IMPLICATIONS OF EUROCODE FOR STEEL PORTAL FRAMES IN SRI LANKA

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Thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Structural Engineering Design

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Abstract

Portal frame structures are widely used all over the world and in Sri Lanka for warehouses and factory buildings as they allow a large column free area with a maximum open space. They are basically made out of steel. Speedy construction, flexibility in use and easy maintenance are the main advantages in steel portal frames. Up until now in Sri Lanka, steel portal frames were designed mainly according to the British standards. But Eurocode is a more updated set of guidelines formed through research and experience.

This paper investigates the implications of Eurocode for steel portal frames in Sri Lanka. A field survey was carried out via questionnaires and responses in interviews to get a firsthand understanding of portal frame structures prevalent in Sri Lanka. With this experience, 48 different portal frames were selected for the parametric study to suit the Sri Lankan conditions varying the span range from 20m to 50m, eaves height from 4.5m to 6.0m and frame spacing from 4.5m to 9.0m. They were analysed to find the implications of Eurocode based on the methods proposed by the Steel Construction Institute. Results of parametric study were compared with each other and with available literature and publications.

Identified implications are discussed in this paper concerning forces, moments and weight variations. A table was developed to obtain optimum column and rafter sections for selected ranges of parameters. No significant advantages were found in designing portal frames to elastic theory based on Eurocode compared to British standards in terms of weight. Main frame weight as a percentage of ULS axial force of a column (excluding the self weight of frame) was found to be in the range of 10% to 45% for 4.5m eaves height frames and 18% to 45% for 6.0m eaves height frames.

Specially dedicated to my beloved family and friends…

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LIST OF ABBREVIATIONS

Abbreviation	Descrption
A	cross sectional area of the member
A_v	shear area
E	modulus of elasticity
f_y	yield strength
f_u	ultimate strength
G	shear modulus
G_k	nominal value of the permanent actions
Q_k	nominal value of the imposed actions
h	column height
H_{Ed}	design value of horizontal reaction at the bottom of the column due to the horizontal loads and the equivalent horizontal force
I	second moment of area of rafter
I_T	torsional constant of the member
i	radius of gyration about the relevant axis
L_{cr}	developed length of the rafter pair between columns
M_{cr}	elastic critical moment for lateral torsional buckling
$M_{y,Ed}$	design bending moment, y-y axis
$M_{Z,Ed}$	design bending moment, z-z axis
$M_{y,Rd}$	design values of the resistance of bending moment, y-y axis
$M_{z,Rd}$	design values of the resistance of bending moment, z-z axis
$M_{b, Rd}$	lateral torsional buckling resistance
N_{Ed}	design compression force in rafter
$N_{c,Rd}$	design resistance to normal forces of the cross section for uniform compression
$N_{b, y, Rd}$	flexural buckling resistance in the major axis
$N_{b, z, Rd}$	flexural buckling resistance in the minor axis

Abbreviation	Descrption
N_{cr}	elastic critical buckling load for the complete span of the rafter
V_{Ed}	design shear force
$V_{c,Rd}$	design shear resistance
$V_{pl,Rd}$	Plastic design shear resistance
$W_{pl, y}$	plastic section modulus of the member
$W_{el,min}$	minimum elastic section modulus
$W_{eff,min}$	minimum effective section modulus
x-x	axis along a member
y-y	axis of a cross section
z-z	axis of a cross section
α_{cr}	factor to increase the design load to cause elastic instability in a global mode
$\alpha_{cr,s,est}$	estimate of α_{cr} for the sway buckling mode
$\alpha_{cr,r,est}$	estimate of $\alpha_{cr}$for the rafter snap- through buckling mode
α_{LT}	imperfection factor
α_m	reduction factor for the number of columns in a row
χ	reduction factor for the relevant buckling curve
χ_{LT}	reduction factor for lateral torsional buckling
ε	strain
$\delta_{H,Ed}$	maximum horizontal deflection at the top of either column, relative to the base, when the frame is loaded with horizontal loads
δ_{NHF}	lateral deflection at the top of the column due to the NHF
φ	global initial sway imperfection
φ_0	basic values for global initial sway imperfection
φ_{LT}	values to determine the reduction factor χ_{LT}
ψ	ratio of moments of a segment
γ_m	partial factor

Abbreviation	Descrption
γ_{m1}	partial factor for resistance of members to instability(member checks)
γ_{m2}	partial factor for resistance of cross sections in tension to fracture
ν	Poisson’s ratio
λ_1	slenderness value to determine the relative slenderness
$\bar{\lambda}$	non dimensional slenderness
$\bar{\lambda}_{LT}$	non dimensional slenderness for lateral torsional buckling

LIST OF APPENDICES

1. INTRODUCTION

1.1 Background

Portal frame structures are usually low rise wide span buildings. These structures are ideal for warehouses and factory buildings as they allow a large column free area with a maximum open space. They are basically made out of steel. Longer spans can be achieved with comparatively thinner sections in steel portal frames. Two or three spans of portal frames can be used according to the requirement to accommodate larger spans economically. Speedy construction, flexibility in use and easy maintenance are the main advantages in steel portal frames.

Portal frame structures are widely used in Sri Lanka in the industrial zone for factory buildings, ware houses and commercial buildings like vehicle showrooms and food cities. British standards were used for the design, fabrication and erection of most of the steel structures in Sri Lanka.

The Eurocodes are a complete set of up-to-date design standards that include the main construction materials, fields of structural engineering and a wide range of structures while retaining the flexibility of meeting local needs and best practice with regard to safety level, loading and durability. The flexibility of meeting local need is addressed by the use of National Annexes which contain Nationally Determined Parameters (NDPs) to be adopted in the relevant country. Eurocodes could soon be adopted in Sri Lanka.

Guidelines are available to design portal frames to Sri Lankan conditions based on British standards, but not for Eurocode. It is a question whether similar sections proposed for the British standards satisfy the Eurocode designs.

Typical structural elements of a portal frame are shown in the figure 1.1.

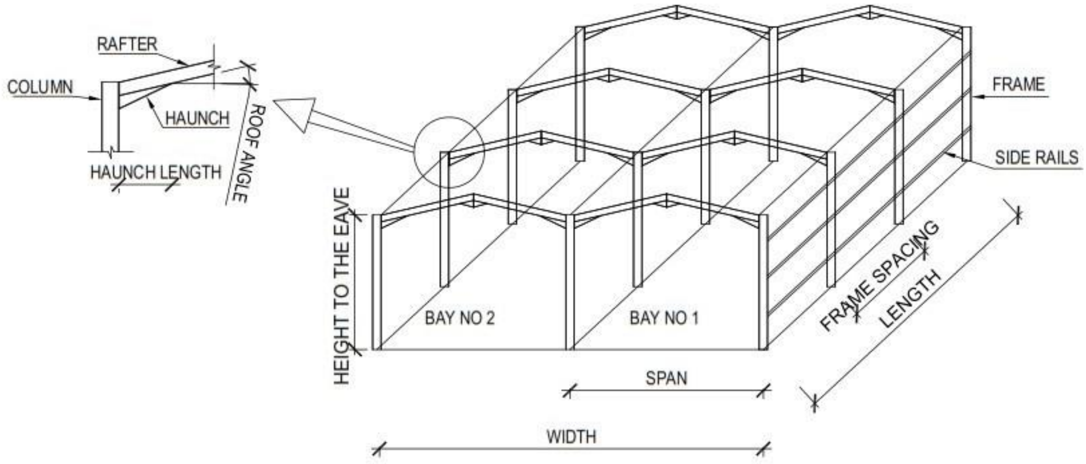


Figure 1.1 Structural elements of a steel portal frame

Portal frame structures comprise several frames braced longitudinally as shown in figure 1.1. Primary steel work consists of columns and rafters connected by moment resisting joints. Rigidity of the connections and the bending stiffness of the members resist the vertical and lateral loads applied on the structure controlling the deflection. Secondary steel works consist of light gauge purlins and side rails.

1.2 Research Objective

The objective is to identify the implications of Eurocode for steel portal frame structures in Sri Lanka focusing on actions, analysis, design, weight etc.

1.3 Scope of the work

Parametric study was carried out to design the portal frame structures based on the Eurocode for Sri Lankan Conditions. Parameters were limited to hot rolled steel sections and commonly used spans, eaves heights, frame spacings, roof angles, base conditions and wind zones in Sri Lanka.

1.4 Methodology

Literature review was carried out parallel to a field study. Portal frame structures were designed and analysed to Eurocode using selected parameters based on the field survey. Results were compared with each other, field survey data, available publications and literature.

Literature review was mainly focused on differences between the British standards and Eurocode in relation to the steel portal structures, design and analysis methods and their effects (second order effects, plastic and elastic analysis), deflection limits and portal frame structures in Sri Lanka.

Field study on the existing steel portal frame structures in Sri Lanka was carried out through a survey using a questionnaire and responses in interviews. Parameters including height, width, roof angle, haunch length, number of bays, member types, wind zone, loading and design codes were mainly considered in the data collection.

Details pertaining to about 100 existing portal frames were collected and analyzed through the field survey.

Data collected via field survey were categorised and analysed to identify the design data and commonly used dimensions of the steel portal frame structures. Parameter ranges most commonly used in Sri Lanka were selected for the parametric study.

Actions and combinations of actions confirmed to Eurocode 1 were applied on steel portal frames and analysed using a commercially available computer software package. Preliminary sizing was done based on the guidelines issued by the Steel Construction Institute. Eaves haunches were modeled in the frame analysis, but not the apex haunches as they are generally used to facilitate a bolt connection. Designs were carried out based on the Eurocode 3.

Results obtained from the analysis and design of portal frames were assessed and compared with each other, data obtained from the field survey, available publications and the British standard related literature done for Sri Lanka.

The assessed data were used to identify the effects of using the Eurocode for the design of steel portal frames.

Literature review and its findings are present in Chapter 2. Field survey and its results are analysed and discussed in Chapter 3 and the questionnaire used for the survey is attached in Appendix A. Chapter 4 briefly discusses about the design of portal frames and a detailed description is attached in Appendix B.

Analysis, results and discussion of the parametric study are presented in Chapter 5. Specimen design calculation of a portal frame used to carry out the parametric study is given in Appendix C.

Conclusion and recommendation of the research study is discussed in Chapter 6.

Property table for steel universal beams and initial design tables used are given in Appendix D and Appendix E respectively.

2. LITERATURE REVIEW

Literature review was carried out pertaining to portal frame structures in Sri Lanka, differences between the British Standards and Eurocode in relation to the steel structures, design and analysis methods and their effects (second order effects, elastic critical buckling factor, etc.), deflection limits and optimisation of structures.

2.1 Portal frame structures in Sri Lanka

Perera, et al.[1] have carried out a parametric analysis on optimum frame spacing for steel portal frames used in Sri Lanka. They have followed plastic analysis method to BS5950:1 (1990) for the study. Perera et al. [1] have developed a table to obtain optimum column and rafter section sizes for a range of pitched roof single bay portal frame structures with eaves heights ranging from 4.5m to 6m, spans from 20m to 50m and frame spacing from 4.5m to 9m.

Cost of portal frame structures comprised two components, cost of material and cost of labour. They have found that the frame spacing of 9m provides least overall weight of steel frames per unit area for all the cases in the parametric study. Cost of purlins and accessories per unit area decreased with increase of span. Labour cost is inversely proportional to frame spacing as larger frame spacing requires fewer frames. Generally overall cost per unit area is increased with the increase of the span and the eaves height. An optimum frame spacing of 7.5m was recommended for all the spans. [1]

2.2 Eurocode vs. BS 5950

Lim, et al,[2] have compared the BS5950 and Eurocode 3 with regard to in-plane stability of portal frames. Early versions of BS5950-1 (specifically BS1985, 1990) provided limits of defined parameters relating to sway stiffness. This permitted the design of portal frames plastically ignoring the second order effects. It was found to be unsafe or over optimistic. In BS5950-1:2000, design rules were revised focusing on the global stability and second order effects which again proved to be uneconomical for certain frames. Eurocode 3 has not provided any simple method of plastic design to avoid the need for second order elastic- plastic analysis. They have instead proposed design rules based on the Merchant-Rankine reduction method to integrate in-plane stability to Eurocode 3 when designing a single story steel building plastically. [2]

Loads and combinations - Eurocode 3 and BS5950

Lim, et al,[2] state that partial load factors and load combinations given in the codes are different to each other. Generally the critical load combination in Eurocode 3 includes a lateral wind load component whereas only vertical load combination is critical for BS 5950-1:2000. Due to this reason, design rules given in BS 5950-1:2000 to integrate global stability and second order effects cannot be directly used for the designs done based on Eurocode. [2]

Load combinations defined for ultimate limit state in the BS5950-1:2000 [3] and BS EN1991-1-1:2002 [6] are shown in table 2.1 and table 2.2.

Table 2.1 Factors for design combinations at ULS for BS5950-1:2000 [3]

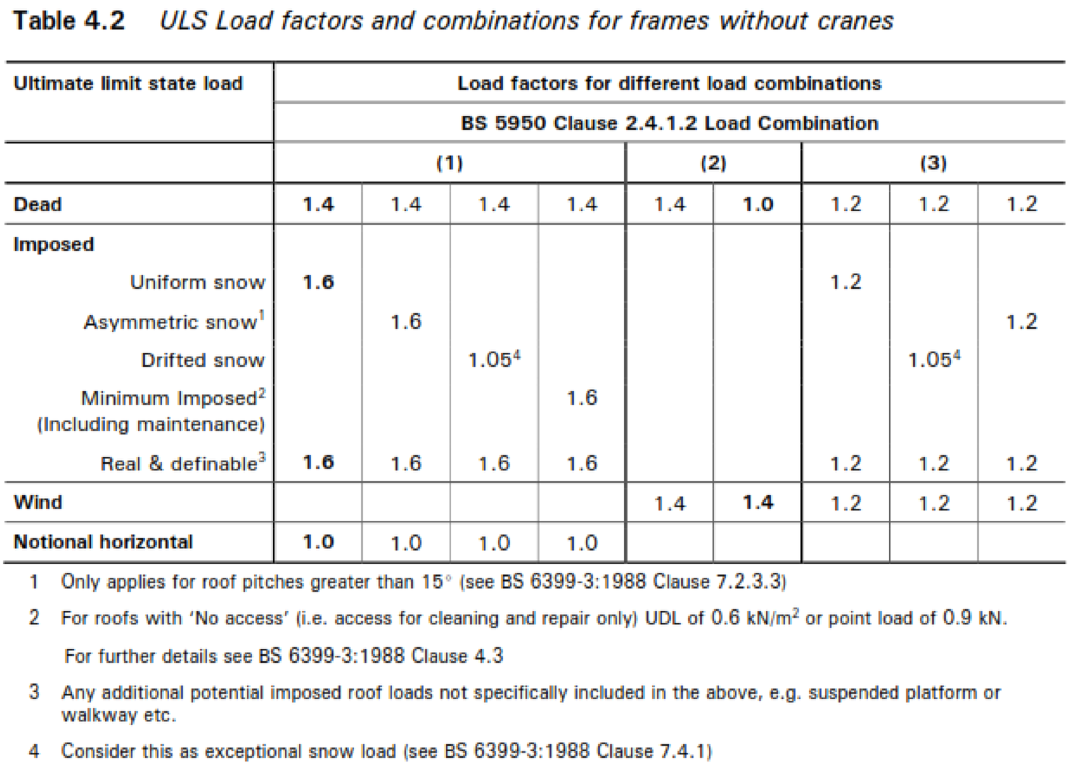


Table 4.2 *ULS Load factors and combinations for frames without cranes*

Ultimate limit state load	**Load factors for different load combinations**								
	BS 5950 Clause 2.4.1.2 Load Combination								
	(1)				**(2)**		**(3)**		
Dead	**1.4**	1.4	1.4	1.4	1.4	**1.0**	1.2	1.2	1.2
Imposed									
Uniform snow	**1.6**						1.2		
Asymmetric snow[1]		1.6							1.2
Drifted snow			1.05[4]					1.05[4]	
Minimum Imposed[2] (Including maintenance)				1.6					
Real & definable[3]	**1.6**	1.6	1.6	1.6			1.2	1.2	1.2
Wind					1.4	**1.4**	1.2	1.2	1.2
Notional horizontal	**1.0**	1.0	1.0	1.0					

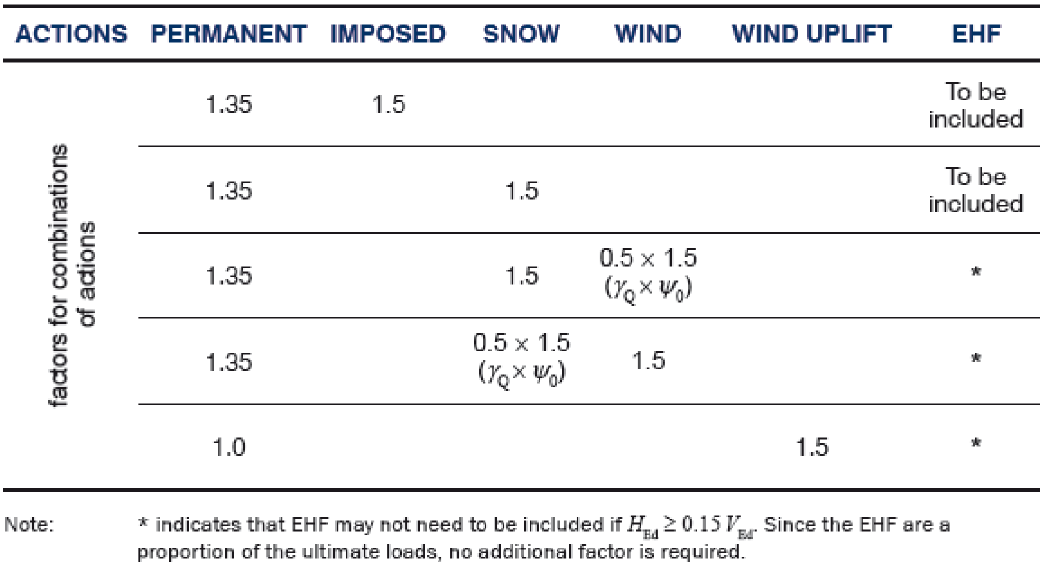
1 Only applies for roof pitches greater than 15° (see BS 6399-3:1988 Clause 7.2.3.3)

2 For roofs with 'No access' (i.e. access for cleaning and repair only) UDL of 0.6 kN/m^2 or point load of 0.9 kN. For further details see BS 6399-3:1988 Clause 4.3

3 Any additional potential imposed roof loads not specifically included in the above, e.g. suspended platform or walkway etc.

4 Consider this as exceptional snow load (see BS 6399-3:1988 Clause 7.4.1)

Table 2.2 Factors for design combinations at ULS for Eurocode [7]



ACTIONS	PERMANENT	IMPOSED	SNOW	WIND	WIND UPLIFT	EHF
factors for combinations of actions	1.35	1.5				To be included
	1.35		1.5			To be included
	1.35		1.5	0.5 × 1.5 $(\gamma_Q \times \psi_0)$		*
	1.35		0.5 × 1.5 $(\gamma_Q \times \psi_0)$	1.5		*
	1.0				1.5	*

Note: * indicates that EHF may not need to be included if $H_{Ed} \geq 0.15\, V_{Ed}$. Since the EHF are a proportion of the ultimate loads, no additional factor is required.

Imposed roof loads are not considered in combination with either wind actions or snow loads in Eurocode 1. [6]

Partial factors

Global partial factors γ_M are defined in Eurocode where the user can change them depending on material properties and other variables. Numerical values for γ_M are recommended in the national annexes of respective countries. National Annexes are developed in Sri Lanka for several Eurocodes and few are under development stage. Table 2.3 gives the partial factors given in different standards.

Table 2.3 Partial factors given in Eurocode and British standards

Design standard	Factor	Value used for calculations
Eurocode 3 [5]	γ_{M0} Resistance of cross section whatever the class	1
	γ_{M1} Resistance of members to instability assessed by member checks	1
	γ_{M2} Resistance of cross section in tension to fracture	1.25
BS 5950-1:2000 [3]	γ_{M} Material factor (cl 2.1.3)	1 for yield strength 1.2 for tensile strength
BS5950-1:1990 [4]	γ_{M} Material factor (cl 2.1)	1

Column and beam design

Yusoff [10] has carried out a research to find the advantages of using Eurocode over BS5950.-1:2000. He has designed and compared multi-storey steel structures to Eurocode 3 and BS5950 and compared the weight of the structures. It was found that structures designed to Eurocode are heavier than that of to BS5950.

