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A Parametric Study for the Design of Truss Building Subjected To Wind Actions

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Abstract: Kingdom of Saudi Arabia (KSA) is rapidly developing with the government's vision (Saudi 2030 Vision) of working towards a developed nation by the year 2030. Government have implemented various types of development such as schools, government offices, hospitals, factories and, housing schemes, etc. It also aims to implement green building designs thereby using a recyclable structural material for construction such as *Steel* is a positive step. The use of truss design, construction, erection and installation is very popular due to it conventional configurations and popularity within the industry for its long spanning and lightness. Nevertheless, its selection and optimum design still needs proper attention. This paper addresses a parametric study to analyze and design single-story industrial buildings located in *Madinah City* using structural analysis and design software SAP2000®. Standards of the Saudi Building Codes SBC are used for the design purpose. The basic data related to wind intensity; dead and live loadings are obtained from SBC301. The dimensions of the two industrial buildings are (20mx35m and 30mx60m) with single span of 20m and 30m. Two different types of trusses are used thus gave rise to four cases. The design is carried out for the maximum capacities (compression, tension and shearing) of structural members to guarantee the structural safety. The purpose of this study is to understand the design philosophy and to give useful recommendations for the optimal and safe design of trusses with minimization of construction cost and time savings of industrial buildings.
Keywords: Parametric Analysis, Steel Trusses, Industrial Buildings, Analysis, Design

INTRODUCTION

The modern industrial buildings generally are steel framed structures or combined skeletal structures. The choice of the truss configuration depends on operating conditions, considerations related to saving on major construction materials. Industrial buildings may be categorized as normal type industrial buildings and special type industrial buildings. Normal types of industrial building are shed type buildings with simple roof structures on open frames. These buildings are used for workshop; warehouses etc. and require large areas unobstructed by the columns. The large floor area provides sufficient flexibility and facility for later change in the production layout without major building alterations. Special types of industrial buildings are steel mill buildings used for the manufacture of heavy machines, production of power etc. Typically the bays in industrial buildings have frames spanning in the longitudinal direction. Several such frames are arranged at a suitable spacing to get the required length of the building. Depending upon the requirement, several bays may be constructed adjoining each other. The choice of structural configuration depends upon the span between the rows of columns, the headroom or clearance required and the nature of roofing material. If the span is less, portal frames such as steel bents or gable frames can be used, but if the span is large then buildings with trusses are used. Different types of the floor are required for their use such as production, workshop, stores, amenities, and administration. The service condition varies widely in these areas, so different floors types are required. Industrial floors shall have sufficient resistance to abrasion, impact, acid action and temperatures depending on the type of activity carried out. Trusses are comprised of assemblies of tension and compression elements. Under gravity loads, the top and bottom chords of the truss provide the compression and tension resistance to overall bending, and the bracing resists the shear forces.

The use of industrial buildings in Saudi Arabia is quite common due to existence of huge industry. The design of such buildings needs attention as they are temporary structures and generally build for a life spanning 10-20 years. Some advantages of using trusses in industrial buildings are for example time-saving (delay minimization and fast construction), cost saving (easily remodelled, repairs and maintenance), materials saving (less material, high bearing strength and recyclability) and labour saving (construction time reduction) [1]. The material used in the design of trusses such as steel is also recyclable which is the aim of green building design and which is one of the goal of 2030 vision of Saudi Arabia [2]. Standards of the Saudi Building Codes SBC, SBC301 [3] are used for the design purpose. This study deals with the design and analysis of trusses under wind actions using SAP 2000 software ® [4]. In this study industrial buildings, made of two different type of trusses (20mx35m and 30mx60m), were aim to analyze with specific spans of 20m and 30m located in the vicinity of *Madinah Al-Munawarah City*, the Eastern Part of Al-Hijaz Region that is located in the Western Part of the Kingdom of Saudi Arabia (KSA). The basic material adopted is steel due to its properties of high strength, stiffness, toughness, and ductility. It is one of the most common materials used in commercial and industrial building construction. The basic data related to wind intensity; dead and live loadings are obtained from the Saudi Building Code [3, 5, 6].

A wide range of truss forms can be created and each can vary in overall geometry and in the choice of the individual elements profile and its connections. Two different types of trusses are used in this study: *a)* Pratt Truss and *b)* Warren Truss connected through the use of bolts and gusset plates. The dead loads consist of the weight of all materials of construction incorporated into the building including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items and fixed service equipment including the weight of cranes. Nevertheless as there are no other loadings except the self-weight therefore in this case the deck metal 18 gages: 0.15 kN/m^2 is adopted. Live loads are produced by the use and occupancy of the building or other structure and do not include construction or environmental loads such as wind load, rain load, earthquake load, flood load, or dead load. Therefore in this case Live loads on roof are *(1)* during maintenance by workers, equipment, and materials, and *(2)* during the life of the structure by movable objects and are equals 1.0KN/ m^2.

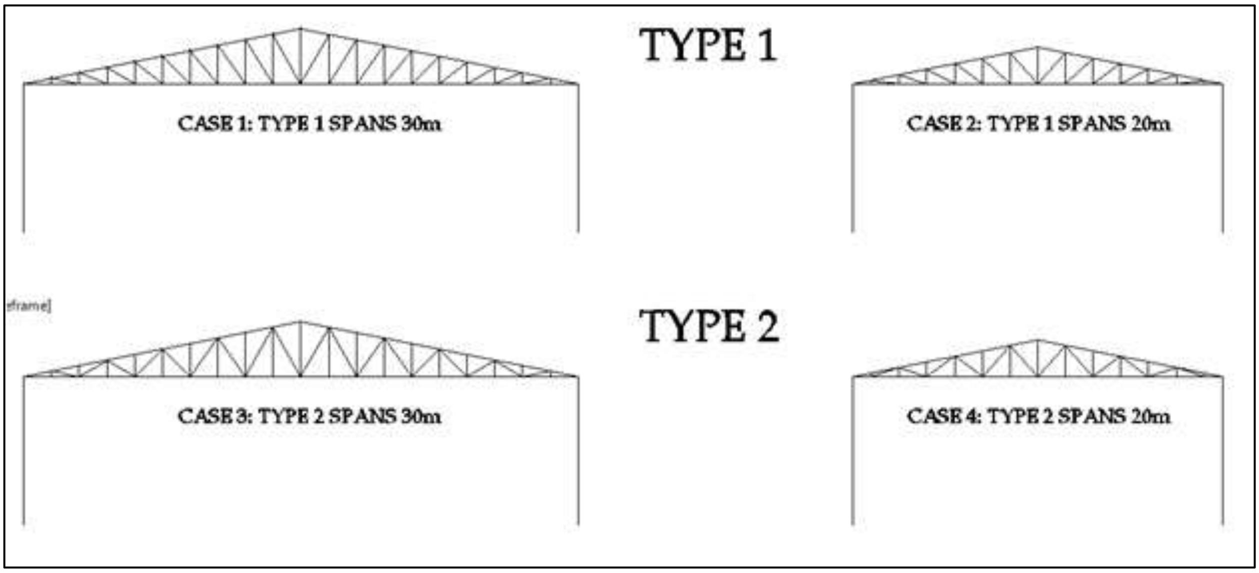


Fig-1: Different cases considered for the design purpose Type 1 Pratt Truss and Type 2 Warren Truss

Wind load is calculated for a wind speed of 25m/sec. The uplift is less than the vertical downward forces, therefore, are not considered in this study. Nevertheless, for the design of connections and foundations, it is necessary to consider [7] the uplift forces and therefore appropriate connections are required to transfer member forces from one to another member safely and effectively. Four different cases are studied in this project as shown in Figure 1. The calculated loads on a typical purlin of truss are shown in Figure-2.

