



DESIGN OF STEEL TRUSSES TO EUROCODE 3

Omotoriogun Victor Femi

Copyright © Structures centre, 2021. All rights reserved

structurescentre@gmail.com



Summary

This article is a detailed explanation on steel truss design based on the recommendations of BS EN 1993-1-1 and the U.K National Annex.

1.0 INTRODUCTION

Many problems of long spanning in structural engineering can be resolved through the use of steel trusses. These days, long span structures are very popular- from sport arena roofs to auditorium galleries, airport terminals, railway stations etc. Trusses are arguably the most structurally efficient and cost-effective solution available. Compared with the I and H sections as alternatives, they are very light weighted and basically stiffer which means that their deflection is invariably less under any given load arrangement. Also, where a system of concentrated actions is to be sustained over a large span, such as in transfer structures found in multi-storey buildings, they are highly effective.

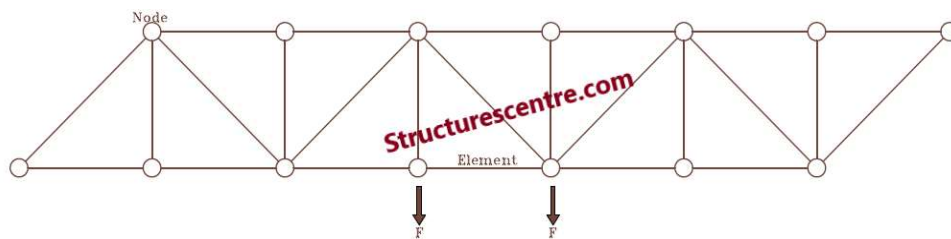


Figure 1: Geometry of a Simple Truss

In a previous article titled- “Structural Aspects of Selecting Plane Steel Trusses” the concept of steel trusses was laid. The article focused on the alternative forms of planar steel trusses available. The article also justified where the alternative forms of steel trusses might be appropriate with emphasis on the design considerations. It is important to state here, that the list of structural forms provided in that article is not exhaustive, there are several other forms available, however, these are only an extension of those listed. In this article, the design of steel trusses will be explained in details

2.0 PRELIMINARY ASPECTS OF DESIGN

The first step in the process of designing a steel truss is to select this structural form. This depends on several factors. For the structural engineer, safety, buildability, economy and structural behaviour under loads are the most important. In-fact structural behaviour

under loads is arguably the most determining factor. Rightly so, safety and economy will depend on the truss behaviour under the system of applied loads. With respect to behaviour, the basic principle in selecting a truss form is to restrict the longer steel elements in tension while the shorter elements in compression. This requires a critical examination of all design load-cases and load combinations. The most onerous load combination should always dictate the structural form to use.

Having selected the truss form which must've being in addition to the structural engineer's considerations selected on the basis of the architectural requirement. The next step would be to immediately consider aspects of serviceability. The overall depth of a truss must be chosen to satisfy the serviceability limit state of deflection. This is easily satisfied by pre-sizing the truss using a span: depth value of between 10-15.

In addition to the above points, a good truss form, for efficiency must allow the following extra consideration.

- The inclination of the diagonal members in the truss relative to the chords should be within the range of 35° - 55° .
- Point loads should as much as possible be applied only at the nodes, except under stringent conditions.
- Joints should be detailed to allow all applied & internal forces to be concentric with the steel connections

3.0 ACTIONS ON STEEL TRUSSES

The actions a steel truss will be required to sustain will depend on the intended function/use of the steel truss. For example, roof trusses in most instances are required to sustain dead, imposed wind and even snow actions whereas a transfer truss may only be required to sustain only dead and imposed actions.

