, correspondence shear V2 and axial force P to “Bolt Connections” file.

NOTE

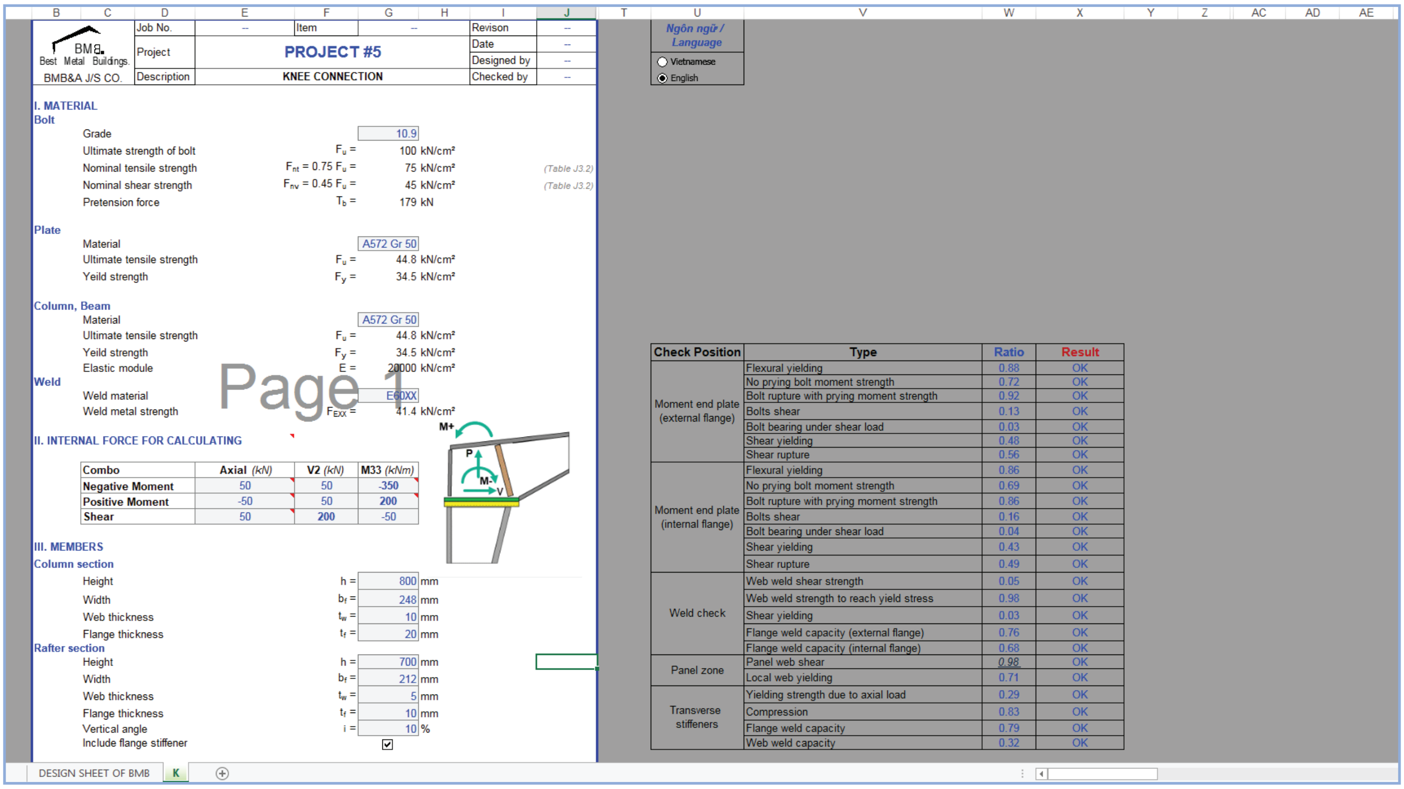
Sign of force in sap will be the same sign of force in calculation file.

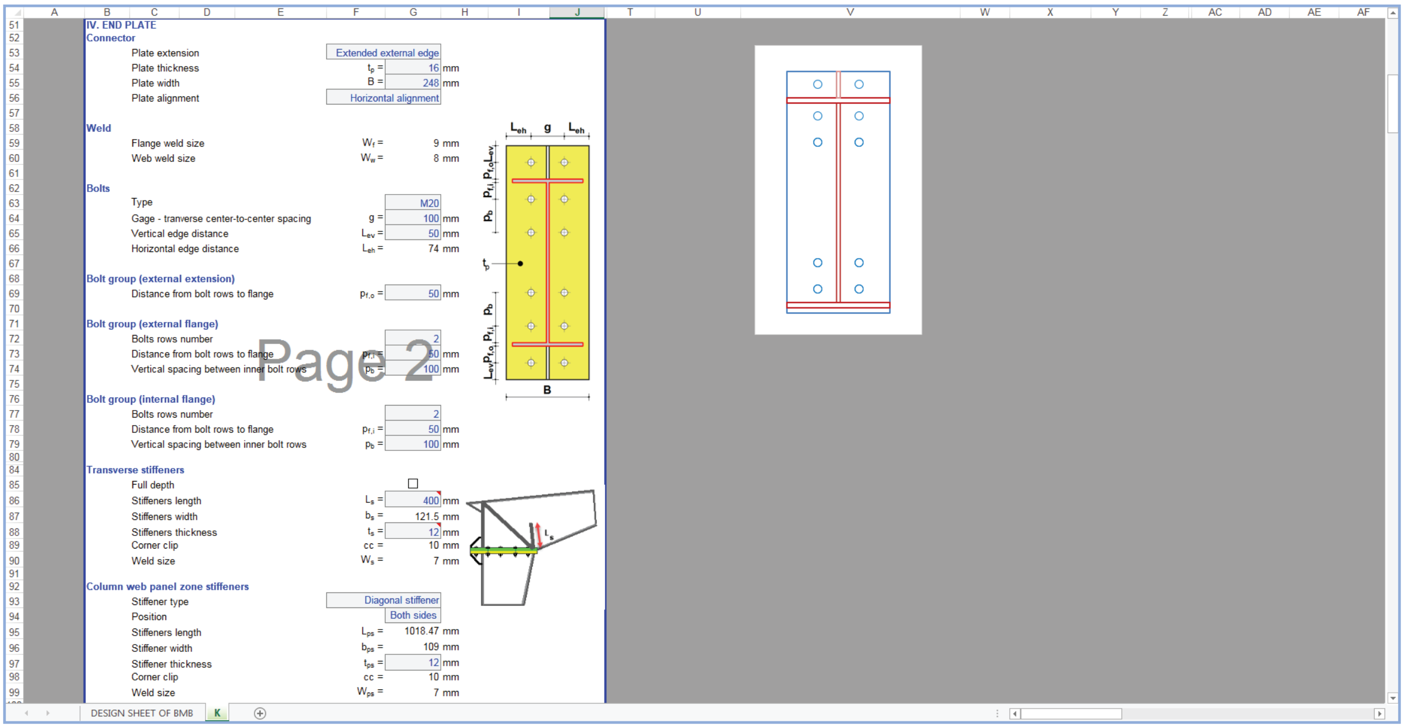
a. Filter the largest value in “M3” column and enter this value, correspondence shear V2 and axial force P to “Bolt Connections” file.

b. Filter the largest value in “V2” column and enter this value, correspondence moment M3 and axial force P to “Bolt Connections” file.

4.4.1.2 Material and geometry

In order to illustrate a K-type connection with full of dangerous cases, we assume information as figure below.





4.4.2. Determind internal force for K connection

Because interal force are in the end of rafter so we have to transfer this load into internal force at top of column

Internal force for Combo of negative moment

Axial force	$P = P_1 \sin\alpha + V_1 \cos\alpha = 50\sin 5.71^o + 50\cos 5.71^o$	54.73 kN
Shear force	$V = P_1 \cos\alpha - V_1 \sin\alpha = 50\cos 5.71^o - 50\sin 5.71^o$	44.78 kN
Moment	$M = M_1$	-350 kNm
Tension force at external flange	$PufTop = -\frac{M}{h_{fk}} + \frac{P}{2} = -\frac{-350}{0.78} + \frac{54.73}{2}$	476.08 kN
Tension force at internal flange	$PufBot = \frac{M}{h_{fk}} + \frac{P}{2} = \frac{-350}{0.78} + \frac{54.73}{2}$	-421.35 kN

Internal force for Combo of positive moment

Axial force	$P = P_1 \sin\alpha + V_1 \cos\alpha = -50\sin 5.71^o + 50\cos 5.71^o$	44.78 kN
Shear force	$V = P_1 \cos\alpha - V_1 \sin\alpha = -50\cos 5.71^o - 50\sin 5.71^o$	-54.73 kN
Moment	$M = M_1$	200 kN
Tension force at external flange	$PufTop = -\frac{M}{h_{fk}} + \frac{P}{2} = -\frac{200}{0.78} + \frac{44.78}{2}$	-234.02 kN
Tension force at internal flange	$PufBot = \frac{M}{h_{fk}} + \frac{P}{2} = \frac{200}{0.78} + \frac{44.78}{2}$	278.80 kN

Internal force for Combo of shear max

Axial force	$P = P_1 \sin\alpha + V_1 \cos\alpha = 50\sin 5.71^o + 200\cos 5.71^o$	203.98 kN
Shear force	$V = P_1 \cos\alpha - V_1 \sin\alpha = 50\cos 5.71^o - 200\sin 5.71^o$	29.85 kN
Moment	$M = M_1$	-50 kN
Tension force at external flange	$PufBot = \frac{M}{h_{fk}} + \frac{P}{2} = \frac{-50}{0.78} + \frac{203.98}{2}$	166.09 kN
Tension force at internal flange	$PufBot = \frac{M}{h_{fk}} + \frac{P}{2} = \frac{-50}{0.78} + \frac{203.98}{2}$	37.89 kN

Combo	Axial **(kN)**	V2 **(kN)**	M33 **(kNm)**	PufTop **(kN)**	PufBot **(kN)**
Negative Moment	54.73	44.78	-350.00	476.08	-421.35
Positive Moment	44.78	54.73	200.00	-234.02	278.80
Shear	203.98	29.85	-50.00	166.09	37.89

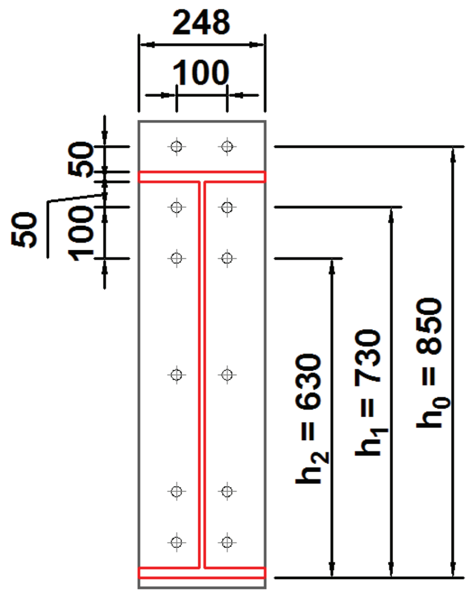
4.4.3. Checking moment end plate (external flange)

Flexural yielding

Calculate total length of yielding line of external flange from Table 4-4 DG16:

$$Y=\frac{b_p}{2}\left(\frac{h_1}{p_{f,i}}+\frac{h_2}{s}+\frac{h_0}{p_{f,o}}-\frac{1}{2}\right)+\frac{2}{g}\left[h_1\left(p_{f,i}+0.75p_b\right)+h_2\left(s+0.25p_b\right)\right]+\frac{g}{2}$$
$$=\frac{248}{2}\left(\frac{730}{50}+\frac{630}{78.74}+\frac{850}{50}-\frac{1}{2}\right)+\frac{2}{100}\left[730\left(50+0.75x100\right)+630\left(78.74+0.25x100\right)\right]+\frac{100}{2}$$
$$=8030.65mm$$

With: $s=\frac{1}{2}\sqrt{b_p g}=\frac{1}{2}\sqrt{248x100}=78.74\ \text{mm}$

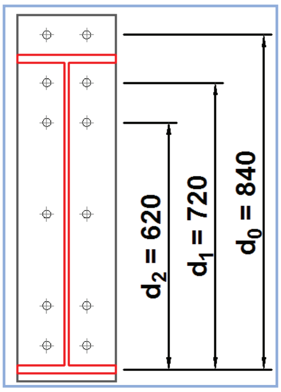


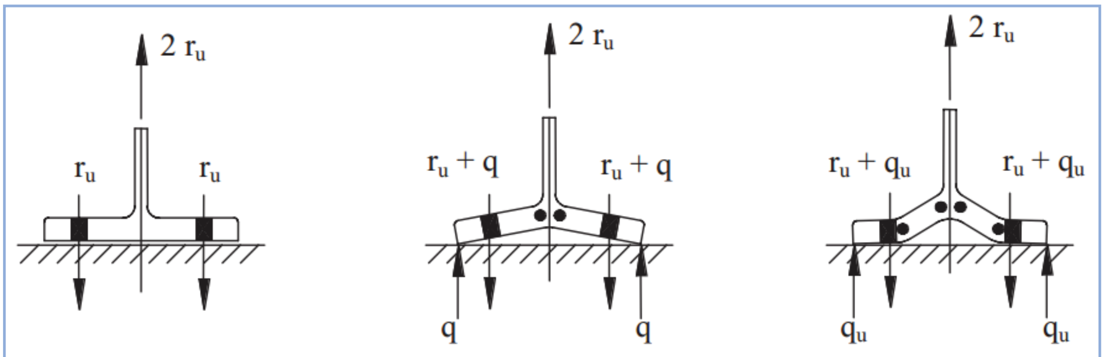
Flexural moment capacity of external end plate	$\frac{M_{pl}}{\gamma_r\Omega}=\frac{1}{\gamma_r\Omega}F_y t_p^2 Y=\frac{1}{1x1.67}34.5x1.6^2x803.065$ ($\gamma_r=1$ for extended connections)	424.71 kNm
Demand flexural moment	$M_{an}=\lvert M\rvert+P\frac{h_{fk}}{2}=\lvert-350\rvert+54.73\frac{0.78}{2}$	371.34 kNm
Demand/Capacity ratio	$D/C=\frac{371.34}{424.71}=0.88$	OK

No prying bolt moment strength

Tension capacity of one bolt	$P_t=F_{nt}\frac{\pi d_b^2}{4}=75\frac{\pi x2^2}{4}$	235.62 kN
Flexural moment capacity of bolt with no prying force	$\frac{M_{np}}{\Omega}=\frac{1}{\Omega}2P_t\sum d_n=\frac{1}{2}2\times 235.62(0.840+0.720+0.620)$	513.65 kNm

Demand/Capacity ratio	$D/C=\dfrac{371.34}{513.65}=0.72$	OK
Conclusion	$0.9\dfrac{M_{pl}}{\Omega}=0.9x424.39=381.95<\dfrac{M_{np}}{\Omega}=513.65$ Thin plate behavior Exist prying force in bolts when end plate is flexed	





(a) thick (b) intermediate (c) thin

Bolt rupture with prying moment strength

Maximum prying force of inner bolts:

$$Q_{\max,i}=\frac{w't_p^2}{4a_i}\sqrt{F_{py}^2-3\left(\frac{F_i'}{w't_p}\right)^2}=\frac{101x16^2}{4x45.72}\sqrt{3450^2-3\left(\frac{93.94}{101x16}\right)^2}=47.10\ \text{kN}$$

With: $w'=b_p/2-d_b=248/2-22=102\ \text{mm}$

$$a_i=25.4\left(3.682\left(\frac{t_p}{d_b}\right)^3-0.085\right)=25.4\left(3.682\left(\frac{16}{20}\right)^3-0.085\right)=45.72\ \text{mm}$$

$$F_i'=\frac{t_p^2F_y\left(0.85\dfrac{b_p}{2}+0.80w'\right)+\dfrac{\pi d_b^3F_{nt}}{8}}{4p_{f,i}}=\frac{16^2x0.345\left(0.85\dfrac{248}{2}+0.8x102\right)+\dfrac{\pi x20^3x0.75}{8}}{4x50}=94.30\ \text{kN}$$

Maximum prying force of outer bolts:

$$Q_{\max,o}=\frac{w't_p^2}{4a_o}\sqrt{F_{py}^2-3\left(\frac{F_o'}{w't_p}\right)^2}=\frac{101x16^2}{4x45.72}\sqrt{3450^2-3\left(\frac{93.94}{101x16}\right)^2}=47.10\ \text{kN}$$

With: $w'=b_p/2-d_b=248/2-22=102\ \text{mm}$

$$a_o = \min\left(L_{ev}; 25.4\left(3.682\left(\frac{t_p}{d_b}\right)^3 - 0.085\right)\right) = \min\left(50; 25.4\left(3.682\left(\frac{16}{20}\right)^3 - 0.085\right)\right) = 45.72\,\text{mm}$$

$$F_o' = \frac{t_p^2 F_y\left(0.85\frac{b_p}{2} + 0.80w'\right) + \frac{\pi d_b^3 F_{nt}}{8}}{4\,p_{f,o}} = \frac{16^2 x 0.345\left(0.85\frac{248}{2} + 0.8x102\right) + \frac{\pi x 20^3 x 0.75}{8}}{4x50} = 94.30\,\text{kN}$$

Moment strength	$\frac{M_q}{\Omega} = \frac{1}{\Omega}\max\begin{cases}\left[2(P_t - Q_{\max,o})d_0 + 2(P_t - Q_{\max,i})d_1 + 2T_b d_2\right] \\ \left[2(P_t - Q_{\max,o})d_0 + 2T_b(d_1 + d_2)\right] \\ \left[2(P_t - Q_{\max,i})d_1 + 2T_b(d_0 + d_2)\right] \\ \left[2T_b(d_0 + d_1 + d_2)\right]\end{cases}$ $= \frac{1}{2}\max\begin{cases}2(235.62 - 47.10)840 + 2(235.62 - 47.10)720 + 2x179x620 \\ 2(235.62 - 47.10)840 + 2x179x(720 + 620) \\ 2(235.62 - 47.10)720 + 2x179x(840 + 620) \\ 2x179x(840 + 720 + 620)\end{cases}$ With: $P_t = F_{nt}\frac{\pi d_b^2}{4} = 75\frac{\pi \times 2^2}{4} = 235.62\ \text{kN}$	405.07 kNm
Demand/ Capacity ratio	$D/C = \frac{371.34}{405.07} = 0.92$	OK

Bolts shear

Shear strength of six bolts in external flange	$\frac{R_n}{\Omega} = \frac{1}{\Omega}F_{nv}\frac{\pi d_b^2}{4}n = \frac{1}{2}45\frac{\pi x 2^2}{4}6$	424.12 kN
Demand shear strength is the maximum shear force correspondent with negative axial force of top flange		54.73 kN
Demand/Capacity ratio	$D/C = \frac{54.73}{424.12} = 0.13$	OK

Bolts bearing under shear load

Distance from edge of outermost hole to edge of plate:

$$l_{c1} = L_{ev} - d_b/2 = 50 - 22/2 = 39\ \text{mm}$$

Distance from edge of hole to hole:

$$l_c = p_b - d_b = 100 - 22 = 78\,\text{mm}$$

Bolts bearing capacity at outermost bolts:

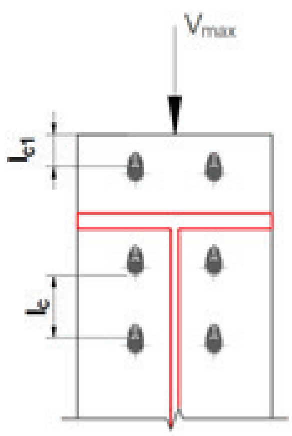
$$r_{n2} = 2(0.6F_u l_c t) = 1.2 l_c t F_u = 1.2x7.8x1.6x44.8 = 670.92\,\text{kN}$$

Bolts bearing capcity between bolts:

$$r_{n2} = 2(0.6F_u l_c t) = 1.2 l_c t F_u = 1.2x7.8x1.6x44.8 = 670.92\,\text{kN}$$

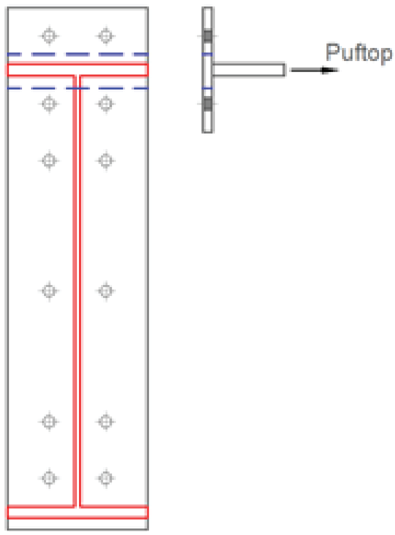
Maximum bearing capacity:

$$r_{n(\max)} = 2(0.6F_u 2dt) = 2.4dtF_u = 2.4x2x1.6x44.8 = 344.06\,\text{kN}$$



Total bearing capacity	$\frac{R_n}{\Omega}=\frac{1}{\Omega}\left[2\min(r_{n1},r_{n(\max)})+2(\frac{n}{2}-1)\min(r_{n2},r_{n(\max)})\right]$ $=\frac{1}{2}\left[2\min(335.46,344.06)+2\left(\frac{6}{2}-1\right)\min(670.92,344.06)\right]$ $=1023.58kN$
Demand shear strength	$V_a=\max(44.78,54.73,29.85)=54.73\ \text{kN}$
Demand/Capacity ratio	$D/C=\frac{54.73}{1023.58}=0.05\quad$: OK

Shear yielding of plate



Shear yielding strength of plate:

$$V_a=\frac{Puftop}{2}=\frac{\max(476.08,234.02,166.09)}{2}=238.04\text{ kN}$$

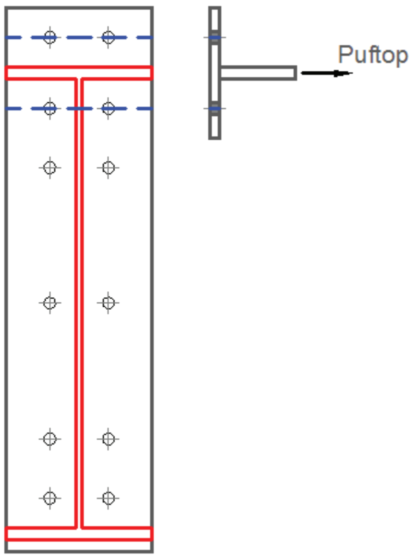
Demand shear strength:

$$V_a=\frac{Puftop}{2}=\frac{\max(476.08,234.02,166.09)}{2}=238.04\text{ kN}$$

Demand/Capacity ratio:

$$D/C=\frac{238.04}{491.84}=0.48\ :\text{OK}$$

Shear rupture of plate



Shear rupture strength of plate:

$$\frac{R_n}{\Omega}=\frac{1}{\Omega}0.6F_u[b_p-2(d_b+1.6)]t_p$$

$$\frac{R_n}{\Omega}=\frac{1}{2}\times0.6\times0.45\times[248-2(22+1.6)]16=431.99kN$$

Demand shear strength:

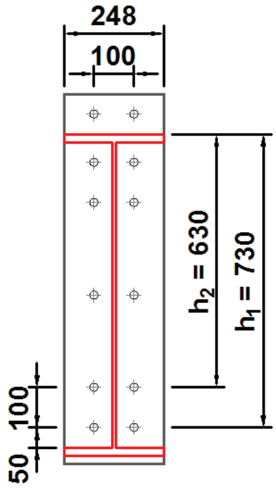
$$V_a=\frac{Puftop}{2}=\frac{\max(476.08,234.02,166.09)}{2}=238.04\ \text{kN}$$

Demand/Capacity ratio: $D/C=\frac{238.04}{431.99}=0.55\ :\text{OK}$

4.4.4. Checking moment end plate (internal flange)

Flexural yielding

Calculate total length of yielding line of internal flange from Table 3-3 DG16:

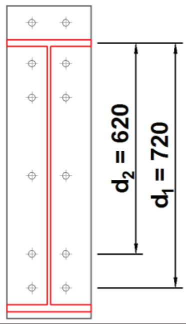
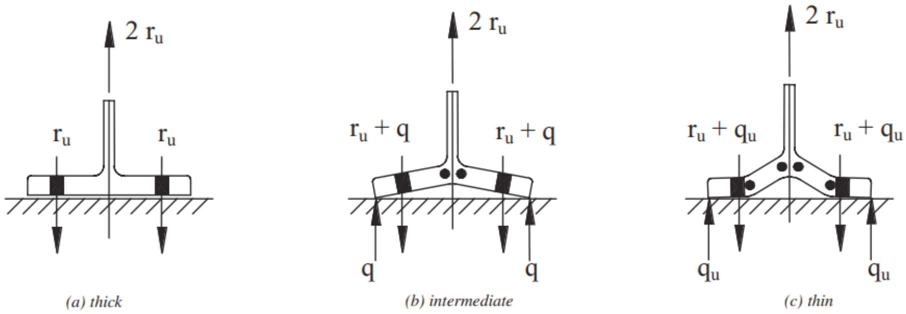


$$Y = \frac{b_p}{2}\left(\frac{h_1}{p_f}+\frac{h_2}{s}\right)+\frac{2}{g}\left[h_1\left(p_f+0.75p_b\right)+h_2\left(s+0.25p_b\right)\right]+\frac{g}{2}$$
$$=\frac{248}{2}\left(\frac{730}{50}+\frac{630}{78.74}\right)+\frac{2}{100}\left[730\left(50+0.75x100\right)+630\left(78.74+0.25x100\right)\right]+\frac{100}{2}$$
$$=5984.65mm$$

With: $s=\frac{1}{2}\sqrt{b_p g}=\frac{1}{2}\sqrt{248x100}=78.74$ mm

Flexural moment capacity of internal end plate	$\frac{M_{pl}}{\gamma_r\Omega}=\frac{1}{\gamma_r\Omega}F_y t_p^2 Y=\frac{1}{1.25x1.67}34.5x1.6^2x598.465=253.20kNm$ $(\gamma_r=1.25$ for flush connections)
Demand flexural moment	$M_{ap}=M+P\frac{h_{fk}}{2}=200+44.78\frac{0.78}{2}=217.46$ kNm
Demand/Capacity ratio	$D/C=\frac{217.46}{253.20}=0.86$: OK

No prying bolt moment strength

		
Tension capacity of one bolt	$P_t=F_{nt}\frac{\pi d_b^2}{4}=75\frac{\pi\times2^2}{4}$	235.62 kN
Flexural moment capacity of bolt with no prying force	$\frac{M_{np}}{\Omega}=\frac{1}{\Omega}2P_t(\sum d_n)=\frac{1}{2}2\times235.62(0.720+0.620)$	315.73 kNm
Demand/Capacity ratio	$D/C=\frac{217.46}{315.73}=0.69$	OK

Conclusion	$0.9\dfrac{M_{pl}}{\Omega} = 0.9x253.20 = 227.88 < \dfrac{M_{np}}{\Omega} = 315.73$ ➡ Thin plate behavior ➡ Exist prying force in bolts when end plate is flexed	

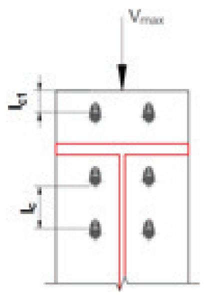
Bolt rupture with prying moment strength

Maximum prying force of inner bolts	$Q_{\max,i} = \dfrac{w' t_p^2}{4a_i}\sqrt{F_{py}^2 - 3\left(\dfrac{F_i'}{w' t_p}\right)^2} = \dfrac{101x16^2}{4x45.72}\sqrt{3450^2 - 3\left(\dfrac{93.94}{101x16}\right)^2}$	47.10 kN
	With: $w' = b_p / 2 - d_b = 248/2 - 22$	102 mm
	$a_i = 25.4\left(3.682\left(\dfrac{t_p}{d_b}\right)^3 - 0.085\right) = 25.4\left(3.682\left(\dfrac{16}{20}\right)^3 - 0.085\right)$	45.72 mm
	$\dfrac{t_p F_y\left(0.85\dfrac{}{2} + 0.80w'\right) + \dfrac{d_b F_{nt}}{8}}{}$ $f\ i$ $\dfrac{16\ \times 0.345\left(0.85\dfrac{248}{2} + 0.8\times 102\right) + \dfrac{20\ \times 0.75}{8}}{4\times 50}$	94.30 kN
Moment strength	$\dfrac{M_q}{\Omega} = \max\left\{\begin{array}{l}\dfrac{1}{\Omega}\left[2(P_t - Q_{\max,i})(d_1 + d_2)\right] \\ \dfrac{1}{\Omega}\left[2T_b(d_1 + d_2)\right]\end{array}\right\}$ $= \max\left\{\begin{array}{l}\dfrac{1}{2}\left[2\left(235.62 - 47.10\right)\left(0.72 + 0.62\right)\right] \\ \dfrac{1}{2}\left[2x179x\left(0.72 + 0.62\right)\right]\end{array}\right\}$	252.62 kN
	With: $P_t = F_{nt}\dfrac{\pi d_b^2}{4} = 75\dfrac{\pi 2^2}{4}$	235.62 kN
Demand/Capacity ratio	$D/C = \dfrac{217.46}{252.62} = 0.86$	OK

Bolts shear

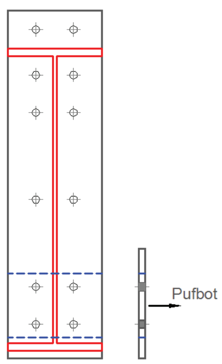
Shear strength of four bolts in internal flange	$\dfrac{R_n}{\Omega} = \dfrac{1}{\Omega}F_{nv}\dfrac{\pi d_b^2}{4}n = \dfrac{1}{2}45\dfrac{\pi 2^2}{4}4$	282.74 kN
Demand shear strength is the maximum shear force correspondent with negative axial force of bottom flange	V_a	44.78 kN
Demand/Capacity ratio	$D/C = \dfrac{44.78}{282.74} = 0.16$	OK

Bolts bearing under shear load

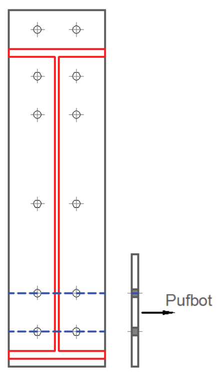
Distance from edge of outermost hole to edge of plate	$l_{c1} = L_{ev} - d_b/2 = 50 - 22/2$	39 mm	
Distance from edge of hole to hole	$l_c = p_b - d_h = 100 - 22$	78 mm	
Bolts bearing capacity at outermost bolts	$r_{n1} = 2(0.6F_u l_{c1} t) = 1.2 l_{c1} t F_u$ $= 1.2 \times 3.9 \times 1.6 \times 44.8$	335.46 kN	
Bolts bearing capcity between bolts	$r_{n2} = 2(0.6F_u l_c t) = 1.2 l_c t F_u$ $= 1.2 \times 7.8 \times 1.6 \times 44.8$	670.92 kN	
Maximum bearing capacity	$r_{n(\max)} = 2(0.6F_u 2dt) = 2.4 dt F_u$ $= 2.4 \times 2 \times 1.6 \times 44.$	344.06 kN	

Total bearing capacity	$\frac{R_n}{\Omega} = \frac{1}{\Omega}\left[2\min(r_{n1}, r_{n(\max)}) + 2(\frac{n}{2} - 1)\min(r_{n2}, r_{n(\max)})\right]$ $\frac{R_n}{\Omega} = \frac{1}{2}\left[2\min(335.46, 344.06) + 2\left(\frac{4}{2} - 1\right)\min(670.92, 344.06)\right]$	679.52 kN
Demand shear strength	$V_a = \max(44.78; 54.73; 29.85)$	54.73 kN
Demand/ Capacity ratio	$D/C = \frac{54.73}{679.52} = 0.08$	OK

Shear yielding of plate

Shear yielding strength of plate	$\frac{R_n}{\Omega} = \frac{1}{\Omega} 0.6 F_y b_p t_p$ $= \frac{1}{1.67} 0.6 \times 34.5 \times 24.8 \times 1.6$	491.84 kN	
Demand shear strength	$V_a = \frac{Pufbot}{2} = \frac{\max(421.35; 278.8; 37.89)}{2}$	210.68 kN	
Demand/ Capacity ratio	$D/C = \frac{210.68}{491.84} = 0.43$	OK	