Dead Load of Sheeting	DLs	0.225 KN/m	Dead Load of beam UPN120	DLb	0.134 KN/m		
Live Load	LL	1.5 KN/m					
Wind Load	WL	2.1 KN/m	L	5 m			
	Total DL	0.359 KN/m				Reactions for Strength	
			Moment			Rleft_comb_1	4.827 KN
Comb 1	1.2DL+L	1.9308 KN/m	6.03375 KNM			Rleft_comb_2	3.702 KN
Comb 2	1.2DL+0.5W	1.4808 KN/m	4.6275 KNM			Rleft_comb_3	10.077 KN
Comb 3	1.2DL+W+L	4.0308 KN/m	12.59625 KNM			Rleft_comb_4	1.077 KN
Comb 4	1.2D	0.4308 KN/m	1.34625 KNM				
						Reactions for Srviceability	
Comb 5	0.9D+W	2.4231 KN/m				Rleft_comb_1	6.05775 KN
Comb 6	D+L	1.859 KN/m				Rleft_comb_2	4.6475 KN

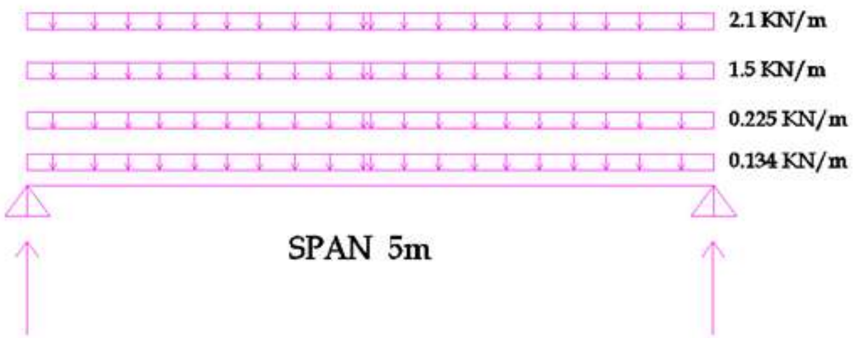


Fig-2: Calculated loads on a typical purlin of truss

The method of sections (*Truss Analysis*) is used to calculate the internal forces in the truss members [8, 9]. The analysis is carried out for several truss members (bottom chord) for Case 4. The reaction of the truss is 150 KN. These values are further verified by using SAP 2000 as shown in Table-1.

Table-1: Comparison between manual and SAP2000 results

Members Label	Members Length (m)	Forces (KN) Manually	Forces (KN) SAP 2000	Nature
13	1	650	650	Tension
32	1.5	590	590	Tension
34	1.5	487	486.4	Tension
36	1.5	386	385.3	Tension

Table-1 shows the comparison between manual and SAP2000 calculations. Such comparisons are necessary prior to carry a complete analysis and design of the structure as these calculations validate the accuracy and the geometry of the model. Hence the model is validated for executing a parametric analysis for design purpose.

NUMERICAL MODELLING

The geometry for all the trusses are drawn in AutoCAD and imported in SAP2000 ®. The modelling assumptions are such that the trusses are assumed to be the primary load bearing whereas purlins are simply supported and are released (pinned connected with the main trusses as shown in Figure-3 left), and therefore only transferring the shear.

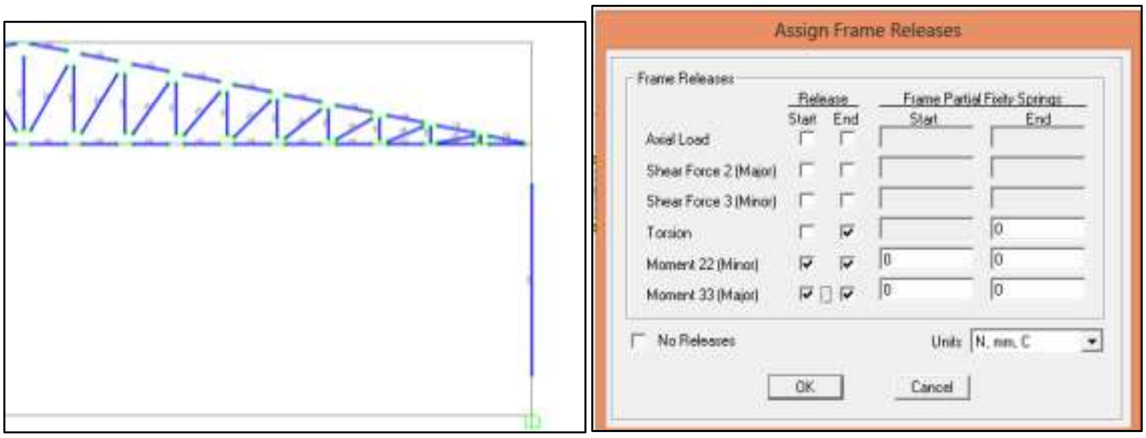


Fig-3: (left) Portion of truss shows member releases, (right) member releases definition in SAP2000

A portion of the the geometry of the trusses is shown in Figure-3 (left) the members of which are released from any moments. Figure-3 (right) shows the definition of the releases. For analysis purpose, a section profile with unit dimension is assumed to check the axial forces in the truss members under the applied ultimate limit state.

The industrial buildings considered for the design purpose are 20mx35m (consists of 8 Trusses) and 30mx60m (consists of 21 Trusses). The purlins are spaced at 5m center-to-center (c/c). The overall height of the 20mx35m building is 10.0 m above ground level with the height of the wall as 8.00 m. Whereas the overall height of the 30mx60m building is 11.0 m above ground level and the height of the wall is 8.00 m. The basic components of the truss are upper chord, lower chord, vertical side members (posts) and diagonal side members. The purlins are kept at a distance of 2.5m along the top chord. The depth of trusses is kept almost 1/10th of the Span. The buildings have a rectangular floor plan with the main truss frames spanning the shorter distance. Both upper and lower chords in the trusses of the building are U (channel) profiles closely spaced built-up members with a spacing of 15mm which governs the thickness of the gusset plate. These chords are hot rolled members and are connected by bolted connections of M16 diameter, grade 8.8 with the said gusset plate thickness of 14mm. The functions of the chords elements are analogous to the function of flanges in an I-beam carrying the applied external forces having compression at one flange while tension in other flange and thus remian in equilibrium due to couple. The flanges in an I-beam are connected usually through another plate, which works as web element to keep the flanges in the proper position, it is also interesting to note that the distance in between the flanges the so-called lever arm, governs the load carrying capacity (the applied moments) of the section. Exactly the same phenomena occur in the case of a truss where the two chords work as flanges and to keep these chords (as previously are flanges in the case of the I-beam), side members (diagonal and verticals) are provided (as previously is the web in the case of the I-beam). The main construction of the roof system consists of secondary beams (purlins) which support the 18 gauge roof system.