Generally, the loads that can be found on a steel truss are:

- a) **Permanent (Dead) Actions:** The permanent actions include all dead loads applied on the steel truss, including the self-weight of the truss and any additional steel element. In the case of a roof truss, the permanent actions will consist of the load from the roof cladding, self-weight of the steel elements (truss members and purlins) and the applied loads from finishes.
- b) **Variable Actions:** Variable actions include all loads applied on the truss that varies in magnitude with time. This includes, imposed loading, wind actions and snow actions. Procedure for derivation can be found in the N.A of Eurocode 1.

4.0 STEEL SECTIONS FOR TRUSSES

Many steel sections are applicable for use as steel truss member. However, the choice will depend on factors ranging between the magnitude of the internal forces, ease of doing the connections, aesthetics and fabrication constraints.

For smaller spans, Double angle or Tee sections are used for the chords and internal members, with the connections being either bolted or welded. With increase in span, or heavier loads, a channel section might become more appropriate with gusset plates at the nodes to connect the members.

In heavily loaded steel trusses such as those used in steel bridges and those found in buildings where they are used as transfer structure, the magnitude of the internal forces would necessitate rolled sections be used. Typically, UKC & UC sections are used and the nodes are often welded. Any connection necessitated by fabrication is completed with bolted spliced connections.

Where steel trusses are to be exposed in buildings, aesthetics becomes much of a big deal. Hollow steel sections are often chosen for their structural efficiency and aesthetic feel. Similar to UKC & UC sections, their Joints are welded and as part of verifying the member

resistance, the resistance of the connections must be verified in accordance with BS EN 1993-1-8 as this might dominate member selection and final geometry.

5.0 DESIGN OF STEEL TRUSSES

The design of the truss involves analysing the truss under the system of loads using statics—either by using the method of joints or sections to obtain the internal forces in the members and then verifying a trial section for adequacy.

In the analysis and design of steel trusses, although it is assumed that the joints in steel trusses are pinned, this is seldom true. The joint thus have some form of rigidity. This follows that some form of bending moment and shear forces would be induced in the steel members. However, for the purpose of simplicity, it is generally satisfactory and even encouraged by design codes to assume that the joints are pinned and hence only verify these members for axial loads only. The assumption on pinned joints is valid regardless of whether bolted or welded connections is used.

Where loads are applied between nodes, bending moment and shear forces will be induced regardless of the pinned joints. Thus, the truss chords are idealized as a continuous beam member and then analysed for bending moment, shear and axial force while the internal members are analysed for axial forces only. In any case the truss nodes are still assumed fully pinned.

Verification of Members in Tension

For a truss member in axial tension under a uniaxial stress state, Clause 6.2.3(1) of BS EN 1993-1-1 gives an expression that must be satisfied. This is defined as:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1$$

Where: N_{Ed} is the design tensile force. $N_{t,Rd}$ is the design tension resistance, how to determine its value has been defined in a previous article. See “Design of Steel Elements in Tension to Eurocode”.

Verification of Members in Compression

For truss members in compression, evaluating the member resistance is slightly more complicated, the different modes of instability must be taken into consideration. In most truss members in axial compression, only flexural buckling in the plane and out of the plane of the truss would need to be verified. This buckling resistance is obtained by applying a reduction factor to the resistance of the cross-section. This reduction factor is obtained from the slenderness of the member which depends on the elastic critical force. This is defined in Clause 6.3.1.1(1) as:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1$$

Where, $N_{b,Rd}$ is the design buckling resistance of the compression member. The expression for determining its value has been defined in a previous article. See “Design of Steel Elements in Axial Compression to EC3”.

In determining the reduction factor, the buckling length of the member needs to be determined, this depends on whether the buckling is occurring in the plane or outside the plane of the truss.

- For buckling in the plane of the truss, the buckling length is taken equal to 90% of the system length (distance between nodes)
- For buckling outside the plane of the truss, the buckling length is equal to the system length

By inspection the roof truss, is predominantly under the influence of gravitational actions, hence the **Pratt truss** is the most effective solution.