Shear rupture of plate

Shear rupture strength of plate	$\frac{R_n}{\Omega} = \frac{1}{\Omega} 0.6 F_u [b_p - 2(d_b + 1.6)] t_p$ $= \frac{1}{2} \times 0.6 \times 0.45 \left[248 - 2(22 + 1.6)\right] 16$	431.99 kN	
Demand shear strength	$V_a = \frac{Pufbot}{2} = \frac{\max(421.35; 278.8; 37.89)}{2}$	310.68 kN	
Demand/ Capacity ratio	$D/C = \frac{210.68}{431.99} = 0.49$	OK	

4.4.5. Weld check

Web weld shear strength

Area of web weld (assume only weld at compresion area assist shear force)	$A_w = \frac{\sqrt{2}}{2} W_w L = \frac{\sqrt{2}}{2} W_w \frac{h_{fk}}{2} = \frac{\sqrt{2}}{2} 8 \times \frac{780}{2}$	2206.17 mm
Shear strength of web weld	$\frac{R_n}{\Omega} = 2\left(\frac{1}{\Omega} 0.6 F_{EXX} A_w\right) = 2\left(\frac{1}{2} 0.6 \times 41.4 \times 22.06\right)$	547.97 kN
Demand shear strength	$V_a = \max(44.78; 54.73; 29.85)$	54.73 kN
Demand/Capacity ratio	$D/C = \frac{54.73}{547.97} = 0.10$	OK

Web weld strength to reach yield stress

Load angle factor when force perpendicular with weld	$\chi = 1 + 0.5 \sin^{1.5}(\theta) = 1 + 0.5 \sin^{1.5}(90)$	1.5
Weld strength of web weld	$\frac{R_w}{\Omega} = 2\left(\frac{1}{\Omega} 0.6 F_{EXX} \chi \frac{\sqrt{2}}{2} W_w\right)$ $= 2\left(\frac{1}{2} 0.6 \times 41.4 \times 1.5 \times \frac{\sqrt{2}}{2} 0.8\right)$	21.07 kN/cm
Demand yield stress	$\frac{R_n}{\Omega} = \frac{1}{\Omega} F_y t_w = \frac{1}{1.67} 34.5 \times 1$	20.64 kN/cm
Demand/Capacity ratio	$D/C = 20.64/21.08 = 0.98$	OK

Shear yielding of web

Shear yielding strength of web	$\frac{R_n}{\Omega} = \frac{1}{\Omega} 0.6 F_y L_p t_w = \frac{1}{1.5} 0.6 \times 34.5 \times 80 \times 1$	1104 kN
Demand shear strength	$D/C = 54.73/1104 = 0.05$	54.73 kN
Demand/Capacity ratio	$D/C = 54.73/1104 = 0.05$	OK

Flange weld capacity (external flange)

Area of outsite flange weld	$A_w = \frac{\sqrt{2}}{2} W_f L = \frac{\sqrt{2}}{2} W_f (2b_f - t_w + 2t_f)$ $= \frac{\sqrt{2}}{2} 9(2 \times 248 - 10 + 2 \times 20)$	3347.44 mm^2
Load angle factor when force perpendicular with weld	$\chi = 1 + 0.5 \sin^{1.5}(\theta) = 1 + 0.5 \sin^{1.5}(90)$	1.5
Weld strength of flange weld	$\frac{R_n}{\Omega} = \frac{1}{\Omega} 0.6 F_{EXX} \chi A_w$ $= \frac{1}{2} 0.6 \times 41.4 \times 1.5 \times 33.47$	623.55 kN

Demand shear strength	$V_a = Puftop$ kN	476.08 kN
Demand/Capacity ratio	$D/C = \frac{476.08}{623.55} = 0.76$: OK	

Flange weld capacity (internal flange)

Area of inside flange weld	$A_w = \frac{\sqrt{2}}{2} W_f L = \frac{\sqrt{2}}{2} W_f (2b_f - t_w + 2t_f)$
Load angle factor when force perpendicular with weld	$\chi = 1 + 0.5\sin^{1.5}(\theta) = 1 + 0.5\sin^{1.5}(90) = 1.5$ $\chi = 1 + 0.5\sin^{1.5}(\theta) = 1 + 0.5\sin^{1.5}(90) = 1.5$
Weld strength of flange weld	$\frac{R_n}{\Omega} = \frac{1}{\Omega} 0.6 F_{EXX} \chi A_w = \frac{1}{2} 0.6x41.4x1.5x33.47 = 623.55$ kN
Demand shear strength	$V_a = Pufbot = 421.35$ kN
Demand/Capacity ratio	$D/C = \frac{421.35}{623.55} = 0.68$: OK

4.4.6. Checking panel zone

Panel web shear

- Shear yield strength of the rafter:

$$P_c = 0.6 F_y A_g = 0.6 F_y (2b_f t_f + h_w t_w) = 0.6x34.5x\left(2x212x10 + 680x5\right)/100 = 1581.48 \text{ kN}$$

For: $P_r = \max\left(Puftop, Pufbot\right) = \max\left(476.08, 421.35\right) = 476.08$
$P_r = 476.08 < 0.4 P_c = 0.4x158148 = 632.59$

- $$\frac{R_n}{\Omega} = \frac{0.6 F_y h t_w}{1.67} = \frac{0.6x34.5x70x0.5}{1.67} = 433.83 \text{ kN}$$

- Shear yield strength of the diagonal stiffeners (consider diagonal stiffeners as compresion elements):

Inertia radius of stiffeners: $r = t_{ps}/\sqrt{12} = 12/\sqrt{12} = 3.46$ mm

Slenderness of stiffeners: $KL/r = 0.65 x 1018.47/3.46 = 191.10$

$$\Rightarrow \frac{KL}{r} = 191.10 > 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{20000}{34.5}} = 113.4$$

$$\Rightarrow F_{cr} = 0.877 F_e = 0.877 \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = 0.877 \frac{\pi^2 x20000}{191.10^2} = 4.74 \frac{kN}{cm^2}$$

Strength of diagonal stiffeners:

$$\frac{R_{nps}}{\Omega} = 2\frac{1}{\Omega}F_{cr}A_g\cos\theta = 2\frac{1}{\Omega}F_{cr}(b_{ps}-cc)t_{ps}\frac{h_c-2t_{fc}}{L_{ps}}$$
$$= 2\frac{1}{1.67}4.74x(109-10)x12x10^{-2}\frac{800-2x20}{1018.47}$$
$$= 50.32kN$$

Total panel web shear resitance: $\frac{R_n}{\Omega}+\frac{R_{nps}}{\Omega} = 433.83+50.32 = 484.15\ \text{kN}$

Demand shear strength: $V_a = \max(Puftop, Pufbot) = \max(476.08, 421.35) = 476.08\ \text{kN}$

Demand/Capacity ratio: $D/C = \frac{476.08}{484.15} = 0.98$

➡ OK

4.4.7. Local web yielding

Web resistance in local web yielding:

$$\frac{}{\Omega} = \frac{1}{\Omega}[0.5(6k+2t_p)+N)F_{yw}t_w = \frac{1}{1.5}\left[0.5\left(6x16+2x16\right)+38\right]34.5x10\quad x5 = 117.3\,\text{kN}$$

With: $k = t_p = 16\ \text{mm}$

$N = t_f + 2W_f = 10+2x9 = 38\ \text{mm}$

Stiffeners resistance in local web yielding:

$$\frac{R_{ns}}{\Omega} = 2\frac{1}{\Omega}F_yA_s = 2\frac{1}{1.67}34.5x10^{-2}x1338 = 552.83\,\text{kN}$$

With: $A_s = t_s(b_s - clip) = 12(121.5-10) = 1338mm^2$

Total resistance of web:

$$\frac{R_n}{\Omega}+\frac{R_{ns}}{\Omega} = 117.3+552.83 = 670.13\,\text{kN}$$

Demand shear strength:

$V_a = \max(Puftop, Pufbot) = \max(476.08, 421.35) = 476.08\ \text{kN}$

Demand/Capacity ratio:

$$D/C = \frac{476.08}{670.13} = 0.71$$

➡ OK

4.4.8. Transverse stiffeners

Yielding strength due to axial load

Gross section area of transverse stiffeners: $A_g = 2\left(b_s - clip\right)t_s = 2x(121.5-10)x12 = 2676mm^2$

Tensile yielding strength of stiffeners: $\frac{V}{\Omega} = \frac{1}{\Omega}F_y A_g = \frac{1}{1.67}34.5x10^{-2}x2676 = 552.83\,\text{kN}$

Demand axial strength is calculated by subtracting web resistance in local web yielding from maximum tension force at bottom flange: $P_{tension} = \max(-421.35, 278.80, 37.89) - 117.3 = 161.5\,\text{kN}$

Demand/Capacity ratio: $D/C = \frac{161.5}{552.83} = 0.29$

OK

Compression

Inertia radius of stiffeners:

$r = t_s / \sqrt{12} = 12/\sqrt{12} = 3.46\,\text{mm}$

Slenderness of stiffeners:

$KL/\text{r} = 0.65\,\text{x}\,400 / 3.46 = 75.14$

$$\frac{KL}{r} = 75.14 < 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{20000}{34.5}} = 113.4$$

$$F_{cr} = \left[0.658^{-}\right]F_y = \left[0.658^{\frac{34.5}{34.96}}\right]34.5 = 22.83\frac{kN}{cm}$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{34.5}{34.96}}\right]34.5 = 22.83\frac{kN}{cm^2}$$

Compressive strength shall be determined based on the limit state of flexural buckling:

$$\frac{P_n}{\Omega} = 2\frac{1}{\Omega}F_{cr}A_g = 2\frac{1}{1.67}22.83x13.38 = 365.83\,\text{kN}$$

Demand axial strength is calculated by subtracting web resistance in local web yielding from maximum compressive force at bottom flange:

$P_{compression} = \left|\min(-421.35, 278.80, 37.89)\right| - 117.3 = 304.05\ \text{kN}$

Demand/Capacity ratio:

$$D/C = \frac{304.05}{365.83} = 0.83$$

OK

Welding stiffeners with rafter flange

Area of flange weld:

$$A_w = \frac{\sqrt{2}}{2} W_s L = \frac{\sqrt{2}}{2} W_s \left(b_{st} - clip - W_s\right) = \frac{\sqrt{2}}{2} 7\left(121.5 - 10 - 7\right) = 517.25 mm^2$$

Load angle factor when force perpendicular with weld

$$\chi = 1 + 0.5\sin^{1.5}(\theta) = 1 + 0.5\sin^{1.5}(90) = 1.5$$

Weld strength of total flange weld:

$$V_a = \max\left(P_{tension}, P_{compression}\right) = \max\left(161.5, 304.05\right) = 304.05 \text{ kN}$$

Demand shear strength:

$$V_a = \max\left(P_{tension}, P_{compression}\right) = \max\left(161.5, 304.05\right) = 304.05 \text{ kN}$$

Demand/Capacity ratio:

$$D / C = \frac{304.05}{385.27} = 0.79$$

➡ OK

Welding stiffeners with rafter web

Area of web weld:

$$\frac{R_n}{\Omega} = 4\left(\frac{1}{\Omega} 0.6 F_{EXX} \chi A_w\right) = 4\left(\frac{1}{2} 0.6 x 41.4 x 1.0 x 18.96\right) = 941.93$$

Load angle factor when force parallel with weld $\chi = 1 + 0.5\sin^{1.5}(\theta) = 1 + 0.5\sin^{1.5}(0) = 1.0$

Weld strength of total web weld:

$$\frac{R_n}{\Omega} = 4\left(\frac{1}{\Omega} 0.6 F_{EXX} \chi A_w\right) = 4\left(\frac{1}{2} 0.6 x 41.4 x 1.0 x 18.96\right) = 941.93 \text{ kN}$$

Demand shear strength: $V_a = \max\left(P_{tension}, P_{compression}\right) = \max\left(161.5, 304.05\right) = 304.05 \text{ kN}$

Demand/Capacity ratio: $D / C = \dfrac{304.05}{941.93} = 0.32$

➡ OK

4.4.9. Checking diagonal stiffeners

Yielding strength due to axial load

Section area of diagonal stiffeners: $A_g = 2\left(b_{ps} - clip\right)t_{ps} = 2\left(109 - 10\right)12 = 2376\,\text{mm}^2$

Tensile yielding strength of stiffeners: $\dfrac{R_n}{\Omega} = \dfrac{1}{\Omega}F_y A_g = \dfrac{1}{1.67}34.5x23.76 = 490.85\,\text{kN}$

Demand axial strength is calculated by subtracting web resistance in panel web shear from maximum of tensile force at bottom flange and compressive force at top flange:

$$P_{tension} = \frac{\max\left[\left|\min\left(476.08, -234.02, 166.09\right)\right|, \max(-421.35, 278.80, 37.89)\right] - 433.83}{\cos\theta} = \frac{-155.03}{\cos\theta} < 0\,\text{kN}$$

Because required strength is negative so it's not necessary to check tensile yielding strength of diagonal stiffeners.

Checking compressive strength of diagonal stiffeners

Inertia radius of stiffeners: $r = t_{ps} / \sqrt{12} = 12 / \sqrt{12} = 3.46\,\text{mm}$

Slenderness of stiffeners: $\dfrac{KL}{r} = 191.33 > 4.71\sqrt{\dfrac{E}{F_y}} = 4.71\sqrt{\dfrac{20000}{34.5}} = 113.4$

$$\Rightarrow \quad \frac{KL}{r} = 191.33 > 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{20000}{34.5}} = 113.4$$

$$\Rightarrow \quad F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} = \frac{\pi^2 x 20000}{191.33^2} = 5.39\frac{kN}{cm^2}$$

$$\Rightarrow \quad F_{cr} = 0.877 F_e = 0.877x5.39 = 4.73\frac{kN}{cm^2}$$

Compressive strength shall be determined based on the limit state of flexural buckling:

$$\frac{P_n}{\Omega} = 2\frac{1}{\Omega}F_{cr}A_g = 2\frac{1}{1.67}4.74x11.88 = 67.44\,\text{kN}$$

Demand axial strength is calculated by subtracting web resistance in panel web shear from maximum of compressive force at bottom flange and tensile force at top flange:

$$P_{compression} = \frac{\max\left[\max\left(476.08, -234.02, 166.09\right), \left|\min(-421.35, 278.80, 37.89)\right|\right] - 433.83}{\cos\theta}$$

$$= \frac{42.25}{\left(h_c - 2t_{fc}\right)/L_{ps}} = \frac{42.25}{\left(800 - 2x20\right)/1018.47} = 56.62kN$$

Demand/Capacity ratio: $D/C = \dfrac{56.62}{67.44} = 0.84$

$\Rightarrow$ OK

Welding stiffeners with rafter flange

Area of flange weld: $A_w = \frac{\sqrt{2}}{2} W_{ps} L = \frac{\sqrt{2}}{2} W_{ps}\left(b_{ps} - clip - W_{ps}\right) = \frac{\sqrt{2}}{2} 7\left(109 - 10 - 7\right) = 455.38 mm^2$

Load angle factor when force perpendicular with weld: $\chi = 1 + 0.5\sin^{1.5}(\theta) = 1 + 0.5\sin^{1.5}(90) = 1.5$

Weld strength of total flange weld: $\frac{R_n}{\Omega} = 4\left(\frac{1}{\Omega} 0.6 F_{EXX} \chi A_w\right) = 4\left(\frac{1}{2} 0.6x41.4x1.5x4.55\right) = 339.07$ kN

Demand shear strength: $V_a = \max\left(P_{tension}, P_{compression}\right) = \max\left(0, 56.62\right) = 56.62$ kN

Demand/Capacity ratio: $D/C = \frac{56.62}{339.07} = 0.17$

➡ OK

Welding stiffeners with rafter web

Area of web weld: $A_w = \frac{\sqrt{2}}{2} W_{ps} L = \frac{\sqrt{2}}{2} W_{ps}\left(L_{ps} - clip - W_s\right) = \frac{\sqrt{2}}{2} 7\left(1018.47 - 10 - 7\right) = 4957.02 mm^2$

Load angle factor when force parallel with weld $\chi = 1 + 0.5\sin^{1.5}(\theta) = 1 + 0.5\sin^{1.5}(0) = 1.0$

Weld strength of total web weld: $V_a = \max\left(P_{tension}, P_{compression}\right) = \max\left(0, 56.62\right) = 56.62$ kN

Demand shear strength: $V_a = \max\left(P_{tension}, P_{compression}\right) = \max\left(0, 56.62\right) = 56.62$ kN

Demand/Capacity ratio: $D/C = \frac{56.62}{2462.64} = 0.02$

➡ OK

4.5. Single Plate Connection Design

4.5.1. Input

4.5.1.1 Loading

Run the model and specify all beam having section B1. Select the **Display > Show Tables** command to show **Choose Tables for Display** form. Make sure that **the Table: Element Forces - Frames i**tem and **ENVE** combination in the **Select Load Cases** form are selected before click the OK button.

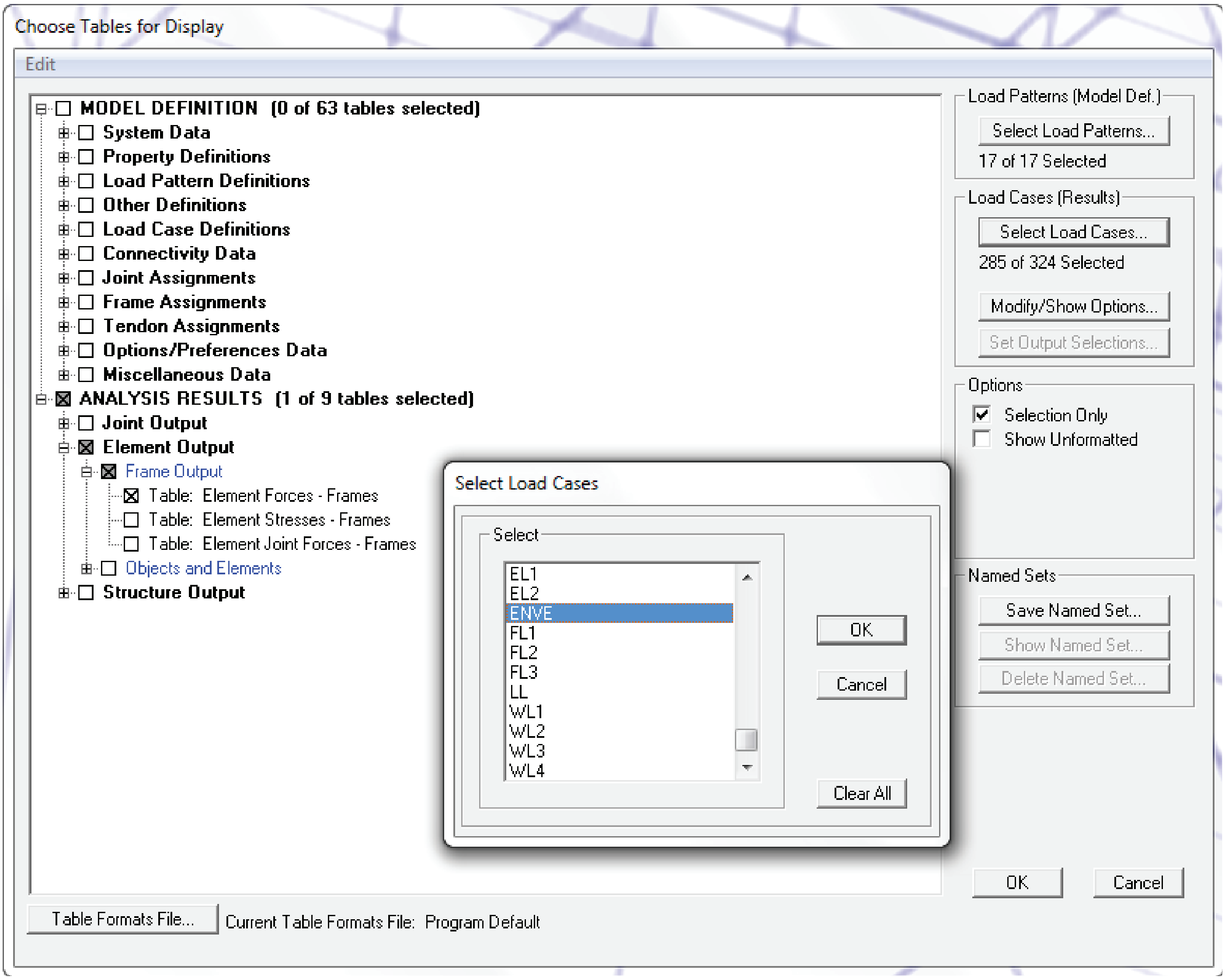
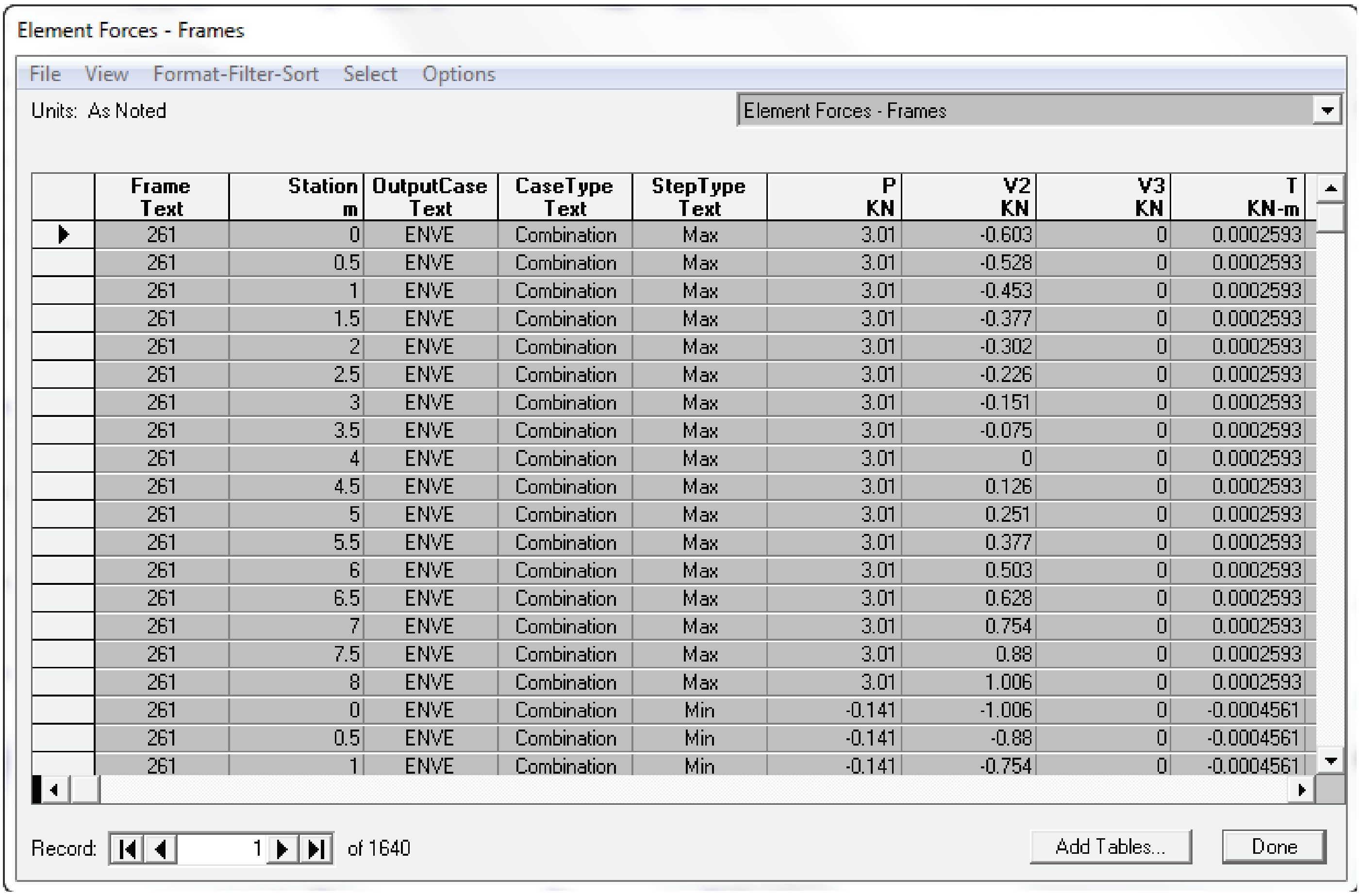


Figure 4 118 Choose Tables for Display form

In Element Forces tables, select File > Export Current Table > To Excel.



Frame Text	Station m	OutputCase Text	CaseType Text	StepType Text	P KN	V2 KN	V3 KN	T KN-m
261	0	ENVE	Combination	Max	3.01	-0.603	0	0.0002593
261	0.5	ENVE	Combination	Max	3.01	-0.528	0	0.0002593
261	1	ENVE	Combination	Max	3.01	-0.453	0	0.0002593
261	1.5	ENVE	Combination	Max	3.01	-0.377	0	0.0002593
261	2	ENVE	Combination	Max	3.01	-0.302	0	0.0002593
261	2.5	ENVE	Combination	Max	3.01	-0.226	0	0.0002593
261	3	ENVE	Combination	Max	3.01	-0.151	0	0.0002593
261	3.5	ENVE	Combination	Max	3.01	-0.075	0	0.0002593
261	4	ENVE	Combination	Max	3.01	0	0	0.0002593
261	4.5	ENVE	Combination	Max	3.01	0.126	0	0.0002593
261	5	ENVE	Combination	Max	3.01	0.251	0	0.0002593
261	5.5	ENVE	Combination	Max	3.01	0.377	0	0.0002593
261	6	ENVE	Combination	Max	3.01	0.503	0	0.0002593
261	6.5	ENVE	Combination	Max	3.01	0.628	0	0.0002593
261	7	ENVE	Combination	Max	3.01	0.754	0	0.0002593
261	7.5	ENVE	Combination	Max	3.01	0.88	0	0.0002593
261	8	ENVE	Combination	Max	3.01	1.006	0	0.0002593
261	0	ENVE	Combination	Min	-0.141	-1.006	0	-0.0004561
261	0.5	ENVE	Combination	Min	-0.141	-0.88	0	-0.0004561
261	1	ENVE	Combination	Min	-0.141	-0.754	0	-0.0004561

Figure 4 118 Choose Tables for Display form

NOTE

Take absolute value for both shear and axial force to enter into "Bolt Connections" file.

In excel file just exported, in Station column, select stations at two ends of beam. Filter the largest value in "V2" column and largest value in "P", enter these value to "Bolt Connections" file.

4.5.1.2 Material and geometry

In order to illustrate a single plate connection with full of dangerous cases, we assume information as figure below.

BMB&A J/S CO.	Description	SINGLE PLATE CONNECTION	Checked by	--

◉ English

I. MATERIAL

Bolt

Grade		10.9		
Ultimate strength of bolt	F_u =	100	kN/cm²	
Nominal tensile strength	F_{nt} =	75	kN/cm²	*(Table J3.2)*
Nominal shear strength	F_{nv} =	45	kN/cm²	*(Table J3.2)*

Plate

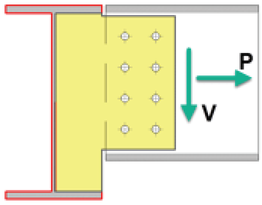
Material		A572 Gr 50	
Tensile strength	F_u =	44.8	kN/cm²
Yeild strength	F_y =	34.5	kN/cm²

Beam, Girder

Material		A572 Gr 50	
Ultimate tensile strength	F_u =	44.8	kN/cm²
Yeild strength	F_y =	34.5	kN/cm²
Elastic module	E =	20000	kN/cm²

II. INTERNAL FORCE FOR CALCULATING

Shear	V =	200	kN
Axial	P =	50	kN



Page 1

III. DIMENSION

Beam section

Height	h =	500	mm
Flange width	b_f =	200	mm
Web thickness	t_w =	10	mm
Flange thickness	t_f =	20	mm
Beam setback	sb =	10	mm

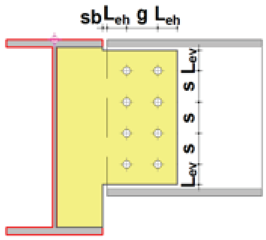
Structural

Plate

Beam

Girder section

Flange width	b_f =	212	mm
Web thickness	t_w =	8	mm



Plate

Thickness	t_p =	14	mm
Height	L_p =	305	mm

Bolt

Type		M20	
Bolt rows	n =	4	
Bolt columns	n_c =	2	
Gage - transverse spacing	g =	100	mm
Pitch - longitudinal spacing	s =	75	mm
Vertical edge distance	Lev =	40	mm
Horizontal edge distance	Leh =	40	mm
Distance between weld and bolts	a =	152	mm
Eccentric distance	e =	202	mm

IV. CALCULATING

1. Plate (beam side)

DESIGN SHEET OF BMB | K | SP | TAP BS

4.5.2. Plate (beam side)

4.5.2.1 Bolt shear

Shear strength of bolt group:

$$\frac{R_n}{\Omega} = \frac{CF_{nv}A_b n_v}{\Omega} = \frac{3.08x45x\dfrac{\pi x 2^2}{4}x1}{2} = 217.71\ \text{kN}$$

Where:

– C: coefficient for eccentrically loaded bolt groups (represents the number of bolts that are effective in resisting the eccentric shear force) is selected from table 7-8 AISC Steel Construction Manual 13th, with n=4, s=75mm (3in), e_x=202mm (7.95in), using interpolation method we find out C value is approximately **3.08**.

7–38 DESIGN CONSIDERATIONS FOR BOLTS

Table 7–8
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

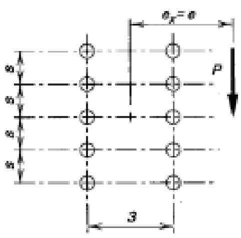
$R_n = C \times r_n$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where
P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0
	3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5
	4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8
	6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8
	7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8
	8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8
	9	0.28	0.86	1.60	2.65	3.87	5.34	6.97	8.75	10.7	12.7	14.7	16.8
	10	0.26	0.78	1.46	2.42	3.53	4.90	6.42	8.10	9.91	11.8	13.8	15.9
	12	0.22	0.66	1.24	2.06	3.01	4.19	5.51	7.01	8.63	10.4	12.2	14.2
	14	0.19	0.57	1.08	1.78	2.62	3.66	4.82	6.15	7.61	9.19	10.9	12.7
	16	0.17	0.51	0.95	1.57	2.32	3.24	4.27	5.47	6.79	8.23	9.78	11.4
	18	0.15	0.45	0.85	1.41	2.07	2.90	3.83	4.92	6.11	7.43	8.85	10.4
	20	0.14	0.41	0.77	1.27	1.88	2.63	3.48	4.47	5.55	6.76	8.07	9.48
	24	0.12	0.34	0.65	1.07	1.58	2.21	2.93	3.77	4.69	5.72	6.85	8.06
	28	0.10	0.29	0.56	0.92	1.36	1.90	2.53	3.25	4.05	4.95	5.93	7.00
	32	0.09	0.26	0.49	0.80	1.19	1.67	2.22	2.86	3.57	4.36	5.23	6.18
	36	0.08	0.23	0.43	0.72	1.06	1.49	1.98	2.55	3.18	3.90	4.67	5.52
	C'	2.94	8.33	15.8	26.0	38.7	54.2	72.2	93.1	117	143	172	204

– n_v: number of shear plane through bolt. In this case, n_v=1.