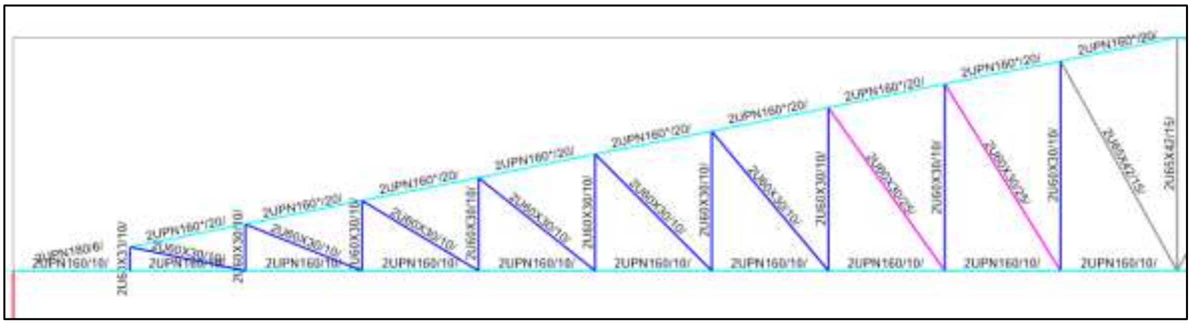


Fig-4: Portion of sections of truss (type 1 and spanning 30m)

The purlins are the secondary members made of hot rolled steel UPN 120 profiles. The span of each purlin is 5000mm connected with the flange of the upper chord. The upper chord (2-UPN-160) is continuous (closely spaced built-up member of 12m) along the connections with an internal splice at 12m from the supports. The members are interconnected by mean of a gusset plate 90x60x18mm. The bottom chord (2-UPN-120) is continuous (closely spaced built-up member of 12m) along the connections with an internal splice of 12m provided at the mid of the truss. All truss connections are provided by 18mm gusset plate with f_u equals 430Mpa of S-275 steel. The bolts are M16 grade 8.8 except the connection where the two chords meet (i.e. at the supports) and at splice connections where M20 grade 8.8 bolts are used to avoid high numbers of bolts as here the gusset has a higher length.

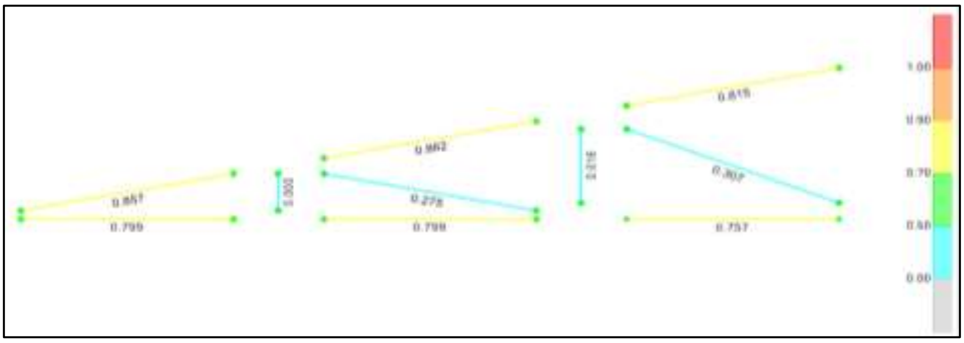
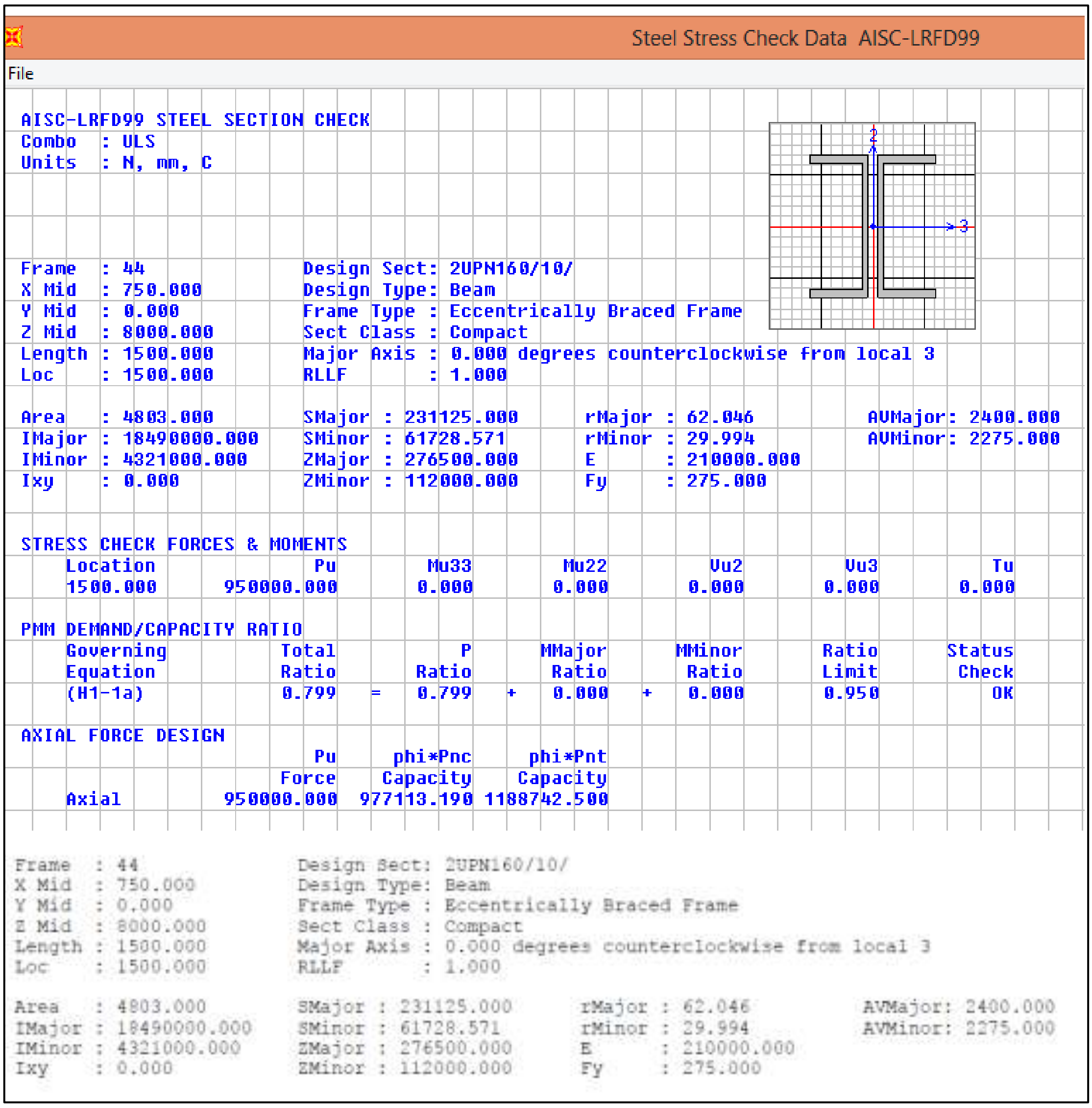


Fig-5: Demand to capacity ratios for edge members of case 1 - verification purpose



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Steel Stress Check Data  AISC-LRFD99
File