Table 2.4 and table 2.5 show the comparison of beam design and column design based on EC3 and BS5950-1:2000.[10]

Under cross sectional classification, “ε” value is reduced in Eurocode 3 compared to BS 5950-1-2000. Limits given for ‘web subject to bending’ are reduced while the ‘flange subject to compression’ limits are extended.

Table 2.4 Criteria to be considered in structural beam design [10]

BS5950-1:2000	Criteria	EC3
Flange subject to compression	**Cross sectional classification**	**Flange subject to compression**
9ε	Class 1 Plastic	10ε
10ε	Class 2 Compact	11ε
15ε	Class 3 Semi-compact	15ε
Web subject to bending (neutral axis at mid depth)		**Web subject to bending (neutral axis at mid depth)**
80ε	Class 1 Plastic	72ε
100ε	Class 2 Compact	83ε
120ε	Class 3 Semi-compact	124ε
$\varepsilon = (275/p_y)^{0.5}$		$\varepsilon = (235/f_y)^{0.5}$
$P_v = 0.6\ p_y\ A_v$ $A_v = Dt$	**Shear capacity**	$V_{pl,Rd}=f_y\ A_v/\sqrt{3}x\ \gamma_{MO}$ $\gamma_{MO} = 1.05$ A_v from section table
	Moment capacity	
	Class 1,2	$M_{c,Rd} = W_{pl}\ f_y/\gamma_{MO}$
	Class 3	$M_{c,Rd} = W_{el}\ f_y/\gamma_{MO}$
	Class 4	$M_{c,Rd} = W_{eff}\ f_y/\gamma_{M1}$
		$\gamma_{MO} = 1.05$, $\gamma_{M1} = 1.05$

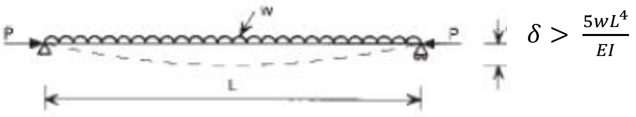
Table 2.5 Criteria to be considered in structural column design [10]

BS5950-1:2000	Criteria	EC3
Flange subject to compression	**Cross sectional classification**	**Flange subject to compression**
9ε	Class 1 Plastic	10ε
10ε	Class 2 Compact	11ε
15ε	Class 3 Semi-compact	15ε
Web(combined axial and bending		Web(combined axial and bending)
$80\varepsilon/ 1+ r_1$	Class 1 Plastic	$396\varepsilon/(13\alpha-1)$
$100\varepsilon/ 1+1.5\ r_1$	Class 2 Compact	$456\varepsilon/(13\alpha-1)$

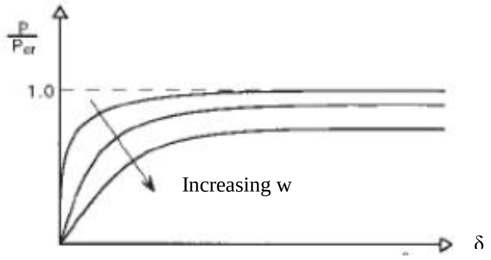
2.3 Second order effects

Lim, et al.[2] discuss the deflection of a simply supported beam to illustrate the second order effects. Deflection of a simply supported uniformly loaded beam is increased when an axial compression is introduced. The first order elastic theory defines that the maximum deflection is “$5wL^4/ 384\ EI$” with the applied uniformly distributed load. The central deflection will be increased exponentially with the increase of the axial load and the failure will occur due to buckling instability. The axial load at the failure is called Euler strut buckling load which is given by, “π^2 EI /L^2 “. Second order analysis is required to predict the deflection accurately when axial load is applied to the same beam. [2]

(a) Simply- supported beam subjected to uniformly distributed load



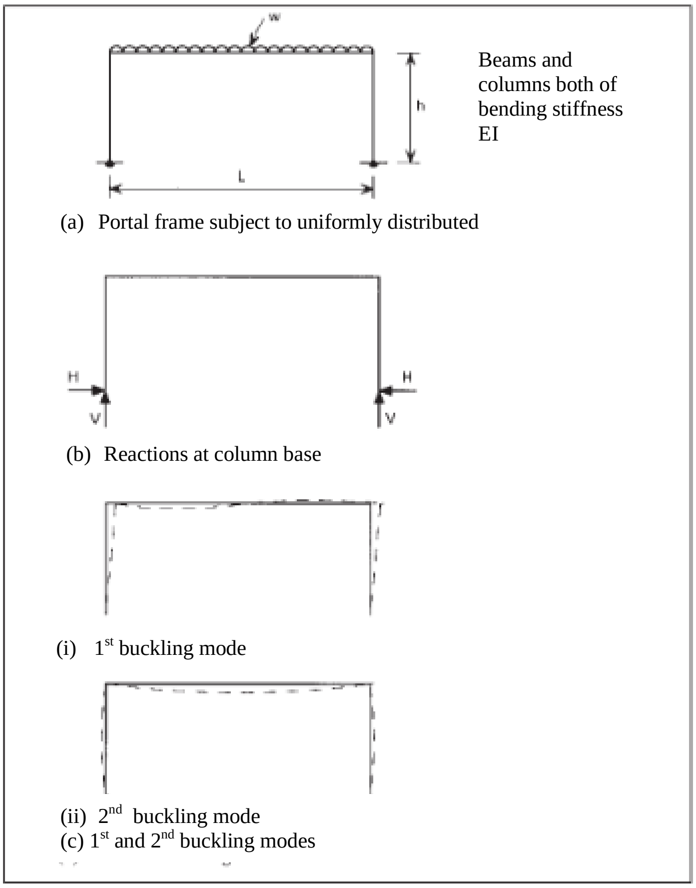
(b) Simply- supported beam subjected to axial load and uniformly distributed load



(c) Variation of axial load with deflection

Figure2.1 Second order effects of axially loaded beams [2]

Portal frames of a structure are subjected to uniformly distributed load and axial load. Horizontal reaction and the axial compression force in the rafter are increased with the increase of ‘L/h” ratio. The axial forces result in in-plane buckling of the portal frames as shown in the figures 2.1 and 2.2. It implies that the portal frames are sensitive to the second order effects. The sensitivity in the elastic range depends on the ratio of the applied load to the load which causes elastic critical buckling of the frame. [2]



Beams and columns both of bending stiffness EI

(a) Portal frame subject to uniformly distributed

(b) Reactions at column base

(i) 1st buckling mode

(ii) 2nd buckling mode

(c) 1st and 2nd buckling modes

Figure 2.2 Second order effects on portal frames [2]

Plastic theory is used to design portal frames as it is more economical. But first order plastic theory does not consider the second order effects. Hence the design tends to overestimate the load which forms plastic hinges in the collapse mechanism. Lim, et al.[2] states that the formal method of integrating the influence of second order effects on plastic collapse of steel frames is by means of a second order analysis which successfully trace the formation of plastic hinges as the load increased. This requires sophisticated software which may not be necessary in many cases.

Elastic critical buckling factor

Lim, et al.[2] proposes simple design rules based on the Merchant-Rankine reduction method which will enable engineers to take in to account in-plane stability when designing single story steel portal frames plastically to Eurocode 3, without the need to resort to second-order elastic-plastic analysis software. The parametric study reveals that the second-order elastic-plastic collapse factor can be predicted using either Merchant-Rankine or a reduced Merchant-Rankine applied to first order plastic analysis for many portal frames. This approach can be used only for single span portal frames.

Elastic critical buckling load factor is required to apply the Merchant-Rankine. They proposed an equation to estimate the elastic critical buckling load factor without carrying out any computer software analysis.

$$\alpha_{cr,s,est} = 0.8\left\{1-\left(\frac{N_{R,ULS}}{N_{R,cr}}\right)max\right\}\alpha_{cr,H}$$

Where
$$N_{cr} = \frac{\pi^2 E I_y}{L_{cr}^2}$$

$$\alpha_{cr,H} = \left(\frac{h}{V_{uls}}\right)\left(\frac{H_{EHF}}{\delta_{EHF}}\right)$$

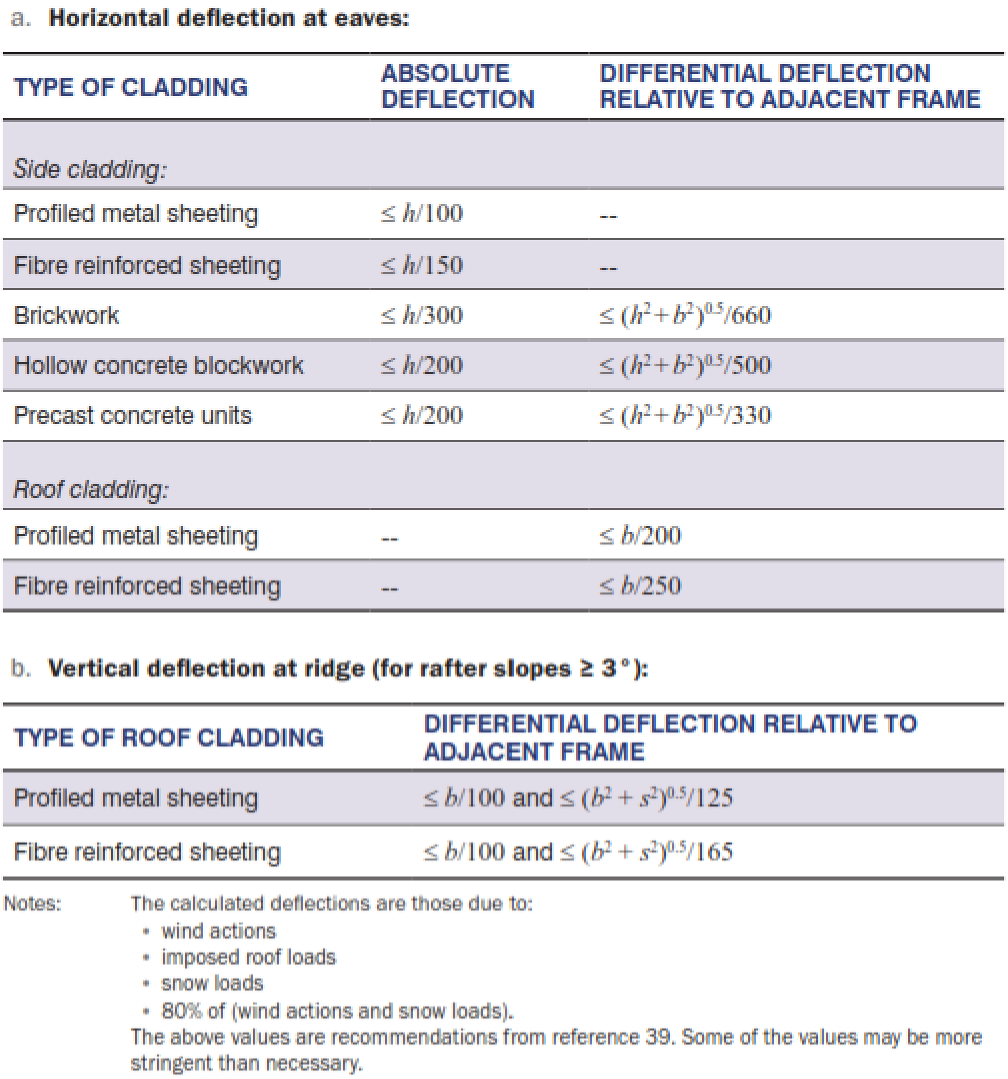
2.4 Optimisation of steel portal frames

Hradi, et al. [8] have studied about the advantages and disadvantages of using more sophisticated methods for portal frames over the commonly used formulas. They have found that the most effective and sustainable method to optimize the structures is advanced 3D modeling. Portal frames made out of slender welded tapered plates save more steel compared to the hot rolled steel sections. Fabrication cost can be achieved by using modern technology. Since the slender frames are prone to lateral instability, special care should be taken in designs.

2.5 Deflection limits

Hradi, et al.[8] have derived a simplified way to check deflection limits based on Eurocodes. Accordingly vertical deflection limit at serviceability is taken as ‘span/200’ at the apex and the horizontal deflection limit at serviceability at the top of the column end is taken as ‘height/100’.

Phan, et al. [9] states that serviceability state deflection limits for portal frames are not specified in the British standards and the judgment of the limit is left on the hand of the design engineer. They state that the Steel Construction Institute has proposed deflection limits intending to avoid problems of tearing in cladding fixing due to differential deflections. It is generally regarded that these limits are too conservative for portal frames with no gantry cranes. The limits proposed by the Steel Construction Institute are shown in the table below.



a. **Horizontal deflection at eaves:**

TYPE OF CLADDING	ABSOLUTE DEFLECTION	DIFFERENTIAL DEFLECTION RELATIVE TO ADJACENT FRAME
Side cladding:		
Profiled metal sheeting	$\leq h/100$	--
Fibre reinforced sheeting	$\leq h/150$	--
Brickwork	$\leq h/300$	$\leq (h^2+b^2)^{0.5}/660$
Hollow concrete blockwork	$\leq h/200$	$\leq (h^2+b^2)^{0.5}/500$
Precast concrete units	$\leq h/200$	$\leq (h^2+b^2)^{0.5}/330$
Roof cladding:		
Profiled metal sheeting	--	$\leq b/200$
Fibre reinforced sheeting	--	$\leq b/250$

b. **Vertical deflection at ridge (for rafter slopes ≥ 3°):**

TYPE OF ROOF CLADDING	DIFFERENTIAL DEFLECTION RELATIVE TO ADJACENT FRAME
Profiled metal sheeting	$\leq b/100$ and $\leq (b^2 + s^2)^{0.5}/125$
Fibre reinforced sheeting	$\leq b/100$ and $\leq (b^2 + s^2)^{0.5}/165$

Notes: The calculated deflections are those due to:
- wind actions
- imposed roof loads
- snow loads
- 80% of (wind actions and snow loads).

The above values are recommendations from reference 39. Some of the values may be more stringent than necessary.

Figure 2.3 Recommended deflection limits for Eurocode [7]

It has been demonstrated that the weight of the portal frames designed considering the deflection limits published by the Steel Construction Institute is two times heavier than in the designs without these limits. [9]

3. FIELD SURVEY

3.1 Questionnaire

A field study on the existing portal frame structures in Sri Lanka was carried out through a survey consisting of a questionnaire and responses in interviews.

Parameters of the portal frame structures including the dimensions, location and wind zone, design standards, analysis methods, materials, foundation details, purposes of the building were mainly considered in the data collection.

Details of 128 existing portal frames were collected and analyzed through the field survey.

Questionnaire used for the field survey is given in the appendix A.

3.2 Analysis and results

Collected data was categorized and analyzed to find the common parameters and other details currently used in Sri Lanka. Based on this, data ranges were selected for the parametric study.

General details of the portal frame structures

Table 3.1 gives summary of results related to general details of the portal frames.

Table 3.1 Summary of general details of the portal frames

Parameter	Types	Number of structures		Percentage
Wind Zone	Zone 1	5	128	4%
	Zone 2	14		11%
	Zone 3	109		85%
Purpose	factory	45	128	35%
	warehouse	47		37%
	other	36		28%

Wind zones

Most of the portal frame structures are located in the Western province which falls under the wind zone 3 of the “Wind loading zones- Sri Lanka” map [15].

Purpose

Steel portal frames were used mainly as factory buildings and warehouses (72%) and remaining are used for other purposes such as vehicle showrooms, indoor stadia, vehicle service centers and supermarkets.

Design standards and analysis details

Table 3.2 shows summary of the field survey results related to design and analysis.

Table 3.2 Summary of design standards and analysis method of portal frames

Parameter	Range	Number of structures		Percentage
Design standard	BS 5950 -1990	92	128	72%
	BS 5950-2000	36		28%
	EC3	0		0%
Analysis method	Elastic	110	128	86%
	Plastic	18		14%

Design Standards

Of the 128 steel portal frames surveyed, none were designed using Eurocode. Majority of the frames (72%) were designed to BS 5950-1:1990 while the rest (28%) were designed to BS 5950-1:2000.

Analysis method

Of the structures surveyed, 110 were designed using elastic theory confirming that elastic theory was more widespread.

Base condition

All of the portal frames were designed assuming pinned bases.

Dimensions

Summary of dimension ranges is shown in the table 3.3. Since all portal frames were single bay structures, span length and width of the building are equal.

Total length of the building lies in the range of 10m to 100m where 62% of the building length ranges from 31m to 50m. 14% of the buildings have a length of above 50m and 7% of the buildings have a length less than 20m.

61% of the structures have a span of 21m -30m representing the most common span range in Sri Lanka. 20% of the structures have a span greater than 31m and 16% of structures have a span range from 11m to 20m.

Common eave height used in Sri Lanka is less than 6m which is seen in 79% of cases and the common roof angle ranges from 6^0 to 10^0.

Table 3.3 Summary of dimensions of portal frames

Parameter	Range of parameter	Number of structures	Percentage
length (m)	<20	9	7%
	21-30	22	17%
	31-50	79	62%
	>50	18	14%
Span (m)	<10	4	3%
	11-20	20	16%
	21-30	78	61%
	>31	26	20%
Frame spacing(m)	<5	15	12%
	5-6	66	52%
	6-7	40	32%
	>7	7	5%
Eve height (m)	<6	101	79%
	7-10	10	8%
	>11	17	13%
Roof angle (Degree)	<6	30	23%
	6-10	93	73%
	10-13	3	2%
	>13	2	2%

Materials used for the structure

All the portal frames were constructed using hot rolled steel universal beam sections of grade S245 and S275. No portal frame structures were made using fabricated steel sections. Zn/AL roofing sheets and asbestos sheets were commonly used for the roof.

3.3 Selected building parameters for the parametric study

Steel portal frames were categorized according to their parameters based on the survey data. Parameter ranges most commonly used in Sri Lanka were selected for the parametric study.

Table 3.4 and 3.5 show the range of parameters selected for the analysis and design.

Table 3.4 Variable parameters selected for the parametric study

Variable parameters	Span (m)	20, 25, 30, 35, 40, 45,50
	Height (m)	4.5,6.0
	Frame spacing (m)	4.5, 6.0,7.5,9.0

Table 3.5 Fixed parameters selected for the parametric study

Fixed parameters	Length	90m
	Roof angle	10 ^{0}
	Base condition	Pinned
	Member type	Hot rolled UB sections
	Zone	Zone 3
	Number of bays	01
	Haunch length	10% of the span

All portal frames are single bay, pitched roof steel portal frames. Length of the structure was taken as 90m for the convenience of calculation.

Materials

Portal frame structures are designed using S355 hot rolled UB sections and the haunches and apexes were provided from the tapered UB sections used for the rafter. The length of the haunch is taken as 1/10 span from the eave. The haunch at the eave was approximately twice the rafter depth. Cold formed purlins are used for the structure with a standard purlin spacing of 1m-1.3m. The roofing material was considered to be Zn/Al sheets.

Sri Lankan context

Loads

Loads used in Sri Lanka for the analysis are different from the European countries. Snow loads are not applicable and the imposed loads recommended in European standards are higher due to the snow effects. Generally imposed load considered in Sri Lanka range from 0.3kN/m^2 to 0.4kN/m^2 and European guidelines recommend 0.6kN/m^2 to 0.75kN/m^2 minimum imposed load for roof with no access. Wind actions considered for European countries are higher than the Sri Lankan values.

Dimensions

Common eaves height used in Sri Lanka is less than 6m. According to the preliminary design tables given in publications of the Steel Construction Institute, the eaves height ranges from 6m to 12m.

Analysis method

Elastic analysis is commonly used in Sri Lanka for portal frame designs.

Steel

S275 and steel having yield strength of 245N/mm^2 are commonly used for steel portal frames in Sri Lanka. S355 steel was selected for the parametric study as there are many advantages of using S355 over S 275 including higher strength, weight saving and small sections. Carbon content is less in S355 which result smaller carbon foot print. Most countries are presently using S355 steel.