Demand shear strength for bolt group:

$$V_a = \sqrt{P^2 + V^2} = \sqrt{50^2 + 200^2} = 206.16\ \text{kN}$$

Demand/Capacity ratio

$$D/C = \frac{206.16}{217.71} = 0.95 : \text{OK}$$

4.5.2.2 Shear yielding/buckling and flexure yielding

Shear yielding strength of plate	$V_c = \frac{1}{\Omega} 0.6 F_y A_g = \frac{1}{1.5} 0.6 \times 34.5 \times 30.5 \times 1.4$	589.26 kN
Plastic section modul of plate	$Z_{pl} = \frac{t_p L_p^{\ 2}}{4} = \frac{1.4 x 30.5^2}{4}$	325.59 cm²
Flexural yielding strength of plate	$M_c = \frac{1}{\Omega} F_y Z_{pl} = \frac{1}{1.67} 34.5 \times 325.59 \times 10^{-3}$	67.26 kNm
Demand moment strength	$M_a = Ve = 200 \times 0.202$	40.4 kNm
Demand shear strength	V_a	200 kN
Demand/Capacity ratio	$D/C = \left(\frac{200}{589.26}\right)^2 + \left(\frac{40.4}{67.26}\right)^2 = 0.48$	OK

4.5.2.3 Bolt bearing under shear load

Distance from edge of outermost hole to edge of plate	$l_{c1} = L_{ev} - d_h / 2 = 40 - 22/2$	29 mm
Distance from edge of hole to hole	$l_c = p_b - d_h = 75 - 22$	53 mm
Bolts bearing capacity at outermost bolts	$r_{n1} = 2(0.6 F_u l_{c1} t) = 1.2 l_{c1} t F_u = 1.2 \times 2.9 \times 1.4 \times 44.8$	218.27 kN
Bolts bearing capcity between bolts	$r_{n2} = 2(0.6 F_u l_c t) = 1.2 l_c t F_u = 1.2 \times 5.3 \times 1.4 \times 44.8$	398.90 kN
Maximum bearing capacity	$r_{n(\max)} = 2(0.6 F_u 2dt) = 2.4 dt F_u = 2.4 \times 2 \times 1.4 \times 44.8$	301.06 kN
Total bearing capacity	$\frac{R_n}{\Omega} = \frac{1}{\Omega} \mathrm{C} \min(r_{n1}, \mathrm{r}_{n2}, r_{n(\max)})$ $= \frac{1}{2} 3.08 \times \min\left(218.27; 398.90; 301.06\right)$	336.14 kN
Demand shear strength	V_a	200 kN
Demand/Capacity ratio	$D/C = \frac{200}{336.14} = 0.59$	OK

4.5.2.4 Shear yielding

Shear yielding strength of plate	$\frac{R_n}{\Omega}=\frac{1}{\Omega}0.6F_yA_g=\frac{1}{1.5}0.6\times34.5\times30.5\times1.4$	589.26 kN
Demand shear strength	V_a	200 kN
Demand/Capacity ratio	$D/C=\frac{200}{589.26}=0.34$	OK

4.5.2.5 Shear rupture

Net area of plate	$A_{nv}=[\mathrm{L}_p-\mathrm{n}(\mathrm{d}_b+1.6)]t=\left[305-4\left(22+1.6\right)\right]14$	2948.4 mm²
Shear rupture strength of plate	$\frac{R_n}{\Omega}=\frac{0.6F_uA_{nv}}{\Omega}=\frac{0.6\times44.8\times29.48}{2}$	396.21 kN
Demand shear strength	V_a	200 kN
Demand/Capacity ratio	$D/C=\frac{200}{396.21}=0.50$	OK

4.5.2.6 Block shear rupture

Net area subject to shear	$A_{nv}=[(n-1)s+L_{ev}-(n-0.5)(d_b+1.6)]\mathrm{t}$ $=\left[\left(4-1\right)75+40-\left(4-0.5\right)\left(22+1.6\right)\right]14$	2553.6 mm²
Net area subject to tension	$A_{nt}=[L_{eh}+(n_c-1)g-(n_c-0.5)(d_b+1.6)]t$ $=\left[40+\left(2-1\right)100-\left(2-0.5\right)\left(22+1.6\right)\right]14$	1464.4 mm²
Gross area subject to shear	$A_{gv}=[(n-1)s+L_{ev}]t=\left[\left(4-1\right)75+40\right]14$	3710 mm²
Block shear rupture strength of plate	$\frac{R_n}{\Omega}=\frac{1}{\Omega}[\mathrm{U}_{bs}F_uA_{nt}+\min(0.6F_yA_{gv};0.6F_uA_{nv})]$ $=\frac{1}{2}\left[0.5\times44.8\times14.64+\min\left(0.6\times34.5\times37.1;0.6\times44.8\times25.54\right)\right]$ (U_{bs} is taken equal to 0.5 due to the tension stress is nonuniform for two of column of bolts)	507.4 kN
Demand shear strength	V_a	200 kN
Demand/Capacity ratio	$D/C=\frac{200}{507.4}=0.39$	OK

4.5.2.7 Flexure rupture

Plastic section modul of net section will be calculated as below	$Z_{net}=\frac{t}{4}\left[\mathrm{L}_p^2-\frac{s^2n(n^2-1)(\mathrm{d}_b+1.6)}{L_p}\right]$ $=\frac{14}{4}\left[305^2-\frac{75^2x4x(4^2-1)(22+1.6)}{305}\right]$	234186 mm³
Flexure rupture strength of plate	$\frac{M_n}{\Omega_b}=\frac{F_uZ_{net}}{\Omega_b}=\frac{44.8x234.19}{2}$	52.46 kNm

Demand moment strength	$M_a = V_a a = 200 \times 0.152$	30.4 kNm
Demand/Capacity ratio	$D/C = \dfrac{30.4}{52.46} = 0.58$	OK

4.5.2.8 Bolt bearing under axial load

Distance from edge of outermost hole to edge of plate	$l_{c1} = L_{eh} - d_b/2 = 40 - 22/2$	29 mm
Distance from edge of hole to hole	$l_c = g - d_b = 100 - 22$	78 mm
Bolts bearing capacity at outermost bolts	$r_{n1} = 2(0.6F_u l_{c1} t) = 1.2 l_{c1} t F_u = 1.2 \times 2.9 \times 1.4 \times 44.8$	218.27 mm
Bolts bearing capcity between bolts	$r_{n2} = 2(0.6F_u l_c t) = 1.2 l_c t F_u = 1.2 \times 7.8 \times 1.4 \times 44.8$	587.06 kN
Maximum bearing capacity	$r_{n(\max)} = 2(0.6F_u 2dt) = 2.4dtF_u = 2.4 \times 2 \times 1.4 \times 44.8$	301.06 kN
Total bearing capacity	$\dfrac{R_n}{\Omega} = \dfrac{1}{\Omega} \mathrm{C} \min(r_{n1}, \mathrm{r}_{n2}, r_{n(\max)}) = \dfrac{1}{2} 3.08 \times \min(218.27; 587.06; 301.06)$	336.14 kN
Demand axial strength	P_a	50 kN
Demand/Capacity ratio	$D/C = \dfrac{50}{336.14} = 0.15$	OK

4.5.2.9 Tension yielding

Tension yielding strength of plate	$\dfrac{R_n}{\Omega} = \dfrac{1}{\Omega} F_y A_g = \dfrac{1}{1.67} 34.5 \times 30.5 \times 1.4$	882.13 kN
Demand axial strength	P_a	50 kN
Demand/Capacity ratio	$D/C = \dfrac{50}{882.13} = 0.06$	OK

4.5.2.10 Tension rupture

Net area subject to tension	$A_{np} = [\mathrm{L}_p - \mathrm{n}(\mathrm{d}_b + 1.6)]t = \lfloor 305 - 4(22 + 1.6) \rfloor 14$	2948.4 mm²
Axial rupture strength of plate	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{np}}{\Omega} = \dfrac{44.8x29.48}{2}$	660.35 kN
Demand axial strength	P_a	50 kN
Demand/Capacity ratio	$D/C = \dfrac{50}{660.35} = 0.08$	OK

4.5.2.11 Tear out under axial load

Net area subject to shear	$A_{nv} = 2[(n_c - 1)g + L_{eh} - (n_c - 0.5)(d_b + 1.6)]t$ $= 2\left[(2-1)100 + 40 - (2-0.5)(22+1.6)\right]14$	2928.8 mm²
Net area subject to tension	$A_{nt} = (n-1)[s - (d_b + 1.6)]t$ $= (4-1)\left[75 - (22+1.6)\right]14$	2158.8 mm²
Gross area subject to shear	$A_{gv} = 2[(n_c - 1)g + L_{eh}]t = 2\left[(2-1)100 + 40\right]14$	3920 mm²
Block axial rupture strength of plate	$\frac{R_n}{\Omega} = \frac{1}{\Omega}[U_{bs} F_u A_{nt} + \min(0.6F_y A_{gv}; 0.6F_u A_{nv})]$ $= \frac{1}{2}\left[1\times 44.8\times 21.59 + \min\left(0.6\times 34.5\times 39.2; 0.6\times 44.8\times 29.29\right)\right]$ (U_{bs} is taken equal to 1 because the tension stress is uniform for all of rows of bolts)	877.27 kN
Demand axial strength	P_a	50 kN
Demand/Capacity ratio	$D/C = \frac{50}{877.27} = 0.06$	OK

4.5.3. Beam checking

Checking bolt bearing under shear load and tension load, shear yielding, tension rupture and tear out under axial load similarly for beam. The other limit states are not necessary.

CHAPTER 5. CRANE SYSTEMS DESIGN

5.1. Crane Types

The most common crane systems used in steel buildings are given in Table 5 1.

These categories of crane service classification have been established in Table 5 2.

This document refers only to Classes A through D.

Table 5 1 General Range of Crane Types

Crane Type	Power Source	Description	Span or Reach (m)	Capacity (Tons)
Under hung or Monorails	Hand Geared or Electric	Single Girder	3 ÷ 15.24	1/2 ÷ 10
Top Running	Hand Geared or Electric	Single Girder	3 ÷ 15.24	1/2 ÷ 10
	Electric	Double Girder	6 ÷ 18.29	5 ÷ 25
	Electric	Box Girder Pendant-Operated 4-Wheel End Truck	6 ÷ 27.43	5 ÷ 25
	Electric	Box Girder Cab Operated 4-Wheel End Truck	15.24 ÷ 30.48	Up to 60
	Electric	Box Girder Cab Operated 8-Wheel End Trucks	15.24 ÷ 30.48	Up to 250
JIB Cranes	Hand Geared or Electric	Floor Mounted 280° ÷ 360° Column Mounted 180°	2.43 ÷ 6	1/4 ÷ 5

Table 5 2 CMAA Crane Service Classes[5]

Class	Usage range	Description
CLASS A (Standby or Infrequent Service)	Power houses, public utilities, turbine rooms, motor rooms, and transformer stations, etc.	slow speeds and with long periods of idling between each lift.
CLASS B (Light Service)	light assembly facilities, repair shops, service buildings, and warehousing, etc.	• slow speeds and light service. • 2 to 5 lifts per hour, • average lift distance of 10 feet
CLASS C (Moderate Service)	manufacturing, machine shops, or paper mill machine rooms, etc.	• average loads is 50% of the rated capacity, • 5 to 10 lifts per hour • average lift distance of 15 feet
CLASS D (Heavy Service)	heavy machine shops, foundries, fabricating plants, steel warehouses, container yards, lumber mills, etc.	• quick speeds and moved constantly • loads approaching 50% of the capacity throughout the workday but not over 65%, • 10 to 20 lifts per hour • average heights of 15 feet
CLASS E (Severe Service)	Magnet/bucket combination cranes for heavy items like crap yards or production mills, cement mills, lumber mills, fertilizer plants, and container handling, etc.	• loads approaching rated capacity throughout their lifetime • more than 20 lifts per hour • require more frequent maintenance and provide the highest reliability possible.
CLASS F (Continuous Severe Service)	industrial settings, etc.	• high capacity loads with constant frequency • require the ability to handle the most extreme working conditions.

[5] Crane Manufacturers Association of America CMAA (2008), Specification No. 70, Charlotte, NC.

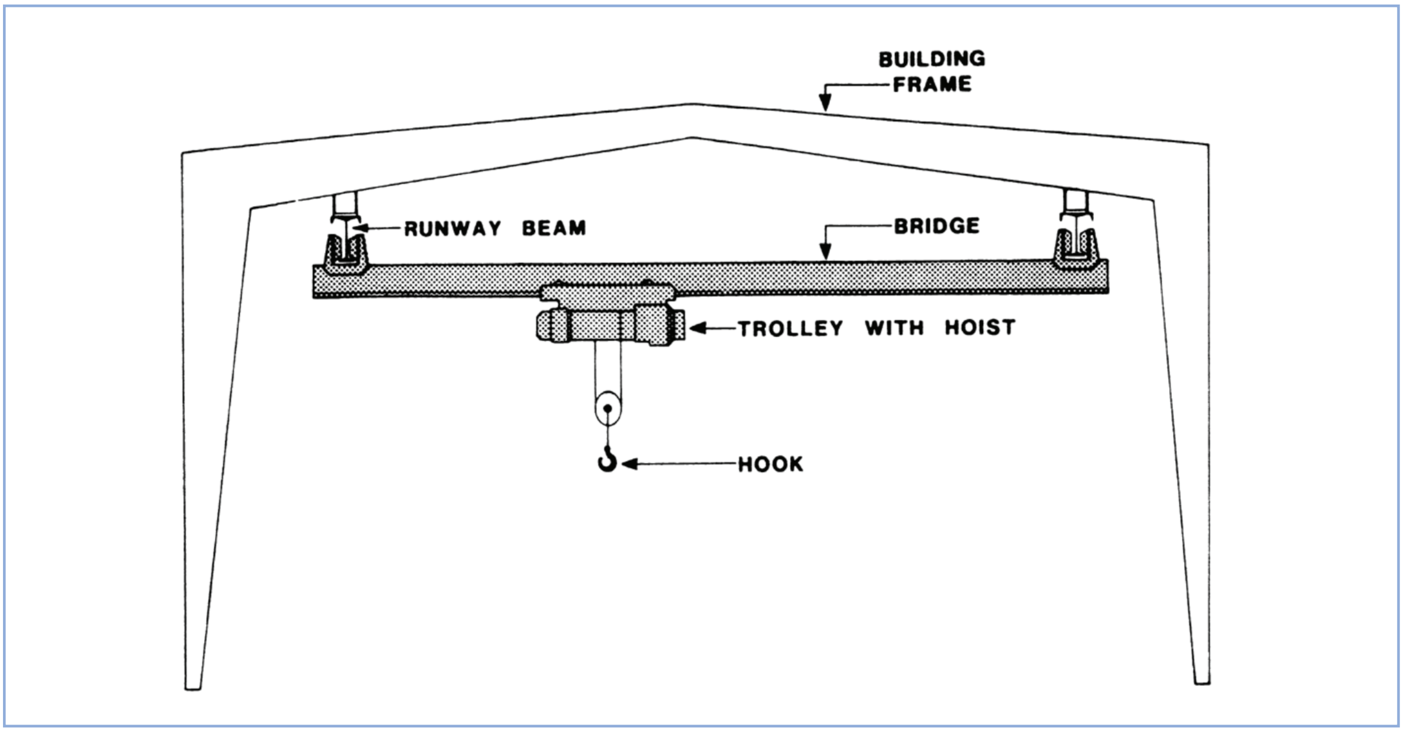


Figure 5 1 Underhung Bridge Crane

The monorail cranes are characterized by the hoist being suspended from the lower flange of a single supporting runway beam (See Figure 5 2). Monorails are used where the need to lift and move items can be confined to one direction.

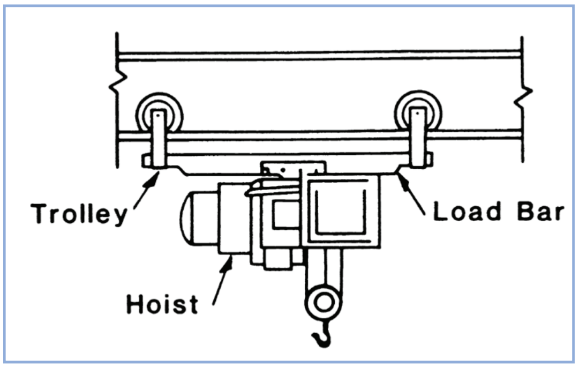


Figure 5 2 Monorail Crane

Top Running Cranes as show in Figure 5 3 are generally used in workshops and warehouses where lifting capacity is required over a large span of the floor area.

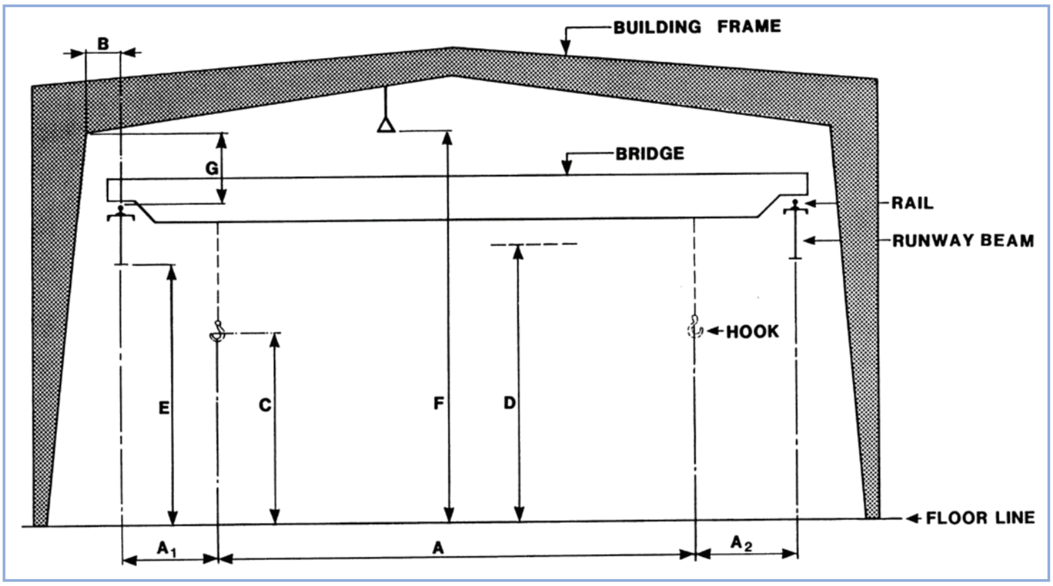
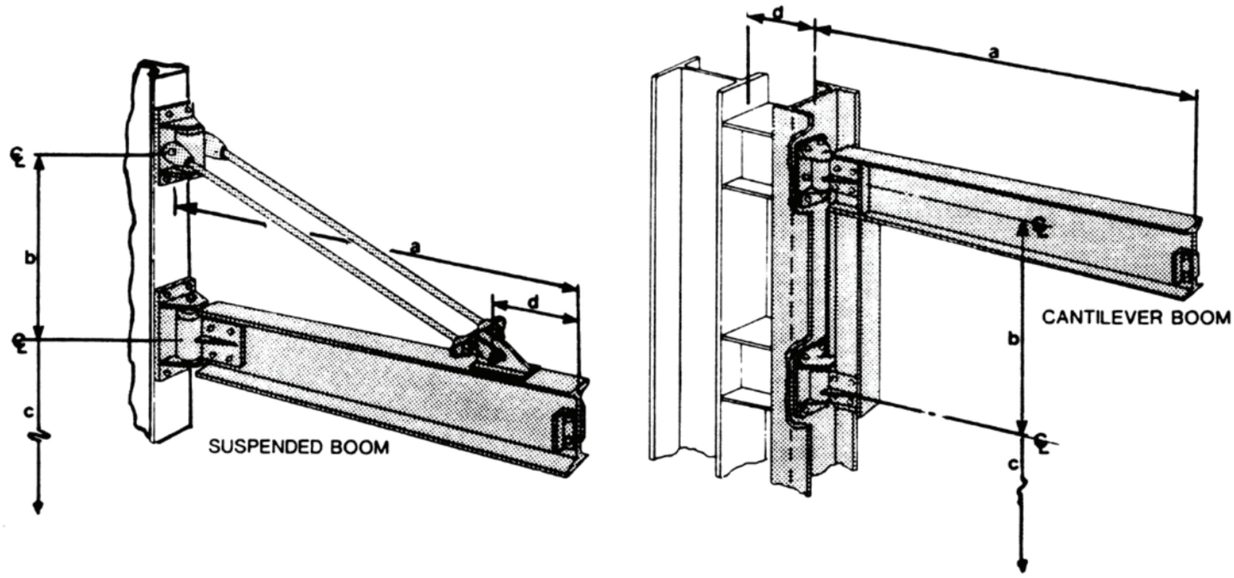


Figure 5 3 Overhead Crane in Design Example

Top Running Cranes usually provide greater hook height and clearance below the crane girder than underhung cranes.

The JIB crane is a crane that has a rotating horizontal boom attached to a fixed.



Column Mounted Jib Crane

Column Mounted Jib Crane with Supplemental Column

Figure 5 4 Jib Crane

Bridge cranes can be designed having either single girder, double girder or box girder.

Single girder is generally used on shorter spans and lower capacities or service classifications. Double girder cranes provide greater hook height, but are no more durable than single girder cranes. However, crane and accessories such as bridge cranes, hoist, and trolley, etc. are provided by the manufacturer, not by BMB&A.

5.2. Design procedure

The clients will usually show their basic requirements in the design brief. Then the designers need to establish various parameters that will influence the structural design of the building.

These may include like below:

(1) Crane type (top running, underhung, etc.);

(2) Capacity (rated in tons) and service classification (CMAA);

(3) Power source: most use electric instead of hand geared.

For electric powered cranes, method of operation (pendant, cab, or radio);

(4) Crane load:

a. The self-weight: Bridge weight (CW) and weight of trolley with hoist (HT);

b. Maximum wheel load without impact;

c. Special allowances for vertical impact, lateral force, longitudinal force, or the loads factored for dynamic effects and lateral loads, if required.

(5) Dimensions:

a. The building layouts

b. Number wheelbase (NWb), distance between cranes (LCr), end-truck length (N), distance wheelbase(W);

c. Level of the top of rail (TOR), the clearance above the top of the rail, Horizontal clearance, vertical clearance, and clearance beneath the runway beam or hook height.

(6) Deflection limits for the crane runway beam and portal frame.

Utilization and state of loading for fatigue assessment.

NOTE

There can be a significant difference in wheel loads and geometry between single and double girder cranes. If the designer cannot establish the make of the crane, then a contingency of 10% load could be added to the load provided by one manufacturer to allow for other make which might be adopted.

The speed of hand-geared cranes is low, and the impact forces which supporting structures may resist are low compared to the faster electric powered cranes.

Once the crane wheel loads and overall geometry have been established, the general design procedure is as given below:

(1) Design the runway beams

(2) Determine the crane load reactions on the bracket and load combinations

(3) Design of main frames.

5.3. Design the runway beam

It is assumed the crane runway beams are simple beams supported at the brackets that cantilever from the main portal columns.

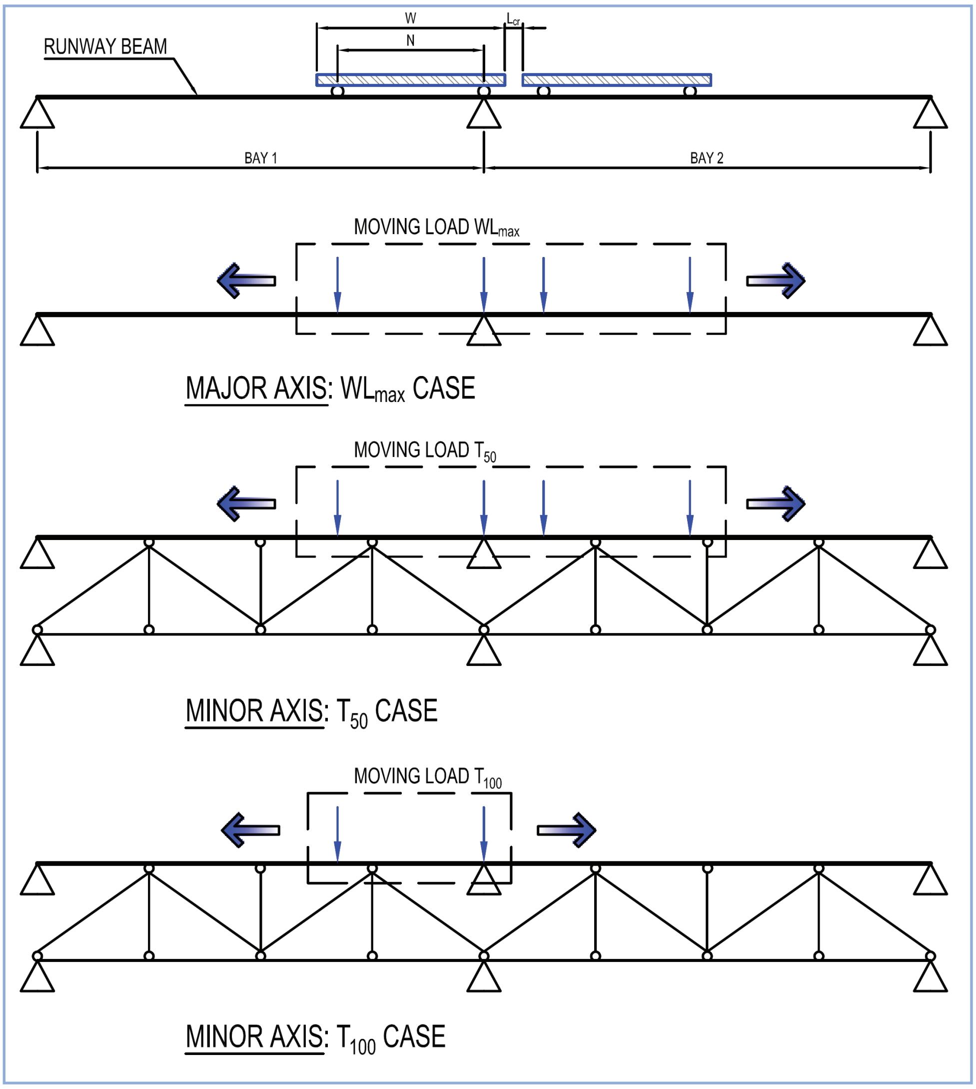


Figure 5 5 Schematics of Crame Runway Beam

NOTE

Bracing

When runway beam depth is less than **900mm**, top flange beam need to be braced by I-260x164x5x6, and V50x50x5 or larger if necessary.

When runway beam depth is not less than 900mm, need to use both top and bottom flange bracing, by I-260x164x5x6, and V50x50x5 or larger if necessary.

Width of runway beam's top flange

When length of building is greater than **50m**, the width of runway beam's top flange need to be greater than **200mm** to avoid errors in rail installation.

5.3.1. Crane Wheel Load

Wheel Load

The maximum wheel load (WL_{max}) and the minimum wheel load (WL_{min}) can be provided by the crane manufacturer or may be conservatively approximated from the crane loads as follows:

$$WL_{max} = \frac{RC + HT + 0.5CW}{NWb}$$	(2012MBSM, Eq. 2.4.1-1)
$$WL_{min} = \frac{0.5CW}{NWb}$$	

where,

WL = Maximum wheel load

RC = Rated capacity of the crane

HT = Weight of hoist with trolley

CW = Weight of the crane excluding the hoist with trolley

NWb = Number of end truck wheels at one end of the bridge

Vertical Impact Force

The maximum wheel load (WL_{max}) used for the design of runway beams, including monorails, their connections and support brackets, shall be increased by the percentage given below to determine the induced vertical impact or vibration force.

Vertical impact shall not be required for the design of frames, support columns, or the building foundation.

Crane Type	%
Monorail cranes (powered)	25
Cab-operated or radio operated bridge cranes (powered)	25
Pendant-operated bridge cranes (powered)	10
Bridge cranes or monorail cranes with hand-geared bridge, trolley and hoist	0

Lateral Force

The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed with due regard to the lateral stiffness of the runway beam and supporting structure.

The lateral force in each end-truck wheel on crane runway beams with electrically powered trolleys shall be calculated as below:

$$T = 20\% \frac{RC + HT}{NWb}$$

Longitudinal Force

The longitudinal force, acting horizontally at the top of the rails and in each direction parallel to each runway beam on crane runway beams, shall be calculated as 10 percent of the maximum wheel loads excluding vertical impact:

$$P = 10\% WL_{max}$$

5.3.2. Design runway beam using SAP 2000

Step-by-step moving-load analysis is initiated through the following process:

- Define Vehicles type through *Define > Moving Loads > Vehicles*:
  VEH-X: defines the maximum wheel load.
  VEH-Y-T50: defines 100% Lateral Forces.
  VEH-Y-T100: defines 50% Lateral Forces.
- Define The Paths through *Define > Moving Loads > Paths*.
  PATH-X: defines the lane in X-axis.
  PATH-Y: defines the lane in Y-axis.
- Define Moving-load Patterns and Cases through *Define > Load Patterns* and *Load Cases*:
  DEAD: defines self-weight of model.
  WLmax: defines moving-load **VEH-X** on **PATH-X** lane.
  T100: defines moving-load **VEH-Y-T100** on **PATH-Y** lane.
  T50: defines moving-load **VEH-Y-T50** on **PATH-Y** lane.
  Set the load-case type to Moving Load, and then specify the vehicles and paths assigned to this moving load, as shown below:

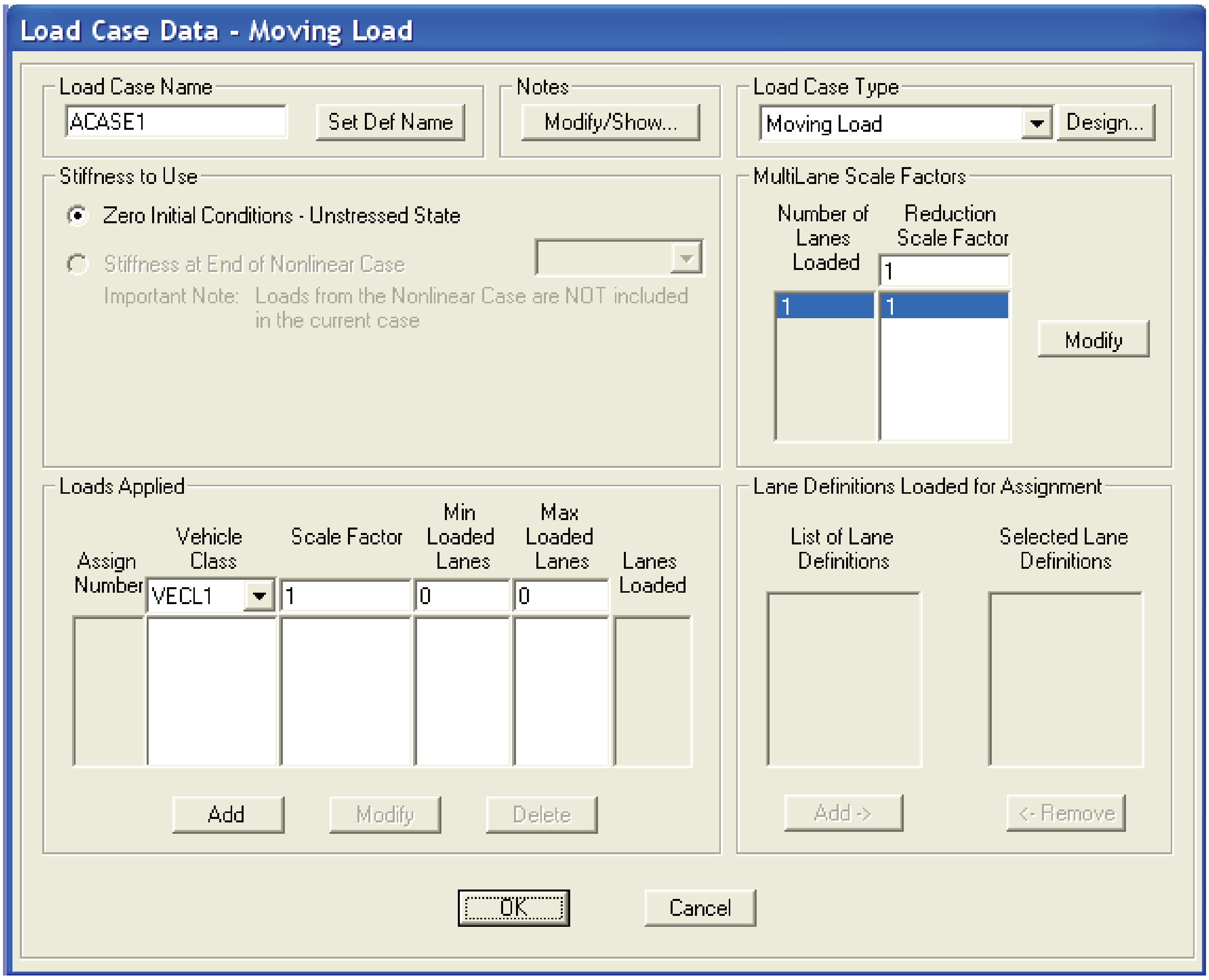


Figure 5 6 Load-case Data

Combinations

Checklist	Combinations	Load case	Output
Strength	M33	DL + WLmax	M33 & M22
	M22-T100	T-100	M22 & V33
	M22-T50	T-50	M22 & V33
Deflection	CV-X	DL + WLmax/α One crane on single aisle: α = Vertical Impact I Other case: α = 1	Δx
	CV-Y-T100	T-100	Δy
	CV-Y-T50	T-50	Δy

Analysis

- Set Analysis Options: *Plane Frame*.
- Sum ratios of runway beam both axis should be less than 1.0.
- Check Deflection as table below:

Table 5 3 Types Deflection Limitations for Top Running Cranes[6]

	DEFORMATION	REMARK
Top Running Cranes		
1/ Runway Beam (vertical deflection)	L/600 (Classes A/B/C) L/800 (Class D) L/1000 (Class E/F)	CV-X
2/ Runway Beam (horizontal deflection)	L/400	CV-Y-T100 & CV-Y-T50
Underhung and Monorail Crane		
Runway Beam (vertical deflection)	L/450 (Classes A/B/C)	CV-X
Crane Bracket (horizontal deflection) 1/ Cab or Radio - Operator cranes 2/ Pendant - Operator cranes	**DL + CR or WL 10yr.** H/240 or ≤ 5.08cm H/100	Crane lateral or WL 10yr.