5.1 Actions

a) Permanent Actions

i) Self-weight of roof cladding (0.6mm sheets) = 0.022kN/m^2

ii) Self-weight of purlins & Truss members = 0.4kN/m^2

iii) Services & finishes say = 0.3kN/m^2

Permanent Actions = $g_k = 0.772\text{kN/m}^2$

b) Variable Actions (Imposed)

The roof is inaccessible except during maintenance and repairs, thus from Table 6.9 of BS EN 1991-1-1:2001

i) Imposed load $q_k = 0.75\text{kN/m}^2$

c) Variable Actions (Wind)

Wind action has been determined according to the recommendations of BS EN (1991 (Part 4). For the sake of simplicity, the detailed calculation is not presented here. See: "Derivation of Wind Loading on Buildings"

i) Overall wind loads $w_k = -1.2\text{kN/m}^2$

5.2 Structural Analysis

5.2.1 Permanent Actions

The permanent action is first converted from pressure to uniformly distributed loading on the purlins, the vertical reaction from the purlins is then determined and applied to the nodes of the steel truss. The truss is then analysed for the internal forces using either the method of joints or sections. Only the axial forces are presented here.

It should be noted that at the first and last nodes, the loads are always halved since the tributary load width would be half the value (1.25/2).

The sign convention for the internal forces is positive for axial compressive forces and negative for tensile forces

Load transferred to purlins = $0.772 \times 1.25 = 0.97\text{kN/m}$

Vertical reaction $G_k = 0.97 \times \frac{4.39}{2} = 2.13\text{kN}$

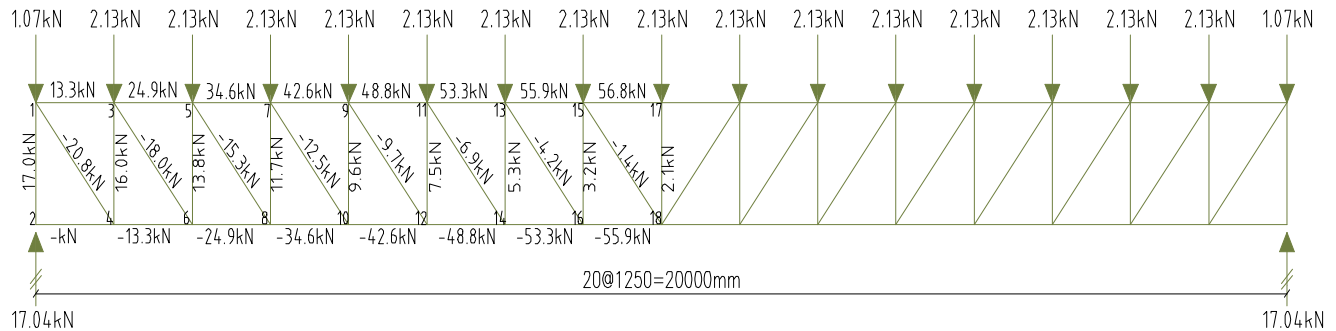


Figure 4: Load case 1 (Permanent Actions)

5.2.2 Variable Actions (Imposed)

Similar procedure as in permanent actions is followed here.