4.0 DESIGN OF PORTAL FRAMES

4.1 Design considerations

S355 hot rolled ‘I’ sections were used for the primary steel work of portal frames for the parametric study. 48 portal frame structures were selected and designed based on Eurocode using the elastic theory.

Haunches are cut from the same size rafters as required and it is welded to the underside of the rafters. They are used at the eave to increase the moment resistance of the eave column connection. Hence the depth of the rafter can be reduced gaining a greater economy.

Initial design was carried out considering the vertical loads and in later design stages lateral stability and buckling resistance is checked by providing lateral restraints. Commercially available computer analysis software was used to analyse the portal frames and to determine the moments and forces.

Steel Construction Institute guidelines were used to design the portal frame structures.

A spread sheet was developed to design portal frames based on Eurocode using the elastic analysis and attached in Appendix C. 48 selected portal frames were designed using this spread sheet.

Wind loads ware calculated for the Eurocode based on BS EN 1991-1-4: 2005[13], Draft National Annex to Eurocode 1[14] and a report on recent development of wind code in Sri Lanka [15].

Following procedures and design checks were carried out to find out the suitable sections for portal frames.

1. Basic design information and frame geometry
2. Actions and combinations (permanent, imposed and wind actions)
3. Preliminary sizing
4. Initial analysis
5. Sensitivity to second order effects
6. Frame imperfections
7. Analysis – using computer analysis software

8. Design
    a. Cross section verification (column and rafter section)
    b. Resistance of the cross section (column and rafter)
        i. Shear resistance
        ii. Bending and shear interaction
        iii. Compression resistance
        iv. Combined bending and axial force
        v. Bending resistance
    c. Buckling verification (column)
        i. Flexural buckling resistance about minor axis
        ii. Lateral torsional buckling resistance
        iii. Adequacy of restraint arrangement
        iv. Interaction of axial force and bending moment
    d. Buckling verification (zone A, zone B and zone C of the rafter)
        i. Flexural buckling resistance about minor axis
        ii. Lateral torsional buckling resistance
        iii. Interaction of axial force and bending moment
    e. Haunch calculations (5 cross sections were selected for calculations)
        i. Calculation of properties
        ii. Cross sectional classification
        iii. Bending resistance
        iv. Shear resistance
        v. Bending and shear interaction
        vi. Compression resistance
        vii. Bending and axial force interaction
        viii. Buckling resistance
    f. Deflection

Procedures and design of portal frames are discussed further in Appendix B and specimen calculation is attached in Appendix C.

5.0 ANALYSIS, RESULTS AND DISCUSSION

5.1General

In this study, different sizes of portal frames are analyzed and designed based on Sri Lankan conditions to discover the behavior of portal frame structures and the impact of Eurocode to those structures. Portal frames are designed using the elastic theory based on Eurocode. A spread sheet is developed for calculation and is attached in Appendix C.

Forces acting on the structures, weight of the structures and member sizes are compared with each other and with available literature.

Parameters were selected based on the results of the field survey and are shown in Table 5.1 and Table 5.2.

Table 5.1 Selected variable parameters and their range used for the parametric study

	Span (m)	Frame spacing			
4.5m eaves height	20	4.5	6	7.5	9.0
	25				
	30				
	35				
	40				
	45				
	50				
6.0m eaves height	30	4.5	6.0	7.5	9.0
	35				
	40				
	45				
	50				

Table 5.2 Fixed parameters used for the parametric study

Fixed parameters	
Total length of the building	90m
Roof angle	10^{0}
Base condition	Pinned
Member type	Hot rolled UB sections
Wind zone	Zone 3
Number of bays	01

Purlin sizes

Simply supported cold formed purlins of “C” type were used with maximum spacing of 1.3m for roof. Table 5.3 shows the purlin sizes used for different frame spacing.

Table 5.3 Purlin details

Frame spacing	Purlin size	Weight (kg/m)
4.5 m	C10019	3.29 kg/m
6.0 m	C15019	4.51 kg/m
7.5 m	C20019	5.74 kg/m
9.0 m	C25024	8.16 kg/m

5.2 Results

Portal frames were analysed and designed to elastic theory based on Eurocode and results are numerically and graphically presented below. S355 grade was used for the design.

Maximum forces and moments of the critical combination were used for the analysis and design. Tables given in section 5.2.1 show the maximum forces and moments obtained after amplifying to accommodate second order effects and frame imperfections. For most of the cases critical combination was combination 1 which includes only vertical loads (permanent and imposed actions).

To find out the implications of Eurocode,

1. Analysis results and weight of the structures obtained from the parametric study were compared with each other
2. Weight of the structures obtained from the parametric study were compared with the
    a. Available literature on portal frames done to British standards,
    b. Field survey data and
    c. Publications of the Steel Construction Institute (P399 and P252 [11],[12]).

Results of parametric study are presented as tables in section 5.2.1.

Table 5.4 to table 5.11 show the results obtained from parametric study carried out according to elastic theory based on Eurocode.

Table 5.4 shows the optimum column and rafter sections obtained for portal frames and table 5.5 shows their respective axial forces and bending moments.

Tables 5.6, 5.7, 5.8 and 5.9 show the variation of forces and moments of columns and rafters with change in span length, frame spacing and eaves height. Weight of individual elements, frames and total structure and the variation of weight with change in span, frame spacing and eaves height are shown in table 5.10 and table 5.11.

Sections 5.2.2 to 5.2.4 present graphs plotted using the above tables with elaborate description and analysis of findings.

Section 5.2.5 compares load effects of Eurocode and British Standards. Section 5.2.6 compares the weight of the structures due to steel grades based on parametric study. Under section 5.2.7, table 5.14 to 5.18 compares the results of parametric study with the field survey data, available literature and publication by the Steel Construction Institute. Evaluations, comparisons and discussions are carried out in this section related to the weight of the structures.

5.2.1 Results – Tables

Table 5.4 Section sizes of portal frames designed to Eurocode - Parametric study

Span (m)	Frame spacing (m)	4.5 m eaves height		6.0 m eaves height	
		Column section	Rafter section	Column section	Rafter section
20	4.5	305x127x37	254x102x25		
	6	305x165x40	254x102x28		
	7.5	305x165x46	305x102x33		
	9	356x171x51	305x127x37		
25	4.5	406x140x46	356x127x33		
	6	356x171x51	356x127x39		
	7.5	406x178x60	305x165x40		
	9	457x152x67	356x171x51		
30	4.5	406x178x54	305x165x40	457x152x52	305x165x40
	6	406x178x60	356x171x51	406x178x67	356x171x51
	7.5	457x191x67	356x171x51	457x191x67	406x178x54
	9	457x191x82	406x178x54	533x210x82	406x178x60
35	4.5	457x152x67	356x171x51	457x152x67	356x171x51
	6	533x165x75	406x178x60	533x165x75	406x140x53
	7.5	533x210x82	457x191x67	457x191x89	457x152x60
	9	533x210x92	457x191x74	610x178x100	457x191x67
40	4.5	457x191x74	457x152x60	533x165x75	406x178x54
	6	533x210x82	457x191x67	533x210x92	457x191x67
	7.5	610x178x100	457x191x82	610x178x100	457x191x74
	9	533x210x109	533x210x92	610x229x113	533x210x82
45	4.5	533x210x82	457x191x67	533x210x82	457x191x67
	6	610x178x100	533x210x82	610x178x100	533x210x82
	7.5	610x229x113	533x210x92	610x229x113	533x210x82
	9	686x254x125	533x210x109	610x229x140	533x210x109
50	4.5	533x210x92	533x210x92	610x178x100	533x210x82
	6	610x229x113	533x210x101	610x229x125	533x210x101
	7.5	686x254x125	533x210x122	686x254x125	610x229x113
	9	686x254x140	610x229x125	610x305x149	533x210x122

Table 5.5 Analysis results of portal frames designed to Eurocode– Parametric study

		4.5 m eaves height					6.0 m eaves height				
Span (m)	Frame spacing (m)	Column			Rafter		Column			Rafter	
		Axial force (kN)	Moment (kNm)	Horizontal force (kN)	Axial force (kN)	Moment (kNm)	Axial force (kN)	Moment (kNm)	Horizontal force (kN)	Axial force (kN)	Moment (kNm)
20	4.5	52	155	34	43	74					
	6	68	202	45	56	96					
	7.5	85	251	55	69	120					
	9	102	299	67	82	140					
25	4.5	66	237	54	63	114					
	6	87	313	69	83	150					
	7.5	108	389	86	103	187					
	9	130	465	104	124	225					
30	4.5	82	341	76	88	163	83	356	59	72	167
	6	107	445	99	115	213	102	408	68	85	244
	7.5	132	515	130	134	225	133	530	88	109	317
	9	158	631	140	164	283	158	681	113	137	320
35	4.5	97	452	100	115	212	96	459	76	90	204
	6	126	592	130	150	277	124	596	99	118	267
	7.5	157	731	162	186	343	157	745	124	148	334
	9	187	870	193	221	408	185	909	149	180	422
40	4.5	112	572	127	144	260	110	601	98	117	269
	6	147	746	166	187	339	147	783	130	152	351
	7.5	183	930	207	234	422	183	976	162	190	438
	9	218	1107	246	278	503	215	1161	191	226	521
45	4.5	132	729	162	181	331	129	754	126	145	336
	6	172	955	212	237	433	170	992	165	191	442
	7.5	212	1177	261	292	534	209	1218	203	234	543
	9	256	1418	316	352	643	251	1460	243	281	651
50	4.5	153	928	207	228	431	152	986	164	187	454
	6	197	1200	267	295	564	198	1284	214	243	592
	7.5	243	1481	329	364	691	238	1545	257	292	712
	9	282	1735	385	426	816	284	1813	302	344	817

Table 5.6 Comparison of column analysis results (Eurocode) -1- Parametric study

Eurocode design -4.5m eaves height				Column					
Span (m)	Frame spacing (m)	Column		% change in axial force when span is increased by 5m	% change in moment when span is increased by 5m	% change in axial force when frame spacing is increased by 1.5m	% change in moment when frame spacing is increased by 1.5m	% change in axial force when eaves height is increased by 1.5m	% change inmoment when eaves height is increased by 1.5m
		Axial (kN)	Moment (kNm)						
20	4.5	52	155						
	6	68	202			31%	30%		
	7.5	85	251			25%	24%		
	9	102	299			20%	19%		
25	4.5	66	237	27%	53%				
	6	87	313	28%	55%	32%	32%		
	7.5	108	389	27%	55%	24%	24%		
	9	130	465	27%	56%	20%	20%		
30	4.5	82	341	24%	44%			1%	4%
	6	107	445	23%	42%	30%	30%	-5%	-8%
	7.5	132	515	22%	32%	23%	16%	1%	3%
	9	158	631	22%	36%	20%	23%	0%	8%
35	4.5	97	452	18%	33%			-1%	2%
	6	126	592	18%	33%	30%	31%	-2%	1%
	7.5	157	731	19%	42%	25%	23%	0%	2%
	9	187	870	18%	38%	19%	19%	-1%	4%
40	4.5	112	572	15%	27%			-2%	5%
	6	147	746	17%	26%	31%	30%	0%	5%
	7.5	183	930	17%	27%	24%	25%	0%	5%
	9	218	1107	17%	27%	19%	19%	-1%	5%
45	4.5	132	729	18%	27%			-2%	3%
	6	172	955	17%	28%	30%	31%	-1%	4%
	7.5	212	1177	16%	27%	23%	23%	-1%	3%
	9	256	1418	17%	28%	21%	20%	-2%	3%
50	4.5	153	928	16%	27%			-1%	6%
	6	197	1200	15%	26%	29%	29%	1%	7%
	7.5	243	1481	15%	26%	23%	23%	-2%	4%
	9	282	1735	10%	22%	16%	17%	1%	4%

Table 5.7 Comparison of Rafter analysis results (Eurocode)-1 –Parametric study

Eurocode design -4.5m eaves height				Rafter					
Span (m)	Frame spacing (m)	Rafter		% change in axial force when span is increased by 5m	% change in moment when span is increased by 5m	% change in axial force when frame spacing is increased by 1.5m	% change in moment when frame spacing is increased by 1.5m	% change in axial force when eaves height is increased by 1.5m	% change in moment when eaves height is increased by 1.5m
		Axial (kN)	Moment (kNm)						
20	4.5	43	74						
	6	56	96			30%	30%		
	7.5	69	120			23%	25%		
	9	82	140			19%	17%		
25	4.5	63	114	47%	54%				
	6	83	150	48%	56%	32%	32%		
	7.5	103	187	49%	56%	24%	25%		
	9	124	225	51%	61%	20%	20%		
30	4.5	88	163	40%	43%			-18%	2%
	6	115	213	39%	42%	31%	31%	-26%	15%
	7.5	134	225	30%	20%	17%	6%	-19%	41%
	9	164	283	32%	26%	22%	26%	-16%	13%
35	4.5	115	212	31%	30%			-22%	-4%
	6	150	277	30%	30%	30%	31%	-21%	-4%
	7.5	186	343	39%	52%	24%	24%	-20%	-3%
	9	221	408	35%	44%	19%	19%	-19%	3%
40	4.5	144	260	25%	23%			-19%	3%
	6	187	339	25%	22%	30%	30%	-19%	4%
	7.5	234	422	26%	23%	25%	24%	-19%	4%
	9	278	503	26%	23%	19%	19%	-19%	4%
45	4.5	181	331	26%	27%			-20%	2%
	6	237	433	27%	28%	31%	31%	-19%	2%
	7.5	292	534	25%	27%	23%	23%	-20%	2%
	9	352	643	27%	28%	21%	20%	-20%	1%
50	4.5	228	431	26%	30%			-18%	5%
	6	295	564	24%	30%	29%	31%	-18%	5%
	7.5	364	691	25%	29%	23%	23%	-20%	3%
	9	426	816	21%	27%	17%	18%	-19%	0%

Table 5.8 Comparison of column analysis results (Eurocode) -2– Parametric study

Eurocode design - (6.0m eaves height)				Column			
		Column					
Span (m)	Frame spacing (m)	Axial (kN)	Moment (kNm)	% change in axial force when span is increased by 5m	% change in moment when span is increased by 5m	% change in axial force when frame spacing is increased by 1.5m	% change in moment when frame spacing is increased by 1.5m
30	4.5	83	356				
	6	102	408			23%	15%
	7.5	133	530			30%	30%
	9	158	681			19%	28%
35	4.5	96	459	16%	29%		
	6	124	596	22%	46%	29%	30%
	7.5	157	745	18%	41%	27%	25%
	9	185	909	17%	33%	18%	22%
40	4.5	110	601	15%	31%		
	6	147	783	19%	31%	34%	30%
	7.5	183	976	17%	31%	24%	25%
	9	215	1161	16%	28%	17%	19%
45	4.5	129	754	17%	25%		
	6	170	992	16%	27%	32%	32%
	7.5	209	1218	14%	25%	23%	23%
	9	251	1460	17%	26%	20%	20%
50	4.5	152	986	18%	31%		
	6	198	1284	16%	29%	30%	30%
	7.5	238	1545	14%	27%	20%	20%
	9	284	1813	13%	24%	19%	17%

Table 5.9 Comparison of rafter analysis results (Eurocode) -2– Parametric study

Eurocode design - (6.0m eaves height)				Rafter			
		Rafter					
Span (m)	Frame spacing (m)	Axial (kN)	Moment (kNm)	% change in axial force when span is increased by 5m	% change in moment when span is increased by 5m	% change in axial force when frame spacing is increased by 1.5m	% change in moment when frame spacing is increased by 1.5m
30	4.5	72	167				
	6	85	244			18%	46%
	7.5	109	317			28%	30%
	9	137	380			26%	20%
35	4.5	90	204	25%	22%		
	6	118	267	39%	9%	31%	31%
	7.5	148	334	36%	5%	25%	25%
	9	180	422	31%	11%	22%	26%
40	4.5	117	269	30%	32%		
	6	152	351	29%	31%	30%	30%
	7.5	190	438	28%	31%	25%	25%
	9	226	521	26%	23%	19%	19%
45	4.5	145	336	24%	25%		
	6	191	442	26%	26%	32%	32%
	7.5	234	543	23%	24%	23%	23%
	9	281	651	24%	25%	20%	20%
50	4.5	187	454	29%	35%		
	6	243	592	27%	34%	30%	30%
	7.5	292	712	25%	31%	20%	20%
	9	344	817	22%	25%	18%	15%

Table 5.10 Comparison of weight of portal frames of 4.5m eaves height designed to Eurocode –Parametric study (Page 30-31)

		Eurocode design-4.5m eaves height									Weight %					
Span (m)	Frame spacing (m)	Column Section	Rafter section	Column weight (kg/m)	Beam weight (kg/m)	Weight/ frame (kg)	Number of frames (90m length building)	Total frame weight (ton)	Total purlin weight (ton)	Total weight (ton) (including purlin weight)	% of Rafter weight to single main frame weight	% change in weight of a single main frame when frame spacing is increased by 1.5m	% change in weight of a single main frame when span is increased by 5m	% change in total weight of the structure when frame spacing is increased by 1.5m	%change in total weight of the structure when span is increased by 5m	% change in total weight of the structure when eaves height is increased by 1.5 m
20	4.5	305x127x37	254x102x25	37	25.2	844.8	21	17.7	7.4	25	61%					
	6	305x165x40	254x102x28	40.3	28.3	1058.3	16	16.9	10.1	27	54%	11%		8%		
	7.5	305x165x46	305x102x33	46.1	32.8	1357.6	13	17.6	12.9	31	49%	15%		13%		
	9	356x171x51	305x127x37	51	37	1669.4	11	18.4	18.4	37	45%	12%		20%		
25	4.5	406x140x46	356x127x33	46	33.1	1254.3	21	26.3	8.9	35	67%		48%		40%	
	6	356x171x51	356x127x39	51	39.1	1604.6	16	25.7	12.2	38	62%	16%	52%	7%	40%	
	7.5	406x178x60	305x165x40	60.1	40.3	1924.5	13	25.0	15.5	41	53%	8%	42%	7%	33%	
	9	457x152x67	356x171x51	67.2	51	2504.3	11	27.5	22.0	50	52%	21%	50%	22%	35%	
30	4.5	406x178x54	305x165x40	54.1	40.3	1714.6	21	36.0	10.1	46	72%		37%		31%	8%
	6	406x178x60	356x171x51	60.1	51	2274.8	16	36.4	13.8	50	68%	22%	42%	9%	33%	11%
	7.5	457x191x67	356x171x51	67.1	51	2560.1	13	33.3	17.6	51	61%	3%	33%	1%	25%	11%
	9	457x191x82	406x178x54	82	54.1	3124.0	11	34.4	25	59	53%	11%	25%	17%	20%	12%

		Eurocode design-4.5m eaves height									Weight %					
Span (m)	Frame spacing (m)	Column Section	Rafter section	Column weight (kg/m)	Beam weight (kg/m)	Weight/ frame (kg)	Number of frames (90m length building)	Total purlin weight (ton)	Total frame weight (ton) (90m length building)	Total weight (ton) (including purlin weight)	% of Rafter weight to single main frame weight	% change in weight of a single main frame when frame spacing is increased by 1.5m	% change in weight of a single main frame when span is increased by 5m	% change in total weight of the structure when frame spacing is increased by 1.5m	%change in total weight of the structure when span is increased by 5m	% change in total weight of the structure when eaves height is increased by 1.5 m
35	4.5	457x152x67	356x171x51	67.2	51	2417.3	21	11.3	50.8	62	75%		41%		35%	8%
	6	533x165x75	406x178x60	74.7	60.1	3032.4	16	15.4	48.5	64	70%	16%	33%	3%	27%	1%
	7.5	533x210x82	457x191x67	82.2	67.1	3617.7	13	19.6	47.0	67	66%	11%	41%	4%	31%	3%
	9	533x210x92	457x191x74	92.1	74.3	4298.4	11	27.9	47.3	75	61%	11%	38%	13%	27%	4%
40	4.5	457x191x74	457x152x60	74.3	59.8	3097.6	21	12.4	65.0	77	78%		28%		25%	1%
	6	533x210x82	457x191x67	82.2	67.1	3711.8	16	17.0	59.4	76	73%	12%	22%	-1%	20%	9%
	7.5	610x178x100	457x191x82	100.3	82	4835.1	13	21.7	62.9	85	69%	22%	34%	11%	27%	1%
	9	533x210x109	533x210x92	109	92.1	5702.8	11	30.8	62.7	94	66%	12%	33%	11%	24%	1%
45	4.5	533x210x82	457x191x67	82.2	67.1	3805.9	21	13.9	79.9	94	81%		23%		21%	6%
	6	610x178x100	533x210x82	100.3	82.2	4959.7	16	19.1	79.4	98	76%	22%	34%	5%	29%	6%
	7.5	610x229x113	533x210x92	113	92.1	5903.4	13	24.3	76.7	101	71%	12%	22%	3%	19%	0%
	9	686x254x125	533x210x109	125.2	109	7234.3	11	34.5	79.6	114	69%	17%	27%	13%	22%	7%
50	4.5	533x210x92	533x210x92	92.1	92.1	5504.9	21	15.1	115.6	131	85%		45%		39%	-2%
	6	610x229x113	533x210x101	113	101	6483.9	16	20.7	103.7	124	79%	12%	31%	-5%	26%	7%
	7.5	686x254x125	533x210x122	125.2	122	8072.1	13	26.3	104.9	131	77%	19%	37%	5%	30%	0%
	9	686x254x140	610x229x125	140.1	125.1	8873	11	37.5	97.6	135	72%	4%	23%	3%	18%	5%