AISC-LRFD99 STEEL SECTION CHECK
Combo  : ULS
Units  : N, mm, C

Frame  : 44                Design Sect: 2UPN160/10/
X Mid  : 750.000           Design Type: Beam
Y Mid  : 0.000             Frame Type : Eccentrically Braced Frame
Z Mid  : 8000.000          Sect Class : Compact
Length : 1500.000          Major Axis : 0.000 degrees counterclockwise from local 3
Loc    : 1500.000          RLLF       : 1.000

Area   : 4803.000          SMajor : 231125.000    rMajor : 62.046        AVMajor: 2400.000
IMajor : 18490000.000      SMinor : 61728.571     rMinor : 29.994        AVMinor: 2275.000
IMinor : 4321000.000       ZMajor : 276500.000    E      : 210000.000
Ixy    : 0.000             ZMinor : 112000.000    Fy     : 275.000

STRESS CHECK FORCES & MOMENTS
    Location             Pu        Mu33       Mu22        Vu2        Vu3         Tu
    1500.000     950000.000       0.000      0.000      0.000      0.000      0.000

PMM DEMAND/CAPACITY RATIO
    Governing           Total          P     MMajor     MMinor      Ratio     Status
    Equation            Ratio      Ratio      Ratio      Ratio      Limit      Check
    (H1-1a)             0.799  =   0.799  +   0.000  +   0.000      0.950         OK

AXIAL FORCE DESIGN
                           Pu     phi*Pnc     phi*Pnt
                        Force    Capacity    Capacity
    Axial          950000.000  977113.190 1188742.500
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Frame  : 44                Design Sect: 2UPN160/10/
X Mid  : 750.000           Design Type: Beam
Y Mid  : 0.000             Frame Type : Eccentrically Braced Frame
Z Mid  : 8000.000          Sect Class : Compact
Length : 1500.000          Major Axis : 0.000 degrees counterclockwise from local 3
Loc    : 1500.000          RLLF       : 1.000