Load transferred to purlins = $0.75 \times 1.25 = 0.94\text{kN/m}$

Vertical reaction $Q_k = 0.94 \times \frac{4.39}{2} = 2.06\text{kN}$

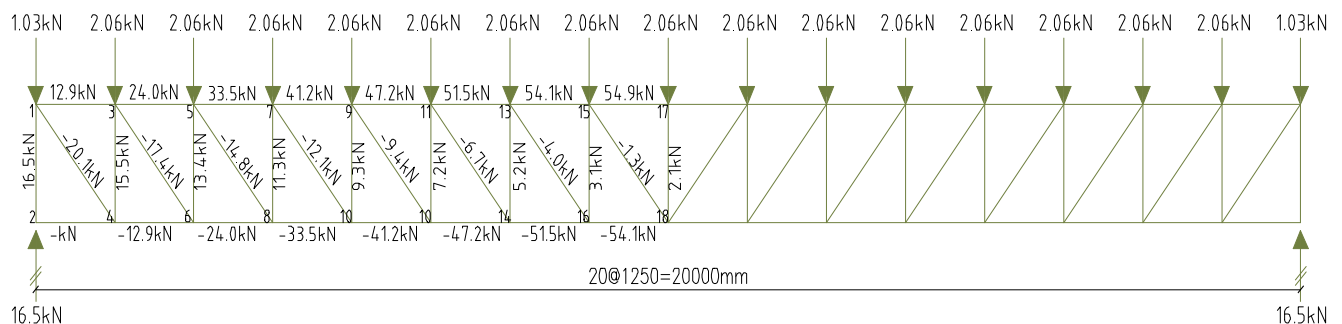


Figure 5: Load case 2 (Variable Imposed Actions)

5.2.3 Variable Actions (Wind)

3-5	24.9kN(c)	24.0kN(c)	38.4kN(t)	69.6kN(c)	24.0kN(t)	1250
5-7	34.6kN(c)	33.5kN(c)	53.5kN(t)	97.0kN(c)	33.6kN(t)	1250
7-9	42.6kN(c)	41.2kN(c)	65.8kN(t)	119.3kN(c)	41.2kN(t)	1250
9-11	48.8kN(c)	47.2kN(c)	75.4kN(t)	136.7kN(c)	47.2kN(t)	1250
11-13	53.3kN(c)	51.5kN(c)	82.3kN(t)	149.3kN(c)	51.5kN(t)	1250
13-15	55.9kN(c)	54.1kN(c)	86.4kN(t)	156.7kN(c)	54.1kN(t)	1250
15-17	56.8kN(c)	54.9kN(c)	87.7kN(t)	159.0kN(c)	54.9kN(t)	1250
2-4	13.3kN(t)	12.9kN(t)	20.6kN(c)	37.3kN(t)	12.9kN(c)	1250
4-8	24.9kN(t)	24.0kN(t)	38.4kN(c)	69.6kN(t)	24.0kN(c)	1250
8-10	34.6kN(t)	33.5kN(t)	53.5kN(c)	97.0kN(t)	33.6kN(c)	1250
10-12	42.6kN(t)	41.2kN(t)	65.8kN(c)	119.3kN(t)	41.2kN(c)	1250
12-14	48.8kN(t)	47.2kN(t)	75.4kN(c)	136.7kN(t)	47.2kN(c)	1250
14-16	53.3kN(t)	51.5kN(t)	82.3kN(c)	149.3kN(t)	51.5kN(c)	1250
16-18	55.9kN(t)	54.1kN(t)	86.4kN(c)	156.7kN(t)	54.1kN(c)	1250
1-2	17.0kN(c)	16.5kN(c)	26.3kN(t)	47.7kN(c)	16.5kN(t)	1500
3-4	16.0kN(c)	15.5kN(c)	24.7kN(t)	44.9kN(c)	15.5kN(t)	1500
5-6	13.8kN(c)	13.4kN(c)	21.1kN(t)	38.7kN(c)	13.1kN(t)	1500
7-8	11.7kN(c)	11.3kN(c)	18.1kN(t)	31.1kN(c)	11.4kN(t)	1500
9-10	9.6kN(c)	9.3kN(c)	14.8kN(t)	26.9kN(c)	9.2kN(t)	1500
11-12	7.5kN(c)	7.2kN(c)	11.5kN(t)	20.9kN(c)	7.2kN(t)	1500
13-14	5.3kN(c)	5.2kN(c)	8.2kN(t)	15.0kN(c)	5.1kN(t)	1500
15-16	3.2kN(c)	3.1kN(c)	4.9kN(t)	9.0kN(c)	3.0kN(t)	1500
17-18	2.1kN(c)	2.1kN(c)	3.3kN(t)	6.0kN(c)	2.1kN(t)	1500
1-4	20.8kN(t)	20.1kN(t)	32.1kN(c)	58.3kN(t)	20.1kN(c)	1953
3-6	18.0kN(t)	17.4kN(t)	27.8kN(c)	50.4kN(t)	17.4kN(c)	1953
5-8	15.3kN(t)	14.8kN(t)	23.6kN(c)	42.9kN(t)	14.7kN(c)	1953
7-10	12.5kN(t)	12.1kN(t)	19.3kN(c)	35.1kN(t)	12.1kN(c)	1953