Output

Using BMB's calculation sheet presents the images and data of the runway-beam design.

5.4. Design Steel Frame

5.4.1. General

The general design main frame procedure is as given below:

1. Determine the load reactions on the bracket.

2. Add the crane runway beam dead load to the dead load and add the following new load case:

a. Crane loads with the maximum wheel load at left column.

Lateral crane loads with maximum at left column and acting from right to left.

b. Crane loads with maximum load at right column.

Lateral crane loads with maximum at right column and acting from left to right

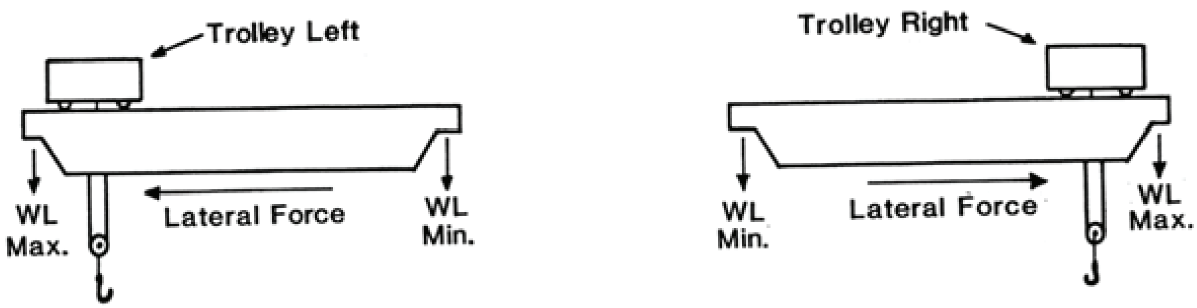
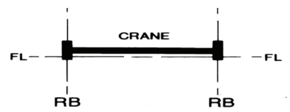
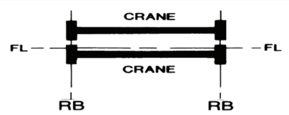
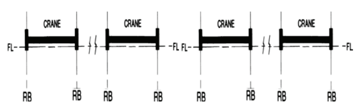
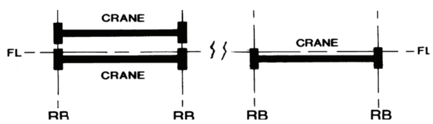


Figure 5 7 Crane Loading Conditions

Crane buildings have single or multiple cranes acting in one or more aisles shall be designed with the crane or cranes located longitudinally in the aisle or aisles in the positions that produce the most unfavorable effect. Unbalanced loads shall be applied as induced by a single crane operating in a crane aisle, and by a crane or cranes operating in one crane aisle of a building with multiple crane aisles. See Table 5 4 for a summary of these provisions.

Table 5 4 Loading for Building Frames and Support Columns

		Schematic	Vertical Impact	Lateral Force
Multiple aisles with multiple cranes	Any one crane in any aisle		0%	100%
	Any two adjacent cranes in any aisle		0% Both cranes	50% Both cranes, or 100% Either crane
	Any one crane in any two adjacent aisles		0% Both cranes	50% Both cranes, or 100% Either crane
	Any two adjacent cranes in any aisle and one crane in any other nonadjacent aisle		0% All cranes	50% All three cranes, or 100% Any one crane

5.4.2. Bracket System

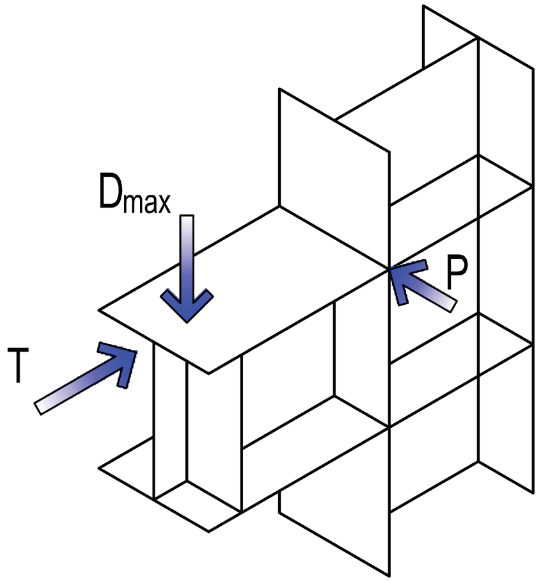


Figure 5 8 Loading on bracket

5.4.3. Bracing

The lateral load on a crane runway beam is transmitted to the main frame column by the bracing systems in the minor axis.

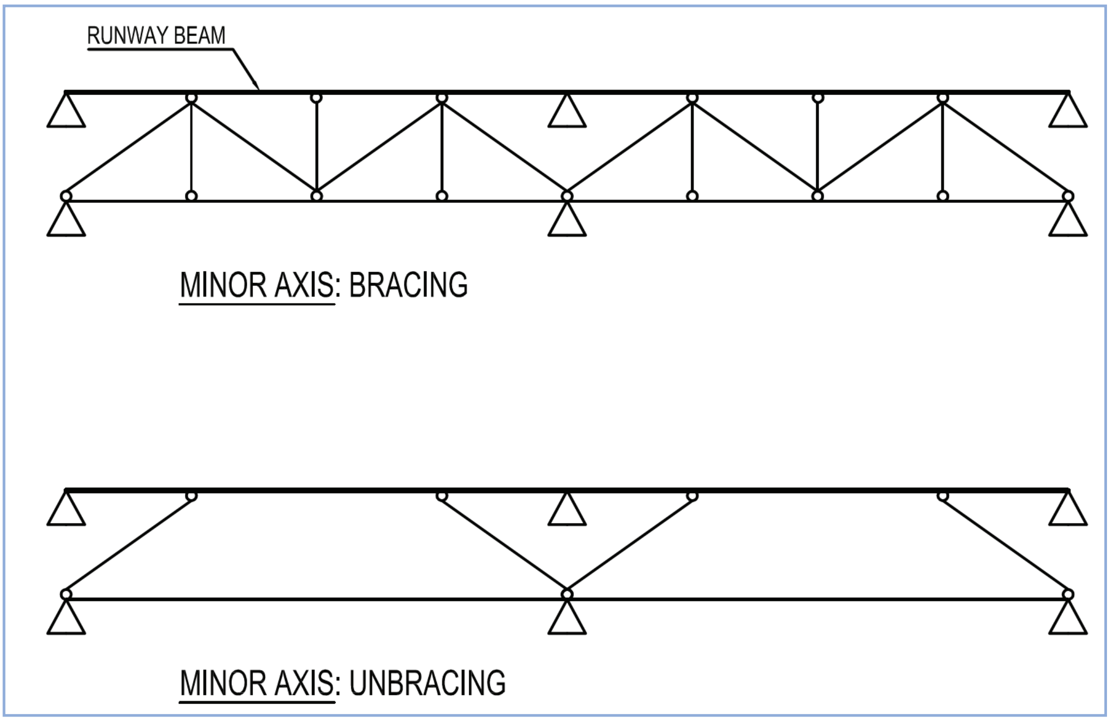


Figure 5 9 Bracing of minor axis

5.5. Design example - One Crane per Aisle

5.5.1. Input

Bay spacing	B = 7.50 m
Crane Data	5 Tons Top Running Crane, 13.72 m span, Class C, top of rail 5.75 m
Bridge weight	CW = 2.79 Tons
Hoist and trolley weight	HT = 0.5 Tons
Service classification	Class C, Radio Operation
2 End-truck wheel	End-truck length N = 2.184 m, Distance Wheelbase W = 1.829 m

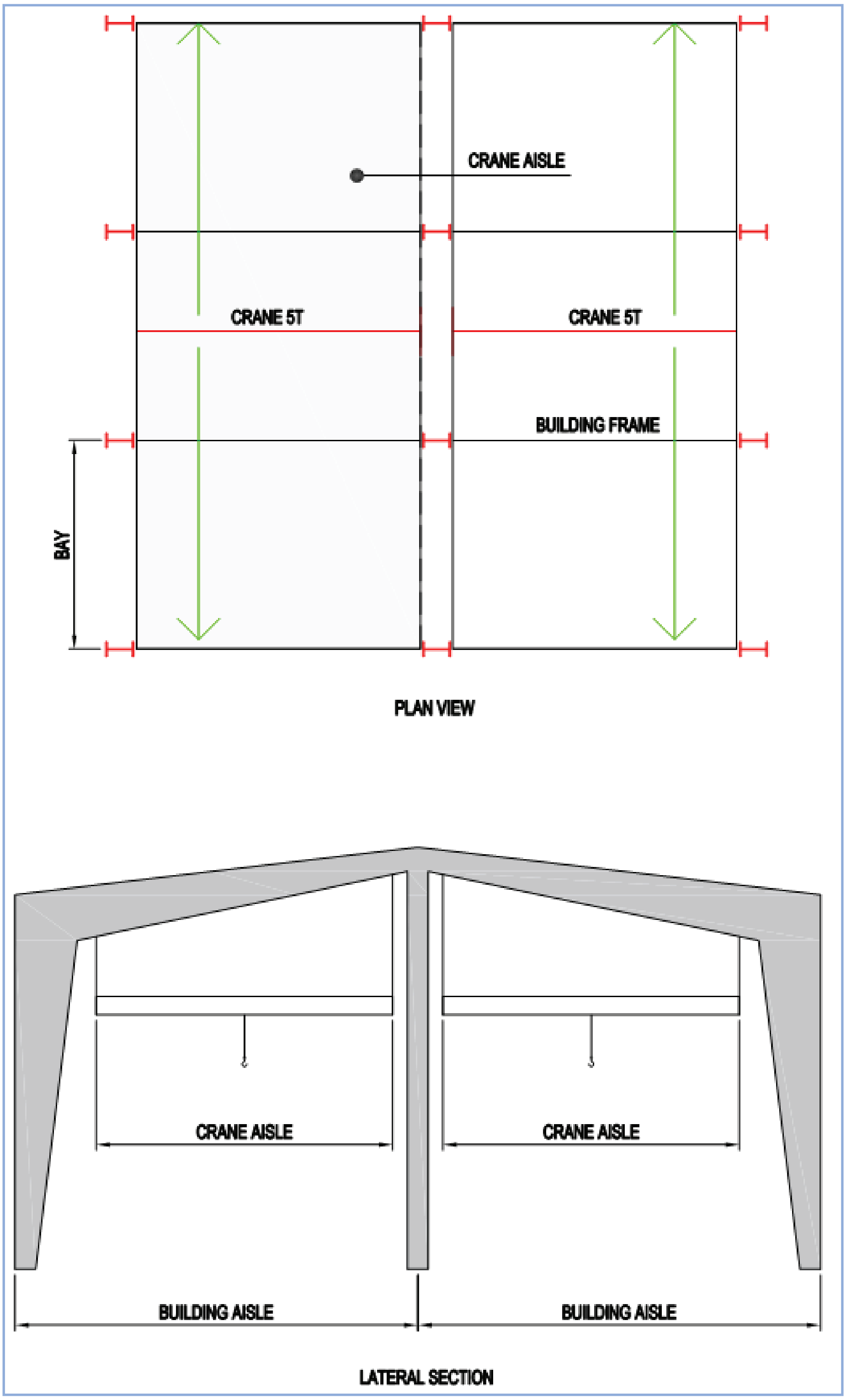


Figure 5 10 Building Layout

5.5.2. Crane Load

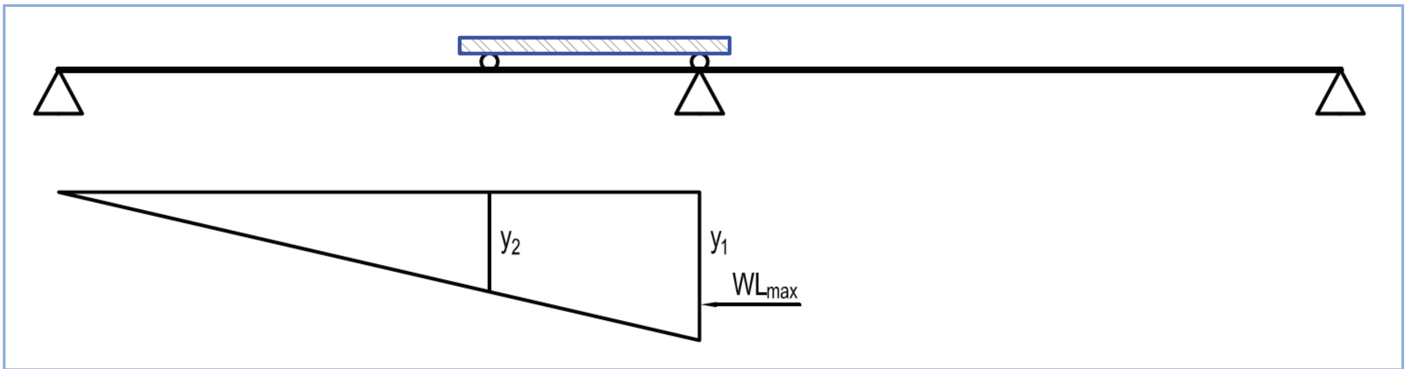
Crane Wheel Load

Crane Wheel Load	$WL\max = (5+0.5+0.5\times 2.79)/2\times 10$	34.5 (kN)
	$WL\min = 2.79/(2\times 2)\times 10$	7.0 (kN)
Lateral Force	$T = 20\%(5+0.5)/2\times 10$	5.5 (kN)
Longitudinal Force	$P = 10\%\times 34.5$	3.45 (kN)

Loading for Runway Beam

Vertical impact	I (Radio Operation)	1.25
Maximum vertical wheel load	$WL\max\times I = 34.5\times 1.25$	43.1 (kN)
100% Lateral Force	$T100 = T = 5.5$	5.5 (kN)
Longitudinal Force	$P = 0.1\times WL\max\times NWb \ = 0.1\times 34.5\times 2$	6.9 (kN)

Loading for Building Columns



Vertical load on column near trolley	$D\max = WL\max\times(1+(B-W)/B)$	60.6 (kN)
	$= 34.5\times(1+(7.5-1.829)/7.5)$	
Vertical load on column away from trolley	$D\min = WL\min\times(1+(B-W)/B)$ $= 7.0\times(1+(7.5-1.829)/7.5)$	12.3 (kN)
Lateral load on building columns	$T\max = T\times(1+(B-W)/B)$ $= 5.5\times(1+(7.5-1.829)/7.5)$	9.66 (kN)
Longitudinal load on building columns	$P = 0.1\times WL\max\times NWb$	6.9 (kN)

5.5.3. Runway Beam Modeling

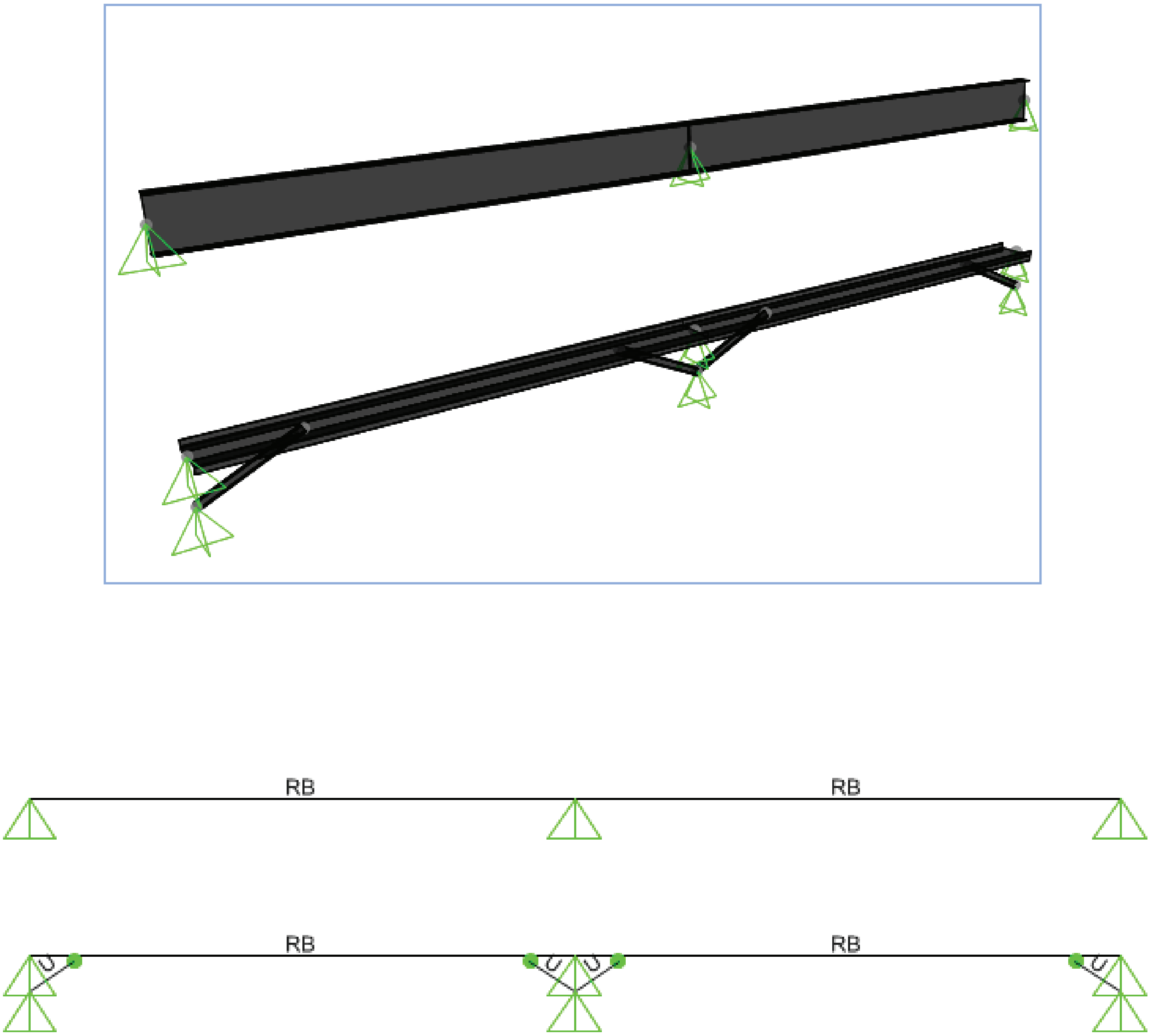
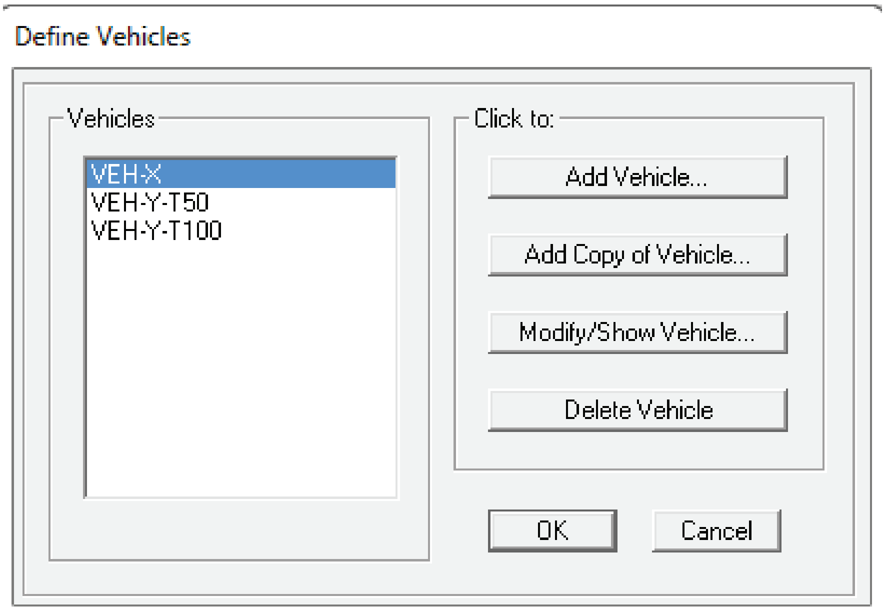
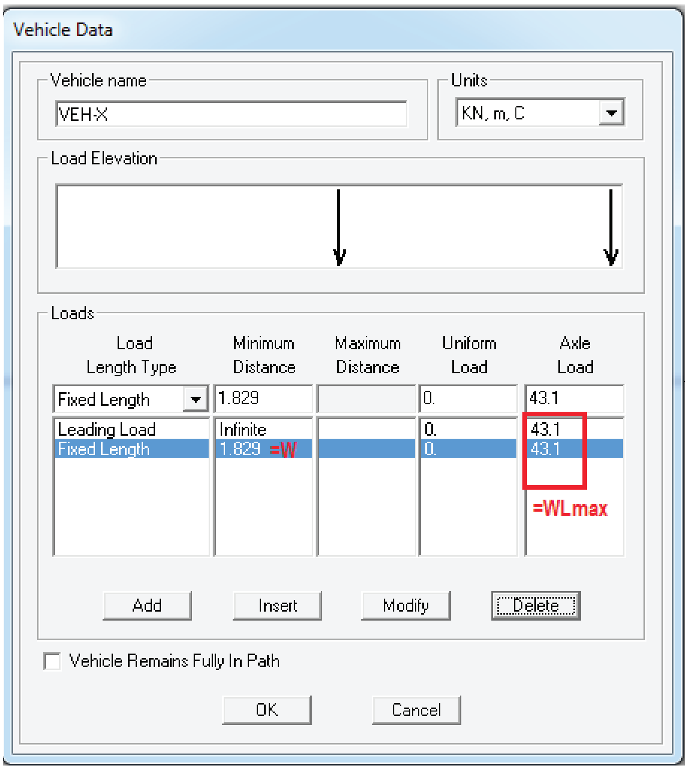


Figure 5 11 Runway Beam in SAP2000 Modeling
Section: RB (I – 766x184x8x10), U (U 120x52x4.8x7.8)

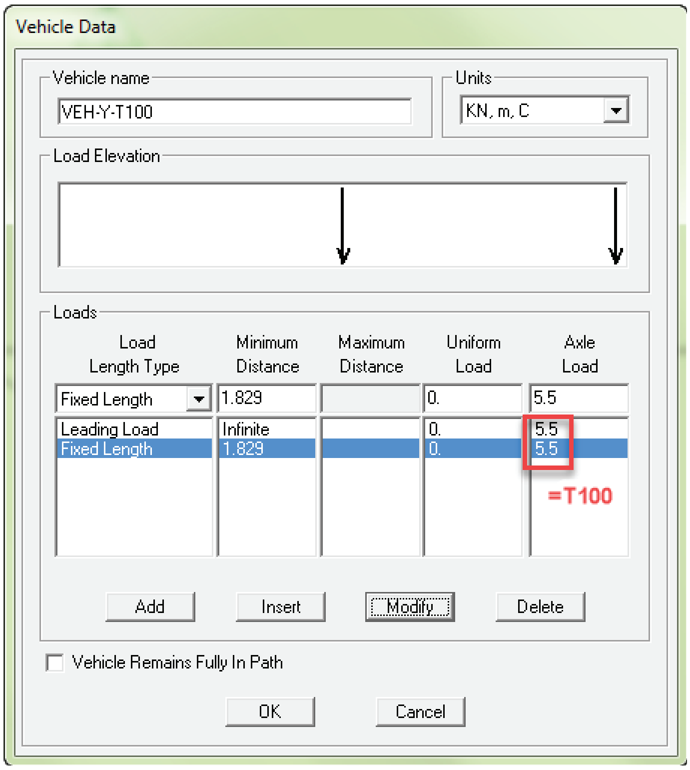
Define Vehicles type



(a) Define Vehicles



(b) Vehicles "VEH-X" defined for WLmax



(c) Vehicles "VEH- Y-T100" defined for T100

Figure 5 12 Define Vehicles

Define Paths

*Figure 5 13 Define Paths of Moving Loads **

PATH-X defined the path on the Major-axis (beams labeled 1 and 2)

PATH-Y defined the path on the Minor-axis (beams labeled 3 and 4)

NOTE

The **discretization** is as same as output-station spacing of frame. The effect of refining path discretization is apparent in the influence lines which follow (Figure 5 14):

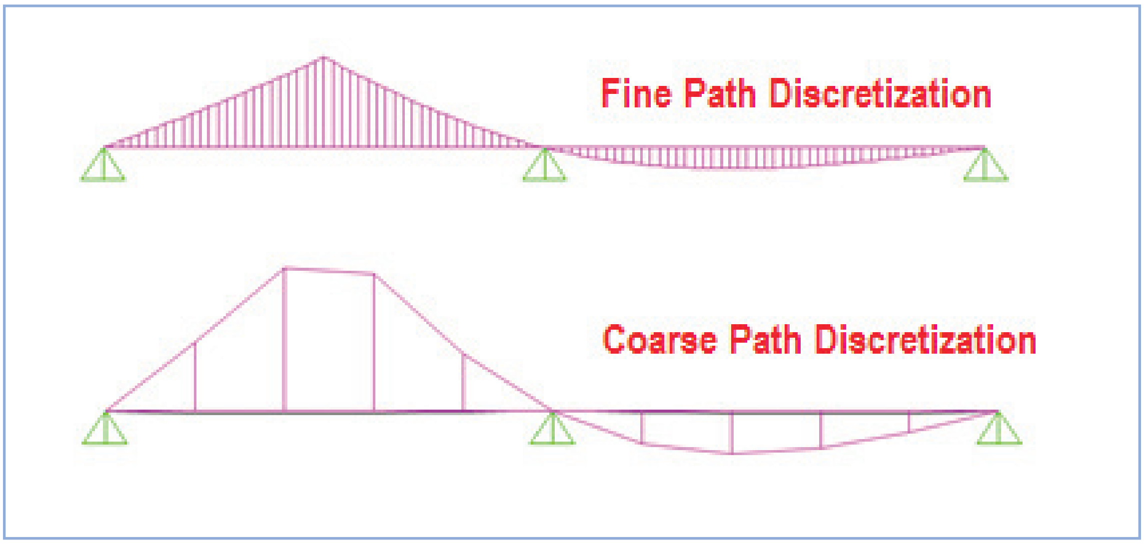
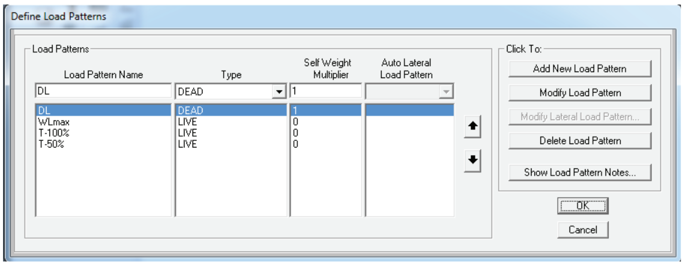
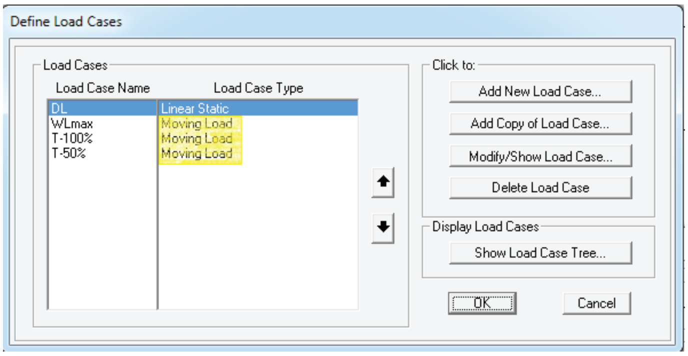


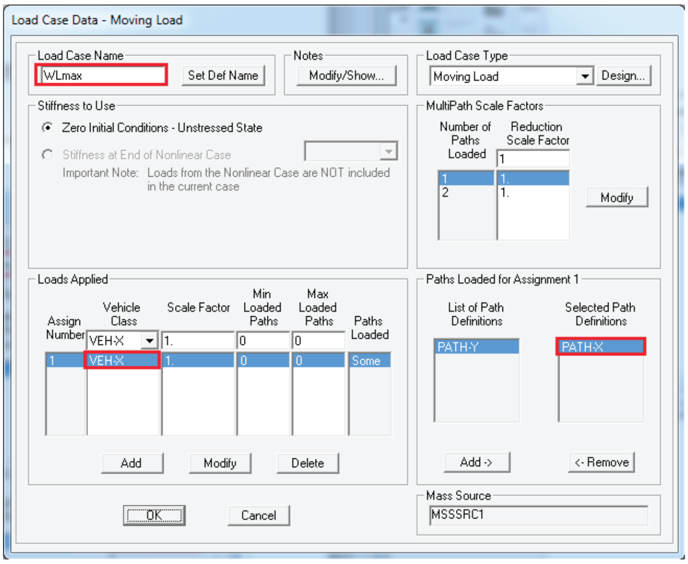
Figure 5 14 Influence line



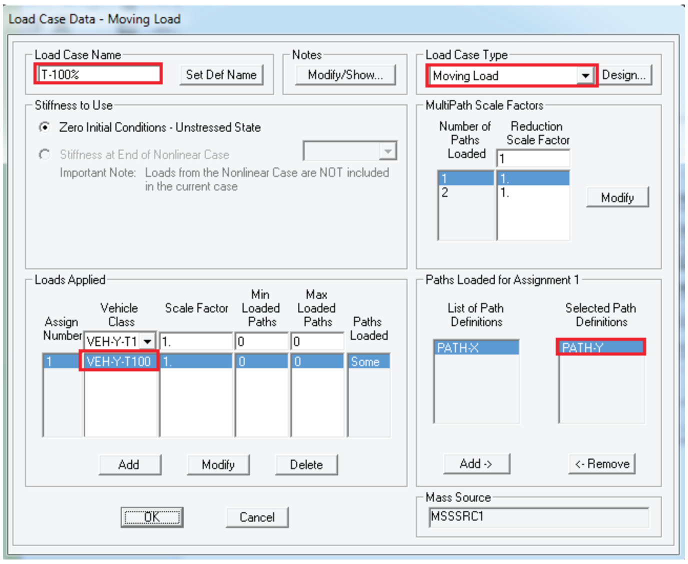
Define Load-Patterns



Define Load-Case



Load-case WLmax Data



Load-case T100 Data

Output

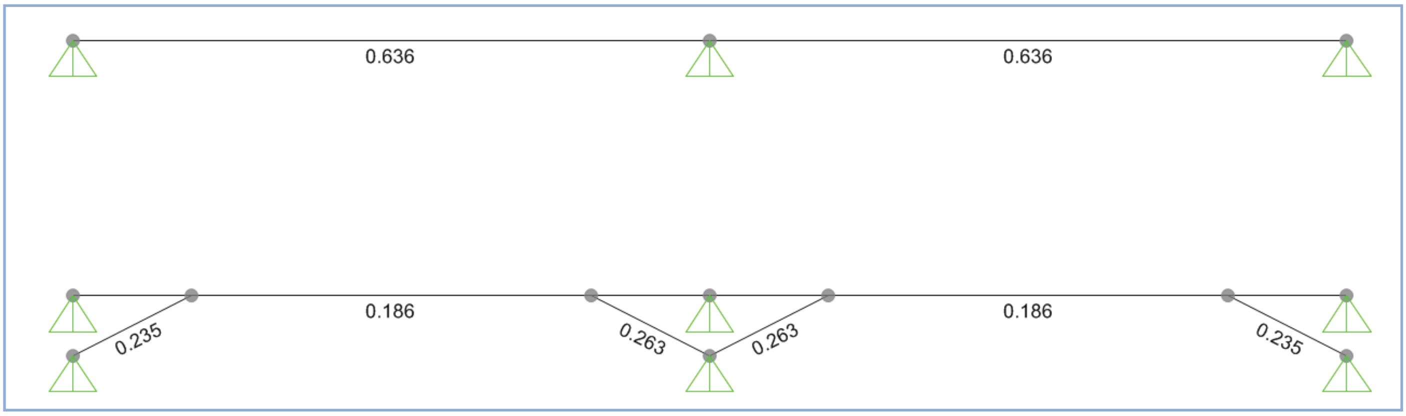


*Figure 5 15 Steel P-M Interaction Ratios of Runway Beam**

NOTE

This ratio is only used for reference, since the SAP model does not include the longitudinal force. The total ratio of the major and minor axis must be less than **1.00**.