Area   : 4803.000          SMajor : 231125.000    rMajor : 62.046        AVMajor: 2400.000
IMajor : 18490000.000      SMinor : 61728.571     rMinor : 29.994        AVMinor: 2275.000
IMinor : 4321000.000       ZMajor : 276500.000    E      : 210000.000
Ixy    : 0.000             ZMinor : 112000.000    Fy     : 275.000
```


Fig-6: SAP 2000 report for the selected member.

The connection of truss to the column is provided by extending the gusset plate of the chord connections vertically and thus rested on the cap plate (which is shop welded with the cleats) through-bolted connection with the cleats (M20 8.8 bolts). The top cap plate is then bolted using 4 number of M20 grade 8.8 bolts with another cap plate which is shop welded at the top of the column. Thus two cap plates both of 10mm thickness are provided at this connection. The lower cap plate is shop welded with the column while the upper cap plate is shop welded with the cleats (L-75x75x6mm). The columns are HEB-200 profile of 8000mm height. The truss to column connection is a pinned connection. The dimension of the base plate is 700x520x35mm, with stiffeners 300x180x20mm to have a full strength and rigid connection. Tweleve numbers of M30 grade 8.8 anchor bolts are used, 6 bolts at tension side and 6 bolts at compression side. The length of the anchor bolts is 700mm properly embedded in trapezoidal isolated pad (footing) of class C25/30 concrete. The external wall envelope is provided by the cladding system being 1000/32mm reverse-steel cladding system with a thickness of 0.9 mm. The cladding sheets are supported by cold-formed double channel profiles. In order to ensure an acoustic isolation, a layer of 100mm mineral wool is introduced between two cladding sheets.

Figure-4 shows the sections of truss (type 1 and spanning 30m). Figure 5 shows the demand to capacity ratios for edge members of Case 1 for the verification purpose, whereas Figure-6 shows a detail report of the design for the selected member.

Figure-7 shows the portion of the deformed shape of the trusses for deflection check for case 1 and 2. Specifically left portion of this Figure shows the deflection of the truss under ULS for case 1 in vertical direction in which deflection equals 111.3mm (type 1 and spanning 30m). The right portion of Figure-7 shows the deflection under ULS for case 2 in vertical direction in which deflection equals 60.2mm (type 1 and Spanning 20m).

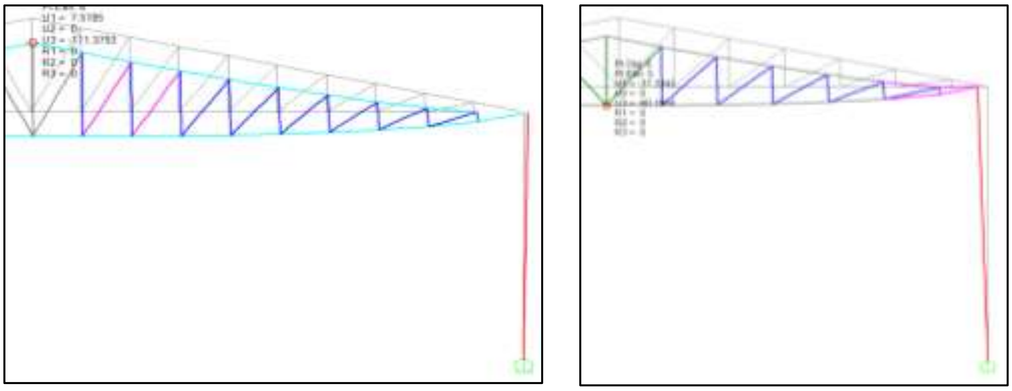


Fig-7: Portion of the deflected shape, (left) deflection under ULS for case 1 (right) deflection under ULS for case 2

Figure-8 shows portion of the deformed shape of the trusses for deflection check for case 3 and case 4; the left side of the figure shows the deflection in vertical directions under ULS for case 3 in which deflection equals 84.5mm (type 2 and spanning 30m). The right side of Figure 8 shows the deflection under ULS for case 4 in vertical direction in which deflection equals 62.0mm (type 2 and spanning 20m).

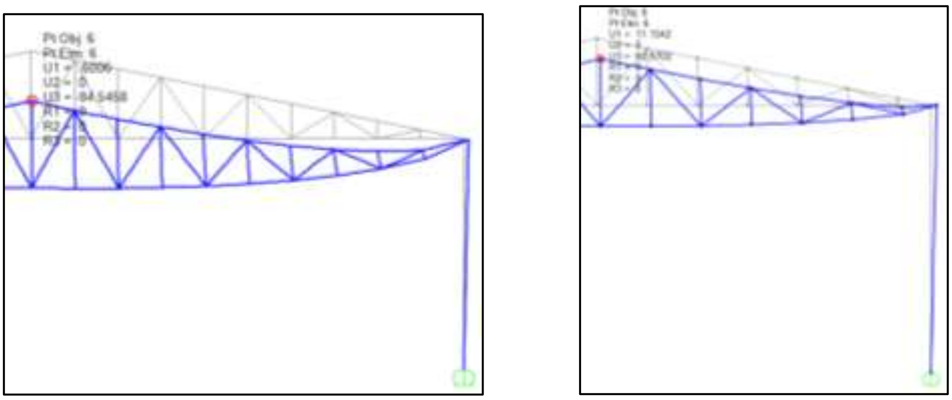


Fig-8: Portion of the deflected shape, (left) deflection under ULS for case 3 (right) deflection under ULS for case 4

The adapted material for the steel sections is S275 with its properties as mentioned in Table-2. The design is carried out using these material properties.

Table-2: Mechanical features of the adopted material design purpose

Part	Material	f_y (MPa)	f_u (MPa)	□$_u$ (%)	E (MPa)	Poisson's Ratio
Truss members/Columns	S-275 Steel Grade	275	430	10	210000	0.3

Table-3 shows the base reactions for all four adopted cases. Since the ultimate load case is considered where downward wind load is considered therefore the only vertical reactions are shown here.

Table-3: Base reactions for all four cases

Case Type	Global FZ (Kgf)
1	4093.78
2	2804.30
3	5419.55
4	3099.9