9-12	9.7kN(t)	9.4kN(t)	15.0kN(c)	27.2kN(t)	9.4kN(c)	1953
11-14	6.9kN(t)	6.7kN(t)	10.7kN(c)	19.4kN(t)	6.8kN(c)	1953
13-16	4.2kN(t)	4.0kN(t)	6.4kN(c)	11.7kN(t)	3.9kN(c)	1953
15-18	1.4kN(t)	1.3kN(t)	2.1kN(c)	3.8kN(t)	1.3kN(c)	1953

5.3 Design of Members

Since the truss would likely be visible in the gallery, from an aesthetic point of view, hollow steel section would be more efficient. Therefore, the trial section would be SHS.

Top Chords

By inspection the critical force occurs on member 15-17. Hence the design values will be used to size the top chords.

Max Tensile Force, $N_{Ed,t} = 54.9\text{kN}$

Max Compressive Force, $N_{Ed,c} = 159.0\text{kN}$

Try 50 x 50 x 5mm SHS – Section

Properties:

Depth, $h=50\text{mm}$; Width, $b=50\text{mm}$; Thickness, $t= 5\text{mm}$; Radius of gyration about major & major axis $i_{yy}=i_z=1.82\text{cm}$; Second moment Area $I=28.9\text{ cm}^4$; Area of section $A = 8.73\text{cm}^2$

Section Classification

$$c = h - 3t = 50 - 3(5) = 35\text{mm}$$

$$\frac{c}{t} = \frac{35}{5} \leq 33\epsilon \text{ (class 1)} = 33 \cdot \sqrt{\frac{235}{275}}$$

7.00 < 30.5 Hence section is class 1

Tensile resistance

$$N_{t,Rd} = N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{873 \times 275}{1.0} \times 10^{-3} = \mathbf{240.1kN}$$

$$\frac{N_{Ed,t}}{N_{t,Rd}} = \frac{54.9}{240.1} = 0.23 \leq 1 \quad \mathbf{O.K}$$

Compression resistance

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{873 \times 275}{1.0} \times 10^{-3} = \mathbf{240.1kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{159.0}{240.1} = 0.66 \leq 1 \quad \mathbf{O.K}$$

Flexural buckling resistance

Since the truss is pin-jointed, buckling of the top-chord is critical outside the plane of the truss, for which the buckling length is equal to the system length. Hence $L_{cr} = 1250\text{mm}$

$$\lambda = \frac{L_{cr}}{i} \cdot \frac{1}{93.9\epsilon} = \frac{1250}{18.2} \cdot \frac{1}{93.9 \sqrt{\frac{235}{275}}} = 0.79$$

For hot finished SHS section, buckling curve a applies, hence $\alpha = 0.21$

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.21(0.79 - 0.2) + 0.79^2] = 0.87$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{0.87 + \sqrt{0.87^2 - 0.79^2}} = 0.81 \leq 1.0$$

$$N_{b,Rd} = \chi \frac{Af_y}{\gamma_{M1}} = 0.81 \times \frac{873 \times 275}{1.0} = \mathbf{194.5kN}$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{159.0}{194.5} = 0.82 \leq 1 \quad \mathbf{O.k}$$

Adopt a 50 x 50 x 5 Square Hollow Section (SHS for Top Chords)

Bottom Chords

By inspection the critical force occurs on member 16-17. The bottom chords are less critical compared to the top chords. However, for simplicity and to ease fabrication, same section as the top chord will be utilized.