Check Runway Beam by Calculation Sheet

I. INPUT DATA

1. Material

Steel		A572 Grade 50
Yield strength of steel	F_y =	345 MPa
Modulus of elasticity of steel	E =	200000 MPa

2. Section

Section of runway beam

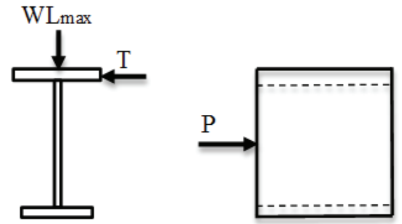
Section	Height	Web thickness	Compression Flange		Tension Flange	
Built-up member	d (mm)	t_w (mm)	b_1 (mm)	t_{f1} (mm)	b_2 (mm)	t_{f2} (mm)
I - 766x184x8x10	766	8	184	10	184	10

Factor

Length L_b (mm)	Length L (mm)	Factor K	Stiffeners @a (mm)	Factor C_b
4700	7500	1	2000	1.088

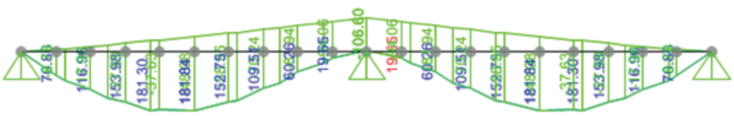
II. FORCE

Loading for runway beam			Crane 1
Maximum vertical wheel load	WL_{max} =	kN	43.11
100% side thrust in each end-truck wheel max	T=	kN	6.88
Longitudinal force	P =	kN	6.90
Vertical impact	I		1.25



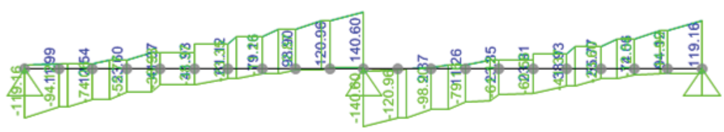
Force of runway beam				
Axial force	Moment		Shear force	
P (kN)	M_{33} (kNm)	M_{22} (kNm)	V_{22} (kN)	V_{33} (kN)
-6.90	181.84	5.76	140.6	10.95

Resultant Moment M33, kNm

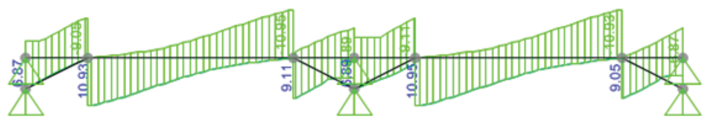


Resultant Moment M22, kNm

Resultant Shear V22, kN



Resultant Shear V33, kN



NOTE

*Length Lb: the largest un-bracing length of runway beam

*Factor Cb: get from Detail of Steel Stress Check Data

III. CHECK OF MENBERS FOR STRENGTH CAPACITY

1. CHECK OF RUNWAY BEAM FOR STRENGTH CAPACITY

a. Design of runway beam for Flexure

Design of runway beam for flexure - major axis

M_{cx} = 228.48 kNm

$M_{cx/No\ LTB}$ = 432.92 kNm *[Chapter F5]*

Design of runway beam for flexure - minor axis

M_{cy} = 37.29 kNm *[Chapter F6]*

b. Design of runway beam for Compression

P_c = 488.71 kN *[Chapter E]*

c. Design of runway beam for Shear

Design of runway beam for shear - major axis

V_{cx} = 435.52 kN *[Chapter G2]*

Design of runway beam for shear - minor axis

V_{cy} = 456.14 kN *[Chapter G7]*

d. Design of runway beam for Combined Forces and Torsion

Design of runway beam for combined flexure and axial force

AXL	B-MAJ	B-MIN	D/C ratio
Pr/Pc	(M33/Mcx)^2	M22/Mcy	H1.3b,H1-2
0.014	**0.633**	**0.154**	**0.802**

[Satisfactory] *[Chapter H]*

Design of runway beam for Shear

$$\frac{V_r}{V_{cy}} = \frac{V_{22}}{V_{cy}} = \quad \mathbf{0.323}$$ [Satisfactory]

$$\frac{V_r}{V_{cx}} = \frac{V_{33}}{V_{cx}} = \quad \mathbf{0.024}$$ [Satisfactory]

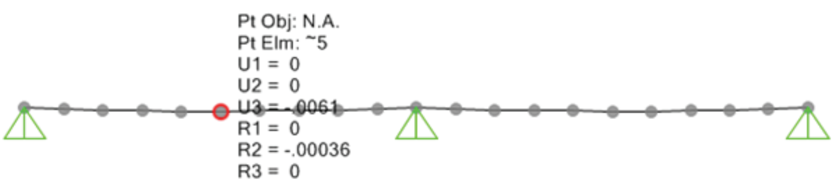
2. CHECK OF BRACING FOR STRENGTH CAPACITY

Design of bracing for Compression

Pc = 41.2 kN [Satisfactory] *[Chapter E5]*

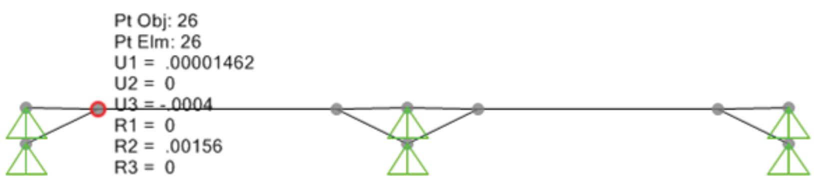
III. CHECK DEFORMATION

1. Vertical Defection



Δ_x = 6.100 mm < Δ_{limit} = L/600 = 12.5 mm [Satisfactory]

2. Horizontal Defection



Δ_x = 0.400 mm < Δ_{limit} = L/400 = 18.75 mm [Satisfactory]

Check Bolt Connection between Runway Beams to the Bracket

I. INPUT DATA

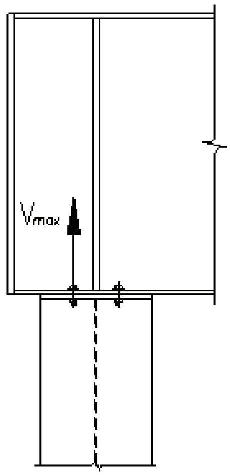
1. Bolt

Grade		10.9	
Type		M24	
Ultimate strength of bolt	F_u =	100	kN/cm^2
Ultimate tensile strength	$F_{nt} = 0.75\ F_u$ =	75	kN/cm^2
Shear strength	$F_{nv} = 0.45\ F_u$ =	45	kN/cm^2
No. of bolt	n =	4	

(Table J3.2)

2. Force

Reaction at support V_{max} = 75.7 kN



II. CALCULATING

Bolt resisting tension

Tensile resistance of bolt

$$\frac{R_{nt}}{\Omega} = \frac{F_{nt} \times A_b}{\Omega} = 169.65\ \text{kN}$$

(J3-1)

Where: Ω = 2

Max. Tension in outermost bolt

$$N_{blM} = \frac{V_{max}}{m_r} = 37.9\ \text{kN}$$

Where: mr is no. of bolt on 1 row

Total of tension on 1 bolt

T_b = 37.9 kN

Hence: $\frac{R_{nt}}{\Omega} \geq T_b$ OK

Check Welding at Bracket

I. MATERIAL

1. Bracket

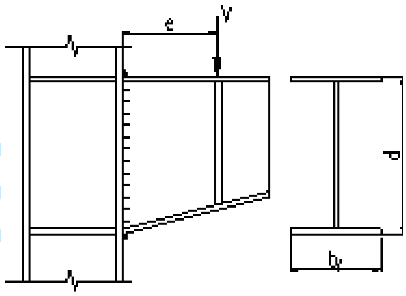
ASTM specifications for Steel		A572 Grade 50	
Yeild strength	F_y =	345	MPa

2. Weld

		E60XX	
Strength of the weld	F_{EXX} =	414	MPa
Height of the fillet vertical weld	h_v =	5	mm
Height of the fillet horizontal weld	h_h =	7	mm
Total length of vertical weld	l_v =	900	mm
Total length of horizontal weld	l_h =	800	mm

II. INTERNAL FORCE FOR CALCULATING

Shear	V =	75.70	kN



III. DIMENSION

Web height of bracket	d =	500	mm
Width flange	b_f =	212	mm
Distance	e =	300	mm

IV. CALCULATING

Eccentric moment $M = V \times e =$ 22.7101 kNm

Allowable shear stress of the weld

$$\frac{R_n}{\Omega} = \frac{F_{nw} \times 1m^2}{\Omega} = \quad 103{,}500 \text{ kN/m}^2$$

Where: $F_{nw} = 0.6F_{EXX} =$ 207000 kN/m^2 (J2-5)

$\Omega =$ 2

Weld area

$A_{we} =$ 0.006 m^2

Moment of inertia of the weld

$I_w =$ 0.000308 m^4

Section modulus

$$W_w = \frac{2I_w}{(d+h)} = \quad 0.0012 \text{ m}^3$$

Checking stress of the weld

$$\tau = \sqrt{\tau_M^2 + \tau_V^2} = \sqrt{\left(\frac{M}{W_w}\right)^2 + \left(\frac{V}{A_{we}}\right)^2} = \quad 22469 \text{ kN/m}^2$$

Hence: $\tau \leq \frac{R_n}{\Omega}$ OK

Check the weld length to weld size ratio

$l_w/h =$ 90.00 < 100

$\beta = 1.2 - 0.002(l_w/h) =$ 1.02 > 1 (J2-1)

$\rightarrow \beta =$ 1.00

The capacity of the weld

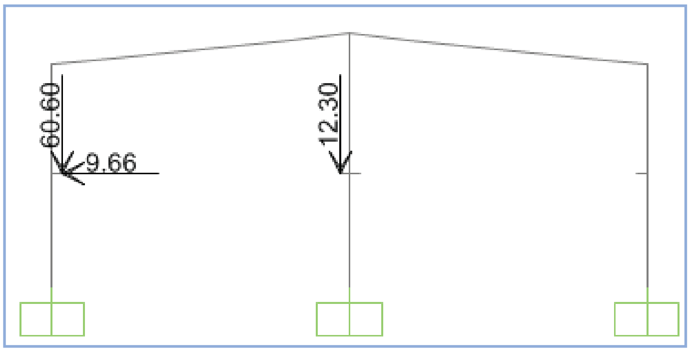
$$\frac{R_{n1}}{\Omega} = \frac{\beta R_n}{\Omega} = \quad 103{,}500 \text{ kN/m}^2$$

Hence: $R_{n1}/\Omega \geq \tau$ OK

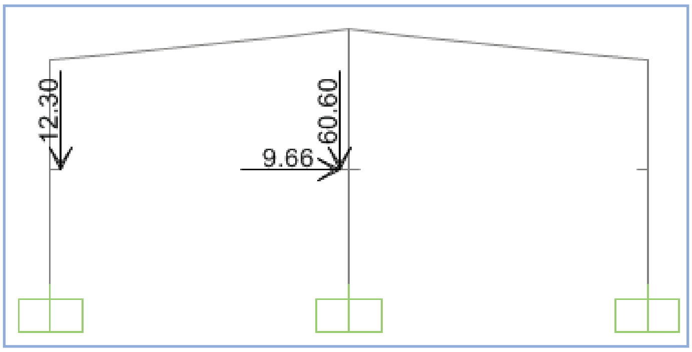
5.5.4. Main Frame

The load frame

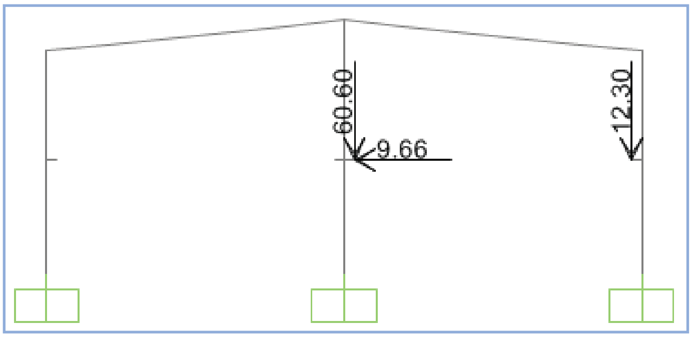
Columns supporting crane fixed about the major axes and pinned about minor axes.



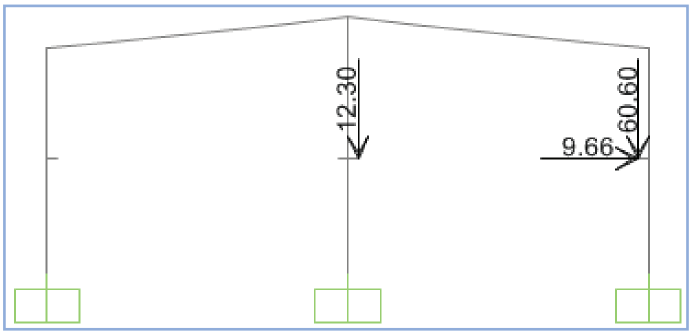
CR1 Case



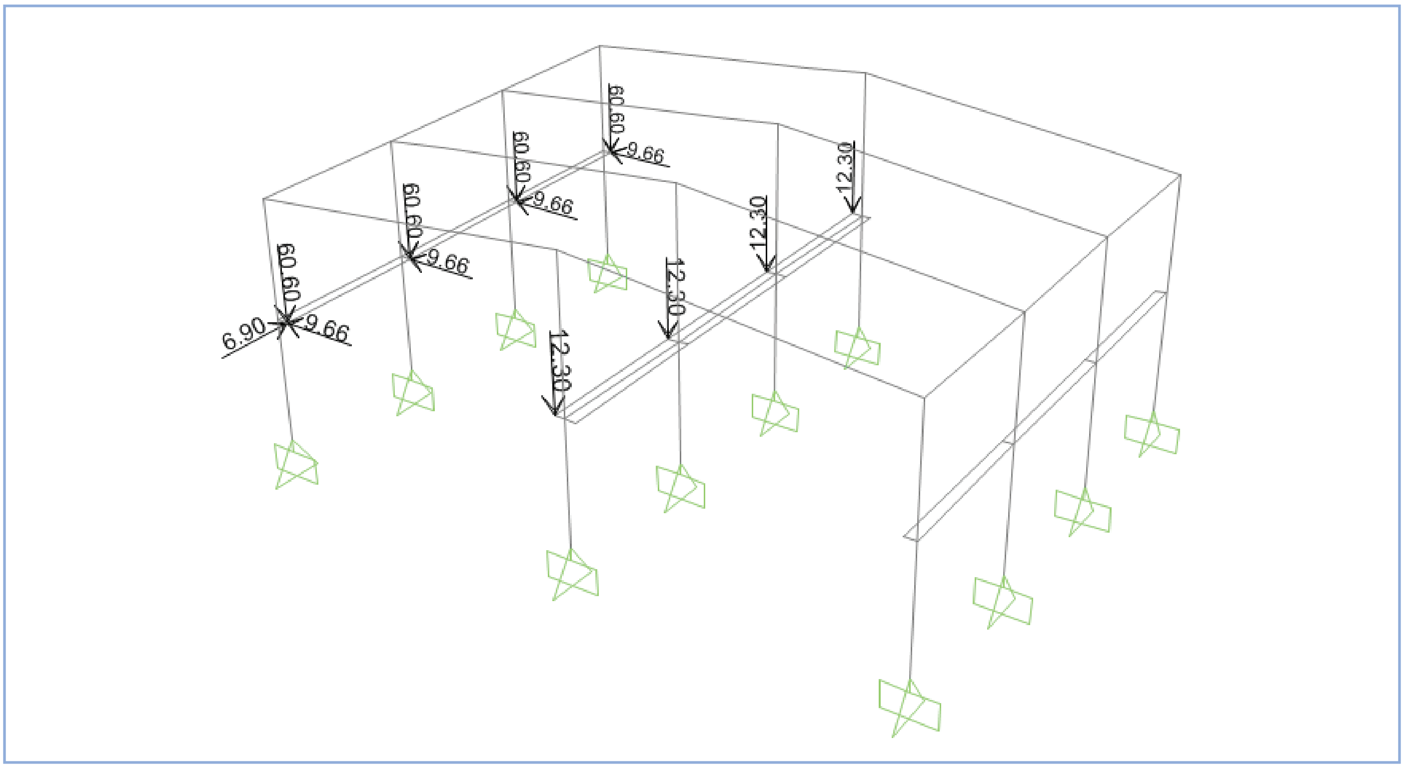
CR2 Case



CR3 Caset



CR4 Case



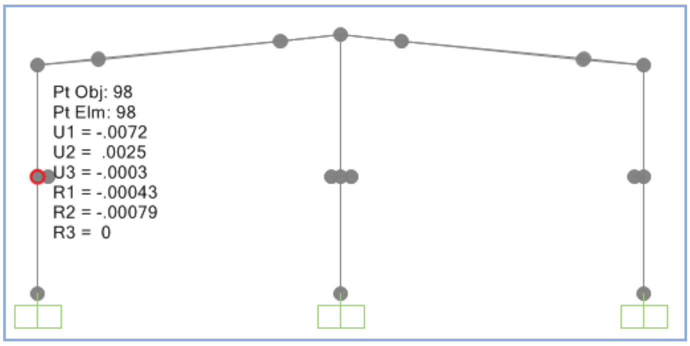
Typical Longitudinal Force for "CR1 Case"

The crane runway beam and rail are to be included as dead loads, applied at the bracket. Note that for clarity, dead loads (DL), live roof loads (LL), wind loads (WL) are not shown in this example.

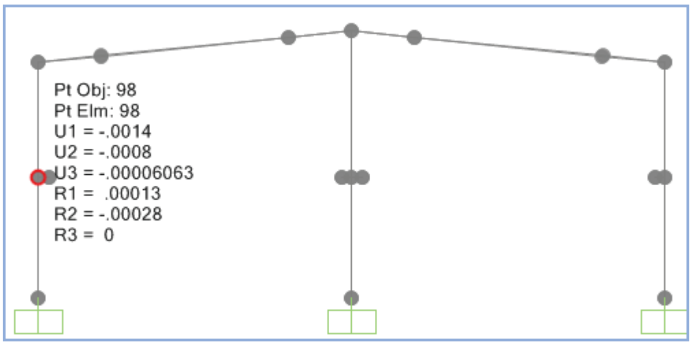
The combinations

COMBO 1	DL + LL	COMBO 16	DL + 0.75CR3 + 0.45WL1
COMBO 2	DL + 0.6WL1	COMBO 17	DL + 0.75CR4 + 0.45WL1
COMBO 3	DL + 0.6WL2	COMBO 18	DL + 0.75CR1 + 0.75CR3 + 0.45WL1
COMBO 4	DL + 0.75LL + 0.45WL1	COMBO 19	DL + 0.75CR1 + 0.75CR4 + 0.45WL1
COMBO 5	DL + 0.75LL + 0.45WL2	COMBO 20	DL + 0.75CR2 + 0.75CR3 + 0.45WL1
COMBO 6	DL + CR1	COMBO 21	DL + 0.75CR2 + 0.75CR4 + 0.45WL1
COMBO 7	DL + CR2	COMBO 22	DL + 0.75CR1 + 0.45WL2
COMBO 8	DL + CR3	COMBO 23	DL + 0.75CR2 + 0.45WL2
COMBO 9	DL + CR4	COMBO 24	DL + 0.75CR3 + 0.45WL2
COMBO 10	DL + CR1 + CR3	COMBO 25	DL + 0.75CR4 + 0.45WL2
COMBO 11	DL + CR1 + CR4	COMBO 26	DL + 0.75CR1 + 0.75CR3 + 0.45WL2
COMBO 12	DL + CR2 + CR3	COMBO 27	DL + 0.75CR1 + 0.75CR4 + 0.45WL2
COMBO 13	DL + CR2 + CR4	COMBO 28	DL + 0.75CR2 + 0.75CR3 + 0.45WL2
COMBO 14	DL + 0.75CR1 + 0.45WL1	COMBO 29	DL + 0.75CR2 + 0.75CR4 + 0.45WL2
COMBO 15	DL + 0.75CR2 + 0.45WL1	COMBO 30	0.6DL + 0.6WL1
		COMBO 31	0.6DL + 0.6WL2

Deflection Checking



Case 'DL+CR+WL 10yr.': Y=0.72cm



Case 'DL + WL 10yr.': Y=0.14cm

Figure 5 16 Crane Bracket Horizontal Deflection

$$[Y] = \min[5.08cm; (575/240)] = 5.08cm$$

Steel stress check

NOTE

When check steel column need to overwrite Factor "Unbraced Length Ratio (Major)" to 1.00 as Figure 5 17.

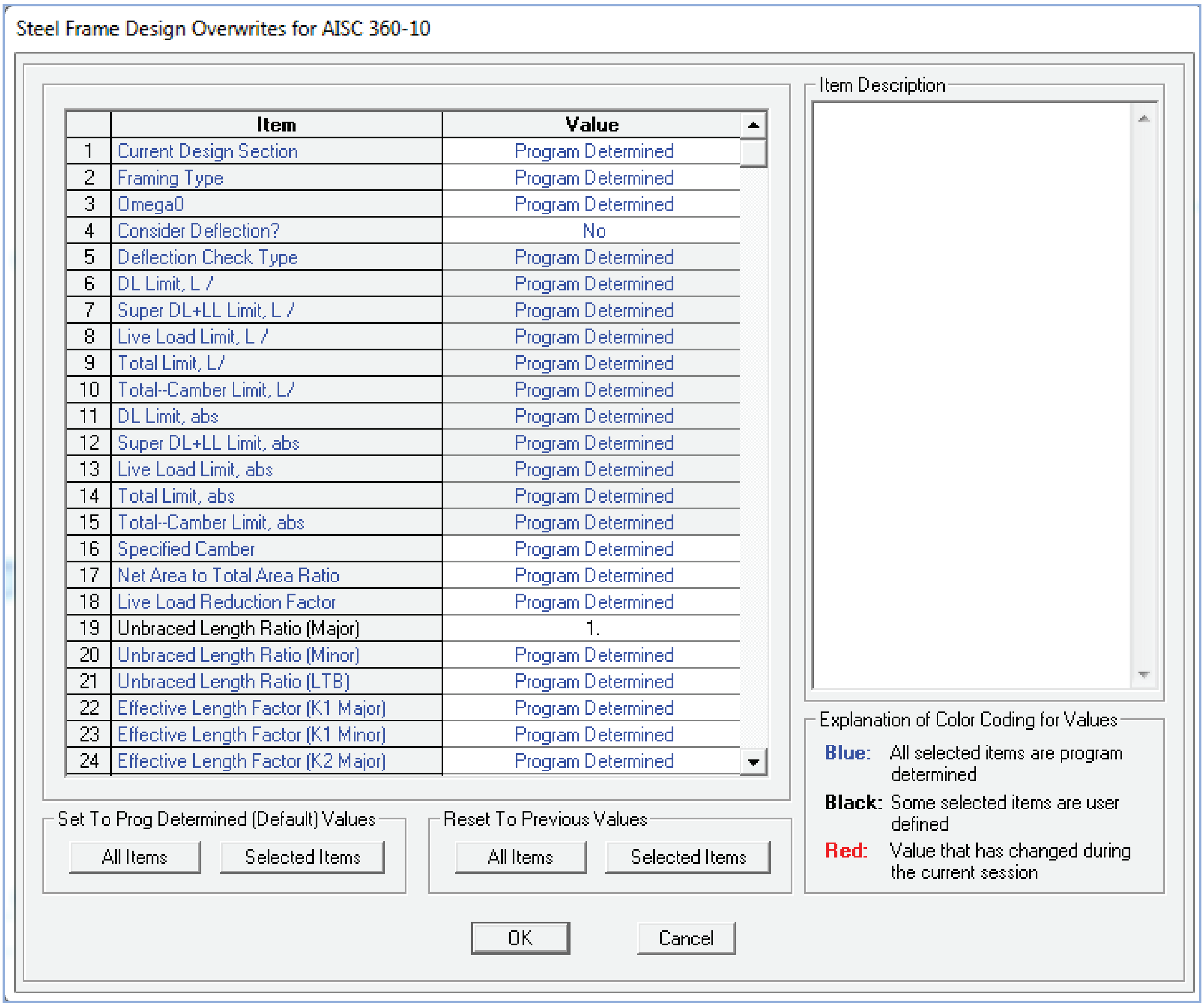


Figure 5 17 Define ratio for steel stress check

CHAPTER 6. MEZZANINE FLOOR DESIGN

6.1. General

Mezzanines and platforms are often required in industrial buildings. The type of usage dictates design considerations. For proper design the designer needs to consider the following design parameters:

1. **Occupancy or Use**.
2. **Design Loads** (Uniform and Concentrated).
   The dead load includes the weight of panel, concrete slab, finish floor and, wall on floor, and self-weight of joist.
   The live load depends on the purpose of the floor. Refer Chapter 2 for more details.
3. **Type of slab**: composite slab, checkered plate, gratings, expanded metals, etc.
   Each type of floor requires different design of joists.
   Mezzanine joists are analyzed and designed as simple span members.
   Joist beam spacing needs being less than **2000mm** and depend on bay spacing for equal distances.
4. **Stair, Opening, Guard rail requirements**.
5. **Design Criteria** (if required): Deflection limitation, Vibration Control, Lateral Stability Requirements.
6. **Future Expansion**.

Design Procedure

1. Design of Joists:
2. Design of Flooring
3. Design of Main Frames

6.2. Composite Slab

Composite Metal Deck Slabs – most commonly used today. Advantages:

- Stay in place form.
- Slab shoring typically not required.
- Metal deck serves as positive reinforcement.
- Metal deck serves as construction platform.

Shear connector can use Steel Headed Stud or Steel Channel Anchors.

Steel headed stud anchors shall be welded through the deck to the steel cross section. Such anchorage shall be provided by steel headed stud anchors, a combination of steel headed stud anchors and arc spot (puddle) welds, or other devices specified by the contract documents.

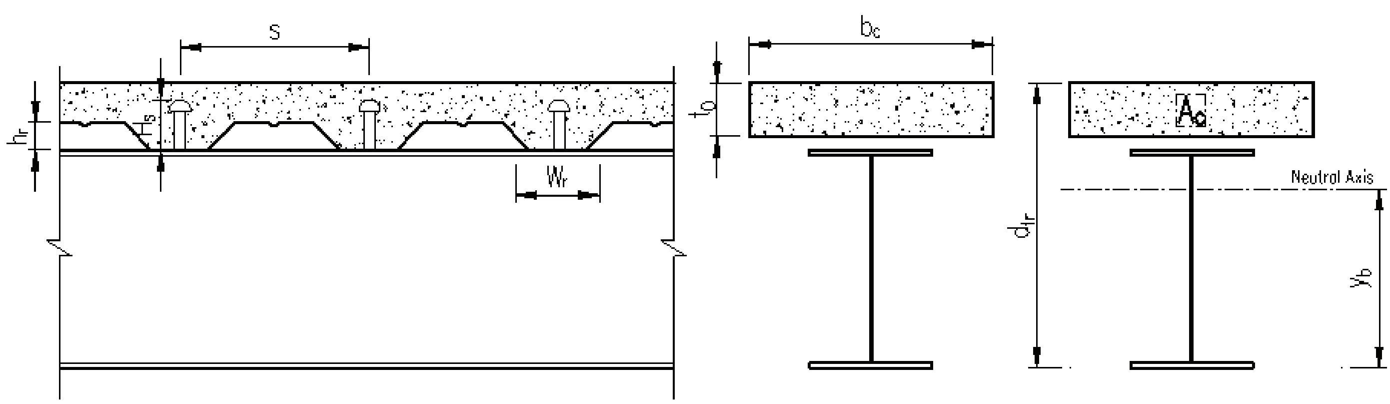


Figure 6 1 Composite Beam Dimensions with Ribs Perpendicular to Beam Span

6.2.1. Section Properties

The section properties are reported with respect to the section local axes (2-3). Furthermore the section properties are reported assuming that the entire section is transformed into an equivalent area of the specified base material.

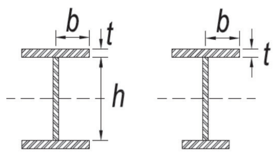
Cross-section area	$A = 2 \times bf \times tf + tw \times hw$
Moment of Inertia	$Ix = \sum \left(bh^3 / 12 \right) + \sum A y_i^2 = \frac{t_w \times h_w^3}{12} + bf \times \frac{h^3 - h_w^3}{12}$
Plastic Section Modulus	$Zx = \sum A_i y_i = 2 \times bf \times tf \times \frac{H - tf}{2} + tw \times \frac{hw^2}{4}$
Elastic Section modulus	$Sx = \frac{Ix}{H / 2}$
Radius of Gyration	$rx = \sqrt{Ix / A}$
Center of gravity	$x_C = \frac{\sum F_i x_i}{\sum F_i}; y_C = \frac{\sum F_i y_i}{\sum F_i}$

Compact and Non-compact Requirements

For flexure, sections are classified as compact, non-compact or slender-element sections.

Width-to-Thickness Ratios of I-shaped built-up sections are shown as below:

Ratio: b / t

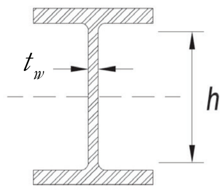


Limiting Width-to-Thickness Ratios for Flanges[1]

Compact Section Limits: $b / t \le \lambda_p = 0.38\sqrt{E / Fy}$

Noncompact Section Limits: $b / t \le \lambda_r = 0.95\sqrt{k_c E / F_L}$

Ratio: h / t_w

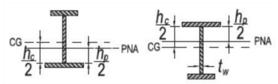


Limiting Ratios for Webs of Doubly-symmetric I-shaped sections[2]

Compact Section Limits: $h / tw \le \lambda_p = 3.76\sqrt{E / Fy}$

Noncompact Section Limits: $h / tw \le \lambda_r = 5.70\sqrt{E / Fy}$

Ratio: $h_c \quad t_w$



Limiting Ratios for Webs of Singly-symmetric I-shaped sections[3]

Compact Section Limits:

$$\frac{h}{t_w} \le \frac{\frac{h_c}{h_p}\sqrt{E / F_y}}{\left(0.54 M_p / M_y - 0.09\right)^2} \le 5.70\sqrt{E / F_y}$$

Noncompact Section Limits: $h / tw \le \lambda_r = 5.70\sqrt{E / Fy}$

where:

$Fy = 345MPa$: Modulus of elasticity of steel

$Fy = 345MPa$: Specified minimum yield stress of ST345

k_c : Coefficient for slender unstiffened elements

$$k_c = \frac{4}{\sqrt{h / tw}}; 0.35 \le k_c \le 0.76$$

F_L : Magnitude of flexural stress in compression flange at which flange local buckling or lateral-torsional buckling is influenced by yielding

For major axis bending of compact and noncompact web built-up I-shaped members, with:

$S_{xt} / S_{xc} \ge 0.7 : F_L = Fy \times 0.75$

$S_{xt} / S_{xc} < 0.7 : F_L = Fy \times S_{xt} / S_{xc} \ge 0.5Fy$

$S_{xt} = Ix / h_t$: Section modulus about the x axis of the outside fiber of the tension flange, mm³.

$M_y = S_x F_y$: Section modulus about the x axis of the outside fiber of the compression flange, mm³.

$M_y = S_x F_y$: the yield moment of the extreme fiber. *(AISC F4-1)*

$M_p = Z_x F_y$: the plastic bending moment. *(AISC F4-2)*

[7] AISC 360-10, Table B4.1 Case 11.

[8] AISC 360-10, Table B4.1 Case 15.

[9] AISC 360-10, Table B4.1 Case 16.