Table 5.11 Comparison of weight of portal frames of 6.0m eaves height designed to Eurocode- Parametric study (Page 32-33)

		Eurocode design (6.0m eaves height)									Weight %				
Span (m)	Frame spacing (m)	Column Section	Rafter section	Column weight (kg/m)	Beam weight (kg/m)	Weight/ frame (kg)	Number of frames (90m length building)	Total purlin weight (ton)	Total frame weight (ton) (90m length building)	Total weight (ton) (including purlin weight)	% of Rafter weight to single main frame weight %	% change in weight of a single main frame when frame spacing is increased by 1.5m	% change in weight of a single main frame when span is increased by 5m	% change in total weight of the structure when frame spacing is increased by 1.5m	%change in total weight of the structure when span is increased by 5m
30	4.5	457x152x52	305x165x40	52.3	40.3	1855.3	21	11	39.0	50	66%				
	6	406x178x67	356x171x51	67.1	51	2358.8	16	15	37.7	52	66%	27%		6%	
	7.5	457x191x67	406x178x54	67.1	54.1	2453.2	13	19	31.9	50	67%	4%		-4%	
	9	533x210x82	406x178x60	82.2	60.1	2817.2	11	26	31.0	57	65%	15%		14%	
35	4.5	457x152x67	356x171x51	67.2	51	2618.9	21	12	55.0	67	69%		41%		35%
	6	533x165x75	406x140x53	74.7	53.3	2790.7	16	16	44.7	61	68%	7%	18%	-9%	16%
	7.5	457x191x89	457x152x60	89.3	59.8	3196.9	13	21	41.6	62	66%	15%	30%	2%	23%
	9	610x178x100	457x191x67	100.3	67.1	3588.3	11	29	39.5	69	66%	12%	27%	11%	20%
40	4.5	533x165x75	406x178x54	74.7	54.1	3093.8	21	13	65.0	78	71%		18%		17%
	6	533x210x92	457x191x67	92.1	67.1	3830.6	16	18	61.3	79	71%	24%	37%	1%	30%
	7.5	610x178x100	457x191x74	100.3	74.3	4221.4	13	23	54.9	78	71%	10%	32%	-2%	25%
	9	610x229x113	533x210x82	113	82.2	4694.7	11	32	51.6	84	71%	11%	31%	8%	22%

		Eurocode design(6.0m eaves height)									Weight %				
Span (m)	Frame spacing (m)	Column Section	Rafter section	Column weight (kg/m)	Beam weight (kg/m)	Weight/ frame (kg)	Number of frames (90m length building)	Total purlin weight (ton)	Total frame weight (ton) (90m length building)	Total weight (ton) (including purlin weight)	% of Rafter weight to single main frame weight %	% change in weight of a single main frame by increasing frame spacing by 1.5m	% change in weight of a single main frame by increasing span by 5m	% change in total weight of the structure by increasing frame spacing by 1.5m	%change in total weight of the structure by increasing span by 5m
45	4.5	533x210x82	457x191x67	82.2	67.1	4052.5	21	15	85.1	100	76%		31%		28%
	6	610x178x100	533x210x82	100.3	82.2	4959.7	16	20	79.4	99	76%	22%	29%	0%	25%
	7.5	610x229x113	533x210x82	113	82.2	5112.1	13	25	66.5	92	73%	3%	21%	-8%	18%
	9	610x229x140	533x210x109	139.9	109	6659.5	11	36	73.3	109	75%	30%	42%	19%	30%
50	4.5	610x178x100	533x210x82	100.3	82.2	5377.0	21	16	112.9	129	78%		33%		29%
	6	610x229x125	533x210x101	125.1	101	6629.1	16	22	106.1	128	77%	23%	34%	-1%	29%
	7.5	686x254x125	610x229x113	125.2	113	7239.6	13	27	94.1	121	79%	9%	42%	-5%	32%
	9	610x305x149	533x210x122	149.2	122	7984.5	11	39	87.83	127	78%	10%	20%	4%	16%

5.2.2 Axial forces on columns and rafters

Figure 5.1 and figure 5.2 show the column axial force variation against frame spacing and span for 4.5m and 6.0m eaves height portal frames. Given axial forces are the amplified forces to accommodate second order effects and frame imperfections.

Increase in span results in higher axial forces to columns which are equal to ‘wL/2’, where ‘w’ is uniformly distributed load acting on rafter and ‘L’ is the length of span. Hence axial force variation with span and frame spacing tends to follow a straight line (y=mx+c). Trend line equations with R^2 values are shown in figures 5.1 and 5.2. R^2 value is 0.98 or more for all the cases which indicates that axial forces vary in a linear behaviour as expected.

Tangent of the graphs increases in the range of 1 when frame spacing increases by 1.5m. Tangent is almost similar for 4.5m and 6.0m eaves height portal frames with similar frame spacing. Axial force differs up to 5% when eaves height changes from 4.5m to 6.0m.

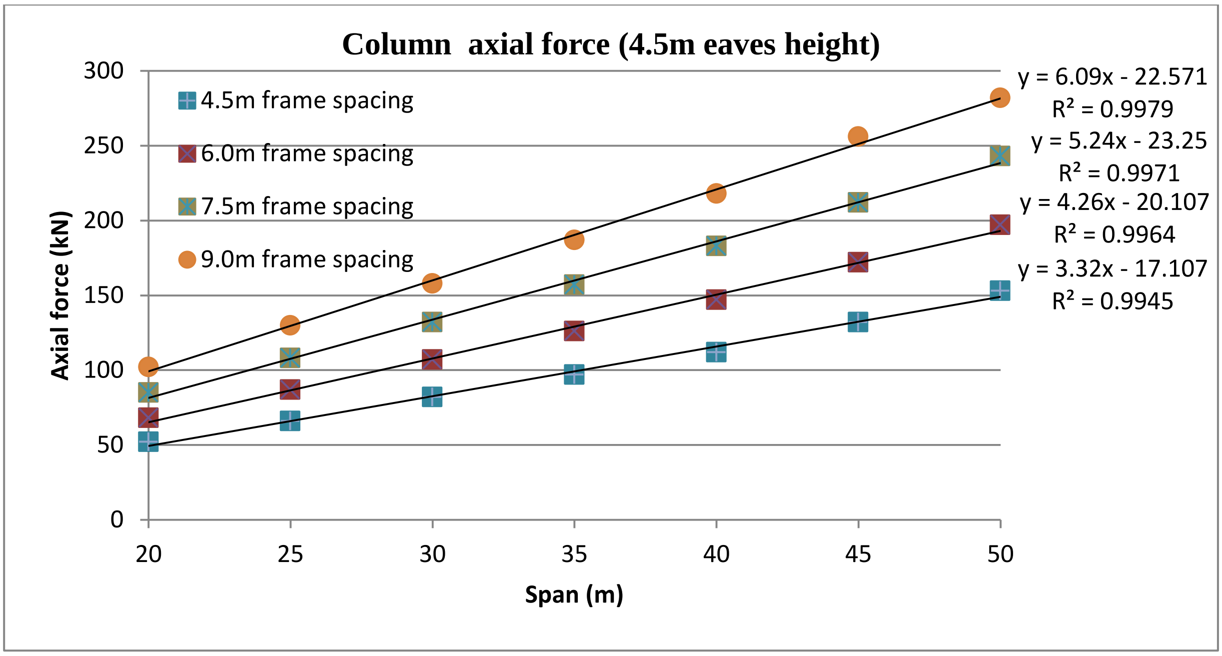


Figure 5.1 Column axial force variations of 4.5m eaves height portal frames designed to Eurocode- Parametric study

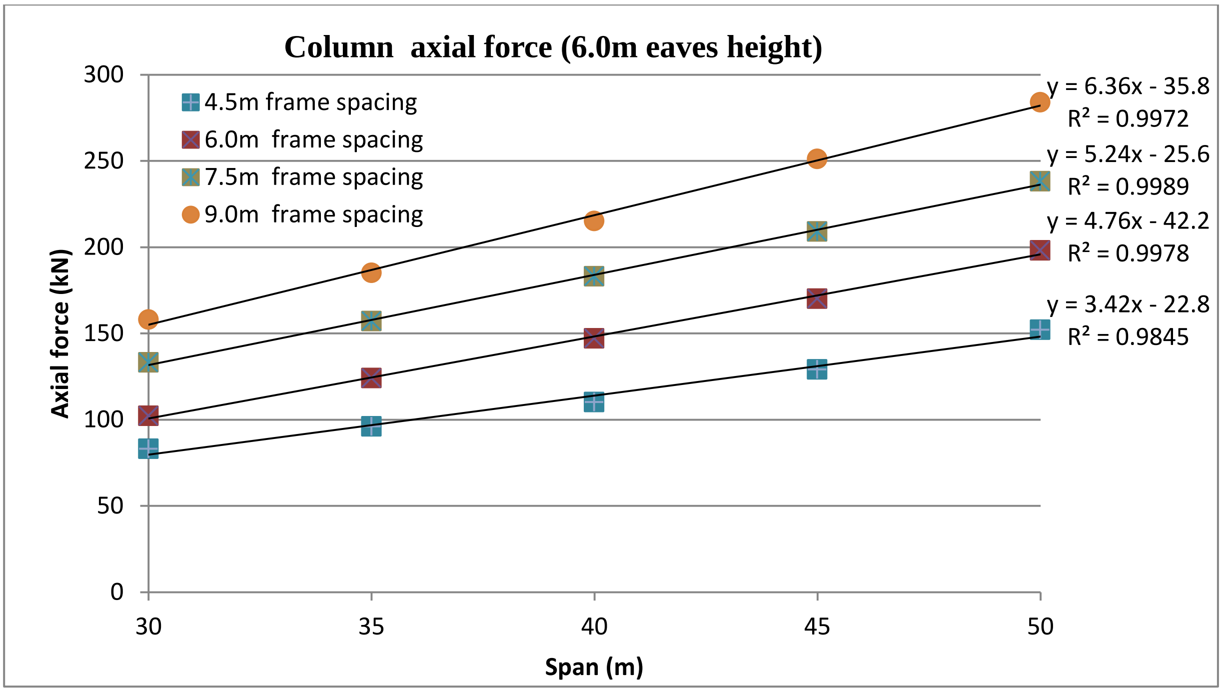


Figure 5.2 Column axial force variations of 6.0m eaves height portal frames designed to Eurocode 3- Parametric study

Figure 5.3 and figure 5.4 show axial force variation in rafters with frame spacing and span. Trend lines and R^2 values are also shown in the figures. R^2 value is 0.97 or more for all the cases which indicates that axial forces vary in a linear behavior as expected.

Higher axial forces in rafters are due to the higher intensity of loads resulting from larger frame spacing and larger spans as discussed under column axial force variation.

For all the cases, axial force in rafters reduces in the range of 15% to 25% when eaves height increases from 4.5m to 6.0m.

This is probably due to reduction of horizontal reaction at the bottom of the column with increase of eaves height from 4.5m to 6.0m.

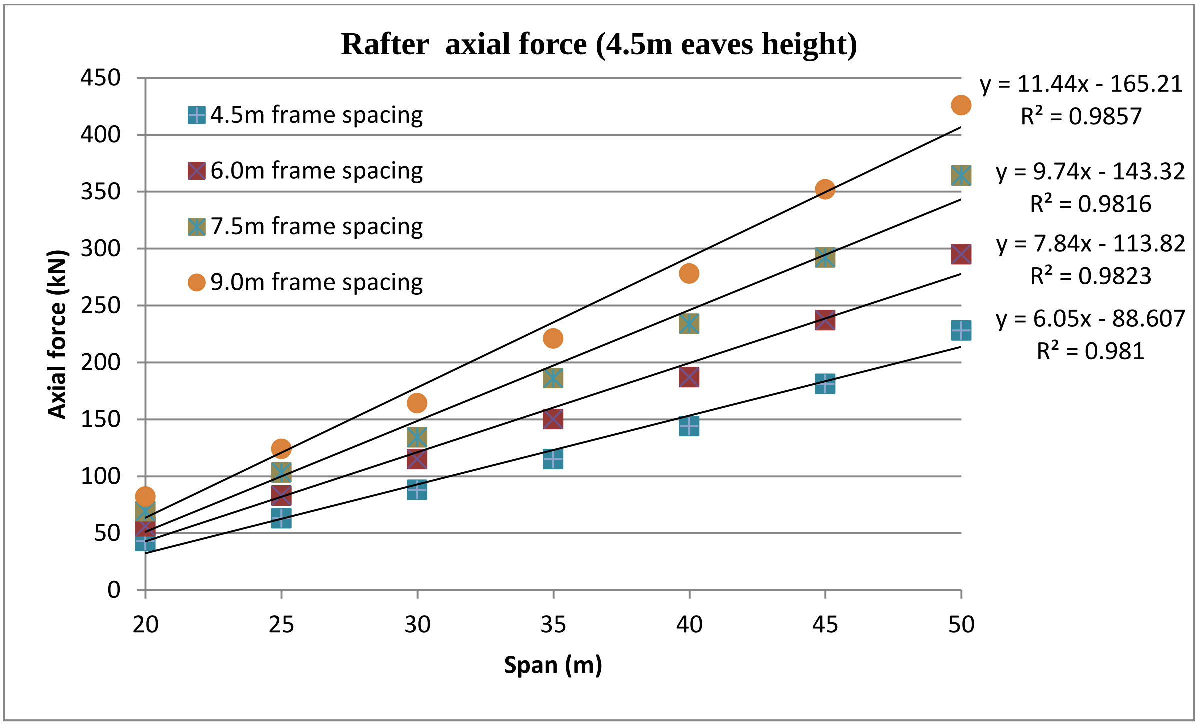


Figure 5.3 Rafter axial force variations of 4.5m eaves height portal frames designed to Eurocode 3- Parametric study

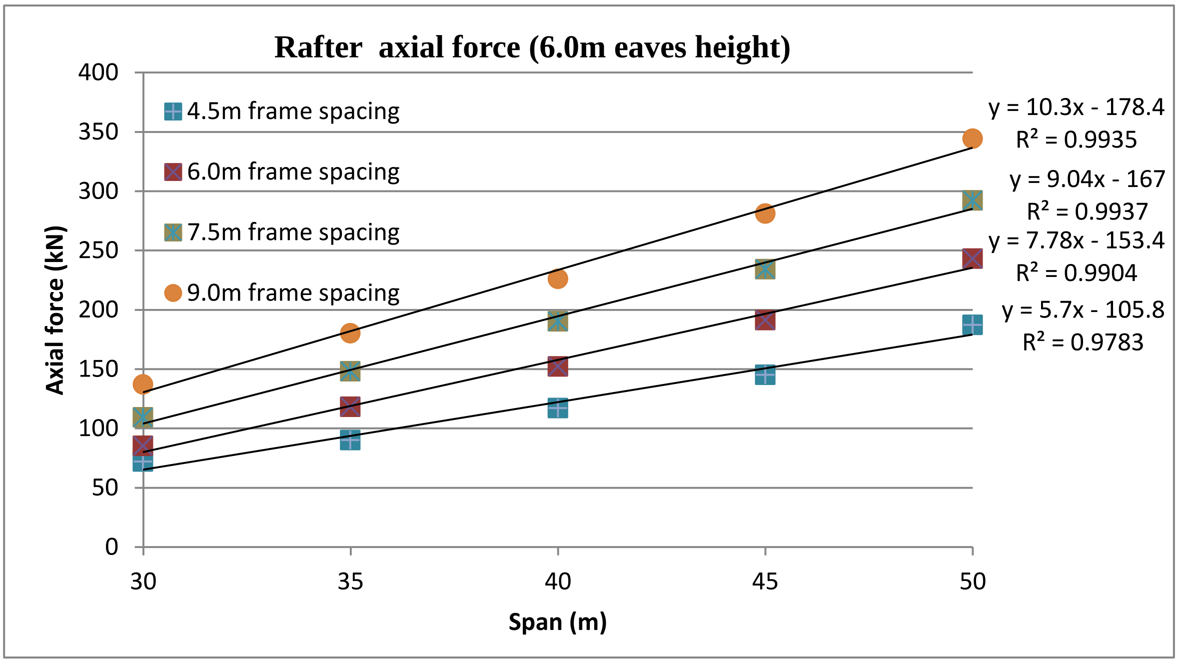


Figure 5.4 Rafter axial force variations of 6.0m eaves height portal frames designed to Eurocode 3- Parametric study

5.2.3 Bending moment of columns

Figure 5.5 and figure 5.6 show bending moment variations at the top of the columns of portal frames having an eaves height of 4.5m and 6.0m.

Bending moment at the top of the column is the maximum moment after amplifying for second order effects and frame imperfections. For most cases critical combination was that consist of permanent actions and variable actions due to imposed load.

Moment at the top of the column increases in a linear fashion with span. Tangent increases in the range of 8 when frame spacing is increased by 1.5 m. According to results, bending moment at the top of column of 6.0m eaves height portal frames are about 10% higher than the 4.5m eaves height portal frames for most cases.

Moment at the top of the column can be obtained from ‘H x h’ where ‘H’ is the horizontal force at the bottom of the column and ‘h’ is the height of the column.

Resultant horizontal forces at the bottom of the column due to actions of critical combination are plotted in the figure 5.7 and 5.8. Horizontal forces increase with span and frame spacing, but reduce with eaves height. Variation of horizontal reaction with eaves height (all other parameters are similar) is less than 25% except for 30m span frames where the variation is about 35%. With span and frame spacing, the intensity of loads increases and as a result, horizontal reaction of the structure increases.