Max Tensile Force, $N_{Ed,t} = 156.7 \text{ kN}$

Max Compressive Force, $N_{Ed,c} = 54.1 \text{ kN}$

Try 50 x 50 x 5mm SHS – Section

Properties:

Depth, $h=50\text{mm}$; Width, $b=50\text{mm}$; Thickness, $t=5\text{mm}$; Radius of gyration about major & major axis $i_{yy}=i_z=1.82\text{cm}$; Second moment Area $I=28.9 \text{ cm}^4$; Area of section $A = 8.73\text{cm}^2$

Section Classification

$$c = h - 3t = 50 - 3(5) = 35\text{mm}$$

$$\frac{c}{t} = \frac{35}{5} \leq 33\varepsilon \text{ (class 1)} = 33 \cdot \sqrt{\frac{235}{275}}$$

7.00 < 30.5 Hence section is class 1

Tensile resistance

$$N_{t,Rd} = N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{873 \times 275}{1.0} \times 10^{-3} = 240.1 \text{ kN}$$

$$\frac{N_{Ed,t}}{N_{t,Rd}} = \frac{156.7}{240.1} = 0.65 \leq 1 \quad \mathbf{O.K}$$

Compression resistance

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{873 \times 275}{1.0} \times 10^{-3} = 240.1 \text{ kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{54.1}{240.1} = 0.23 \leq 1 \quad \mathbf{O.K}$$

Flexural buckling resistance

The buckling length of the truss is equal to the system length. Hence $L_{cr} = 1250\text{mm}$

$$\lambda = \frac{L_{cr}}{i} \cdot \frac{1}{93.9\varepsilon} = \frac{1250}{18.2} \cdot \frac{1}{93.9 \sqrt{\frac{235}{275}}} = 0.79$$

For hot finished SHS section, buckling curve a applies, hence $\alpha = 0.21$

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.21(0.79 - 0.2) + 0.79^2] = 0.87$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{0.87 + \sqrt{0.87^2 - 0.79^2}} = 0.81 \leq 1.0$$

$$N_{b,Rd} = \chi \frac{Af_y}{\gamma_{M1}} = 0.81 \times \frac{873 \times 275}{1.0} = \mathbf{194.5\text{kN}}$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{54.1}{194.5} = 0.28 \leq 1 \quad \mathbf{O.k}$$

Adopt a 50 x 50 x 5 Square Hollow Section (SHS for Bottom Chords)

Verticals

By inspection the critical force occurs on member 1-2.

Max Tensile Force, $N_{Ed,t} = \mathbf{16.5\text{kN}}$

Max Compressive Force, $N_{Ed,c} = \mathbf{47.7\text{kN}}$

Try 40 x 40 x 2.5mm SHS - Section

Properties:

Depth, $h=40\text{mm}$; Width, $b=40\text{mm}$; Thickness, $t=2.5\text{mm}$; Radius of gyration about major & major axis $i_{yy}=i_z=1.52\text{cm}$; Second moment Area $I=8.54\text{ cm}^4$; Area of section $A = 3.68\text{cm}^2$

Section Classification

$$c = h - 3t = 40 - 3(2.5) = 32.5\text{mm}$$

$$\frac{c}{t} = \frac{32.5}{2.5} \leq 33\varepsilon \text{ (class 1)} = 33 \cdot \sqrt{\frac{235}{275}}$$

13.0 < 30.5 Hence section is class 1

Tensile resistance

$$N_{t,Rd} = N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{368 \times 275}{1.0} \times 10^{-3} = \mathbf{101.2\text{kN}}$$

$$\frac{N_{Ed,t}}{N_{t,Rd}} = \frac{16.5}{101.2} = 0.16 \leq 1 \quad \mathbf{O.K}$$