Checking Dimension Requirements

Concrete slabs on formed steel deck connected to steel beams need to be satisfied the following requirements[10] :

(1) The nominal rib height (hr) shall not be greater than 3 in. (75 mm).

(2) The average width of concrete rib or haunch (wr) shall be not less than 2 in. (50 mm), but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.

(3) The slab thickness above the steel deck shall be not less than 2 in. (50 mm).

(4) a. The concrete slab shall be connected to the steel beam with welded steel headed stud anchors, not larger than 19 mm in diameter.

Steel headed stud anchors, after installation, shall be extended not less than 38mm above the top of the steel deck and there shall be at least 13 mm of specified concrete cover above the top of the steel headed stud anchors.

b. The diameter of a steel headed stud anchor shall not be greater than 2.5 times the thickness of the base metal to which it is welded, unless it is welded to a flange directly over a web.

c. Steel anchors shall have at least 1 in. (25 mm) of lateral concrete cover in the direction perpendicular to the shear force, except for anchors installed in the ribs of formed steel decks. The minimum distance from the center of an anchor to a free edge in the direction of the shear force shall be 8 in. (203 mm) if normal weight concrete is used and 10 in. (250 mm) if lightweight concrete is used.

(5) The Stud spacing

a. Steel deck shall be anchored to all supporting members at spacing not to exceed 460 mm.

b. The minimum center-to-center spacing of steel headed stud anchors shall be six diameters along the longitudinal axis of the supporting composite beam and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks oriented perpendicular to the steel beam the minimum center-to-center spacing shall be four diameters in any direction.

c. The maximum center-to-center spacing of steel anchors shall not exceed eight times the total slab thickness or 36 in. (900 mm).

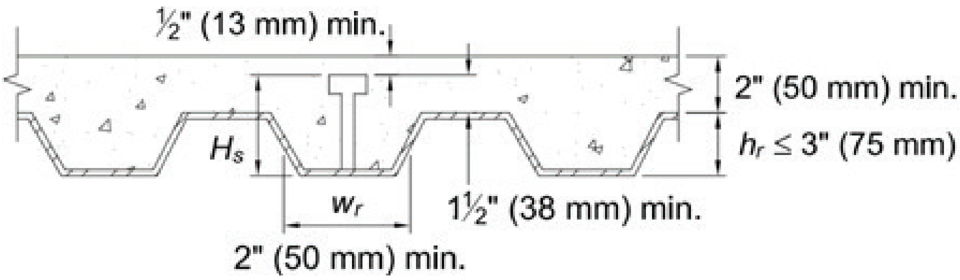


Figure 6 2 Dimension Requirements of Steel headed stud anchors

[10] AISC 360-10, Section I3.2c, Section I8, page 146.

6.2.2. Beam Bending Capacities for Fully Composite Beam

Properties of Transformed Section

To calculate the moment capacity with an elastic stress distribution, first need to calculate the location of the elastic neutral axis (ENA) and the transformed section moment of inertia as below:

Effective Width	$b_{eff} = \dfrac{\min(L/4;B)}{n}$ *(AISC 360-10 I3.1a)*
Effective Thickness of Slab	$t_o = t_s - h_r / 2$ where: $n = E_s / E_c$: Modular ratio E_c : Modulus of elasticity of steel E_c : Modulus of elasticity of concrete
Height of Composite Section	$d_{tr} = H + h_s$
Distance from **Elastic Neutral Axis(ENA)** to steel bottom of composite member	$y_b = \dfrac{\sum A_i y_i}{\sum A_i}$
The transformed section moment of inertia, about its ENA	I_{tr}
Section modulus of bottom edge	$y_b \geq H : S_{tr} = Ix / (0.5h)$ $y_b \geq H : S_{tr} = Ix / (0.5h)$
Section modulus of top edge	$S_t = \dfrac{I_{tr}}{d_{tr} - y_b}$

Check Bending Capacities

Moment Capacity for Positive Bending

$$f_b = \frac{M_{\max}}{S_{tr}} \leq \frac{F_y}{\Omega_b}; \Omega_b = 1.67$$

Moment Capacity for Negative Bending

$$f_c = \frac{M_{\max}}{nS_t} \leq 0.45 f_c^{'}$$

6.2.3. Beam Shear Capacity

Shear Capacity

The nominal shear strength of unstiffened or stiffened webs of all other doubly symmetric shapes and singly symmetric shapes and channels, is determined as follows:

$$V_n = 0.6F_y A_w C_v$$

(AISC I4.2, G2-1)

where:

C_v The web shear coefficient, is given by:

(AISC G2.1b, G2-3)
(AISC G2.1b, G2-4)
(AISC G2.1b, G2-5)

Conditions	C_v
$hw / tw \le 1.10\sqrt{\frac{k_v E}{Fy}}$	1.00
$1.10\sqrt{\frac{k_v E}{Fy}} < hw / tw \le 1.37\sqrt{\frac{k_v E}{Fy}}$	$\frac{1.10\sqrt{k\,E / Fy}}{h \quad tw}$
$hw / tw > 1.37\sqrt{\frac{k_v E}{Fy}}$	$\frac{1.51 k_v E}{(h / tw)^2\, Fy}$

k_v The web plate buckling coefficient, is given by:

(AISC G2.1b)

For unstiffened webs with $hw / tw < 260$: $k_v = 5$

For stiffened webs:

+ $k_v = 5 + \frac{5}{(a / h)^2}$

+ $k_v = 5$ for $a / h > 3.0$ or $a / h > \left(\frac{260}{h / tw}\right)^2$

A_w dt : area of web

h for built-up welded sections, the clear distance between flanges

a clear distance between transverse stiffeners

Checking the Beam Shear

The beam shear at the ends of the beam is checked using the following equation.

$$V_{\max} \le \frac{V_n}{\Omega_v}$$

(AISC 360-10 G2-1)

where:

(AISC G2.1b, 1)

$\Omega_v = 1.67$ Safety factor for shear

$V_{\max}$ The required shear strength

V_n Shear capacity

6.2.4. Steel Anchors

Shear connector capacities are defined for both shear studs and channel shear connectors. Next the equations used for determining the number of shear connectors on the beam are provided.

The nominal shear force between the steel beam and the concrete slab transferred by steel anchors for Positive Flexural Strength:

$$V' = 0.5\min\begin{cases}0.85 f_c' A_c \\ F_y A_s\end{cases} \qquad \text{(AISC 360-10 G2.1b)}$$

where:

A_c $b_{eff} \times t_o \times n$: area of concrete slab within effective width

A_s area of steel cross section

Steel Headed Stud Anchor

The nominal shear strength of one steel headed stud anchor embedded in a solid concrete slab

$$Q_n = \min\begin{cases}0.5 A_{sa}\sqrt{f_c' E_c} \\ R_g R_p A_{sa} F_u\end{cases} \qquad \text{(AISC 360-10 I8.2a)}$$

where:

A_{sa} cross-sectional area of steel headed stud anchor, mm².

E_c $0.043 w_c^{1.5}\sqrt{f'_c}\,(MPa)$: Modulus of elasticity of concrete

w_c Weight of concrete per unit volume, assumed 2300 kg/m³

f_u specified compressive strength of concrete, MPa

f_u specified minimum tensile strength of a steel headed stud anchor

$R_g; R_p$ The table below presents values for R_g and R_g for several cases.
Capacities for steel headed stud anchors can be found in the Manual.

Condition		R_g	R_p
No decking		1.0	0.75
Decking oriented parallel to the steel shape	$w_r / h_r < 1.5$	1.0	0.75
	$w_r / h_r < 1.5$	0.85[(a)]	0.75
Decking oriented perpendicular to the steel shape	1	1.0	0.6[(b)]
Number of steel headed stud anchors occupying	2	0.85	0.6[(b)]
the same decking rib	3 or more	0.7	0.6[(b)]

where:

w_r: nominal rib height

w_r : average width of concrete rib or haunch

(a) for a single steel headed stud anchor

(b) this value may be increased to 0.75 when $e_{mid-ht} \geq 2in. = 51mm$

e_{mid-ht} distance from the edge of steel headed stud anchor shank to the steel deck web, measured at mid-height of the deck rib, and in the load bearing direction of the steel headed stud anchor (in other words, in the direction
of maximum moment for a simply supported beam).

Steel Channel Anchors

The nominal shear strength of one hot-rolled channel anchor embedded in a solid concrete slab shall be determined as follows:

$Q_n = 0.3\left(tf + 0.5tw\right) l_a \sqrt{f_c' E_c}$	(AISC 360-10 I8.2a)

where:

- l_a length of channel anchor
- tf thickness of flange of channel anchor
- tw thickness of channel anchor web

Check Shear Connector Capacity

The number of shear connectors is given as follows:

$\sum Q_n = Q_n N_r \frac{L}{2s} \geq V'$	(AISC 360-10 I3.1c)

where:

- N_r Number of Stud each position
- s Shear Connectors Spacing
- L Length of joist beam

6.2.5. Beam Deflection and Camber

Checking Stress when made camber

Camber	$\Delta_{DL} = \frac{5}{384} \times \frac{q_{DL} L^4}{EI_s}$
Checking Stress	$\frac{M_{DL}}{S_s} < \frac{Fy}{\Omega_b}$

Checking Deflection of Joist Beam

Effective moment of inertia	$I_{eff} = 0.75 I_{tr}$	(AISC 360-10 I3.2)
Deflection by Floor Load	$\Delta_{FL} = \frac{5}{384} \times \frac{q_{FL} L^4}{EI_{eff}} \leq [\Delta] = \frac{L}{360}$	
Deflection by Dead load & Floor load	$\Delta = \frac{5}{384} \times \frac{q L^4}{EI_{eff}} \leq [\Delta] = \frac{L}{240}$	

6.2.6. Check Decking

Decking Properties

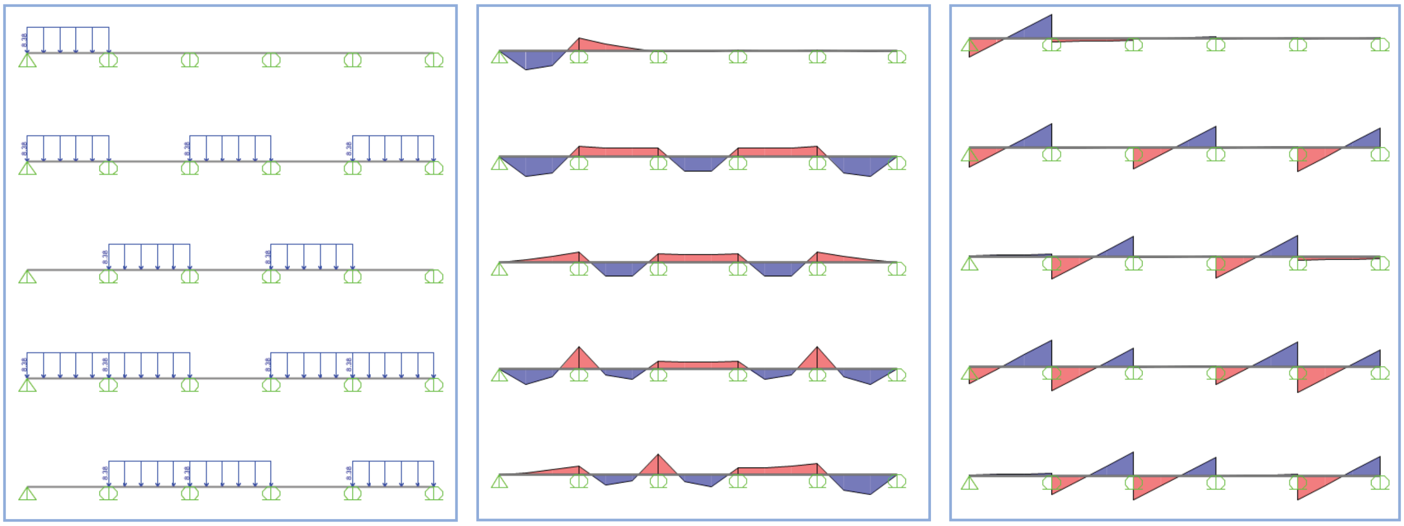
It is necessary to determine decking's material each project information.
Normally, yielding strength of decking is 235 MPa.

Table 6 1 Properties of decking

[width]-[rid depth]-[rib spacing]-[average width rib]	Thickness		
	0.75	0.95	1.15
[1000]-[50]-[334]-[166]	455282 17842	574099 22441	691840 26975
[0870]-[75]-[287]-[148]	922823 23643	1168944 29917	1415089 36178

Checking the decking

Loading impact on decking: $q_c = B_r \times (q_{CL} \times n + G_{concrete} \times t_o)$



Load assign decking (5 cases) *Moment 3-3 Diagram* *Shear 2-2 Diagram*

Figure 6 3 Schematics of Decking

Checking capacity:

$$\frac{M_{\max}}{S_x} \le \frac{Fy}{\Omega_b}; \Omega_b = 1.67$$

Checking deflection :

$$f_{\max} \le [f] = \frac{B}{180}$$

[11]SDI Composite Steel Floor Deck – Slabs, page 8, 2012.

6.2.7. Design Example 8a - Composite Slab

Input

Loading	Floor load (FL)	5.0	kN/m²
	Brick wall 1m height 0.2m thickness Wall load $WL = 18 \times 1 \times 0.2$	3.6	kN/m
Slab thickness	Included: Surface sika, thickness 10*mm*, density 22*kN*/*mm*³ Mortar, thickness 20*mm*, density 18*kN*/*mm*³	120.0	mm
Joist beam	Section Length Spacing	I – 508x148x5x6 8000.0 1500.0	 mm mm
Decking	Type / Thickness (mm) Construction method Yielding strength, Fy Safety factor, n	[B1000 – hr50]/[0.75] Don't Use Support Column 235 1.5	 MPa
Shear Connector	Diameter/Height/Qty. each position/spacing	[16]/[90]/[02]/[334]	
Concrete strength: B20	Compression strength, fc' Elastic modulus, Ec	16 18800	MPa MPa
Steel	**ASTM specifications: A572 Grade 50** Yield strength, F*y* Ultimate strength, F*u* Modulus of elasticity, *E*	 345 450 200000	 MPa MPa MPa

6.2.7.1 Design Composite Joist Beam using Calculation Sheet

Slab Loading

Load Case	Include	Thickness (mm)	G (kN/m³)	Design Load
Dead Load (DL)	Mass of joist Wall load Surface sika	 10	 22	0.334 kN/m 3.60 kN/m
	Concrete slab Mortar	$t_o = 95$ 20	25 18	2.955 kN/m²

Dead load applied on each joist: $q_{DL} = 0.334 + 3.6 + 1.5 \times 2.955 = 8.37(kN/m)$
Hence,

Moment by Dead Load	$M_{DL} = qL^2/8 = 8.37 \times 8^2/8$	66.99 kN/m
Moment by Dead Load & Floor Load	$M_{DL+FL} = qL^2/8 = (8.37 + 1.5 \times 5) \times 8^2/8$	126.93 kN/m
Maximum Shear by (DL+FL)	$V_{DL+FL} = qL/2 = (8.37 + 1.5 \times 5) \times 8/2$	63.48 kN

Section Properties

Cross-section area	$A = 2\times 148\times 6 + 496\times 5$	4256	mm²
Mass	$A\times 78.5$	0.334	kN/m
Moment of Inertia	$Ix = \dfrac{5\times 496^3}{12} + 148\times\dfrac{508^3 - 496^3}{12}$	1.63E8	mm4
Plastic Section Modulus	$Zx = 2\times\left(148\times 6\times\dfrac{502}{2} + 5\times\dfrac{496}{2}\times\dfrac{496}{4}\right)$	753296	mm³
Elastic Section modulus	$Sx = 2\times 1.63E+08/508$	6.41E5	mm³
Radius of Gyration	$rx = \sqrt{1.63E+08/4256}$	195.5	mm

Classification of Filled Composite Sections for Local Buckling

Flanges: Noncompact	$\lambda p = 0.38\sqrt{200000/345}$	9.15
	$\lambda r = 0.95\sqrt{\dfrac{0.45\times 200000}{0.75\times 345}}$	18.65
	$k_C : 0.35 \le \left[k_C = \dfrac{4}{\sqrt{396/5}} = 0.45\right] \le 0.76$	0.45
	$S_{xt}/S_{xc} = 1.00 \Rightarrow F_L = 0.75Fy$	
	$0.5B/t = 0.5\times 148/6 = 12.33 \Rightarrow \lambda_p < (0.5B/t) < \lambda_r$	
Web: Noncompact	$\lambda p = 3.76\sqrt{E/Fy} = 3.76\sqrt{200000/345}$	90.53
	$\lambda r = 5.70\sqrt{200000/345}$	137.23
	$h/tw = 496/5 = 99.2 \Rightarrow \lambda_p < (h/tw) < \lambda_r$	
Class: Noncompact		

Checking Dimension Requirements

(1) The nominal rib height, hr	50 mm	$\le$ 75 mm	OK
(2) The average width of rib, wr	166 mm	$>$ 50 mm	OK
(3) The slab thickness	120 mm	$\ge hr + 50 = 100mm$	OK
(4) Steel headed stud anchors			
Diameter	16 mm	$\le$ 19 mm	OK
Height		$\le 2.5\times tf1 = 2.5\times 6 = 15$	
Stud spacing		$\ge hr + 38 = 88mm$	
	90 mm	$\ge 5\times 16 = 80mm$	
		$< 120 - 13 = 107mm$ (slab cover above)	
		$\le$ 460 mm	
	334 mm	$> Min. = 6\times 16 = 96mm$	
		$< Max. = \max(8\times 120; 900) = 960mm$	

Determining Composite Properties for Plastic Design

Effective Width	$b_{eff} = \min(8000/4;1500)/10.64$	141	mm
Modular ratio	$n = 200000/18800$	10.64	
Effective Thickness of Slab	$t_o = 120 - 50/2$	95	mm
Height of Composite Section	$d_{tr} = 120 + 508 = 628mm$		
Distance from ENA to steel bottom of composite member	$y_b = \dfrac{141\times95\times(508+120-95/2)+4256\times508/2}{141\times95+4256}$	501.77	mm
The transformed section moment of inertia, about its ENA	$I_{tr} = I_s + I_c = 4.24E8 + 9.31E7$	5.17E8	mm4
	$I_s = Ix + A(y_c - y_b)^2 = 1.63E8 + 4256(254-501.77)^2$	4.24E8	mm4
	$I_c = 141\times95^3/12 + 141\times95\times(628-95/2-501.77)^2$	9.31E7	mm4
Section modulus of bottom edge	$y_b = 501.77 < H = 508 : S_{tr} = \dfrac{5.17E8}{501.77}$	1.03E6	mm³
Section modulus of top edge	$S_t = \dfrac{5.17E8}{628-501.77}$	4.1E6	mm³

Check Bending Capacities

Positive Bending	$f_b = \dfrac{126.93}{1.03E6/1000^3}$	1.23E5	kN/m²
$f_b < \dfrac{F_y}{\Omega_b}$: OK	$\dfrac{F_y}{\Omega_b} = \dfrac{345E3}{1.67}$	2.10E5	kN/m²
Negative Bending $f_c < 0.45 f_c'$: OK	$f_c = \dfrac{126.93}{10.64\times4.1E6/1000^3}$	2.9E3	kN/m²
	$0.45 f_c' = 0.45\times16E3$	7.2E3	kN/m²

Check Shear Capacities

The web shear coefficient	$\dfrac{h_w}{t_w} = \dfrac{496}{5} = 99.2 > 1.37\sqrt{\dfrac{5\times200000}{345}} = 73.76$	
	$k_v = 5$ for $a/h = 5.4 > 3.0$ where: $a/h = 2500/496 = 5.4 > 3.0$	
	$\Rightarrow C_v = \dfrac{1.51\times5\times200000}{(496/5)^2\times345}$	0.44
The available shear strength	$\dfrac{V_n}{\Omega_v} = \dfrac{0.6\times345E3\times0.496\times0.005\times0.44}{1.67}$	135.26 kN
Check Shear Capacity	$\dfrac{V_n}{\Omega_v} > V_{\max} = 63.48kN$: OK	

Check Shear Connector Capacity

The required Shear for Shear Connector	$V' = 0.5\min\begin{cases}0.85\times 16E3\times 0.141\times 0.095\times 10.64 = 1938\\ 345E3\times 4256/1000^2 = 1468\end{cases}$	734.16 kN
The nominal shear strength of each stud	$Q_n = \min\begin{cases}0.5\times\dfrac{201.06}{1000^2}\sqrt{16\times 18800}\times 1000 = 55.13kN\\ 1.0\times 0.75\times\dfrac{201.06}{1000^2}\times 450\times 1000 = 67.86kN\end{cases}$	55.13 kN
The number of shear connectors	$\sum Q_n = 55.13\times 2\times\dfrac{8000}{2\times 334} = 1321kN \geq V' = 734.16kN$	: OK
where:	$A_{sa} = \pi\times 16^2/4 = 201.06(mm^2)$ $f_c' = 16MPa$ $E_c = 0.043\times 2300^{1.5}\sqrt{16} = 18800MPa$ $w_r/h_r = 166/50 = 3.32 \geq 1.5 \Rightarrow \begin{cases}R_g = 1.00\\ R_p = 0.75\end{cases}$ $f_u = 450MPa$	

Beam Deflection and Camber

Camber	$\Delta_{DL} = \dfrac{5}{384}\times\dfrac{8.37\times 8^4}{200000\times\dfrac{1.63E8}{1000^4}}$	13.7 mm
Checking Stress	$\dfrac{66.99}{6.41E5/1000^3} = 1.05E5 < \dfrac{345E3}{1.67} = 2.07E6(kN/m^2)$	: OK

Checking Deflection of Joist Beam

Effective moment of inertia	$I_{eff} = 0.75\times 5.17\mathrm{E}8$	3.88E8 mm^4
Deflection by Floor Load	$\Delta_{FL} = \dfrac{5}{384}\times\dfrac{5\times 1.5\times 8^4}{200000\times 3.88E-4} = 5.26mm < [\Delta] = \dfrac{8000}{360} = 22.2mm$	
Deflection by Dead load & Floor load	$\Delta = \dfrac{5}{384}\times\dfrac{(5\times 1.5+8.37)\times 8^4}{EI_{eff}} 10.91mm \leq [\Delta] = \dfrac{8000}{240} 33.3mm$	

Design Data

Components	Types	Ratio	Conclusion
Section	Preliminary dimension Top flange Bottom flange Shear strength	0.842 0.405 0.596 0.464	OK
Studs	Spacing	0.556	OK
Camber	Camber Capacity	14 0.506	mm OK
Deflection	Live load Dead load & Live load	0.232 0.327	OK

6.2.7.2 Design Composite Mezzanine Decking using Calculation Sheet

Decking [**1000**]-[**50**]-[**334**]-[**166**] thickness **0.75**mm: $S_x = 1.78E4(mm^3)$; $S_x = 1.78E4(mm^3)$

Checking the decking

Loading impact on decking	$q_c = B_r \times (q_{CL} \times n + G_{concrete} \times t_o)$ $q_c = 1 \times (4 \times 1.5 + 25 \times 0.095)$	8.375 kN/m
Maximum Moment	$M_{\max} = 1.86(kNm)$	
Checking capacity	$\frac{1.86}{1.78E-5} 1.02E5 \le \frac{Fy}{\Omega_b} = \frac{235000}{1.67} = 1.4E5(kN/m^2)$	: OK
Checking deflection	$f_{\max} = 4.52mm \le [f] = \frac{B}{180} = \frac{1500}{180} = 8.33mm$	: OK

6.3. Checkered Plate Slab

6.3.1. General

Allowable thicknesses of Checkered Plate are 4mm, 5mm, 6mm or 8mm (included ribbed plate).
Checkered Plate Slab can be designed with or without Flat Bar.

NOTE

Incase floor load is not less than 4 kN/m²; designers should use Checkered Plate Slab with Flat Bar under.

If floor load is less than 4 kN/m², Checkered Plate Slab without Flat Bar can be used with the joists having maximum spacing 1.2÷1.5m.

6.3.2. Detailing Requirement

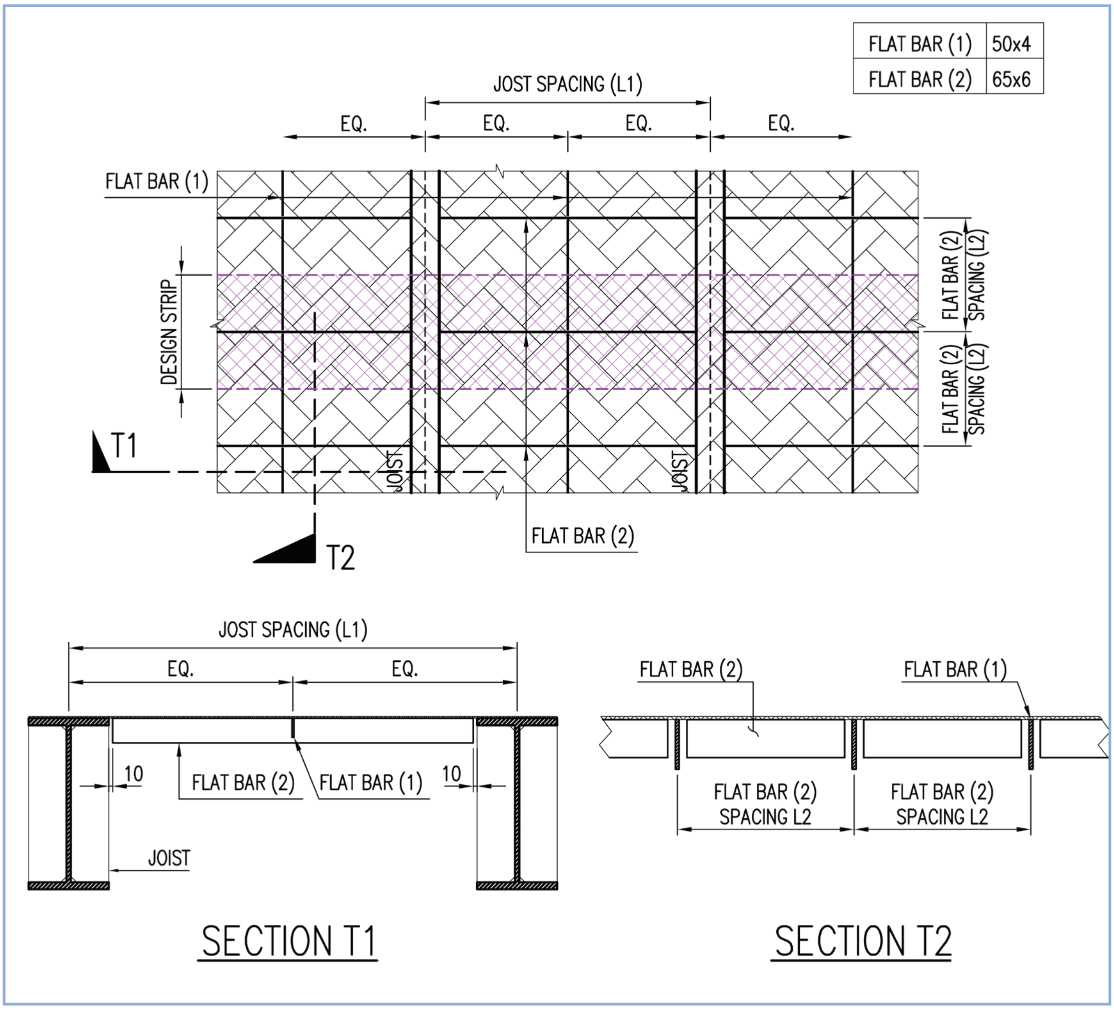


Figure 6 4 Checkered Plate with Flat Bar

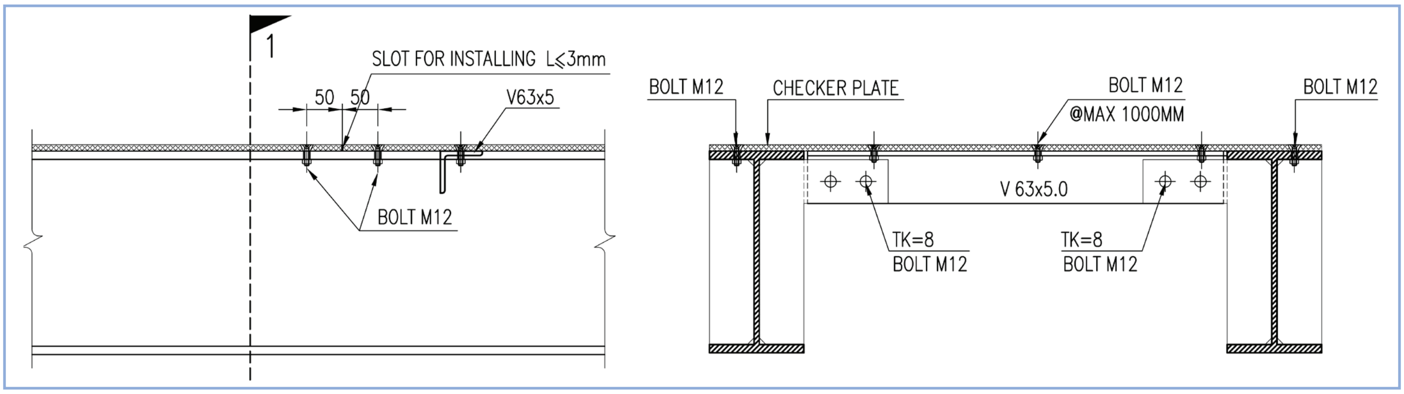


Figure 6 5 Bolt Connection of Checkered Plate and Joists

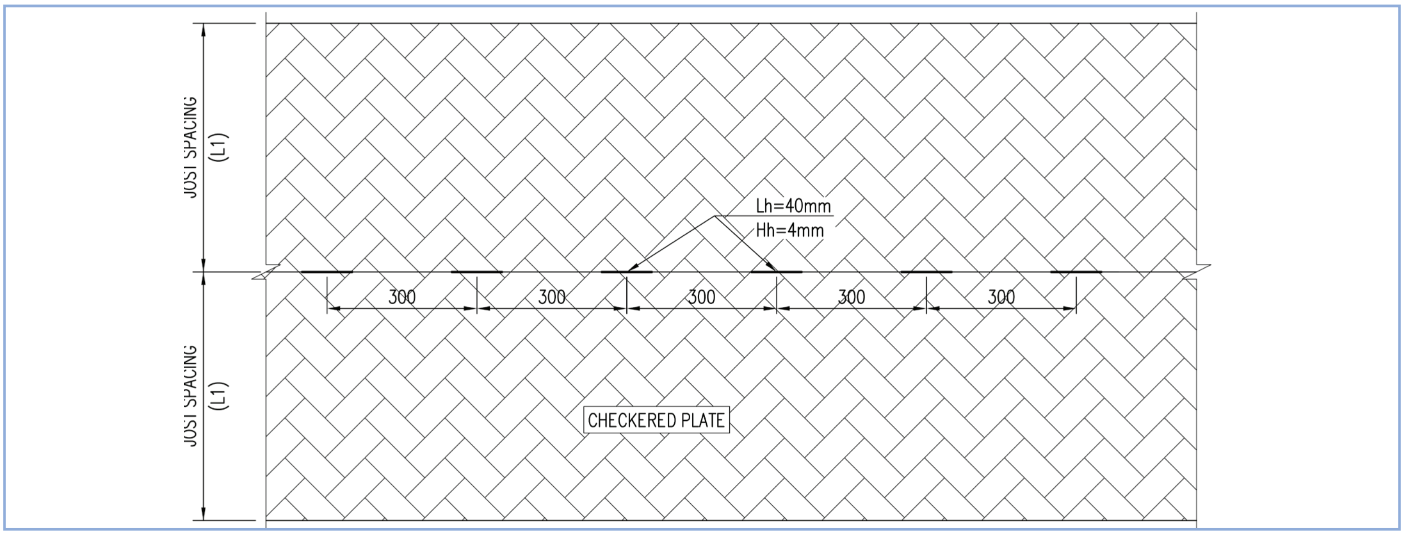


Figure 6 6 Weld connection at the intersection of Checkered Plates

6.3.3. Checkered Plate with Flat Bar

Design Strip Properties

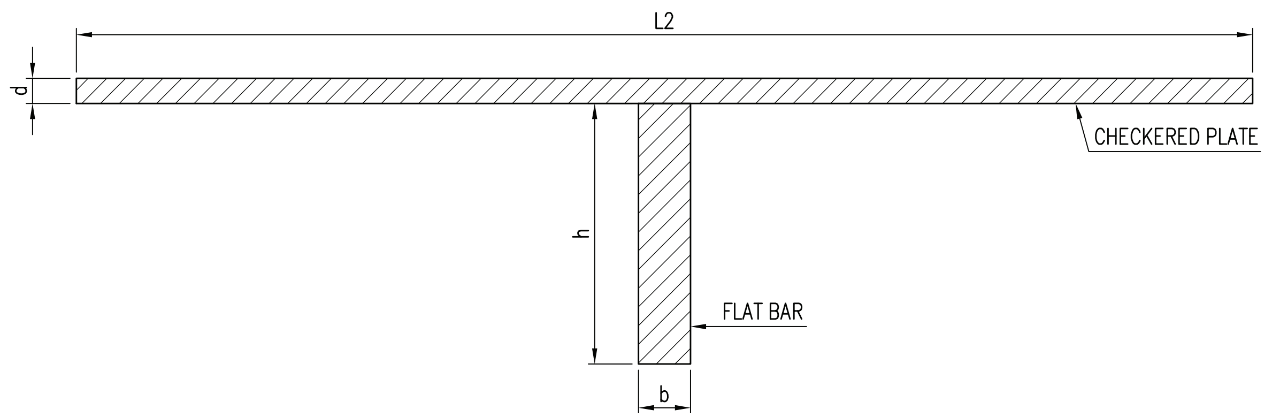


Figure 6 7 Design Strip of Checkered Plate with Flat Bar

Section area	$A = d \times L_2 + b \times h$
Distance from Neutral Axis to bottom	$y_{base} = \sum A_i y_i / \sum A_i$
Inertia moment of strip	$Ix = \sum \left(bh^3 / 12 \right) + \sum Ay_i^2$
Cylindrical rigidity of plate	$D = Eh_e^3 / 12 \times L_2$
where,	E , Young's Modulus h_e, elastic thickness $\upsilon = 0.3$, Poisson's Ratio for steel

Check Deflection

Checkered Plate is considered as a uniform loaded long rectangular plate with longitudinal edges which are free to rotate but cannot move toward each other during bending. The design strip cut out this plate is in the condition of a uniformly loaded bar submitted to the action of an tensile force H, as show in Figure 6-8. The magnitude of H is such as to prevent the ends of bar from moving along the x-axis.