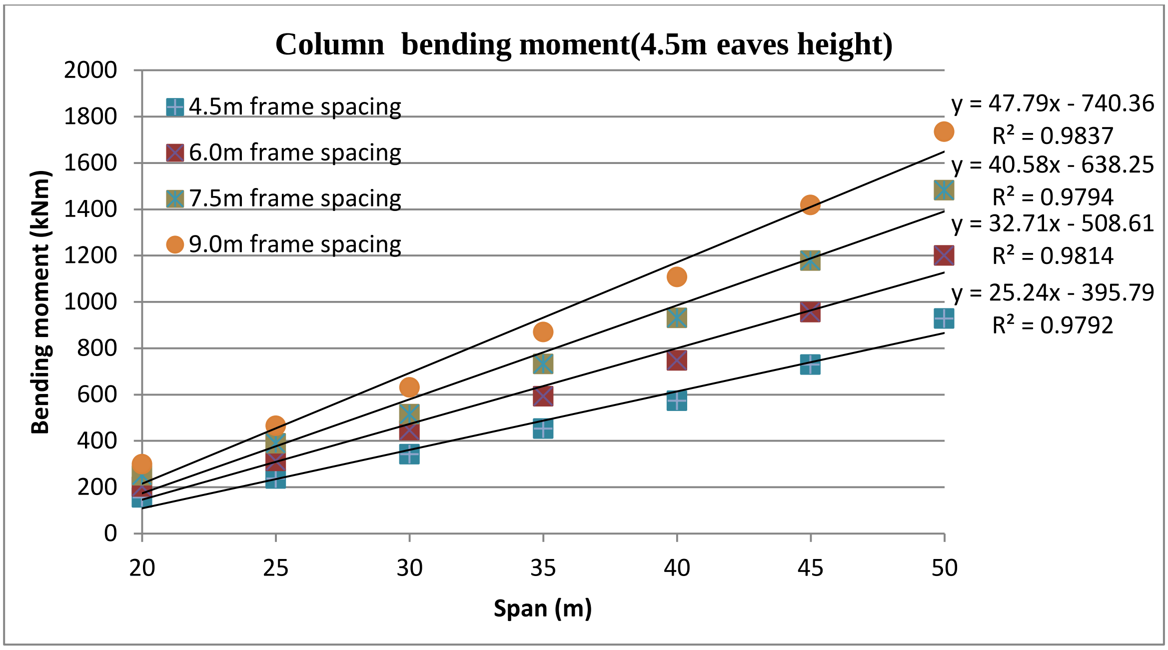


Figure 5.5 Column bending moment variations of 4.5m eaves height portal frames designed to Eurocode- Parametric study

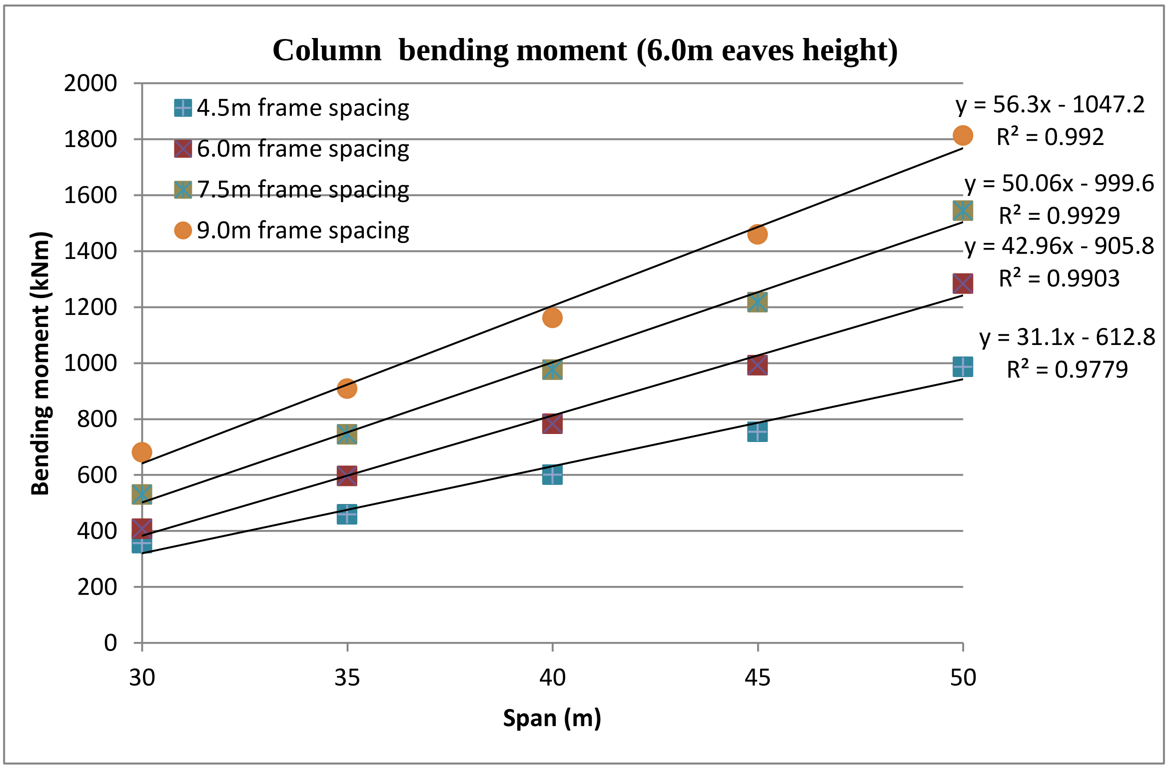


Figure 5.6 Column bending moment variations of 6.0m eaves height portal frames designed to Eurocode- Parametric study

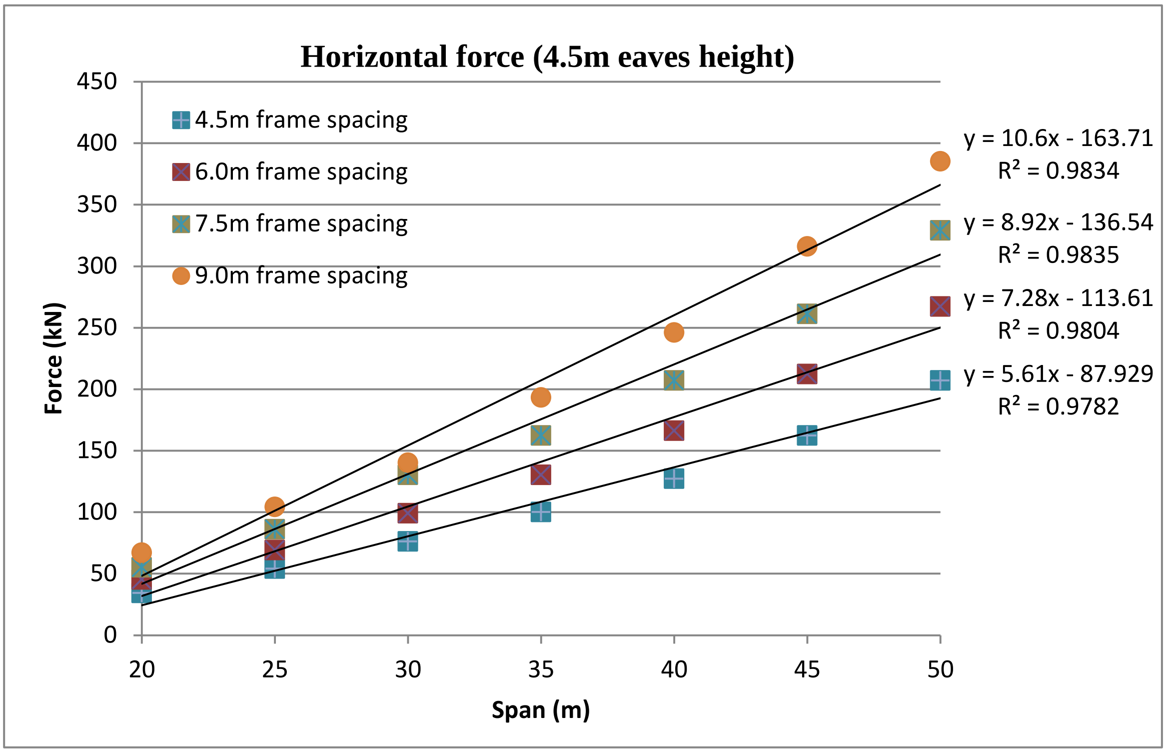


Figure 5.7 Horizontal force variations at the bottom of the column of 4.5m eaves height portal frames designed to Eurocode – Parametric study

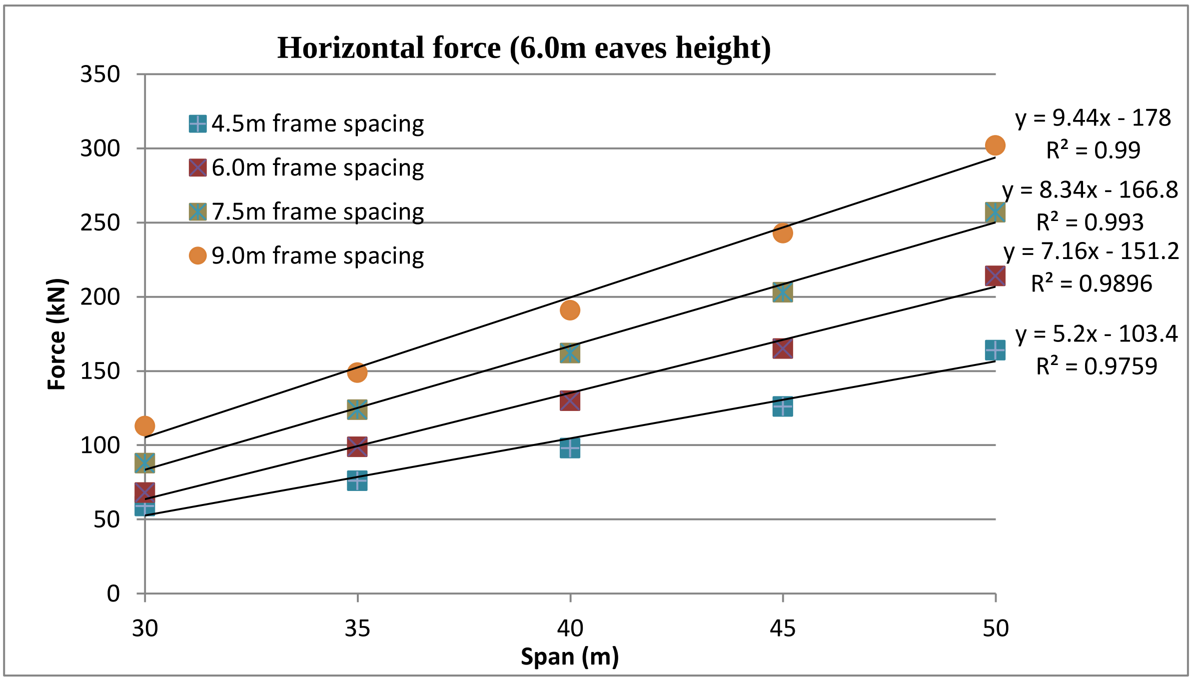


Figure 5.8 Horizontal force variations at the bottom of the column of 6.0m eaves height portal frames designed to Eurocode – Parametric study

5.2.4 Weight comparison – Parametric study

Weight of a single steel frame

Variation of the weight of a single frame against the frame spacing and the span obtained from the parametric study are shown in figure 5.9.

As expected, weight of a single frame increases with the span and the frame spacing. Larger sections are required to cater to higher bending moments and axial forces which result in heavy weight.

Weight is increased in the range of 10% to 20% for most cases when the eaves height increases from 4.5m to 6.0m.

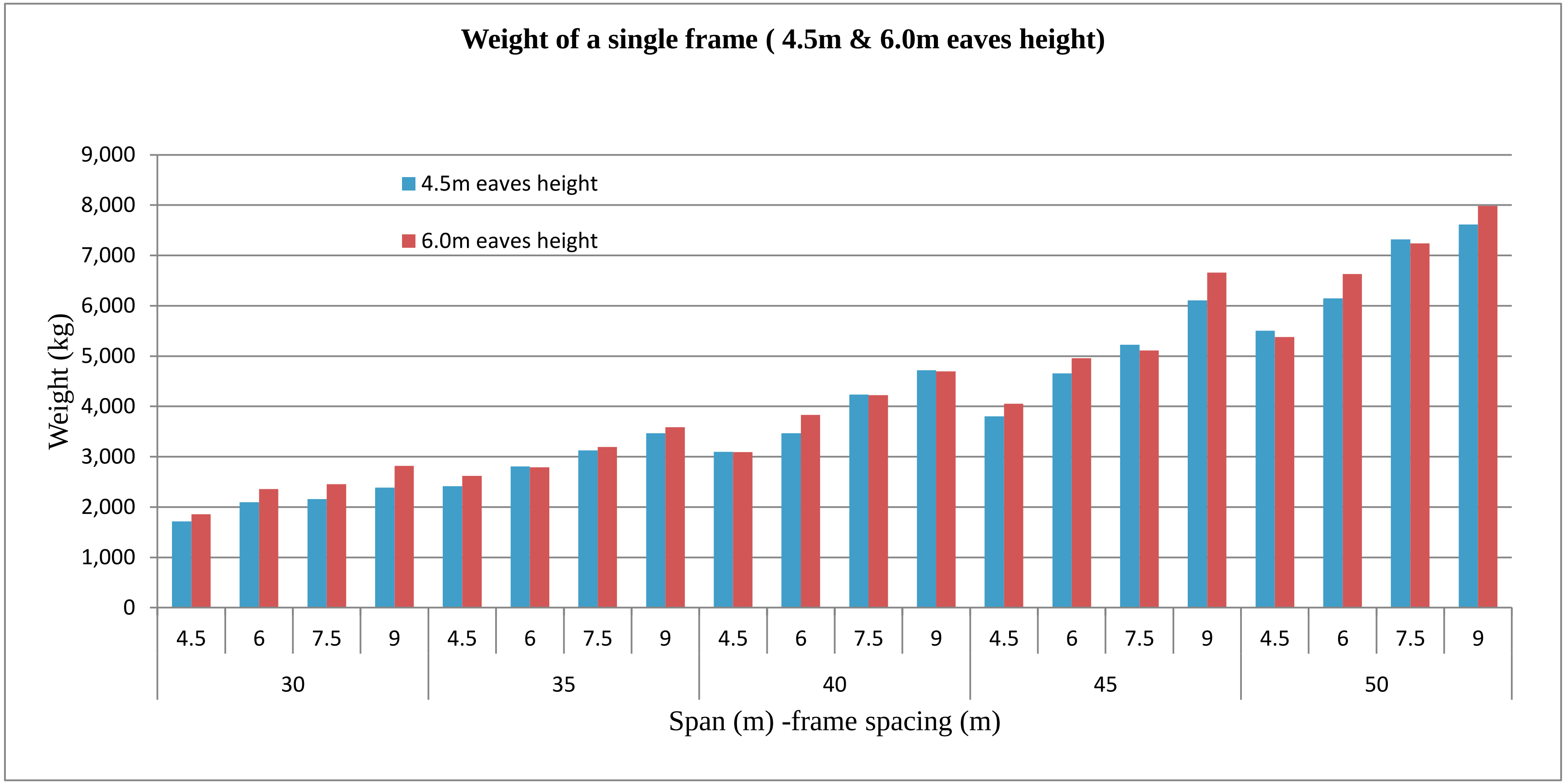


Figure 5.9 Weight of a single main frame designed to Eurocode – Parametric study

Main frame weight as a percentage of ULS axial force on a single column

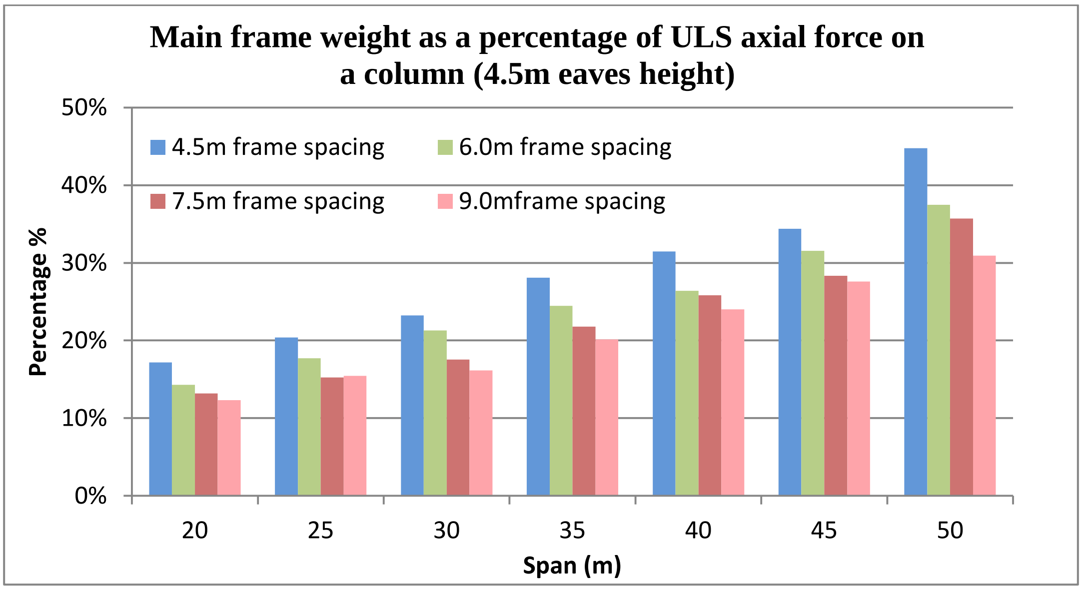


Figure 5.10 Main frame self-weight as a percentage of ULS axial force on a single column of 4.5m eaves height portal frames designed to Eurocode- Parametric study

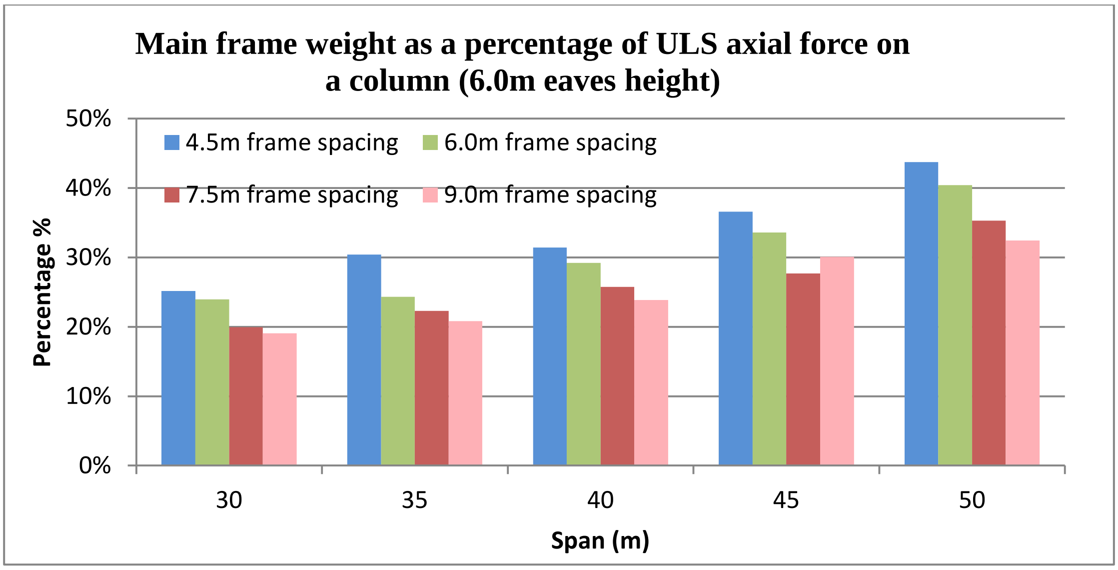


Figure 5.11 Main frame self-weight as a percentage of ULS axial force on a single column of 6.0m eaves height portal frames designed to Eurocode- Parametric study

As shown in figures 5.10 and 5.11, self-weight of the main frame is about 15% to 45% of the ultimate limit state axial force on a single column for all the portal frames having a span of 30m or more.

Rafter weight to weight of a single frame

Weight percentage of the rafter to the weight of a single main frame obtained from parametric study is shown in figure 5.12 and figure 5.13.

As shown in the figure 5.12 (4.5m eaves height frames) the rafter weight percentage out of the weight of a single frame is in the range of 45% to 85%. It reduces with frame spacing and increases with span.

For frames having an eaves height of 6.0m, rafter weight percentage to an individual frame is in the range of 65% - 80%. Percentage weight of rafter to the weight of an individual frame (6.0m eaves height) increase with the span and for most cases, increase with the frame spacing.