Table-4 shows steel design – summary data to AISC-LRFD 99 [10] where the maximum design to capacity ratio in steel section is recorded as 0.84 showing the adequacy of the steel sections used. This table also includes the member number, profile, demand to capacity ratio and load combination. It is also possible to carry out an optimized design with optimum demand to capacity ratio but in that case many profiles will be utilized and thereby it will produce difficulty during fabrication and even erection. Nevertheless such study is required to check the adequacy and the pro-and-cons of truss type on one another but it is considered beyond the scope of the current paper.

Table-4: Steel design – Summary data to AISC-LRFD 99

Frame	Profile	D/C Ratio	Combo
1	HE240B	0.09	ULS
3	HE240B	0.09	ULS
4	2U60X30*/10/	0.36	ULS
7	2U60X30*/10/	0.00	ULS
8	2U60X30*/10/	0.05	ULS
9	2U60X30*/10/	0.00	ULS
10	2U60X30*/10/	0.04	ULS
11	2U60X30*/10/	0.00	ULS
12	2U60X30*/10/	0.03	ULS
14	2U60X30*/10/	0.43	ULS
15	2U60X30*/10/	0.31	ULS
16	2U60X30*/10/	0.73	ULS
17	2U60X30*/10/	0.00	ULS
18	2U60X30*/10/	0.05	ULS
19	2U60X30*/10/	0.00	ULS
20	2U60X30*/10/	0.04	ULS
21	2U60X30*/10/	0.00	ULS
22	2U60X30*/10/	0.03	ULS
23	2U60X30*/10/	0.43	ULS
24	2U60X30*/10/	0.31	ULS
25	2U60X30*/10/	0.73	ULS
26	2U60X30*/10/	0.25	ULS
27	2U60X30*/10/	0.11	ULS
28	2U60X30*/10/	0.10	ULS
29	2U60X30*/10/	0.25	ULS
30	2U60X30*/10/	0.11	ULS
31	2U60X30*/10/	0.10	ULS
13	2UPN140*/6/	0.65	ULS
32	2UPN140*/6/	0.59	ULS
33	2UPN140*/6/	0.59	ULS
34	2UPN140*/6/	0.49	ULS

35	2UPN140*/6/	0.49	ULS
36	2UPN140*/6/	0.39	ULS
37	2UPN140*/6/	0.39	ULS
39	2UPN140*/6/	0.44	ULS
41	2UPN140*/6/	0.44	ULS
42	2UPN140*/6/	0.57	ULS
43	2UPN140*/6/	0.57	ULS
44	2UPN140*/6/	0.70	ULS
45	2UPN140*/6/	0.70	ULS
46	2UPN140*/6/	0.84	ULS
47	2UPN140*/6/	0.76	ULS
2	2UPN140*/6/	0.39	ULS
5	2UPN140*/6/	0.39	ULS
6	2UPN140*/6/	0.49	ULS
48	2UPN140*/6/	0.49	ULS
49	2UPN140*/6/	0.59	ULS
50	2UPN140*/6/	0.59	ULS
51	2UPN140*/6/	0.65	ULS
52	2UPN140*/6/	0.57	ULS
53	2UPN140*/6/	0.57	ULS
54	2UPN140*/6/	0.70	ULS
55	2UPN140*/6/	0.70	ULS
56	2UPN140*/6/	0.84	ULS
57	2UPN140*/6/	0.76	ULS

DESIGN OF CONNECTIONS

The connections of the truss are bolted connections. The verification of the connections is carried out in this section where all the relevant checks are as recommended by Eurocode 3 [11] are carried out. Checks such as shear, bearing, net section (global and local) failure and block tearing are considered in this case. The checks are focused only on one type of truss.

The number of bolts required to be connected is initially fixed by fixing the bolt grade and diameter of the bolt and by fixing the minimum and maximum spacing as suggested by the Eurocode-3 [12]. The Nomenclature of the connections in case 4 truss is summarized in Figure-9. Every connection might not require design but definitely will require fabrication drawings. Generally shear, tension and bearings checks are made for each joint.

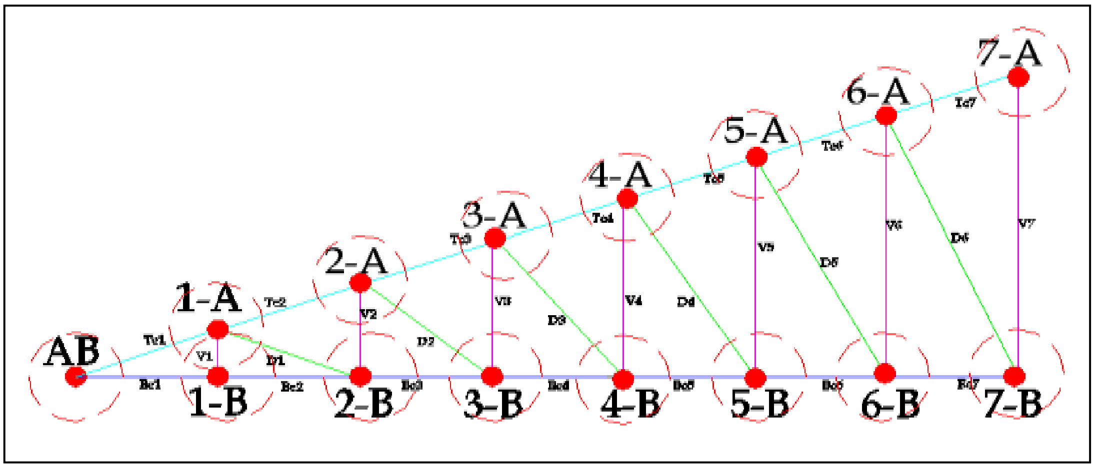
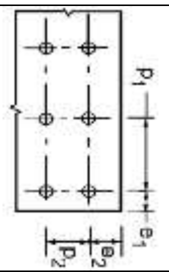


Fig-9: Nomenclature of the connections.

Table-5 shows minimum and maximum spacing of bolts. The minimum spacing is limited by the hole diameter and thus on the corresponding bolt diameter, whereas the maximum spacing is limited by the thickness of the smaller plate or the thickness of connected part of the section whichever is the smaller.

Table-5: Minimum and maximum spacing of bolts

Distance	Formulae min	Formulae max	M16 min	M20 min	Max for thickness (mm) 7.5	7	6	5.5	Symbols for spacing of Fasteners
e_1	1,2*d*0	4*t*+ 40	21.6	26.4	70	68	64	62	
e_2	1,2*d*0	4*t*+ 40	21.6	26.4	70	68	64	62	
p_1	2,2*d*0	Min(14*t*or 200)	39.6	48.4	105	98	84	77	
p_2	2,4*d*0	Min(14*t*or 200)	43.2	52.8	105	98	84	77	

For the shear and bearing of the bolts, Table-6 shows data required such as bolt diameter, grade, hole diameter, thickness of the plate, edge distances and pitches in both directions.