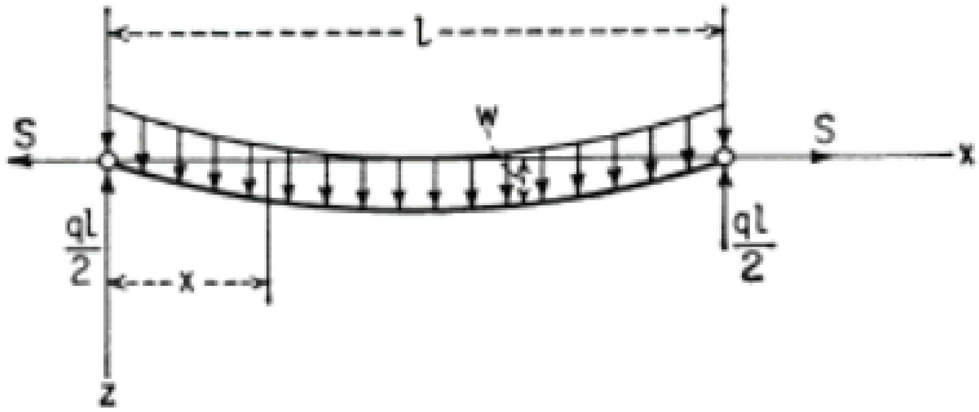


Figure 6 8 Uniformly Loaded Rectangular Plates with Simply Supported Edges

The maximum deflection	$\Delta = \Delta_o \dfrac{1}{1+\alpha} \le [\Delta] = \dfrac{L}{180}$
The maximum deflection caused by uniform load q	$\Delta_o = \dfrac{5}{384} \dfrac{ql^4}{E_1 I_x}$; where $E_1 = \dfrac{E}{1-\upsilon^2}$
The factor for the effect of the tensile reactions at the ends of the strip	$\dfrac{1}{1+\alpha}$
The ratio of the tensile force α to the Euler critical load of the elemental strip, α, is determined from the equation:	$\alpha(1+\alpha)^2 = 3\left(\dfrac{\Delta_o}{t}\right)^2$

Check Bending

The tensile force subject on joint of strip	$H = \dfrac{\pi^2 D\alpha}{l^2}$
The maximum bending moment	$M_{\max} = \dfrac{ql^2}{8} - H\Delta$
Check Stress	$\sigma = \dfrac{H}{A} + \dfrac{M_{\max}}{S_x} \leq \dfrac{F_y}{\Omega}; \Omega = 1.67$

Check Weld Connection

Tensile force subject to weld	$T \leq \dfrac{R_n}{\Omega} = \dfrac{F_{nw} \times A_w}{\Omega}; \Omega = 2.00$
Allowable bearing capacity of weld	$T \leq \dfrac{R_n}{\Omega} = \dfrac{F_{nw} \times A_w}{\Omega}; \Omega = 2.00$
Nominal stress of the weld	$F_{nw} = 0.6 F_{EXX}\left(1 + 0.5 \sin^{1.5}\theta\right)$
Area of the weld	$A_w = \dfrac{\sqrt{2}}{2} \times h_f$

6.3.4. Checkered Plate without Flat Bar

Formulae show ratio of design length and slab thickness(According to formulae of A.L.Teloian):

$$\frac{l}{t} = \frac{4n_o}{15}\left(1 + 75\frac{E_1}{n_o^4 q}\right) \Rightarrow l = t \times \frac{4n_o}{15}\left(1 + 75\frac{E_1}{n_o^4 q}\right)$$

where $\dfrac{1}{n_o} = \left[\dfrac{f}{L}\right] = \dfrac{1}{180}$: Ratio of allowable deflection of slab.

Hence, available joist spacing (distance edge flange to edge flange of joists) $[l_o] \leq l$.

6.3.5. Design Example 8b - Checkered Plate with Flat Bar

Input

Spacing of Joist	L_1	1500	mm
Spacing of Flat bar	L_2	600	mm
Checkered Plate Thickness	d	4	mm
Flat bar	Width, b	4	mm
	Height, h	50	mm
Material	ASTM specifications for Steel	JIS 3101 /SS400	
	Yield strength, F_y	235	MPa
	Ultimate strength, F_u	400	MPa
	Modulus of elasticity of steel, E	200000	MPa
Welding	Welding wire	E60xx	
	Height of fillet weld	3	mm
Floor load	FL	5	kN/m²

Properties Of Design Strip

Section area	$A = 4\times600+4\times50 = 2600mm^2$	2600	mm²
Distance from Neutral Axis to bottom	$y_{base} = \dfrac{600\times4\times52+4\times50\times25}{2600}$	49.9323	mm
Inertia moment of strip	$I = \dfrac{600\times4^3}{12}+600\times4\times(52-y_{base})^2 - \dfrac{4\times50^3}{12}+4\times50\times(25-y_{base})^2$	179451	mm⁴
Section modulus	$S_x = \dfrac{I_x}{0.5y_x} = \dfrac{179451}{0.5\times49.9323}$	7187.8	mm³
Cylindrical rigidity of plate	$D = \dfrac{2E8\times0.004^3}{12}\times0.6$	0.64	kNm

Check Deflection

Modulus E_1	$E_1 = \frac{200000}{1-0.3^2} = 2.2E5(MPa)$	2.2E5	MPa
Dead load	$q_{DL} = \frac{2600}{1000^2} \times 78.5 \times 1.05$	0.214	kN/m²
Uniform load q	$q = q_{DL} + q_{FL} = (0.214+5) \times 0.6$	3.13	kN/m²
The maximum deflection caused by uniform load q	$\Delta_o = \frac{5}{384} \frac{3.13 \times 1.5^4}{2.2E8 \times 1.79E-7}$	5.24E-3	m
The ratio α	$\alpha(1+\alpha)^2 = 3\left(\frac{5.24E-3}{4E-3}\right)^2 = 5.15 \Rightarrow \begin{cases} \alpha = 1.535 \\ \alpha = -0.77 \end{cases}$ Choose $\alpha = 1.535$		
The maximum deflection	$\Delta = (5.4E-3) \times \frac{1}{1+1.535}$	2.1E-3	m
Check deflection	$\Delta \le [\Delta] = \frac{1.5}{180} = 0.0833m$	OK	

Check Bending

The tensile force	$H = \frac{3.14^2 \times 0.64 \times 1.535}{1.5^2}$	4.3	kN
The maximum bending moment	$M_{\max} = \frac{3.13 \times 1.5^2}{8} - 4.4 \times 2.1E-3$	0.87	kNm
Tensile stress	$\sigma = \frac{4.4}{2.6E-3} + \frac{0.87}{7.19E-6}$	1.23E8	kN/m²
Check Stress	$\sigma \le \frac{F_y}{\Omega} = \frac{245E3}{1.67} = 1.47E6\left(kN/m^2\right)$	OK	

Check Weld Connection

Factor β	$\beta = 1.2 - \dfrac{0.002}{3/600} = 0.8 < 1$	0.8
Tensile force subject to weld	$T = \dfrac{H}{n\beta b_{eff}} = \dfrac{4.3}{0.6 \times 0.8}$	8.95 kN/m
Bearing capacity of weld	$\dfrac{R_n}{\Omega} = \dfrac{3.73E5 \times 2.12E-3}{2}$ $A_w = \sqrt{2}/2 \times 0.003 \times 1$ $F_{nw} = 0.6 \times 4.14E5 \times \left(1 + 0.5\sin^{1.5} 90°\right)$ $F_{EXX} = 60(ksi)$	394.96 kN/m 2.12E-3 m² 3.73E5 kN/m² 4.14E5 kN/m²
Check Welding	$T = 8.95 < \dfrac{R_n}{\Omega} = 394.969\left(kN/m^2\right)$	OK

6.3.6. Design Example 8c - Checkered Plate without Flat Bar

Input: as same as Design Example 8b, Checkered Plate without Flat Bar.

Specify Thickness: 4.00 mm.

Dead load	$q_{DL} = \dfrac{4}{1000} \times 78.5 \times 1.05$	0.33 kN/m²
Uniform load q	$q = q_{DL} + q_{FL} = 0.33 + 5.00$	5.33 kN/m²
Joist spacing	$[l_o] \le 0.004 \times \dfrac{4 \times 180}{15}\left(1 + 75\dfrac{2.2E8}{180^4 \times 5.33}\right) = 0.758m$	0.8 m

Then, check Deflection and Bending as same as Example 8a.

Conclusion

The type of checkered plate (with or without flat bar) depends on the load and thickness (if required).

6.4. Grating Slab

6.4.1. General

Open steel rectangular pattern floor is constructed from mild steel and is constructed by a forge-welding process in which the load bearing and transverse bars are heated and joined under pressure.

Gratings is suitable for most floor walkways, gantries, platforms, etc. and could even be used upright as fencing.

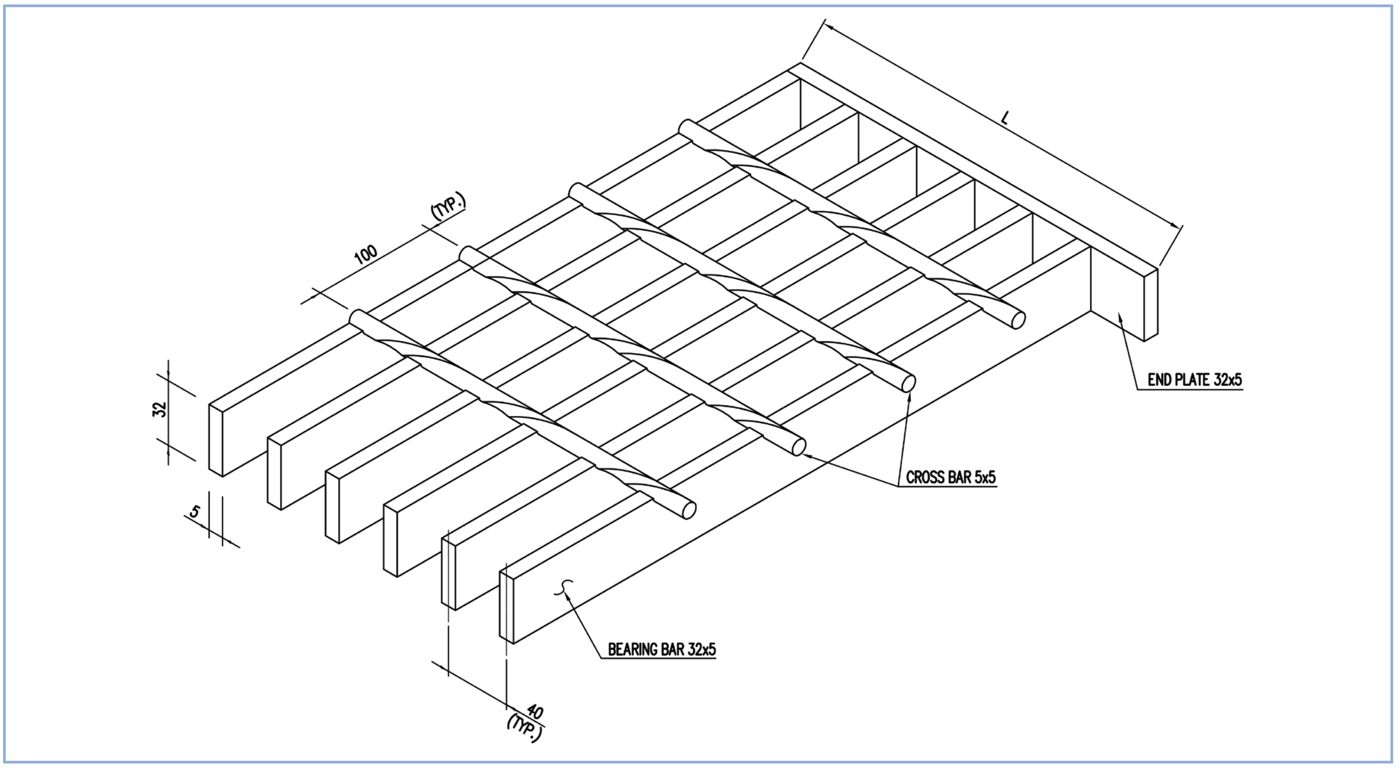


Figure 6 9 Standard Grating

Standard Range – Readily available with either plain or serrated load bearings bars (5mm thick) at 40 pitch and twisted transverse bars at 50 or 100 centers.

6.4.2. Design Example 8c - Grating Slab

Input

Floor load	q_{FL}	3.0	kN
Span/Trench width	b	1300	mm
Bearing bar x pitch		(32 x 5) @max 40	mm
Allowable stress	F_y	235	MPa
Modulus of elasticity of steel	E	200000	MPa
Allowable deflection	$[\Delta] = L/240$	5.41	mm

Consider 1m width of grating slab.

Moment of inertia	$Ix = \frac{5 \times 32^3}{12}$	13653 mm^4
Elastic section modulus	$Sx = \frac{13653}{32/2}$	853 mm^3
Load per each Bearing bar	$P = \frac{q_{FL}}{n \times b} = \frac{3}{1000/40 \times 1.3 \times 1.0}$	0.0923 kN/m
Maximum Moment	$M_{\max} = \frac{ql^2}{8} = \frac{0.0923 \times 1.3^2}{8}$	0.0195 kNm
Maximum Stress	$\sigma = \frac{M_{\max}}{S_x} = \frac{0.0195}{8.53E-7}$	1.83E5 kN/m^2
Check stress	$\sigma \le \frac{F_y}{\Omega} = \frac{245E3}{1.67} = 1.47E6\left(kN/m^2\right)$	OK
Check deflection	$\Delta = \frac{5}{384}\frac{ql^4}{EI} = \frac{5}{384}\frac{0.0923 \times 1.3^4}{2E8 \times 1.36E-8} = 0.00126m < [\Delta]$	OK

CHAPTER 7. ACCESSORIES

7.1. Purlins and Girts Design

7.1.1. General

Purlins and girts are the immediate supporting member for roof and wall sheeting respectively. They act principally as beams, but also perform as struts and as compression braces in restraining rafters and columns laterally against buckling.

Purlins and girts are almost universally zed (Z), channel (C), or double channel section members.

C-sections have equal flanges and may be used in single spans and un-lapped continuous spans in multi-bay buildings. Their freestanding stable shape allows easy handling and storage and is easily adapted for use in small and medium sized buildings as structural framework.

Z-sections feature one broad and one narrow flange allowing the two sections to fit together snugly, making them suitable for lapping. Z sections of the same depth and different thickness' can be lapped in any combination. Purlins and Girts that are lapped form a structurally continuous line along the length of the building, a factor that contributes significantly to the reduction in building costs.

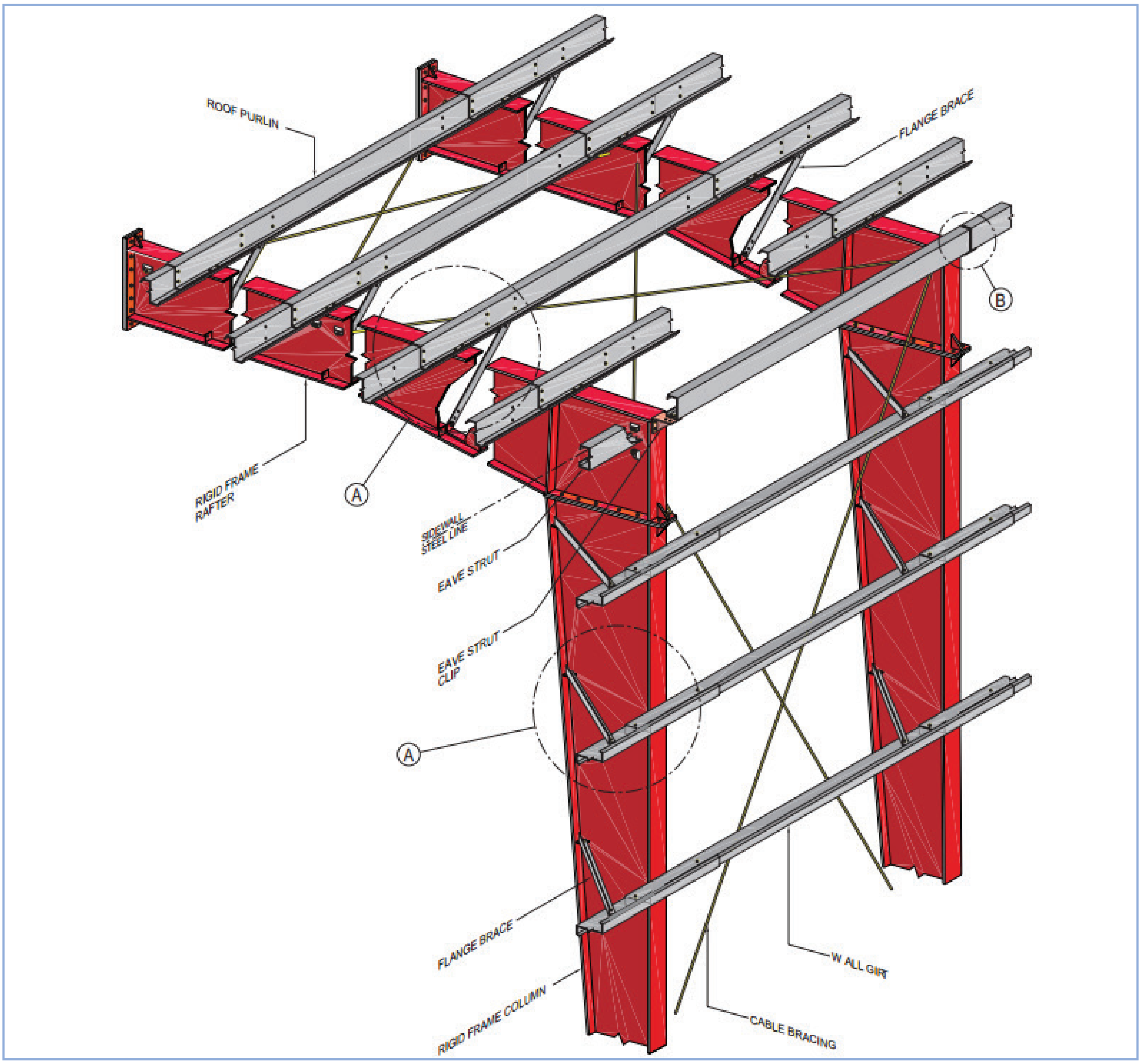


Figure 7 1 Typical Purlins and Girts Details

7.1.2. Design procedure

Purlins and girts can be calculated as simply supported spans, lapped continuous spans or increased thickness end spans in lapped continuous systems.

7.1.2.1 Purlin/Girt spacing

Purlin spacing must be chosen to suit the type of roof sheeting and ceiling system if any. The use of translucent fiberglass roof sheeting will also restrict the purlin spacing. Some suspended ceiling systems require a maximum purlin spacing of 1200mm (DONGIL project). Purlin deflections must also be controlled.

Roof purlins are to be arranged according to the following guide lines as applicable:

- 700 mm between first roof purlin and the eave strut
- Intermediate spacing not exceeding 1600 mm.

7.1.2.2 Loading

The maximum spans are determined not only from wind load considerations, but also from live load requirements. Refer Chapter 2 for more detail.

Collateral load applied on purlin such as Fire Sprinkler, MEP System, Plaster Ceiling System, etc. should be clearly specified as concentrated or uniformed load to identify the most dangerous case.

Components	Load Case	Load Combination
Purlins	Gravity Loads: DL, AL, WL_1 Windload: WL_1, WL_{2E}, WL_{2E}, WL_3	Case 1: $DL + AL + LLr$ Case 2: $\begin{cases} 0.6DL + 0.6AL + 0.6WL_1 \\ 0.6DL + 0.6AL + 0.6WL_{2E} \end{cases}$ Case 3: $\begin{cases} 0.6DL + 0.6AL + 0.6WL_{2F} \\ 0.6DL + 0.6AL + 0.6WL_3 \end{cases}$
Eave struts	Gravity Loads: LLr, LLr Rainload: WL Windload: WL_1, WL_{2E}, WL_{2E}, WL_3	Case 1: $DL + LLr$ Case 2: $DL + P_r$ Case 2: $0.6DL + 0.6WL_3$
Girts	Gravity Loads: AL, AL, WL_4 Windload: WL_4, WL_5	Case 1: $\begin{cases} 0.6DL + 0.6AL + 0.6WL_4 \\ 0.6DL + 0.6AL + 0.6WL_5 \end{cases}$

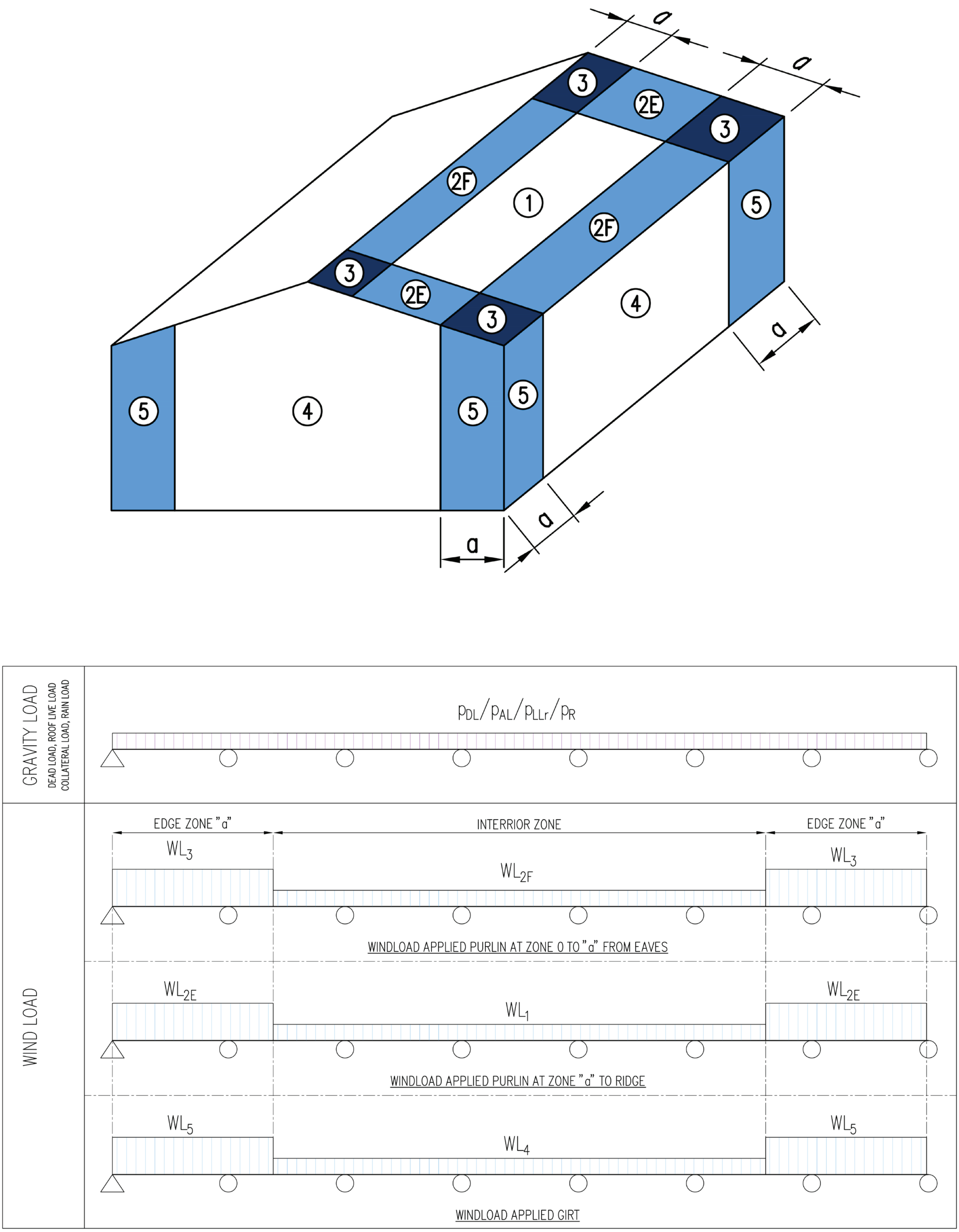


Figure 7 2 Schematics of Purlin/Girt/Eave strut

The peak local pressures zones around the perimeter of the roof govern the purlin spacing these areas, and the purlins in end bays is usually adopted for the rest of the roof because of the difference in loads and bending moments between end bays and internal span purlins. It is therefore advantageous for economical design to consider:

- Increased wall thicknesses in end span purlins, or
- Reduced end bay spacing, or
- Extra purlin spacing, extra purlins, or increase lapped length in end spans, provided this increases the design strength of the purlins.

It is necessary to provide bridging between purlins to reduce the effective lengths to control flexural-torsion buckling. BMB recommends at least one row or purlin bracing in every span, and that un-braced length be restricted to less than 20 times the section depth, or 4000 mm, whichever is less.

Table 7 1 Recommended purlin bracing each bay

Bay Spacing	Number of purlin bracing
< 8m	01
8 ÷ 9m	02
> 9m	03

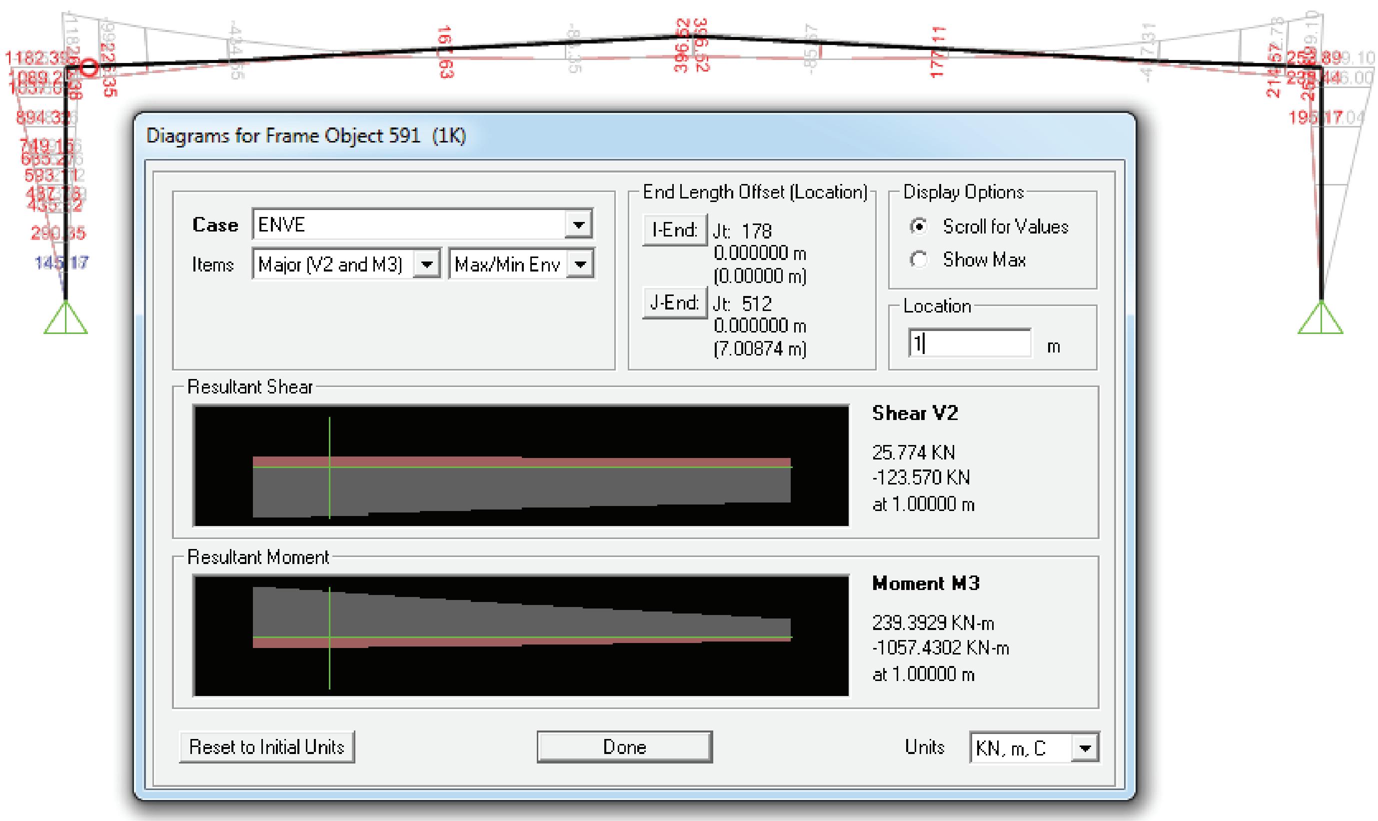
7.1.2.3 Lapped Length

The lapped length for Z-section purlins is a minimum of 10 percent of the span.

7.1.2.4 Purlin bolts

Standard purlins and girts cleats are generally used without analysis or design. Purlin cleats are subjected not only to axial load but also to bending moments. Angel cleats also provide greater robustness during transport and erection. One yardstick for robustness is that girt cleats should not yield when stood on by a heavy worker. This would equate to a 1.1 kN load applied to the tip of the cleat with a 1.5 load factor to allow for dynamic effects as the worker climbs the steel work.

The standard bolt is an M12 Type 4.6. Refer Figure 7 6 for more details.