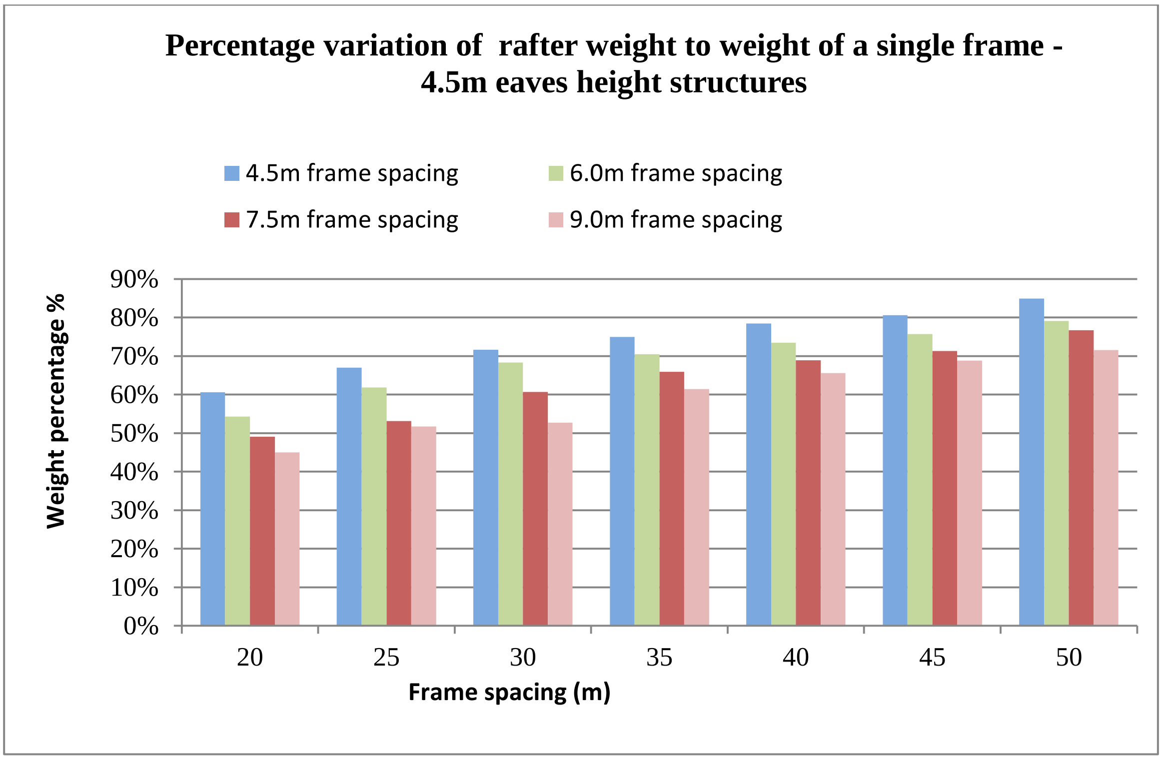


Figure 5.12 Percentage variation of rafter weight to the weight of a single main frame (4.5m eaves height) designed to Eurocode- Parametric study

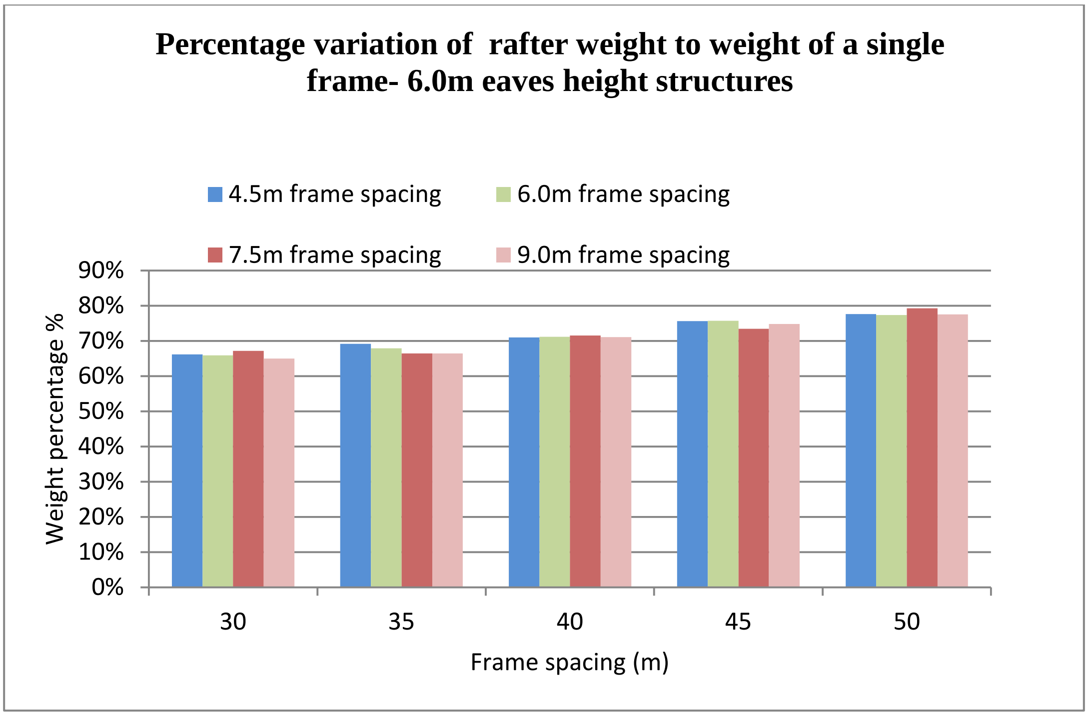


Figure 5.13 Percentage variation of rafter weight to the weight of a single main frame (6.0m eaves height) designed to Eurocode – Parametric study

Total weight of main steel frames of the building

Total weight of main steel frames of a building having a length of 90m is shown in figure 5.14. Total weight consists of column and rafter weight of all the frames in the building.

Total weight of main steel frames of the structure is smaller when frame spacing is 9.0m for all the spans considered in 4.5m eaves height buildings.

For 6.0m eaves height structures, total weight of main steel frames of the structure is lowest when the frame spacing is 9.0m for 30m, 35m, 40m and 50m span structures and 7.5m for 45m span structures.

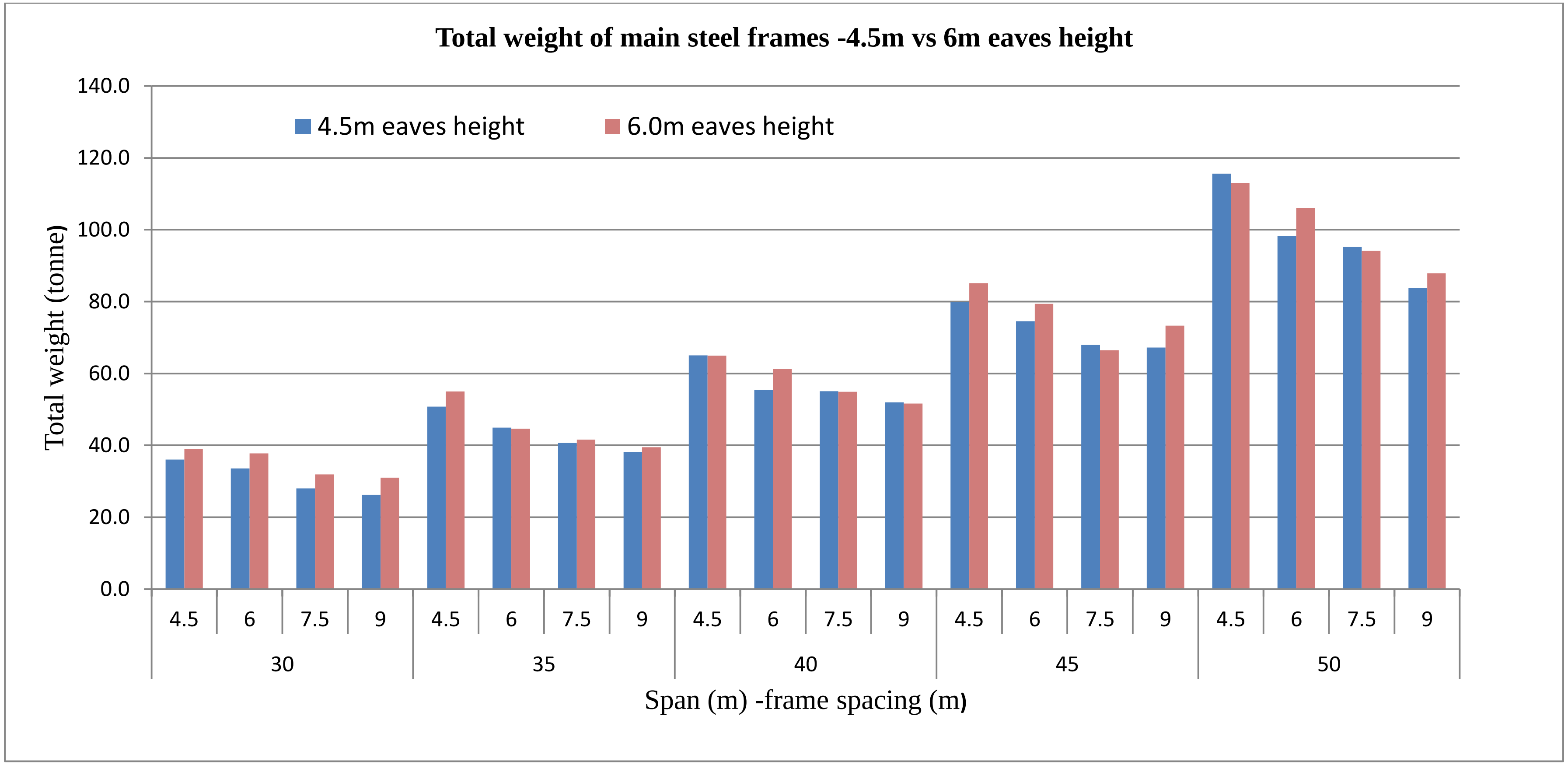


Figure 5.14 Comparison of total weight of the main steel frames designed to Eurocode (90m building length) – Parametric study

Total weight of the building

Total weight variation of the structure against span and frame spacing are shown in figure 5.15. Total weight of the structure includes the weight of main steel frames, purlins and side rails.

Regardless of eaves height of the structure, 9m frame spacing shows the highest weight for all the spans except for 50m span. 4.5m frame spacing gives the highest weight for the 50m span structures. The minimum weight is obtained for frame spacing 6.0m and 7.5m for most cases. For 20m and 25m span frames having an eaves height of 4.5m, total weight of the structure increases progressively with frame spacing.

Structures of 6.0m eaves height show a higher weight up to 12% compared to the 4.5m eaves height structures for most cases.

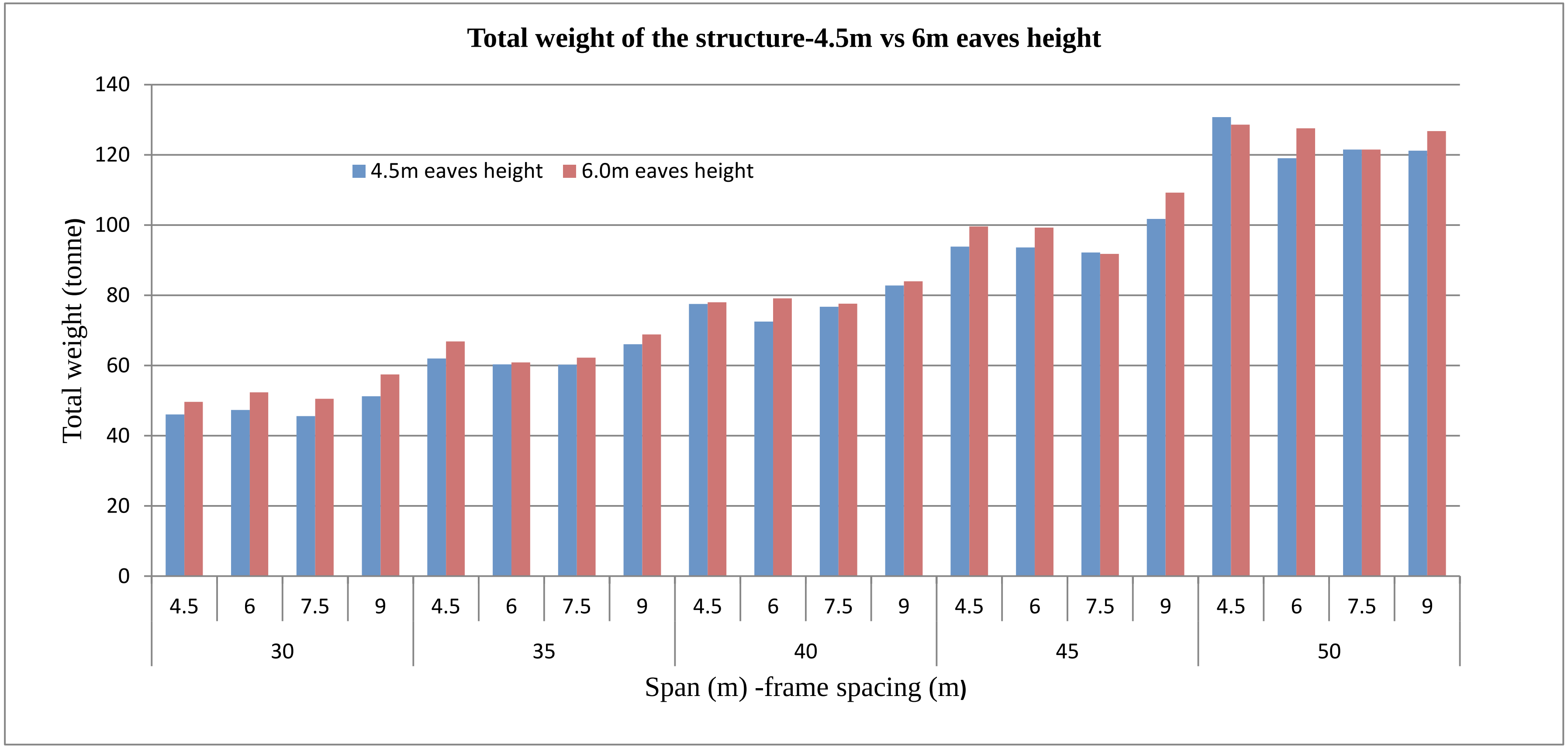


Figure 5.15 Comparison of weight of the structures designed to Eurocode (90m building length) –Parametric study

Purlin and side rail weight as a percentage of total weight of the structure

Percentages of the purlin and side rail weight to total weight of the structure are shown in figure 5.16 and figure 5.17. Weight percentage of purlins increases with frame spacing and reduces with span for all the cases. 10% -50% of the weight of the total structure consists of the purlin weights.

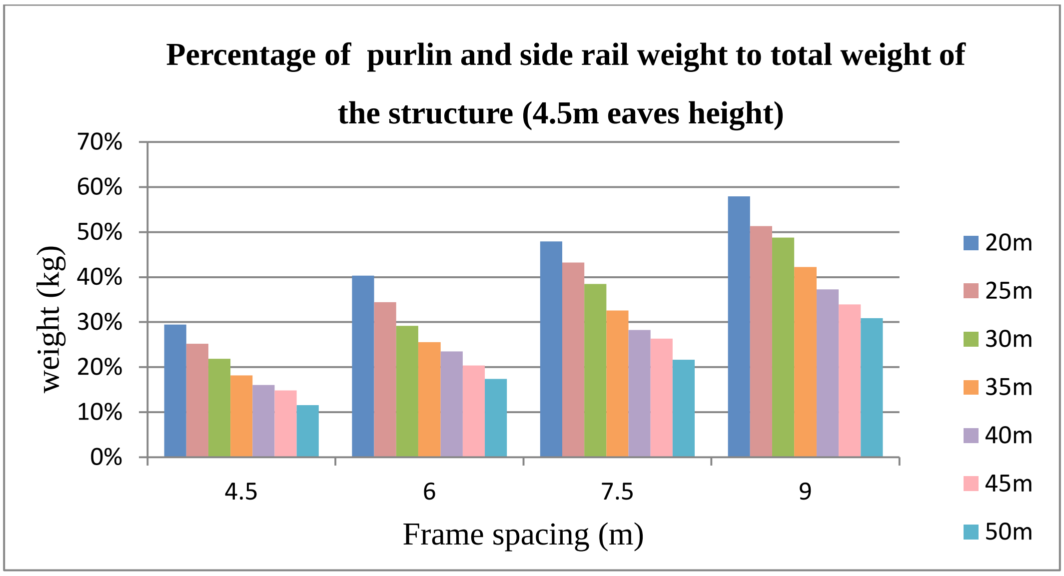


Figure 5.16 Percentage of purlin weight to total weight of the structure (4.5m eaves height) designed to Eurocode- Parametric study

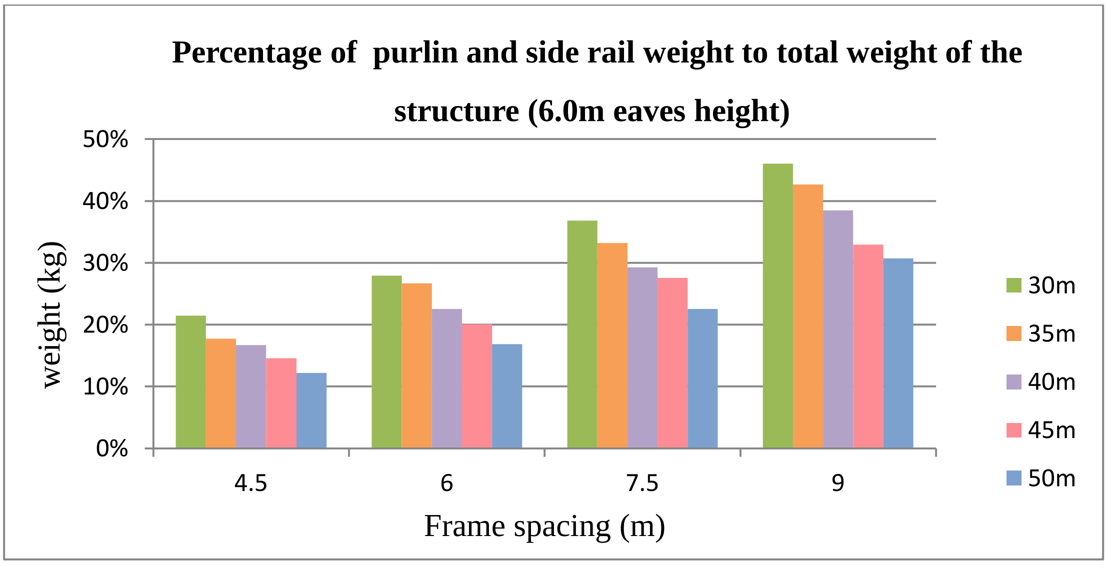


Figure 5.17 Percentage of purlin weight to total weight of structure (6.0 m eaves height) designed to Eurocode- Parametric study

5.2.5 Comparison of load effects- Eurocode and British Standards

In Eurocode the loadings are called as actions and dead loads are named as permanent actions.

Permanent actions / dead load

Dead load (G_k) consists of self weight of the structure, roofing sheets with purlins and services including lighting and Air-conditioning ducts.

Imposed actions / imposed load on roof

Snow loads are not applied for Sri Lankan conditions. Roof imposed load is considered here with no access providing allowance for cleaning and maintenance only.

Wind actions/ wind load on roof

Wind loads ware calculated for the Eurocode based on BS EN 1991-1-4: 2005[13], Draft National Annex to Eurocode 1[14] and a report on recent development of wind code in Sri Lanka [15].

Wind load was calculated for the British standards based on the following documents. CP3: Chapter V: Part 2:1972 1:2000.[19]

Comparison of analysis results

Loads calculated based on Eurocode and British Standards are applied on selected frames and analysed using commercially available software. Results are shown and compared in table 5.12.

Axial force on column and rafter and horizontal force at the bottom of the column calculated based on Eurocode are 5% lower than the British Standard values. Bending moment at the top of the column obtained based on the Eurocode are 6% lower than the results calculated based on British Standards. The reason is the difference in combination factors.

Table 5.12 Comparison of load effects – Parametric study (Eurocode and British Standards)

	Eurocode results					British standard results					% change in			
		Left column					Left column							
Span	Critical Combination	Axial force (kN)	Horizontal force (kN)	Moment at column top (kNm)	Rafter axial force (kN)	Critical Combination	Axial force (kN)	Horizontal force (kN)	Moment at column top (kNm)	Rafter axial force (kN)	Axial force (kN)	Horizontal force (kN)	Moment at column top (kNm)	Rafter axial force (kN)
30	C1	133	89	530	109	C1	140	94	564	115	-5.2%	-5.6%	-6.4%	-5.5%
35	C1	157	124	745	148	C1	165	132	790	155	-5.1%	-6.1%	-6.0%	-4.7%
40	C1	183	162	976	190	C1	193	172	1034	200	-5.6%	-6.2%	-5.9%	-5.2%

5.2.6 Comparison of steel grades- Parametric study

Parametric study was carried out for 3 selected frames using S275 steel based on Eurocode to elastic theory to find the effects of steel grade to the weight of the structure. Table 5.13 shows the weight comparison due to steel grade variation. Structures designed using S275 steel are in the range of 11% heavier than that of the structures designed to S355. Heavy sections incur additional costs due to larger foundations, higher handling cost, etc.