Table-6: Data required for the shear and bearing of bolts

Bolt Dia	Bolt Grade	Hole diameter	Plate Thickness	e_1	e_2	p_1	p_2
16	8.8	18	18	30	30	45	50
20	8.8	22	18	30	30	50	60

All values are in mm.

Table-7 shows bearing strength for the adopted profiles to be used in the design of connections.

Table-7: Bearing strength for the design of connections

Section	Member	t_{min}	$F_{b,Rd}$, KN M16	M20
UPN-140	Top Chord	15	99.90	102.8
UPN-140	Bottom chord	14	93.23	95.98
U 60 x 30	For D6 and V7	12	79.91	82.30
U 65 x 42	Other remaining members	11	73.25	75.40

Finally in Table-8 the required number of bolts under the applied loading when using M16 and M20 grade 8.8 bolts are shown. n_b denotes the number of bolts required.

Table-8: Number of bolts for M16 and M20 grade 8.8 bolts

Member		Axial Force (KN)	M16 Grade 8.8 Bolts					M20 Grade 8.8 Bolts				
			$F_{b,rd}$	n_b	$F_{v,rd}$	n_b	Max n_b	$F_{b,rd}$	n_b	$F_{v,rd}$	n_b	Max n_b
Lower Chord UPN-140	Bc1	805.67	93.23	9	60.3	13	13	95.98	8	94.08	9	9
	Bc2	805.67	93.23	9	60.3	13	13	95.98	8	94.08	9	9
	Bc3	742.82	93.23	8	60.3	12	12	95.98	8	94.08	8	8
	Bc4	678.55	93.23	7	60.3	11	11	95.98	7	94.08	7	7
	Bc5	620.47	93.23	7	60.3	10	10	95.98	6	94.08	7	7
	Bc6	557.77	93.23	6	60.3	9	9	95.98	6	94.08	6	6
	Bc7	495.41	93.23	5	60.3	8	8	95.98	5	94.08	5	5
Upper Chord UPN-140	Tc1	817.44	99.9	8	60.3	14	14	102.8	8	94.08	9	9
	Tc2	753.68	99.9	8	60.3	12	12	102.8	7	94.08	8	8
	Tc3	688.46	99.9	7	60.3	11	11	102.8	7	94.08	7	7
	Tc4	629.53	99.9	6	60.3	10	10	102.8	6	94.08	7	7
	Tc5	565.92	99.9	6	60.3	9	9	102.8	6	94.08	6	6
	Tc6	502.65	99.9	5	60.3	8	8	102.8	5	94.08	5	5
	Tc7	439.57	99.9	4	60.3	7	7	102.8	4	94.08	5	5

Verticals V7- U-65x42 Others U-60x30	V1	0	79.91	0	60.3	0	0	--	--	--	--	--
	V2	10.61	79.91	0	60.3	0	0	--	--	--	--	--
	V3	21.22	79.91	0	60.3	0	0	--	--	--	--	--
	V4	31.83	79.91	0	60.3	1	1	--	--	--	--	--
	V5	42.44	79.91	1	60.3	1	1	--	--	--	--	--
	V6	53.05	79.91	1	60.3	1	1	--	--	--	--	--
	V7	127.32	73.25	2	60.3	2	2	--	--	--	--	--
Diagonals D6- U-65x42 Others U-60x30	D1	98.3	79.91	1	60.3	2	2	--	--	--	--	--
	D2	67.88	79.91	1	60.3	1	1	--	--	--	--	--
	D3	65.28	79.91	1	60.3	1	1	--	--	--	--	--
	D4	75.99	79.91	1	60.3	1	1	--	--	--	--	--
	D5	82.07	79.91	1	60.3	1	1	--	--	--	--	--
	D6	89.27	73.25	1	60.3	1	1	--	--	--	--	--

As the members are continuous, it is needed to find the difference of forces between the members that meet at the junction, and in the studied case, the maximum difference in the top chord occurs in member Tc6 and Tc7 is 64 kN. At this junction, it is required to provide bolts in such a way to meet the gusset plate geometry and it should be higher than the number of bolts as required by the net force. As per the designed number of bolts provided, the drawing of connection (see Figure-10) is made in such a way to meet all the requirement of the code and then all the possible checks are carried out.

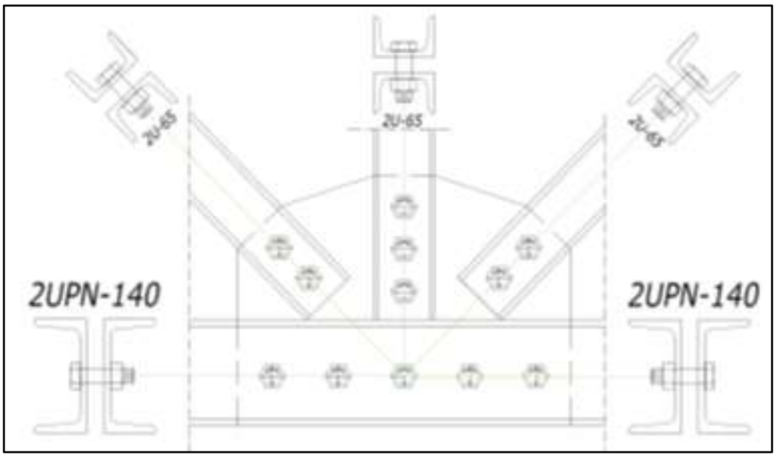


Fig-10: Central connection for type 1 spanning 20m

Results and discussions

In this section results obtained from the analysis and design are mentioned. Figure 11 demonstrates the deflection obtained from the 4 cases under the same load combinations. Case 1 and case 3 span 30m whereas case 2 and case 4 span 20m. Case 1 is deflecting more as compared to case 3 showing that truss adopted in Case 1 is more deformable. Nevertheless, these satisfy the deflection criteria as per the code.

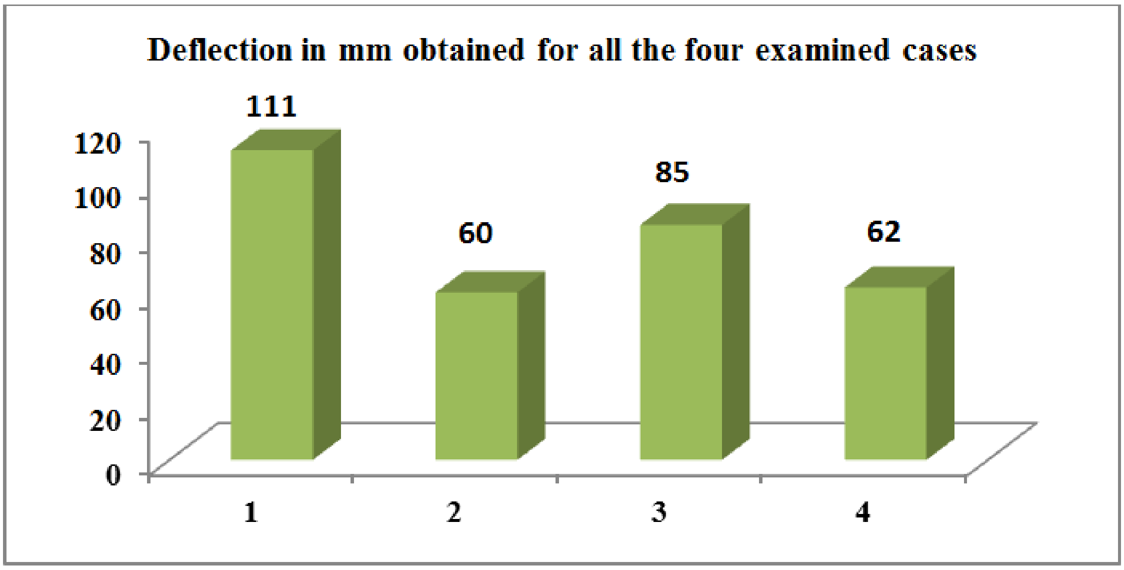


Fig-11: Comparison of Deflections obtained from the analyzed trusses for all the four cases

Figure-12 gives a comparison for the weight of steel to be used in different trusses. Case 3 required more steel compare to case 1 (both trusses are spanning 20m). Therefore case 1 is economical as compared to case 3. Similarly, case 2 is economical than case 4.

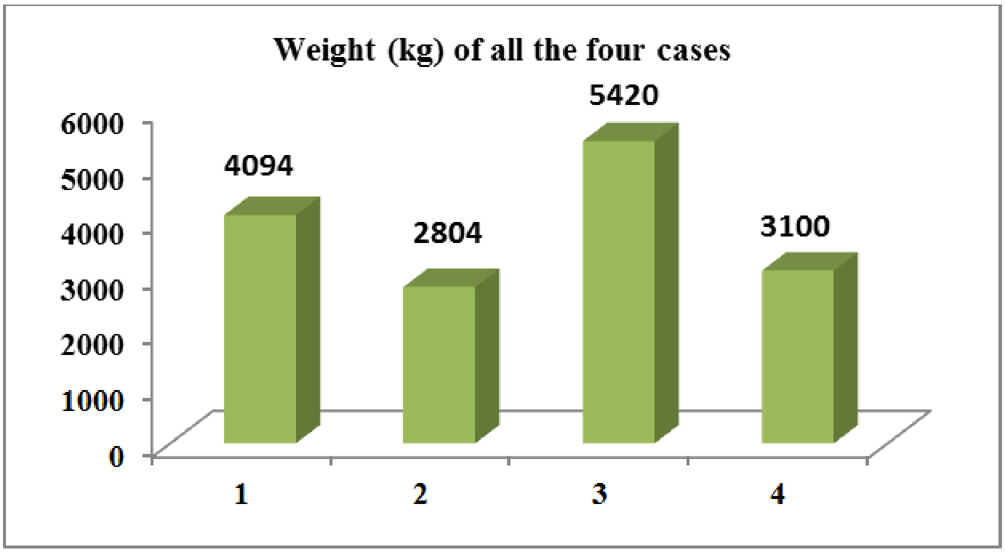