Input

Flange Brace section Yield strength: $F_y = 235MPa$ Elastic modulus: $E = 200000MPa$		V60x60x2.5	
Bending moment	h	1057	kN
Height of rafter	h	1.65	m
Length of Flange Brace	$l = h\sqrt{2}$	2.33	m
Length coefficient	k	1.00	

Section properties

Area	A	2.938E-4	m²
Radius of Gyration	r	0.019	m

Checking ratio of slenderness: (compressive member with pin connection double end)

$$\frac{kL}{r} = \frac{1\times 2.3}{0.019} = 121 < 200$$

Checking capacity the compression

Elastic buckling stress	$F_e = \frac{\pi^2 E}{(kL/r)^2} = \frac{3.14^2 \times 2E8}{121^2} = 1.3E5(kN/m^2)$ *(AISC E3-4)*
Critical stress	$\frac{kL}{r} < 4.71\sqrt{\frac{E}{Fy}} = 4.71\sqrt{\frac{200000}{345}} = 113$ *(AISC E3-2, E3-3)* $F_{cr} = 0.658^{F_y/F_e} \times F_y = 1.14E5(kN/m^2)$
Capacity of the compression member	$[P] = \frac{P_u}{\Omega} = \frac{F_{cr} \times A_g}{1.67} = \frac{1.14E5 \times 2.938E-4}{1.67} = 20kN$ *(AISC E3-1)*
The Transverse Compression load	$P = 2\%\frac{M}{h} = 0.02 \times 1057/1.65 = 12.81kN < [P]$ *(AISC Section J1.4)*

7.1.3. Detailing Requirement

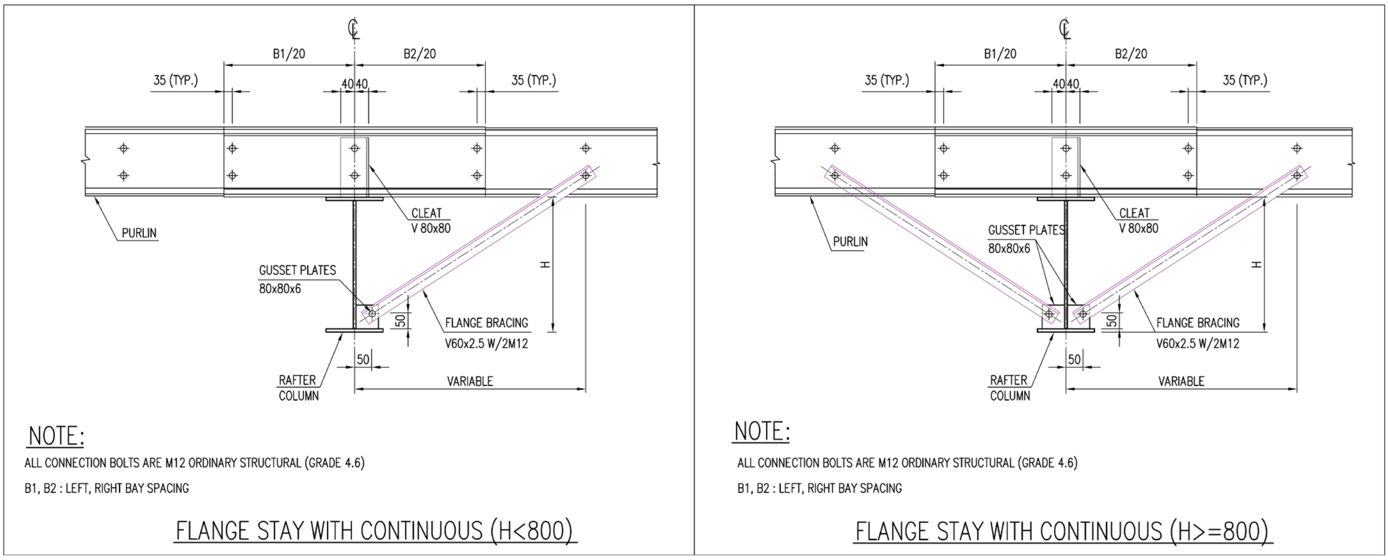


Figure 7 3 Flange Brace and Lapped Length Detail

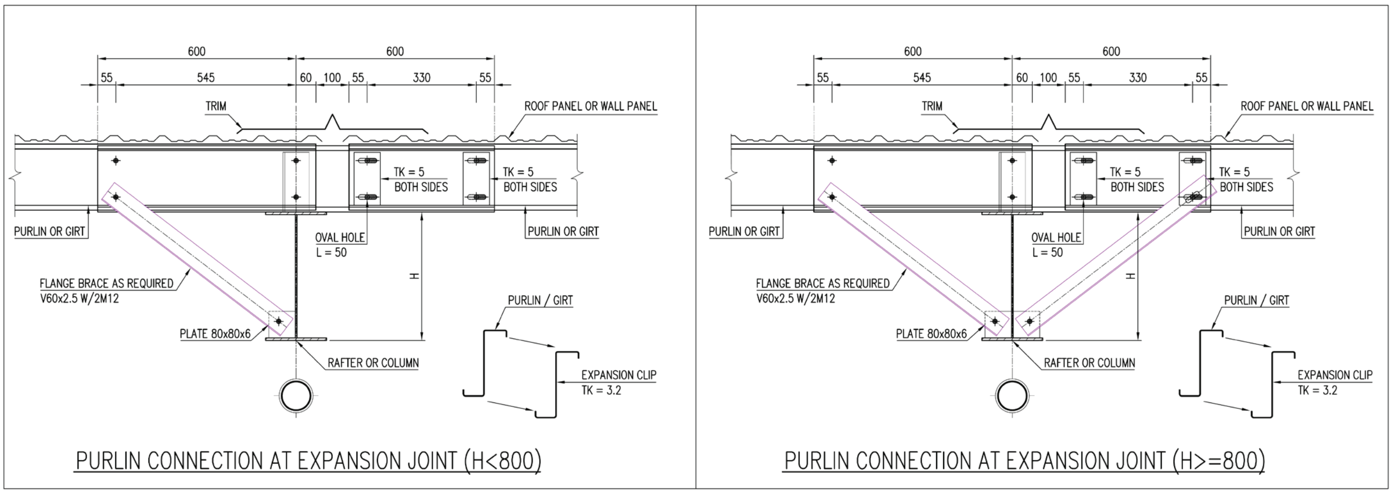


Figure 7 4 Alternative Flange Brace at Expansion Joint Detail

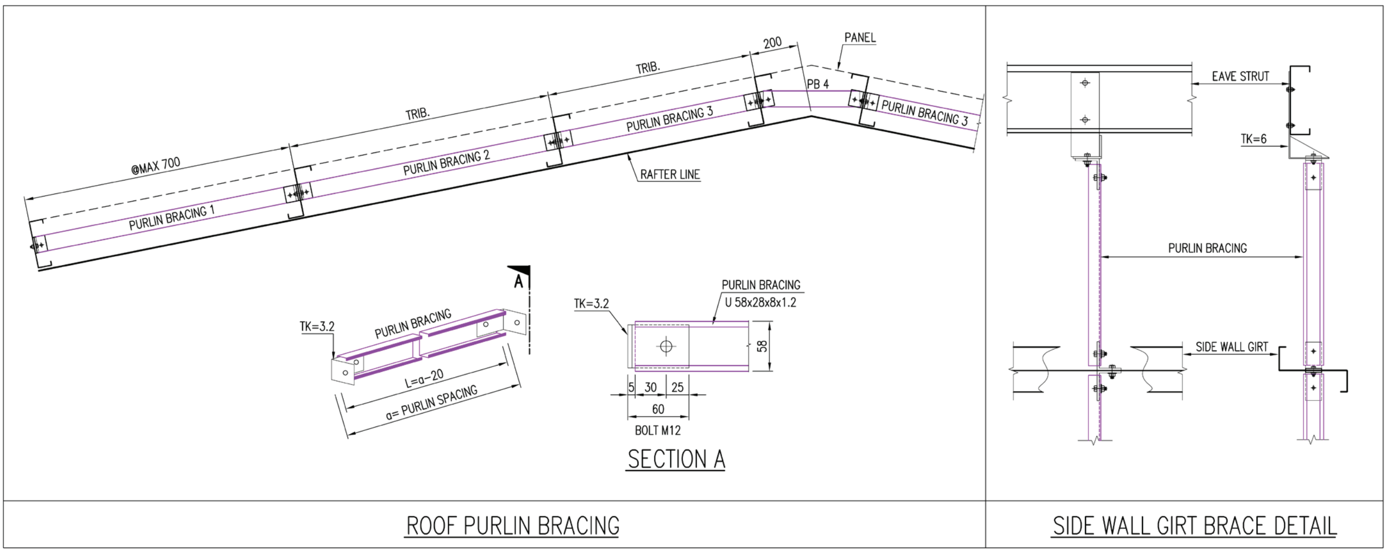


Figure 7 5 Typical Purlin Bracing Details

PURLIN CLEAT SCHEDULE			
PURLIN Z		PURLIN C	
'Y'	CLEAT SIZE	'Y'	CLEAT SIZE
< 500	V80x6	100 – 200	5 MM
> 500	V80x8	201 – 350	6 MM
		351 – 550	8 MM
		551 – 750	10 MM

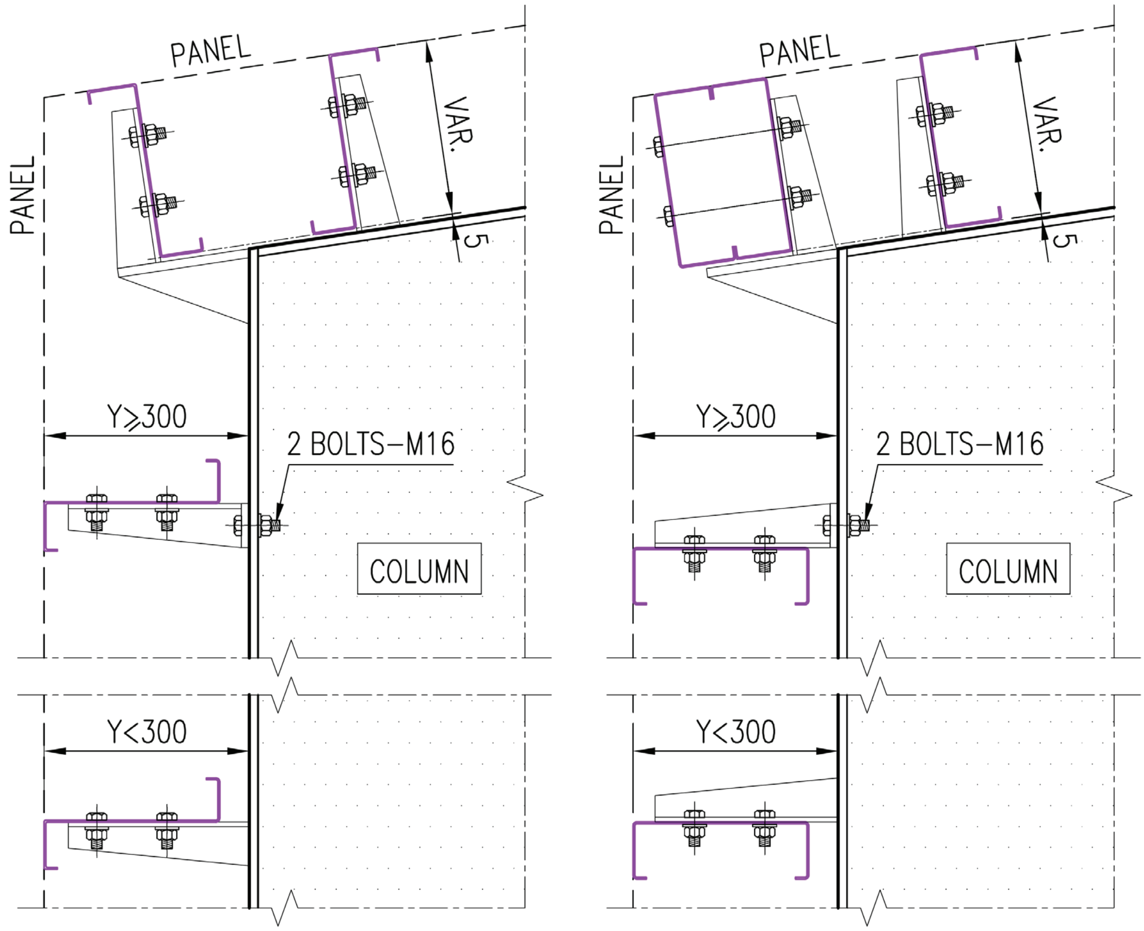


Figure 7 6 Typical Purlin Cleat Details

NOTE
For purlin Z300, C300, use X sag rod bridging.

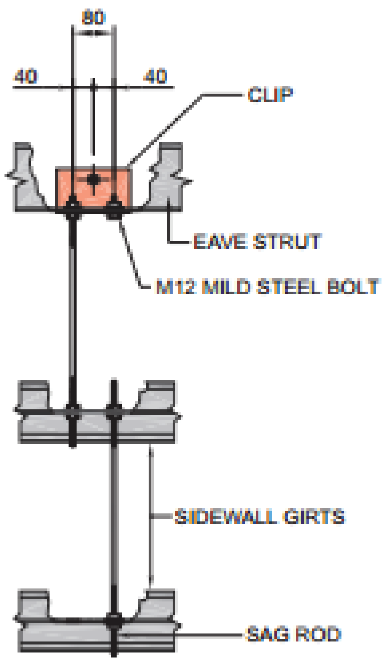


Figure 7.7 Detail A – Sag rod at wall

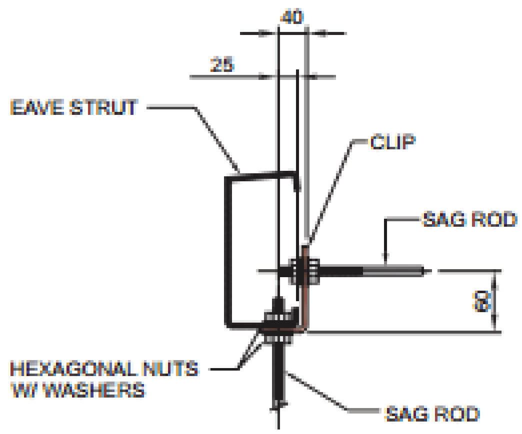


Figure 7.8 Detail B – Sag rod at eave

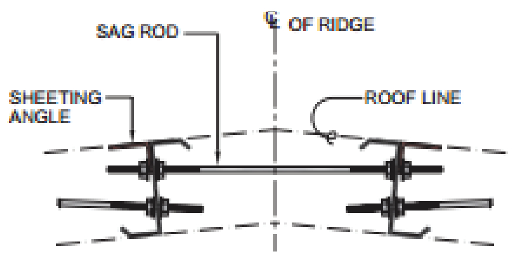


Figure 7.9 Detail C – Sag rod at ridge

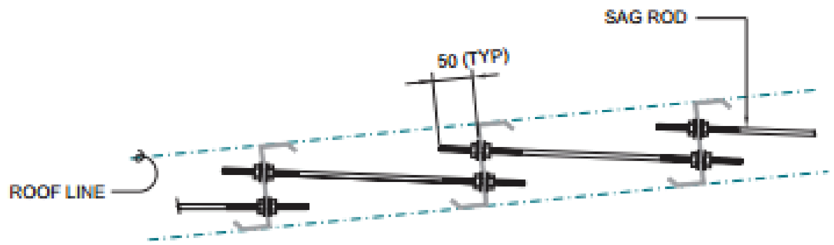


Figure 7.10 Detail D – Sag rod at roof

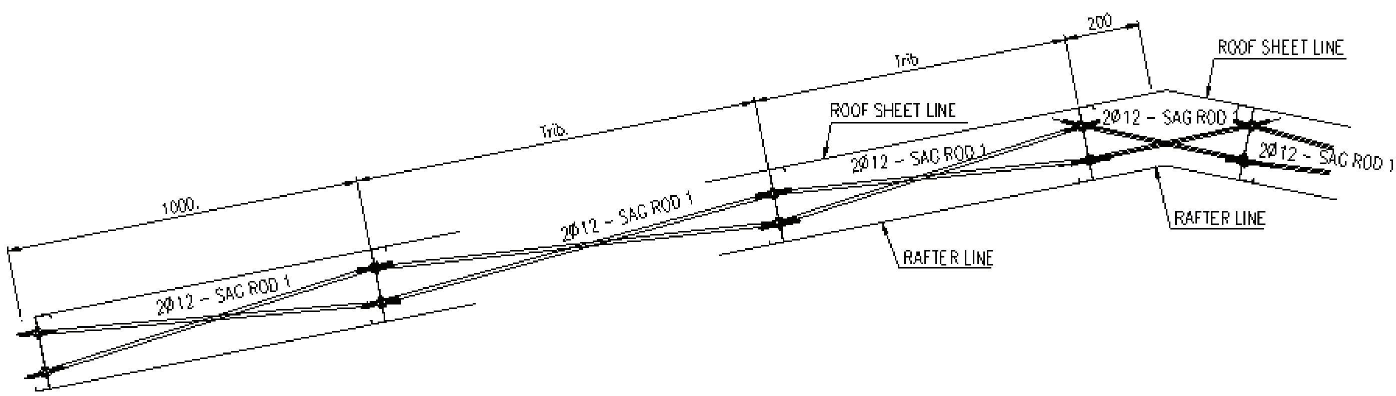


Figure 7.11 X - Sag rod

7.2. Drainage

The purpose of this section is to provide information for gutter capacity, the spacing of drainage downspouts (conductors), and secondary emergency overflow design for the roofs of metal buildings.

Basic equation for rectangular gutter capacity:

$$[L]=\left[\frac{w^{\frac{3}{7}}\times d^{\frac{3}{7}}\times\left(\frac{43200}{R\times I}\right)^{\frac{5}{14}}}{0.481}\right]^{\frac{28}{13}}$$

(2012 MBSM, Eq. A4.3)

Basic equation for downspout capacity:

$$[A]=\frac{I\times R\times L}{1200}$$

(2012 MBSM, Eq. A4.4)

Roof Drainage Area: $L\times W\times B$

where:

- R — Width of roof to be drained, ft.
- L — Length of gutter to be drained, ft.
- w — Gutter width, ft.
- d — Gutter depth, ft.
- I — Rainfall intensity, in./hour. As per BMB&A standard I=250 mm/h.
- B — Multiplier to modify drainage area based on roof pitch

Pitch	Factor "B"
Level to 25%	1.00
> 25% to 40%	1.05
> 40% to 65%	1.10
> 65% to 90%	1.20
> 90% to 100%	1.30

Emergency overflow

Where roof drains are required, secondary (emergency overflow) roof drains or scuppers shall be provided where the roof perimeter construction extends above the roof in such a manner that water will be entrapped if the primary drains allow buildup for any reason.

7.2.2. Example

Building Dimensions: 80m wide x 80m long (10 @ 8m bays).

Symmetrical gable roof configuration. Roof slope: 5%.

Rainfall intensity: I=250 mm/h= 9.84 in./h

Solve

Width of roof to be drained: $L = 8m = 26.25\,ft$

Length of gutter to be drained: $L = 8m = 26.25\,ft$

Factor: $B = 1.00$

Gutter Size 720(width x height): $0.18m \times 0.202m = 0.59\,ft \times 0.66\,ft$

Gutter capacity:

$$[L] = \left[\frac{0.59^{\frac{3}{7}} \times 0.66^{\frac{3}{7}} \times \left(\frac{43200}{131.23 \times 9.84 \times B} \right)^{\frac{5}{14}}}{0.481} \right]^{\frac{28}{13}} \times 0.3048 = 9m \geq L = 8m : OK$$

Downspout capacity: Diameter 220mm thickness 6.6mm

$$[A] = \frac{9.84 \times 131.23 \times 26.25 \times 1}{1200} \times 0.00064516 = 0.018m^2 > A = \pi \frac{0.214^2}{4} = 0.036m^2 : OK$$

APPENDIX A: HOT ROLLED SECTION

Table A.1 I Section

Section	Weight (kg/m)	Section	Weight (kg/m)
I 150 × 75 × 5 ×7	14.0	H100 × 100 × 6 × 8	17.2
I 198 × 99 × 4.5 × 7	18.2	H125 × 125 × 6.5 × 9	23.8
I 200 × 100 × 5.5 × 8	21.3	H 150 × 150 × 7 × 10	31.5
I 248 × 124 × 5 × 8	25.7	H 194 × 150 × 6 × 8	30.6
I 250 × 125 × 6 × 9	29.6	H 175 × 175 × 7.5 × 11	40.4
I 298 × 149 × 5.5 × 8	32.0	H 200 × 200 × 8 × 12	49.9
I 300 × 150 × 6.5 × 9	36.7	H 244 × 175 × 7 × 11	44.1
I 346 × 174 × 6 × 9	41.4	H 250 × 250 × 9 × 14	72.4
I 350 × 175 × 7 × 11	49.6	H 294 × 200 × 8 ×12	56.8
I 400 × 200 × 8 ×13	66.0	H 300 × 300 × 10 × 15	94.0
I 450 × 200 × 9 × 14	76.0	H 350 × 350 × 12 × 19	137.0
I 496 × 199 × 9 × 14	79.5	H 390 × 300 × 10 × 16	107.0
I 500 × 200 × 10 × 16	89.6	H 400 × 400 × 13 × 21	172.0
I 600 × 200 × 11 × 17	106.0	H 482 × 300 × 11 × 15	114.0
I 700 × 300 × 13 ×24	185.0	H 582 × 300 × 12 × 17	137.0
		H 588 × 300 × 12 × 20	151.0

Table A.2 Channel Section

Section	Weight (kg/m)
U 200 x 80 x 7.5 x 11 x 12m	24.60
U 250 x 90 x9 x 13 x 12m	34.60
U 300 x 90 x 9 x 13 x 12m	38.10
U 100 x 50 x5 x 6m	9.36
U 120 x 53 x 5.5 x 12m	12.06
U 140 x 58 x 6 x 12m	14.50
U 150 x 75 x 6.5 x 12m	18.60
U 160 x 56 x 5.2 x12m	12.50
U 160 x 65 x 8,5 x 12m	19.80
U 180 x 67 x 5.4 x 12m	15.00
U 200 x 73 x 7 x 12m	22.60
U 200 x 75 x 9 x 12m	23.60
U 250 x 78 x 7 x 12m	24.60
U 300 x 87 x 9.5 x12m	39.10

Table A.3 Angle Section

Section	Weight (kg/piece)	Section	Weight (kg/piece)
V 25 x 3 x 6m	6.72	V 90 x 6 x 6m	51.0
V 30 x 3 x 6m	8.16	V90 x 7 x 6m	57.7
V 40 x 3 x 6m	11.10	V 90 x 8 x 6m	65.4
V 40 x 4 x 6m	14.52	V 100 x 7 x 6m	64.8
V 50 x 3 x 6m	13.92	V 100 x 8 x 6m	73.2
V 50 x 4 x 6m	18.30	V 100 x 10 x 6m	87.0
V 50 x 5 x 6m	22.62	V 120 x 8 x 12m	14.2
V 63 x 4 x 6m	23.40	V 120 x 10 x 12m	18.3
V 63 x 5 x 6m	28.86	V 120 x 12 x 12m	21.6
V 63 x 6 x 6m	34.32	V 130 x 10 x 12m	19.75
V 70 x 6 x 6m	38.34	V 130 x 12 x12m	23.4
V 70 x 7 x 6 m	44.34	V 150 x 10 x 12m	22.9
V 75 x 6 x 6m	41.34	V 150 x 12 x 12m	27.3
V 75 x 8 x 6m	54.12	V 150 x 15 x 12m	33.6
V 80 x 6 x 6m	44.16		
V 80 x 8 x 6m	51.06		

Table A.4 Pipe Section

Section	Weight (kg/piece)	Section	Weight (kg/piece)
Ø60 x 2.3	Ø76 x 5	Ø114 x 3.2	Ø168.3 x 5.4
Ø60 x 2.6	Ø90 x 2.3	Ø114 x 3.6	Ø168.3 x 5.56
Ø60 x 2.9	Ø90 x 2.5	Ø114 x 4	Ø168.3 x 6.35
Ø60 x 3	Ø90 x 2.7	Ø114 x 4.5	Ø168.3 x 6.55
Ø60 x 3.2	Ø90 x 2.9	Ø114 x 5	Ø168.3 x 7.11
Ø60 x 3.6	Ø90 x 3	Ø114 x 5.4	Ø219.1 x 3.96
Ø60 x 4	Ø90 x 3.2	Ø141.3 x 3.96	Ø219.1 x 4.78
Ø60 x 4.5	Ø90 x 3.6	Ø141.3 x 4.78	Ø219.1 x 5.16
Ø76 x 2.5	Ø90 x 4	Ø141.3 x 5.16	Ø219.1 x 5.56
Ø76 x 2.7	Ø90 x 4.5	Ø141.3 x 5.56	Ø219.1 x 6.35
Ø76 x 2.9	Ø90 x 5	Ø141.3 x 6.35	Ø219.1 x 6.55
Ø76 x 3.2	Ø114 x 2.5	Ø141.3 x 6.55	Ø219.1 x 7.11
Ø76 x 3.6	Ø114 x 2.7	Ø168.3 x 3.96	Ø219.1 x 8.18
Ø76 x 4	Ø114 x 2.9	Ø168.3 x 4.78	
Ø76 x 4.5	Ø114 x 3	Ø168.3 x 5.16	

Table A.5 Tube in Square Section

Section	W(kg/m)	Section	W(kg/m)	Section	W(kg/m)
☐ - 20x20x1.2	0.7	☐ - 80x80x4.5	10.3	☐ - 250x250x9	66.5
☐ - 20x20x1.6	0.87	☐ - 90x90x2.3	6.23	☐ - 250x250x12	86.8
☐ - 25x25x1.2	0.87	☐ - 90x90x3.2	8.51	☐ - 250x250x16	112.4
☐ - 25x25x1.6	1.12	☐ - 90x90x4.5	11.7	☐ - 300x300x6	54.7
☐ - 25x25x2.3	1.53	☐ - 90x90x6	15.1	☐ - 300x300x9	80.6
☐ - 25x25x3.2	1.98	☐ - 100x100x2.3	6.95	☐ - 300x300x12	106
☐ - 30x30x1.2	1.06	☐ - 100x100x3.2	9.52	☐ - 300x300x16	138
☐ - 30x30x1.6	1.38	☐ - 100x100x4	11.7	☐ - 300x300x19	160
☐ - 30x30x2.3	1.73	☐ - 100x100x4.5	13.1	☐ - 350x350x6	64.1
☐ - 30x30x3.2	2.48	☐ - 100x100x6	17	☐ - 350x350x9	94.7
☐ - 40x40x1.6	1.88	☐ - 100x100x9	24.1	☐ - 350x350x12	124
☐ - 40x40x2.3	2.62	☐ - 100x100x12	30.2	☐ - 350x350x16	163
☐ - 40x40x3.2	3.49	☐ - 125x125x3.2	12	☐ - 350x350x19	190
☐ - 50x50x1.6	2.38	☐ - 125x125x4.5	16.6	☐ - 400x400x9	109
☐ - 50x50x2.3	3.34	☐ - 125x125x5	18.3	☐ - 400x400x12	143
☐ - 50x50x3.2	4.5	☐ - 125x125x6	21.7	☐ - 400x400x14	166
☐ - 50x50x4.5	60.2	☐ - 125x125x9	31.1	☐ - 400x400x16	188
☐ - 50x50x6	7.56	☐ - 125x125x12	39.7	☐ - 400x400x19	220
☐ - 60x60x2.3	4.06	☐ - 150x150x4.5	20.1	☐ - 400x400x22	251
☐ - 60x60x3.2	5.5	☐ - 150x150x6	26.4	☐ - 450x450x9	122
☐ - 60x60x4.5	7.43	☐ - 150x150x9	38.2	☐ - 450x450x12	160
☐ - 60x60x6	9.45	☐ - 175x175x4.5	23.7	☐ - 450x450x16	209
☐ - 75x75x2.3	5.14	☐ - 175x175x6	31.1	☐ - 450x450x19	250
☐ - 75x75x3.2	7.01	☐ - 200x200x4.5	27.2	☐ - 450x450x22	286
☐ - 75x75x4.5	9.55	☐ - 200x200x6	46.9	☐ - 500x500x12	181
☐ - 75x75x6	12.3	☐ - 200x200x9	52.3	☐ - 500x500x16	238
☐ - 80x80x2.3	5.5	☐ - 200x200x12	67.9	☐ - 500x500x19	280
☐ - 80x80x3.2	7.51	☐ - 250x250x6	45.2	☐ - 500x500x22	320

Table A.6 Tube in Rectangular Section

Section	W(kg/m)	Section	W(kg/m)	Section	W(kg/m)
□ - 30x20x1.6	1.38	□ - 75x45x2.3	4.06	□ - 150x75x6	19.34
□ - 30x20x2.3	1.53	□ - 75x45x3.2	5.50	□ - 150x100x3.2	12.00
□ - 30x20x3.2	1.98	□ - 75x45x4.5	7.43	□ - 150x100x4.5	16.60
□ - 40x20x1.6	1.38	□ - 75x50x2.3	4.24	□ - 150x100x6	21.70
□ - 40x20x2.3	1.73	□ - 75x50x3.2	5.75	□ - 150x100x9	31.10
□ - 40x20x3.2	2.47	□ - 75x50x4.5	7.79	□ - 200x100x4.5	20.10
□ - 50x20x1.6	1.63	□ - 75x50x6	9.92	□ - 200x100x6	26.40
□ - 50x20x2.3	2.25	□ - 100x50x2.3	5.14	□ - 200x100x9	38.20
□ - 50x30x1.6	1.88	□ - 100x50x3.2	7.01	□ - 200x150x4.5	23.70
□ - 50x30x2.3	2.62	□ - 100x50x4.5	9.55	□ - 200x150x6	31.10
□ - 50x30x3.2	3.49	□ - 100x50x6	12.30	□ - 200x150x9	45.30
□ - 50x30x4.5	4.61	□ - 125x75x2.3	6.95	□ - 250x150x6	35.80
□ - 60x30x1.6	2.13	□ - 125x75x3.2	9.52	□ - 250x150x9	52.30
□ - 60x30x2.3	2.98	□ - 125x75x4.5	13.10	□ - 250x150x12	67.90
□ - 60x30x3.2	3.99	□ - 125x75x6	17.00	□ - 300x200x6	45.20
□ - 60x40x1.6	2.38	□ - 150x50x2.3	6.95	□ - 300x200x9	66.50
□ - 60x40x2.3	3.34	□ - 150x50x3.2	9.53	□ - 300x200x12	86.80
□ - 60x40x3.2	4.50	□ - 150x50x4.5	13.10	□ - 400x200x6	54.70
□ - 75x20x1.6	2.25	□ - 150x50x6	17.00	□ - 400x200x9	80.60
□ - 75x20x2.3	3.16	□ - 150x75x3.2	10.80	□ - 400x200x12	106.00
□ - 75x45x1.6	2.88	□ - 150x75x4.5	14.90		

Table A.7 Downspout

Size	Diameter (m)	Thickness (m)
[]110x95	0.110 x 0.095	-
Ø90 x 2.9	0.090	0.0029
Ø114 x 3.8	0.114	0.0038
Ø168 x 4.3	0.168	0.0073
Ø220 x 6.6	0.220	0.0066
Ø250 x 7.3	0.250	0.0073
Ø280 x 8.2	0.280	0.0082
Ø315 x 9.2	0.315	0.0092
Ø400 x 11.7	0.400	0.0117

Viet Nam Office

Head Office : 148 Phan Xich Long St., Ward 7
Phu Nhuan Dist., HCMC - Viet Nam

Tel : (+8428) 35 172 509/11 - 35 172 601
Website : www.bmbsteel.com.vn
Email : sales@bmbsteel.com.vn

Ha Noi Office : B22 Trung Hoa, Nhan Chinh,
Urban Area, Thanh Xuan, Ha Noi

Tel - Fax : (+844) 39 412 176/77 - 39 412 178

Factory 1 : Capacity 2,500 MT/month
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Binh Duong - Viet Nam

Oversea Office

Myanmar Office : No.103 - Block C - Yuzana High Way,
Complex - Yangon - Myanmar

Tel : (+951) 230 4370
Fax : (+951) 230 4191

Cambodia Office : G16 Golden Street, City 307-309,
Street 271, Tumnup Toeuk quater,
Chamkar mon Dist, Phnom Penh

Tel : (+855) 23 21 54 03/04
Fax : (+855) 23 21 54 14

Middle East Shield International - P.O. Box 144323,
Amman 11814, Jordan - Exclusive Agency

Tel : (+962) 6 585 2552
Fax : (+962) 6 585 8841
Email : info@shield-inter.com

Philippines office : 14th Floor, Unit 1415 AIC Burgundy Empire Tower,
ADB Avenue Corner, Garnet Road, Ortigas Center,
Pasig City, Philippines

Tel : (+632) 470 7633 / 33 / 34

Indonesia office : Cikarang Industrial part, Cibitung - Bekasi,
Jawa Barat province, Indonesia
Email : qth@bmbsteel.com.vn

Thailand office : 54 Sukhumvit Road (Asoke), North Klongtoey,
Wattna, Bangkok 10110

Tel : +66 2664 4493
Fax : +66 2664 4492

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