Table 5.13 Comparison of steel grade effects – Parametric study (S355 and S275)

Span (m)	Frame spacing (m)	Parametric study -6.0m eaves height –S355		Parametric study – 6.0m eaves height S275		Weight of a frame (kg)		% change in weight of a single frame
		Column section	Rafter section	Column section	Rafter section	S355	S275	
30	7.5	457x191x67	406x178x54	533x165x75	406x178x60	2453	2727	10.0%
35	7.5	457x191x89	457x152x60	610x178x92	457x191x67	3197	3491	8.4%
40	7.5	610x178x100	457x191x74	610x229x113	533x210x82	4221	4695	10.1%

5.2.7 Comparison of portal frame weights- Parametric study with available literature

5.2.7.1 Research works done by Perera, et al. [1]

Table 5.14 to table 5.15 compare the weights of portal frames designed to Eurocode (parametric study) with the research works done by Perera, et al.[1] and it is graphically presented in figure 5.18

Portal frames designed to Eurocode are heavier for all the cases. This is mainly due to the method of analysis used. Perera, et al. [1] have designed portal frame structures to plastic theory and the parametric study was carried out to elastic theory. Plastic theory is more economical and elastic theory is more conservative.

Perera et al.[1] used grade 43 steel (yield strength 275N/mm^2) and designs were done based on BS5950- 1:1990 whereas parametric study was done using S355 steel (Yield strength 355N/mm^2 based on Eurocode 3.

Percentage variation is calculated by,

$$\%\ variation = \left(\frac{Weight\ \ (Eurocode\ 3) - Weight\ (British\ \ standard)}{Weight\ (British\ standard)}\right) X\ 100\%$$

Table 5.14 Comparison of sections of 4.5m eaves height portal frames designed to Eurocode (parametric study) and research works by Perera,et al.[1] to BS5950-1:1990

Parametric study (Eurocode)				Research works[1] BS 5950-1:1990		% change in weight of a Single main frame
Span (m)	Frame spacing (m)	Column section	Rafter section	Column section	Rafter section	
20	4.5	305x127x37	254x102x25	254x146x31	254x102x22	16%
	6	305x165x40	254x102x28	254x146x37	254x102x22	20%
	7.5	305x165x46	305x102x33	305x165x40	305x102x25	25%
	9	356x171x51	305x127x37	305x165x40	305x102x28	29%
25	4.5	406x140x46	356x127x33	305x165x40	305x102x25	26%
	6	356x171x51	356x127x39	305x165x46	305x102x28	28%
	7.5	406x178x60	305x165x40	305x165x46	356x127x33	25%
	9	457x152x67	356x171x51	356x171x51	356x127x33	46%
30	4.5	406x178x54	305x165x40	305x165x46	305x102x33	21%
	6	406x178x60	356x171x51	356x171x51	356x127x33	43%
	7.5	457x191x67	356x171x51	406x178x54	406x140x46	14%
	9	457x191x82	406x178x54	457x191x67	406x140x46	19%
35	4.5	457x152x67	356x171x51	356x171x51	406x140x39	31%
	6	533x165x75	406x178x60	406x178x60	406x140x46	29%
	7.5	533x210x82	457x191x67	457x191x67	457x152x52	27%
	9	533x210x92	457x191x74	457x191x74	457x152x60	24%
40	4.5	457x191x74	457x152x60	406x178x60	406x140x46	29%
	6	533x210x82	457x191x67	457x191x67	457x152x52	27%
	7.5	610x178x100	457x191x82	457x191x82	457x152x60	34%
	9	533x210x109	533x210x92	533x210x82	457x191x67	36%
45	4.5	533x210x82	457x191x67	457x191x67	457x152x52	27%
	6	610x178x100	533x210x82	533x210x82	457x152x60	34%
	7.5	610x229x113	533x210x92	533x210x92	457x191x67	34%
	9	686x254x125	533x210x109	610x229x101	457x191x74	42%
50	4.5	533x210x92	533x210x92	457x191x74	457x152x60	49%
	6	610x229x113	533x210x101	533x210x92	457x191x67	45%
	7.5	686x254x125	533x210x122	610x229x101	457x191x74	56%
	9	686x254x140	610x229x125	610x229x113	533x210x82	47%

Table 5.15 Comparison of sections of 6.0m eaves height portal frames designed to Eurocode (parametric study) and research works by Perera,et al.[1] to BS5950-1:1990

Parametric study (Eurocode)				Research works[1] BS 5950-1:1990		% change in weight of a Single main frame
Span (m)	Frame spacing (m)	Column section	Rafter section	Column section	Rafter section	
30	4.5	457x152x52	305x165x40	406x140x46	305x102x33	20%
	6	406x178x67	356x171x51	406x178x54	406x140x39	28%
	7.5	457x191x67	406x178x54	457x152x60	406x140x46	16%
	9	533x210x82	406x178x60	457x191x67	457x152x52	17%
35	4.5	457x152x67	356x171x51	406x178x54	406x140x39	29%
	6	533x165x75	406x140x53	406x178x60	406x140x46	18%
	7.5	457x191x89	457x152x60	457x191x67	457x152x52	20%
	9	610x178x100	457x191x67	533x210x82	457x152x60	15%
40	4.5	533x165x75	406x178x54	457x191x67	406x140x46	16%
	6	533x210x92	457x191x67	457x191x74	457x152x60	15%
	7.5	610x178x100	457x191x74	533x210x82	457x191x67	14%
	9	610x229x113	533x210x82	533x210x92	457x191x74	14%
45	4.5	533x210x82	457x191x67	457x191x74	457x152x52	23%
	6	610x178x100	533x210x82	533x210x82	457x152x67	22%
	7.5	610x229x113	533x210x82	610x229x101	457x191x67	19%
	9	610x229x140	533x210x109	610x229x101	533x210x82	34%
50	4.5	610x178x100	533x210x82	533x210x92	457x152x60	30%
	6	610x229x125	533x210x101	610x229x101	457x191x74	33%
	7.5	686x254x125	610x229x113	610x229x113	533x210x82	31%
	9	610x305x149	533x210x122	610x229x125	533x210x92	29%

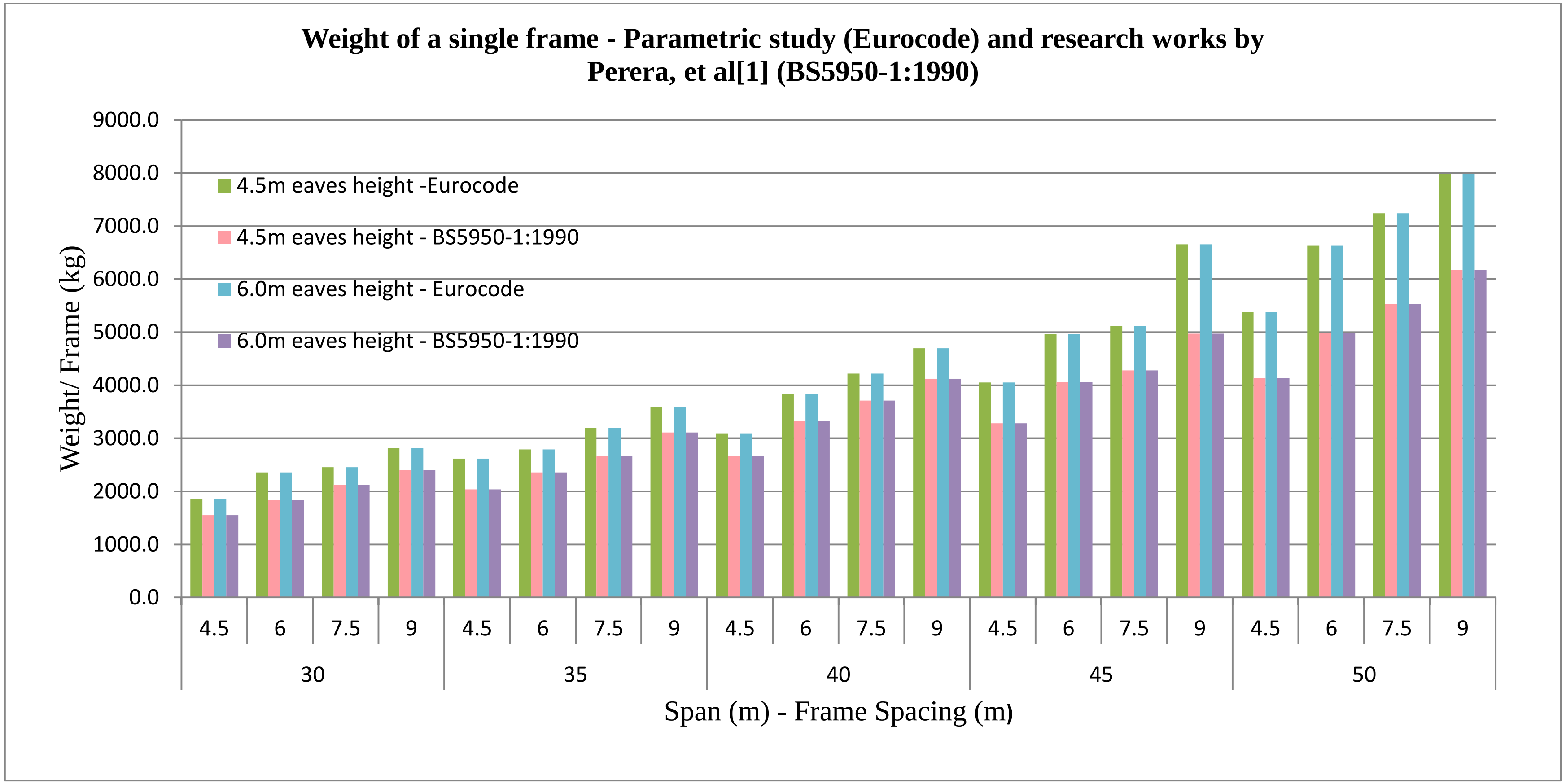


Figure 5.18 Comparison of a single frame weight- Parametric study (Eurocode) and research works by Perera, et al.[1] (BS59501:1990)

Figure 5.19 and 5.20 show percentage variation of weight of a single frame obtained from parametric study (Eurocode 3) to research works done by Perera, et al.[1](BS 5950-1:1990).

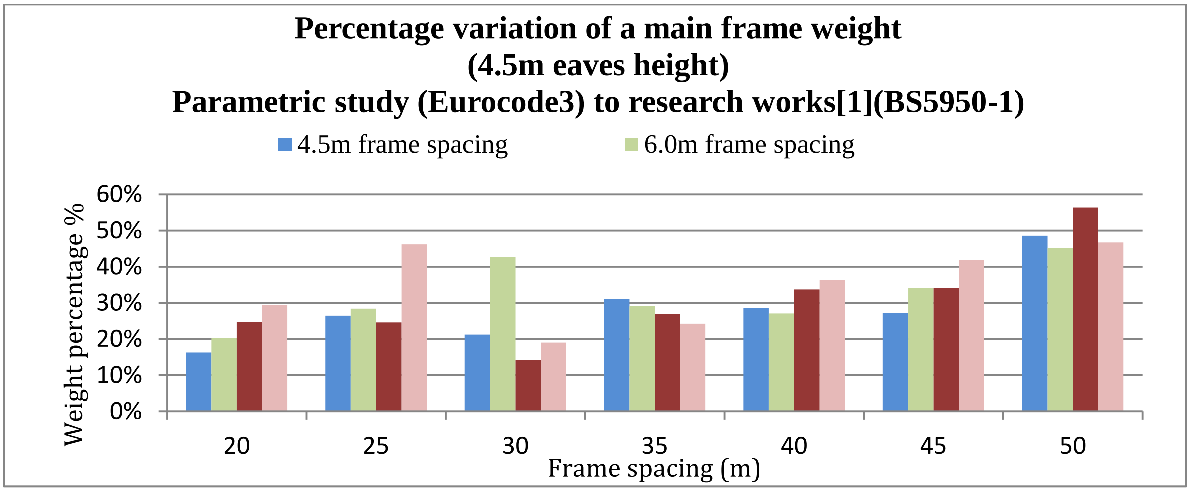


Figure 5.19 Percentage variation of a main frame weight (4.5m eaves height) - Parametric study (Eurocode) to research works by Perera, et al.[1] (BS5950-1:1990)

For all the cases of 4.5m eaves height portal frames, weights of the frames designed to Eurocode are more than 10% greater than those of the British standards.

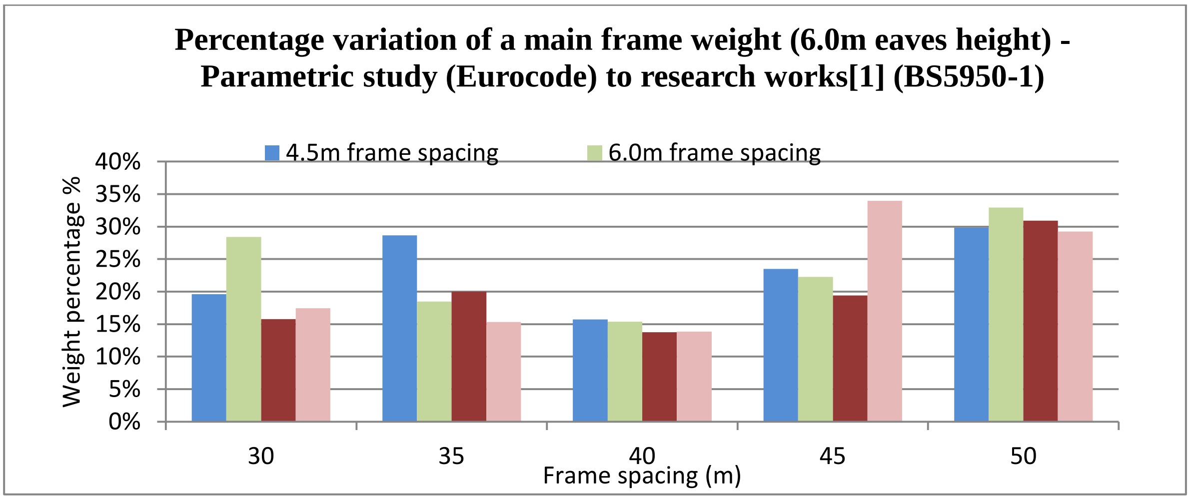


Figure 5.20 Percentage variation of a main frame weight (6.0m eaves height) – Parametric study (Eurocode) to research works by Perera, et al.[1] (BS5950-1:1990)

For 6.0m eaves height portal frames, weights of the frames designed to Eurocode are 15% -35% greater than those of the British standards.

5.2.7.2 Data available in publications of the Steel Construction Institute

Table 5.16 and table 5.17 compare the weight of the sections designed to Eurocode obtained from the parametric study with the preliminary sections proposed by the Steel Construction Institute publications. (SCI- P399 and SCI –P252) [11,12].

SCI guidelines on preliminary sizing provide the section sizes based on the design load on rafters. In parametric study, portal frames having frame spacing of 7.5m matches with the design load given in the SCI document.

Parametric study and preliminary sizes proposed in SCI –P399 [11] were done based on the Eurocode 3 for S355 grade universal beam sections. Single main frame weights obtained from parametric study are about 3% heavier than that of the SCI-P399 [11] proposed sections.

Preliminary sizes given in SCI -P252 [12] were done based on BS5950-1:2000 for S275 grade universal beam sections. Parametric study results are about 3% heavier than the SCI-P252 [12] proposed sections.

SCI publications state that the member sizes given by the Steel Construction Institute are suitable for rapid preliminary design only or at the estimating stage.

Table 5.16 Comparison of the sections obtained from parametric study (Eurocode) with preliminary sizes given by the Steel Construction Institute (P399) [11] – 6.0m eaves height portal frames

Span (m)	Frame spacing (m)	Parametric study -6.0m eaves height –S355		SCI -6.0m eaves height-S355		Weight of a frame (kg)		% change in weight of a single frame
		Column section	Rafter section	Column section	Rafter section	Eurocode	SCI	
30	7.5	457x191x67	406x178x54	457x191x82	356x171x45	2453.2	2355	4%
35	7.5	457x191x89	457x152x60	533x210x92	356x171x57	3196.9	3131	2%
40	7.5	610x178x100	457x191x74	610x229x113	356x171x67	4221.4	4081	3%

Table 5.17 Comparison of the sections obtained from parametric study (Eurocode) with preliminary sizes given by the Steel Construction Institute (P252) [12] – 6.0m eaves height portal frames

Span (m)	Frame spacing (m)	Parametric study -6.0m eaves height –S355		SCI -6.0m eaves height-S275		Weight of a frame (kg)		% change in weight of a single frame
		Column section	Rafter section	Column section	Rafter section	Eurocode	SCI	
30	7.5	457x191x67	406x178x54	533x210x82	406x140x46	2453.2	2388	3%
35	7.5	457x191x89	457x152x60	533x210x92	406x178x60	3196.9	3241	-1%
40	7.5	610x178x100	457x191x74	610x229x113	457x191x67	4221.4	4081	3%

Note: SCI guidelines on preliminary sizing provide the section sizes based on the design load on rafters. In parametric study, portal frames having frame spacing of 7.5m matches with the design load given in the SCI document.

5.2.7.3 Field survey data

Table 5.18 compares the portal frames designed to Eurocode obtained from parametric study with the data collected through the field survey.

Eurocode designed sections are about 5% heavier than the sections used in Sri Lanka for all the cases except for 35m span.

In Sri Lanka most of the portal frames are designed to elastic theory.

It was found through field survey that limited steel section sizes having yield strength of 245N/mm^2 are commonly available in Sri Lanka and it takes long process, significant time and extra cost to import any other sections sizes. Hence engineers in the industry tend to design portal frames to the commonly available sections. It results in limitations to designing portal frames to optimum sizes.

Table 5.18 Comparison of sections obtained from parametric study (Eurocode) and field survey data - 6.0m eaves height portal frames

Span (m)	Frame spacing (m)	Parametric study -Eurocode -6.0m eaves height-S355		Field survey -6.0m eaves height- (Yield strength 245N/mm^2)		Weight of a frame (kg)		% change in weight of a single frame
		Column section	Rafter section	Column section	Rafter section	Eurocode	Field data	
30	6	406x178x67	356x171x51	400x200x66	350x175x49	2358.8	2284.7	3%
35	6	533x165x75	406x140x53	450x200x76	400x200x56	2790.7	2902.2	-4%
40	6	533x210x92	457x191x67	500x200x89	400x200x66	3830.6	3748.7	2%
45	6	610x178x100	533x210x82	600x200x106	450x200x76	4959.7	4744.8	5%
50	6	610x229x125	533x210x101	600x300x151	500x200x89	6629.1	6330.6	5%

5.3 Discussion

Parametric study was carried out for 48 portal frames, which were designed varying the parameters selected through the results of the field survey. Results were compared by means of forces, moments and weight. Steel sections and weights were also compared with available literature and field data. Using these findings the implications of Eurocode for the steel portal frame structures in Sri Lanka were identified.

Self-weight of the steel portal frame varied from 5% to 23% of the ultimate design load on the portal frame (exclusive of self-weight) depending on the span.

Structures designed based on the parametric study using S275 steel are in the range of 11% heavier than that of the structures designed to S355.

Structures designed to Eurocode obtained from the parametric study are heavier than that of the structures designed by Perera, et al. [1]. Parametric study was done following the elastic theory to Eurocode 3 using S355 steel sections. Research works done by Perera, et al. [1] have followed plastic theory to BS5950-1:1990 using S275 steel sections. Main reason for heavier sections is the analysis method. Plastic analysis is more economical and elastic analysis is more conservative resulting heavy sections.