Fig-12: Comparison of weights obtained for all the four examined cases

Figure-13 gives a comparison for the base reaction which can be used to design the footing and bases of the columns for different trusses. Case 3 gives a higher reaction compare to case 1. Therefore it will require bigger size of base plate and footing. Similarly, case 4 will give economical footing compare to case 2. Finally, Table-9 lists the comparisons of deflection, weight and base reactions for all 4 cases.

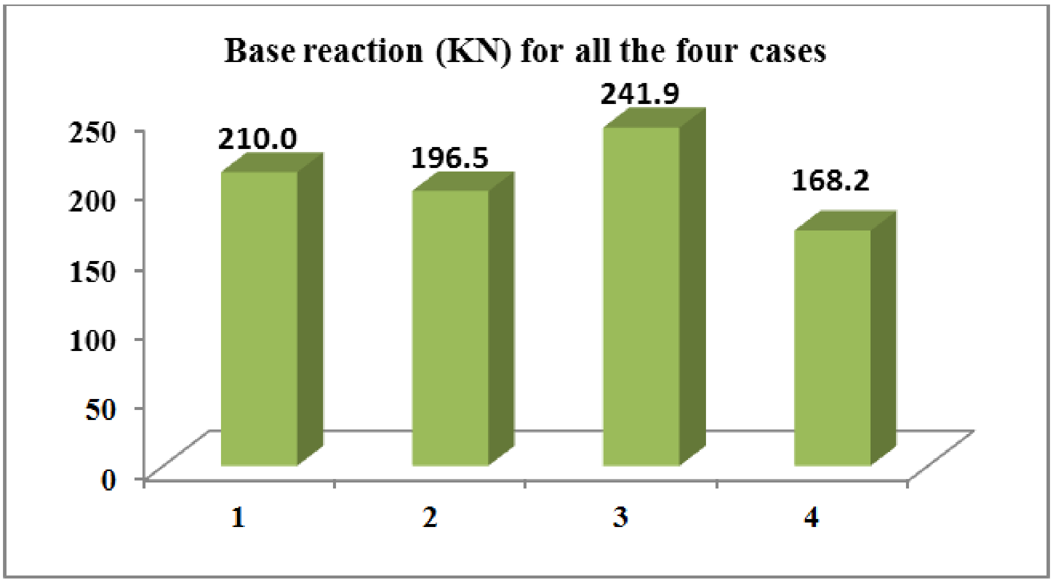


Fig-13: Comparison of Base Reaction obtained for all the four examined cases

Table-9: Comparison of Deflections, Weights and Base Reactions

Case	Span (m)	Deflection (mm)	Weight (kg)	Base Reaction (kN)
1	30	111.3	4093.78	210.00
2	20	60.2	2804.3	196.5
3	30	84.5	5419.55	241.89
4	20	62.0	3099.9	168.2

Table-9 shows comparisons of deflections, weights and base reactions of the analyzed 4 different cases. These values are also reflected in the Figure 11, 12 and 13. In order to give a comparison for all the 4 cases, in this section span is normalized to deflection value whereas weight and reactions are normalized to the span of the trusses. As a result, it is evident that case 3 is much stiffer compares to the other counterparts (See Figure-14).

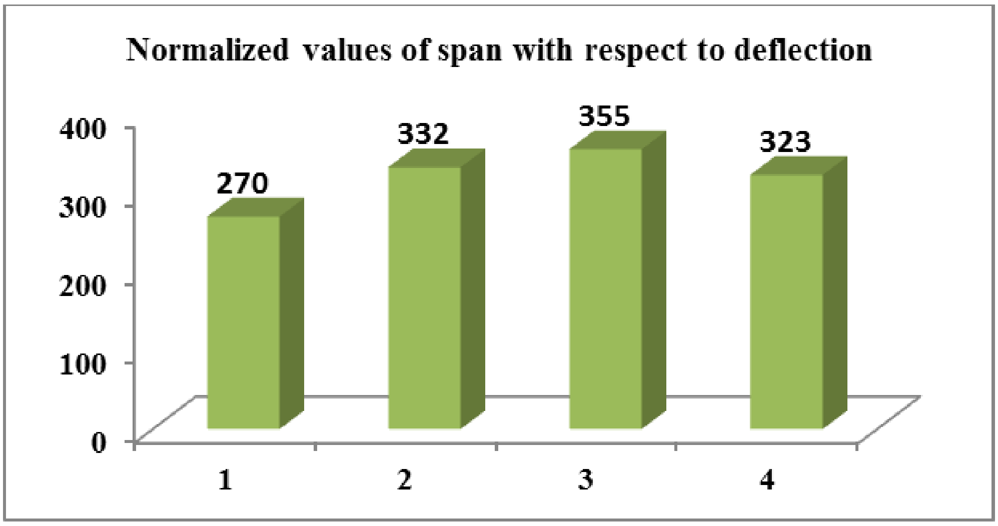


Fig-14: Comparison of deflection normalized to span

Figure-15 shows the results obtained when the weight of the trusses are normalized to its span length. As a result, it is evident that case 1 is optimally designed to compare to the other counterparts.

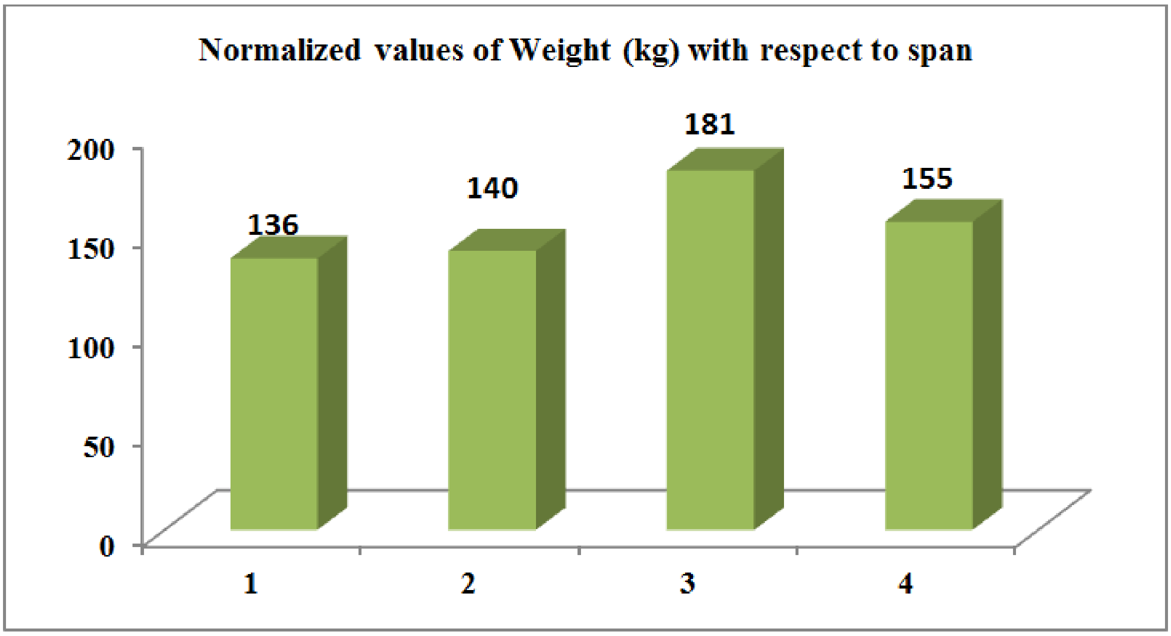


Fig-15: Comparison of weight normalized to span

Finally amongst all the cases, in order to see the consequences of the span on the base reactions of the truss, normalized values are provided here. For example, the base reaction of the truss is normalized to its span length. As a result, it is evident that case 1 is optimally designed to compare to the other counterparts (see Figure-16).

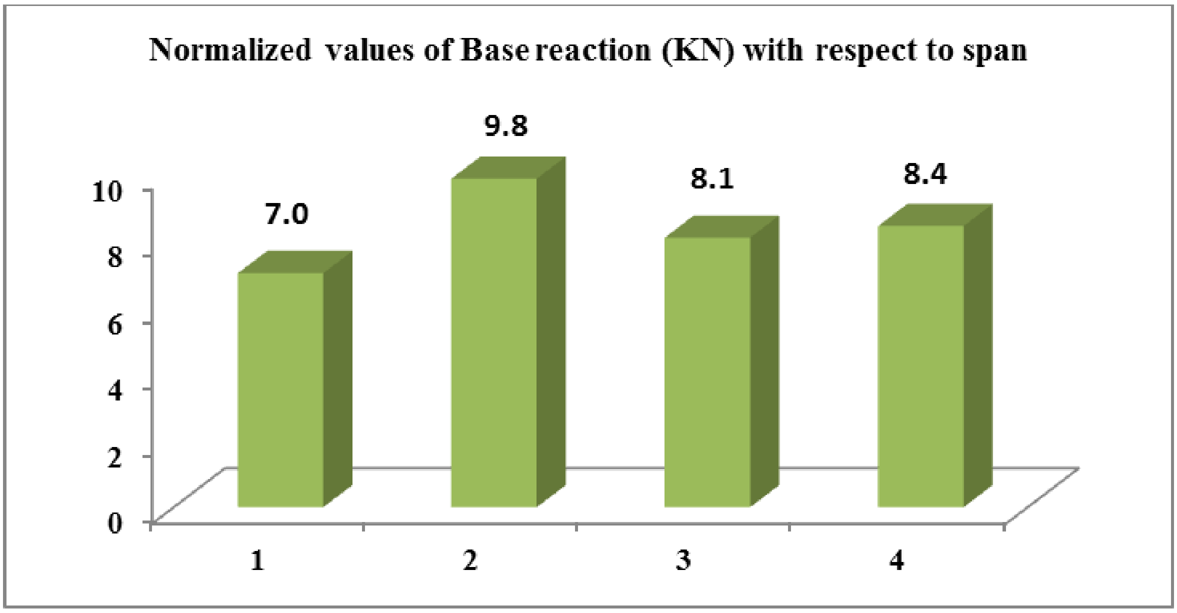


Fig-16: Comparison of base reaction normalized to span

Conclusions

In this study, two different types of trusses having two different spans were analyzed and discussed. The analysis and design were carried out for both manual calculations and SAP 2000 verifications. For the analysis, the method of sections (for truss analysis) was used where the results were same as obtained from SAP 2000 and manual calculations. The loads and load combinations were estimated using SBC 301 whereas for the design AISC/LRFD 99 Code was adopted which is the basis for SBC 301. Also, checks for tension member have been performed using the Code procedure which validated the design rules. From the results and the normalized deflection, weight and reaction values it is evident that case 3 (warren truss) is much stiffer among all the cases. In addition case 2 will require bigger footings and base plate among all the cases and furthermore case 1 is optimally designed to compare to the other counterparts. It is further concluded that simplicity in the design is necessary such as homogonise profiles and few profiles that are easily available. It does not only simplify the fabrication and construction but also reduce the cost of connections as they requies also homoginity. A detailed study is still required to see the consequences of different spans, truss typologies and its fabrications, depth of trusses, connection used, configurations of members and adopted sections etc. This will give a clear guide for the involved engineers and technicians.

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Conflict of Interest

There is no conflict of interest between the authors.

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NOT APPLICABLE

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