Compression resistance

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{368 \times 275}{1.0} \times 10^{-3} = \mathbf{101.2\text{kN}}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{47.7}{101.2} = 0.47 \leq 1 \quad \mathbf{O.K}$$

Flexural buckling resistance

The buckling length of the truss is equal to the system length. Hence $L_{cr} = 1500\text{mm}$

$$\lambda = \frac{L_{cr}}{i} \cdot \frac{1}{93.9\varepsilon} = \frac{1500}{15.2} \cdot \frac{1}{93.9 \sqrt{\frac{235}{275}}} = 1.14$$

For hot finished SHS section, buckling curve a applies, hence $\alpha = 0.21$

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.21(1.14 - 0.2) + 1.14^2] = 1.25$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{1.25 + \sqrt{1.25^2 - 1.14^2}} = 0.57 \leq 1.0$$

$$N_{b,Rd} = \chi \frac{Af_y}{\gamma_{M1}} = 0.57 \times \frac{368 \times 275}{1.0} = 57.7 \text{ kN}$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{47.7}{57.7} = 0.83 \leq 1 \quad \mathbf{O.k}$$

Adopt a 40 x 40 x 2.5 Square Hollow Section (SHS for Verticals)

Diagonals

By inspection the critical force occurs on member 1-4.

Max Tensile Force, $N_{Ed,t} = 58.3 \text{ kN}$

Max Compressive Force, $N_{Ed,c} = 20.1 \text{ kN}$

Try 40 x 40 x 2.5mm SHS - Section

Properties:

Depth, $h=40\text{mm}$; Width, $b=40\text{mm}$; Thickness, $t=2.5\text{mm}$; Radius of gyration about major & major axis $i_{yy}=i_z=1.52\text{cm}$; Second moment Area $I=8.54 \text{ cm}^4$; Area of section $A = 3.68\text{cm}^2$

Section Classification

$$c = h - 3t = 40 - 3(2.5) = 32.5 \text{ mm}$$

$$\frac{c}{t} = \frac{32.5}{2.5} \leq 33\epsilon \text{ (class 1)} = 33 \cdot \sqrt{\frac{235}{275}}$$

13.0 < 30.5 Hence section is class 1

Tensile resistance

$$N_{t,Rd} = N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{368 \times 275}{1.0} \times 10^{-3} = \mathbf{101.2kN}$$

$$\frac{N_{Ed,t}}{N_{t,Rd}} = \frac{58.3}{101.2} = 0.58 \leq 1 \quad \mathbf{O.K}$$

Compression resistance

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{368 \times 275}{1.0} \times 10^{-3} = \mathbf{101.2kN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{20.1}{101.2} = 0.20 \leq 1 \quad \mathbf{O.K}$$

Flexural buckling resistance

The buckling length of the truss is equal to the system length. Hence $L_{cr} = 1953\text{mm}$

$$\lambda = \frac{L_{cr}}{i} \cdot \frac{1}{93.9\varepsilon} = \frac{1953}{15.2} \cdot \frac{1}{93.9 \sqrt{\frac{235}{275}}} = 1.48$$

For hot finished SHS section, buckling curve a applies, hence $\alpha = 0.21$

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.21(1.48 - 0.2) + 1.48^2] = 1.73$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{1.73 + \sqrt{1.73^2 - 1.48^2}} = 0.38 \leq 1.0$$

$$N_{b,Rd} = \chi \frac{Af_y}{\gamma_{M1}} = 0.38 \times \frac{368 \times 275}{1.0} = \mathbf{38.5kN}$$

$$\frac{N_{Ed,c}}{N_{b,Rd}} = \frac{20.1}{38.5} = 0.52 \leq 1 \quad \mathbf{O.k}$$

Adopt a 40 x 40 x 2.5 Square Hollow Section (SHS for Diagonals).

5.4 Serviceability - Check for Deflection

The truss is checked for deflection using the service imposed variable actions. To determine the maximum deflection occurring at midspan, the truss is idealized as a simply supported beam subjected to a concentrated force. The maximum deflection occurring at midspan is then determined using principle of super-position.

$$\Delta_{max} = \sum \frac{Pa(l-x)(2lx-x^2-a^2)}{6EI}$$

Where:

- Δ_{max} = deflection (mm)
- P = Concentrated force at x (N)
- x = point at which deflection is to be found (mm)
- a = point of application of point load from support (mm)
- l = span (mm)
- E = Elastic modulus of members (N/mm²)
- I = Second moment area of the truss (mm⁴)

We must determine the value of I for the entire truss using the parallel axis theorem. If we cut a section through the truss and then using the parallel axis theorem, we have:

$$I = 2I_{chord} + 2A_{chords}y^2$$

$$= 2(28.9 \times 10^8) + 2 \times 873 \left(\frac{1500}{2}\right)^2 = 6.76 \times 10^9 mm^4$$

$$\Delta_{max} = \sum \frac{Pa(l-x)(2lx-x^2-a^2)}{6EI}$$

$$2 \left[\frac{(2.06 \times 10^3) \times 1250(20000 - 10000)[(2 \times 20000 \times 10000) - 10000^2 - 1250^2]}{6 \times 6.76 \times 10^9 \times 200 \times 10^3 \times 20000} \right] +$$

$$2 \left[\frac{(2.06 \times 10^3) \times 2500(20000 - 10000)[(2 \times 20000 \times 10000) - 10000^2 - 2500^2]}{6 \times 6.76 \times 10^9 \times 200 \times 10^3 \times 20000} \right] +$$

$$2 \left[\frac{(2.06 \times 10^3) \times 3750(20000 - 10000)[(2 \times 20000 \times 10000) - 10000^2 - 3750^2]}{6 \times 6.76 \times 10^9 \times 200 \times 10^3 \times 200000} \right] +$$

$$\begin{aligned}
& + 2 \left[\frac{(2.06 \times 10^3) \times 5000(20000 - 10000)[(2 \times 20000 \times 10000) - 10000^2 - 5000^2]}{6 \times 6.76 \times 10^9 \times 200 \times 10^3 \times 20000} \right] \\
& + 2 \left[\frac{(2.06 \times 10^3) \times 6250(20000 - 10000)[(2 \times 20000 \times 10000) - 10000^2 - 6250^2]}{6 \times 6.76 \times 10^9 \times 200 \times 10^3 \times 20000} \right] \\
& + 2 \left[\frac{(2.06 \times 10^3) \times 7500(20000 - 10000)[(2 \times 20000 \times 10000) - 10000^2 - 7500^2]}{6 \times 6.76 \times 10^9 \times 200 \times 10^3 \times 20000} \right] \\
& + 2 \left[\frac{(2.06 \times 10^3) \times 8750(20000 - 10000)[(2 \times 20000 \times 10000) - 10000^2 - 8750^2]}{6 \times 6.76 \times 10^9 \times 200 \times 10^3 \times 20000} \right] \\
& + \left[\frac{(2.06 \times 10^3) \times 100000(20000 - 10000)[(2 \times 20000 \times 10000) - 10000^2 - 10000^2]}{6 \times 6.76 \times 10^9 \times 200 \times 10^3 \times 20000} \right] = \\
& = 0.095 + 0.187 + 0.273 + 0.35 + 0.41 + 0.46 + 0.5 + 0.25 = \mathbf{2.53mm}
\end{aligned}$$

Permissible deflection for roofs (BS EN 1993-1-1) = $\frac{L}{250} = \frac{20000}{250} = \mathbf{80mm}$

6.0 Citations & Further Reading

Steel Construction Institute – Single storey buildings Part 5: Detailed Design of Trusses.

BS EN 1993: Design of steel structures - Part 1-1: General rules and rules for buildings

U.K National Annex to BS EN 1993: Design of steel structures - Part 1-1: General rules and rules for buildings.