Table 5.19 and 5.20 gives the critical design criteria and the sequences, when using sections proposed by Perera, et al. [1] to design portal frames with similar parameters to Eurocode 3 based on the parametric study. The numbers and the critical design criteria referred here are in accordance with the specimen calculation attached in Appendix B. For most of the cases, the sections fail at the cross section classification stage. (Zone A- column face to haunch end, zone B- haunch end to point of contraflexure, zone C- point of contraflexure to ridge)

Sections designed to Eurocode, obtained from the parametric study are about 3% heavier than the sections proposed by the steel Construction Institute.
Parametric study and preliminary sizes proposed in SCI –P399 [11] were done based on the Eurocode 3 for S355 grade universal beam sections. Preliminary sizes given in

SCI -P252 [12] were done based on BS5950-1:2000 for S275 grade universal beam sections. The section dimensions too were found to be very similar.

SCI publications state that the member sizes given by the Steel Construction Institute are suitable for rapid preliminary design only or at the estimating stage.

Eurocode designed sections obtained from the parametric study are about 5% heavier than the sections used in Sri Lanka for all the cases except for 35m span.

Field study revealed that the engineers commonly use elastic theory and limited steel section sizes having yield strength of 245N/mm^2 to design portal frames in Sri Lanka. The process to import any other sections with different steel grades consumes a significant time period and an extra cost. Hence engineers in the industry tend to design portal frames to the commonly available sections. It results in limitations to designing portal frames to optimum sizes.

Eurocode discusses the second order effects and methods to reflect it in the designs. Second order effects inside a certain limit and sway imperfections are incorporated to the design by introducing factors such as “α_{cr}” factor and imperfection factor. Design actions are amplified by those factors accordingly. When second order effects exceed a certain limit, Eurocode guides to check whether second order analysis is required or first order analysis is adequate via calculations. Due to second order effects, forces and moments are increased and this results in heavy sections.

More Complex and time consuming calculations and lengthy procedures are few disadvantages of the Eurocode.

Table 5.19 Critical design criteria and the sequences, when using the sections proposed by Perera, et al [1] for parametric study – 1

Critical design criteria	4.5m eaves height portal frames- span (m), frame spacing (m)																			
	30				35				40				45				50			
	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9
9.2.1Column																				
shear resistance																				
Bending and shear interaction																				
Compression resistance																				
Bending resistance	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
9.2.2 Rafter																				
shear resistance																				
Bending and shear interaction																				
Compression resistance																				
Bending resistance		x								x	x	x			x	x		x	x	x
10.1 Column verification -buckling verification																				
Lateral- torsional buckling resistance, $M_{b,Rd}$	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Flexural buckling resistance - minor axis, $N_{b,z,Rd}$																				
Arrangement 2	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Interaction of axial force and bending moment	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
10.2 Rafter Verification																				
Flexural buckling resistance minor axis, $N_{b, z, Rd}$																				
Lateral -torsional buckling resistance, $M_{b,Rd}$																				
Interaction of axial force and bending moment in accordance with expression 6.62																				

Critical design criteria	4.5m eaves height span (m), frame spacing (m)																			
	30				35				40				45				50			
	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9
10.2.2 Zone B - hogging region																				
Flexural buckling resistance - minor axis, $N_{b,z,Rd}$																				
Lateral torsional buckling resistance, $M_{b,Rd}$	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Interaction of axial force and bending moment in accordance with expression 6.62	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
10.2.3 Resistance to in-plane buckling and bending																				
Flexural buckling resistance - major axis, $N_{b,y,Rd}$																				
Interaction of axial force and bending moment in accordance with expression 6.61														x		x		x	x	x
11 Verification of haunch length																				
11.1 Bending resistance- cross section 1		x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Calculation for the cross section 2						x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Calculation for the cross section 3												x		x	x	x	x	x	x	x
Calculation for the cross section 4								x		x	x	x		x	x	x	x	x	x	x
Shear resistance																				
Bending and shear interaction																				
Compression resistance																				
Bending and axial force interaction		x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
11.2 Buckling resistance																				

Table 5.20 Critical design criteria and the sequences, when using the sections proposed by Perera, et al [1] for parametric study –2

Critical design criteria	6.0m eaves height- span (m), frame spacing (m)																			
	30				35				40				45				50			
	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9
9.2.1 Column																				
Shear resistance																				
Bending and shear interaction																				
Compression resistance																				
Bending resistance	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
9.2.2 Rafter																				
shear resistance																				
Bending and shear interaction																				
Compression resistance																				
Bending resistance															x			x		
10.1 Column verification- buckling verification																				
Lateral- torsional buckling resistance, $M_{b,Rd}$	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Flexural buckling resistance - minor axis, $_{Nb,z,Rd}$																				
Arrangement 2	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Interaction of axial force and bending moment	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
10.2 Rafter Verification																				
Flexural buckling resistance - minor axis, $N_{b, z, Rd}$																				
Lateral -torsional buckling resistance, $M_{b,Rd}$																				
Interaction of axial force and bending moment in accordance with expression 6.62																				

Critical design criteria	6.0m eaves height- span (m), frame spacing (m)																			
	30				35				40				45				50			
	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9	4.5	6	7.5	9
10.2.2 Zone B - hogging region																				
Flexural buckling resistance - minor axis, $N_{b,z,Rd}$																				
Lateral torsional buckling resistance, $M_{b,Rd}$	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Interaction of axial force and bending moment in accordance with expression 6.62	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
10.2.3 Resistance to in-plane buckling and bending																				
Flexural buckling resistance - major axis, $N_{b,y,Rd}$																				
Interaction of axial force and bending moment in accordance with expression 6.61																	x	x		x
11 Verification of haunch length																				
11.1 Bending resistance- cross section 1	x		x	x	x	x	x	x	x		x	x	x	x	x	x	x	x	x	x
Calculation for the cross section 2	x					x	x		x		x	x	x	x	x	x	x	x	x	x
Calculation for the cross section 3	x											x			x		x	x	x	x
Calculation for the cross section 4												x		x	x	x	x	x	x	x
Shear resistance																				
Bending and shear interaction																				
Compression resistance																				
Bending and axial force interaction	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
11.2 Buckling resistance																				

6.0 CONCLUSION AND RECOMMENDATIONS

A parametric study was carried out for the elastic analysis and design of 48 different portal frame structures to find out the implications of the Eurocode for steel portal frames in Sri Lanka.

Conclusions were reached from the parametric study pertaining to analysis, weight and British standards and guide lines published by the Steel Construction Institute.

Analysis

Column axial forces vary in the range of ± 5% when eaves height changes from 4.5m to 6.0m. Axial force increases in a linear manner against the span with a gradient of 3 to 6 for all the cases and the gradient increases by 1 when the frame spacing increases by 1.5m.

For all the cases, axial force in the rafters reduces in the range of 15% to 25% when the eaves height increases from 4.5m to 6.0m. Axial force varies in a linear fashion against the span with a gradient range from 5 to 10.

Resultant horizontal force at the bottom of the column increases with frame spacing and span, but reduces with eaves height of the structure.Variation of horizontal reaction when eaves height changes from 4.5m to 6.0m, is less than 25% except for 30m span frames which is about 35%.

Moment at the top of the column increases in a linear fashion with the span. According to results the bending moment at the top of column of 6.0m eaves height portal frames is about 10% higher than the 4.5m eaves height portal frames for most cases.

Axial forces, moments and horizontal forces on columns and rafters of portal frames of span ranges 20m to 50m can be estimated by the equations given in the graphs in chapter 5.

Weight

Self-weight of the steel portal frame varied from 5% to 23% of the ultimate design load on the portal frame (exclusive of self-weight) depending on the span.

Weight of a single frame increases with span and frame spacing and for most cases, weight is increased in the range of 10% to 20% when eaves height increase from 4.5m to 6.0m. Rafter weight is in the range of 45% to 85% of the weight of a single frame.

Total weight of main steel frames of the structure is lowest when frame spacing is 9.0m for all the spans considered for portal frames except for 45m span 6.0m eaves height structures where the lowest is for 7.5m frame spacing.

9m frame spacing consumes the highest weight of the total structure (including purlins and side rails) for all cases except for 50m span which is 4.5m frame spacing. The minimum weight is obtained with the frame spacing of 6.0m and 7.5m for most cases. Structures of 6.0m eaves height show a higher total weight of up to 12% compared to the 4.5m eaves height structures.

Structures designed based on the parametric study using S275 steel are in the range of 11% heavier than that of the structures designed to S355.

Comparison of parametric study data with other available data

a) BS 5950-1: 1990

Member sizes obtained from the parametric study (elastic theory, S 355) are,

- For 4.5m eaves height portal frames, 15% to 55% heavier than the sections designed for BS 5950-1:1990 using the plastic theory and S275 steel.
- For 6.0m eaves height portal frames, 10% to 35% heavier than the sections designed for BS 5950-1:1990 using the plastic theory and S275 steel.

b) Field survey data

- Member sizes obtained from the parametric study (elastic theory, S 355) are about 5% heavier than the sections used in Sri Lanka (field survey) for all the cases except for 35m span (yield strength of 245N/mm^2 steel).

c) Publications by the Steel Construction Institute

Member sizes obtained from the parametric study (elastic theory, S 355) are

- About 2% to 4% heavier than that of the sections proposed by the Steel Construction Institute (P399) for Eurocode 3 (steel grade S355). And section dimensions were found to be similar.
- About 3% heavier than that of the sections proposed by the Steel Construction Institute (P252) for BS 5950-part 1:2000 (steel grade S275) and section dimensions were found to be similar.

Recommendations

Parametric study was carried out for 48 selected portal frames of span range from 20m to 50m, frame spacing from 4.5m to 9.0m and eaves height from 4.5m to 6.0m. The frames were designed using S355 steel sections to elastic theory.

It is recommended to design portal frames to plastic theory based on Eurocode to identify whether greater economy can be achieved over the structures designed to plastic analysis to BS5950.

Only 3 selected portal frames were designed using S275 steel sections to compare the effect of steel grades to the structures. It is recommended to carry out elastic design for all portal frames in the selected range using S275 steel to compare the effects.

Due to the limited time constraints, effects of connections were not explored here. It is recommended to carry out the connection designs to identify the effects to the structure.

Pinned bases and rigid connections were assumed in the parametric study though they practically act as semi rigid connections. It is recommended to consider the rigidity of the connections practically to identify the effects to the structures.

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Appendix A:

Questionnaire

Research Questionnaire

Introduction

This survey is being conducted for research purposes by Mrs. K.I.S.G.Premachandra, a student following the Master of Science Degree (Structural Design) at the Department of Civil Engineering, University of Moratuwa.

The purpose of the questionnaire is to study about the portal frame structures in Sri Lanka and to identify the implications of the Eurocode for those structures.

Your response to this survey, or any individual question on the survey, is completely voluntary. You will not be individually identified and your responses will be used for statistical purposes only.

1.0 Details of portal frame structures

1.1 Name of theProject/ division of theCompany/ Authority/ Bureau/ Department working for:

....

1.2 Number of Portal frame buildings construction/ design involved in for past 5 years:.....

....

1.3Locations of the buildings:..

Refer Figure 1 for Zone classification.

Zone	Zone 1			Zone 2			Zone 3		
	☐			☐			☐		
Number of buildings	1-5	5-10	>10	1-5	5-10	>10	1-5	5-10	>10
	☐	☐	☐	☐	☐	☐	☐	☐	☐

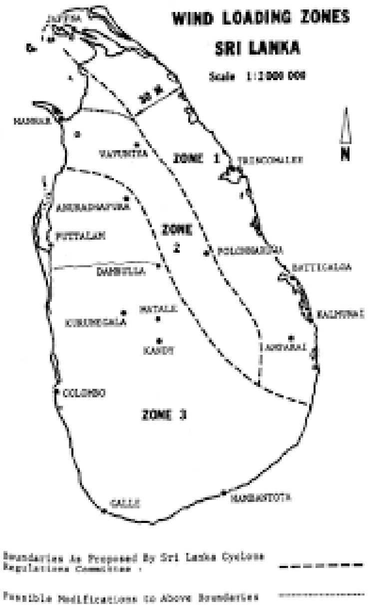

Figure 1

1.4Purposes of the buildings:

Purpose	Factory Building			Warehouse			Vehicle Showroom			Shops			Sports Building		
	☐			☐			☐			☐			☐		
Number of buildings	1-5	5-10	>10	1-5	5-10	>10	1-5	5-10	>10	1-5	5-10	>10	1-5	5-10	>10
	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

1.5Standard code used for the design:

BS5950 -1990	☐
BS5950 – 2000	☐
EUROCODE	☐

Comments:..

1.6 Method of design:

Elastic	☐
Plastic	☐

Comments:..

....

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2.0 Dimensions

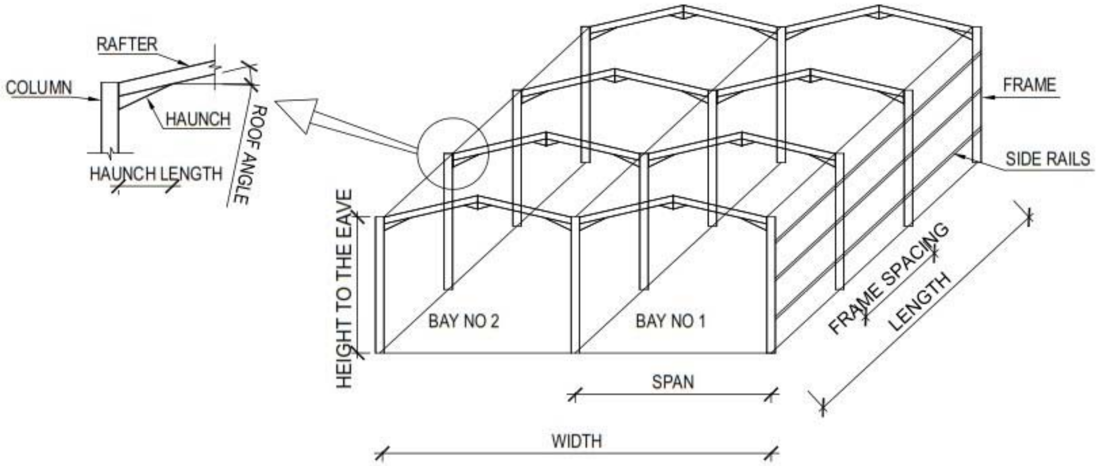


Figure 2.0

2.1 Lengths of the buildings:

Length	<20m (65ft)			21m -30m (68ft -98ft)			31m-50m (101ft -164ft)			>50m (164ft)		
	☐			☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:...

2.2 Width of the building:

Width	<10m (32ft)			11m -20m (36ft -65ft)			21m-30m (68ft -98ft)			>31m (101ft)		
	☐			☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:...

2.3Height to the eave

Height	<6m (32ft)			7m -10m (22ft -32ft)			>11m (36ft)		
	☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:--

2.4 Number of bays

number	1			2			3			>4		
	☐			☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:--

2.5 Span of the building

Span	<10m (32ft)			11m -20m (36ft -65ft)			21m-30m (68ft -98ft)			> 31m (101ft)		
	☐			☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:--

2.6 Roof angle

Span	$<6^0$			6^0-10^0			10^0-13^0			$> 13^0$		
	☐			☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:--

2.7 Haunch details

2.7.1 Have u used haunches for the Portal frame?

Yes	☐
No	☐

Comments:………………………………………………………………………………………

2.7.1 Rafter type with haunches

Uniform Sections with Haunches	☐
Tapered sections	☐

Comments:………………………………………………………………………………………

2.8Frame spacing

Spacing	<5m (16ft)			5m -6m (16ft -19ft)			6m-7m (19ft -22ft)			>7m (22ft)		
	☐			☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

3.0 Footing details

3.1 Footing type

Footing Type	Pad footing			Piles			Other		
	☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:………………………………………………………………………………………

3.2 Column – Footing connection

Footing Type	Pinned			Fixed			Partially fixed		
	☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:……………………………………………………………………………………

4.0 Material Properties

4.1 Material used for the Portal frame

Column	Steel			Concrete			Concrete column & Steel rafter		
	☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:……………………………………………………………………………………

Rafter	Steel			Concrete			Concrete column & Steel rafter		
	☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:……………………………………………………………………………………

4.1.1 For Steel Portal frames

Frame Section	Hot- rolled Universal beams			Fabricated steel sections			Other		
	☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:……………………………………………………………………………………

4.2Cladding material

Material	Brick			Block			Aluminum Cladding			Other		
	☐			☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:--

4.3 Roofing material

Material	Zn/Al roofing sheet			Asbestos roofing sheet			Clay tiles			Other		
	☐			☐			☐			☐		
% of the number of buildings	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51	<20	21-50	>51
	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:--

Thank you for your time taken in completing this questionnaire. The results of this work will be important for the implementation of steel portal frame structures in Sri Lanka. Your comments regarding the questionnaire are highly appreciated. Contact me via the email, sgpgayani@gmail.com.

Appendix B:

Design of portal frames

APPENDIX B
DESIGN OF PORTAL FRAMES

B.1 Design considerations

S355 hot rolled ‘I’ sections were used for theprimary steel work of portal frames for the parametric study. 48portal frame structures were selected and designed based on the Eurocode using the elastic theory.

Haunches are cut from the same size rafters as required and it is welded to the underside of the rafters. They are used at the eave to increase the moment resistance of the eave column connection. Hence the depth of the rafter can be reduced gaining a greater economy. Providing more stiffness to the frame, reducing deflection and facilitating an efficient bolted moment resisting connection are few other advantages of a haunch.

Initial design was carried out considering the vertical loads and in later design stages lateral stability and buckling resistance is checked by providing lateral restraints.

Steel Construction Institute guidelines were used to design the portal frame structures.

Specimen design calculation for a portal frame is given in the Appendix C.

Following procedures and design checks were carried out to find out the suitable sections for portal frames.

1. Basic design information and frame geometry
2. Actions and combinations (Permanent, imposed and wind actions)
3. Preliminary sizing
4. Initial analysis
5. Sensitivity to second order effects
6. Frame imperfections
7. Analysis – using computer analysis software
8. Design
    a. Cross section verification (column and rafter section)
    b. Resistance of the cross section (Column and rafter)
        i. Shear resistance

        ii. Bending and shear interaction
        iii. Compression resistance
        iv. Combined bending and axial force
        v. Bending resistance
    c. Buckling verification (Column)
        i. Flexural buckling resistance about minor axis
        ii. Lateral torsional buckling resistance
        iii. Adequacy of restraint arrangement
        iv. Interaction of axial force and bending moment
    d. Buckling verification (Zone A, Zone B and Zone C of the rafter)
        i. Flexural buckling resistance about minor axis
        ii. Lateral torsional buckling resistance
        iii. Interaction of axial force and bending moment
    e. Haunch calculations (5 cross sections were selected for calculations)
        i. Calculation of properties
        ii. Cross sectional classification
        iii. Bending resistance
        iv. Shear resistance
        v. Bending and shear interaction
        vi. Compression resistance
        vii. Bending and axial force interaction
        viii. Buckling resistance
    f. Deflection

Procedures and design checks are discussed further in the specimen calculation attached in Appendix C.

B.2 Actions

Following actions were considered for the analysis of the frames.

1. Permanent actions – self weight of frame and haunches, weight of secondary steel structure and connections, service loads (lighting and air conditioning ducts) and roof weight were considered. Weight of roofing sheets with purlins

and accessories were taken as 0.3kN/m^2 and 0.05kN/m^2 was provided for the services.

2. Imposed load on roof –
   As per BS EN 1991-1-1, Table 6.10, imposed load for type H roofing with no access except for normal maintenance and repair is taken as 0.4 kN/m^2. [6]
3. Wind actions – The wind action calculations were done based on BS EN 1991-1-1-4:2005, draft national annex and research documents developed for Sri Lanka. [13], [14]

Combination of actions

Following combinations of actions were used for the design.

Table B.1 Combinations of actions

Combination 1	1.35	Permanent	1.5	Imposed
Combination 2	1.35	Permanent	1.5	Wind (positive or negative)
Combination 3	1	Permanent	1.5	Wind (positive or negative)

Imposed roof loads are not considered in combination with either wind actions or snow loads in Eurocode (cl. 3.3.2) [6]

Robustness

Robustness requirements are designed to ensure that any structural collapse is not disproportionate to the cause.

For portal frames, no special provisions are needed to satisfy robustness requirements set by the Eurocode.

B.3 Analysis of portal frames

Commercially available computer analysis software was used to analyse the portal frames and to determine the moments and forces.

Figure B.1 shows typical bending moment diagram of a portal frame under symmetrical vertical loading arrangement.

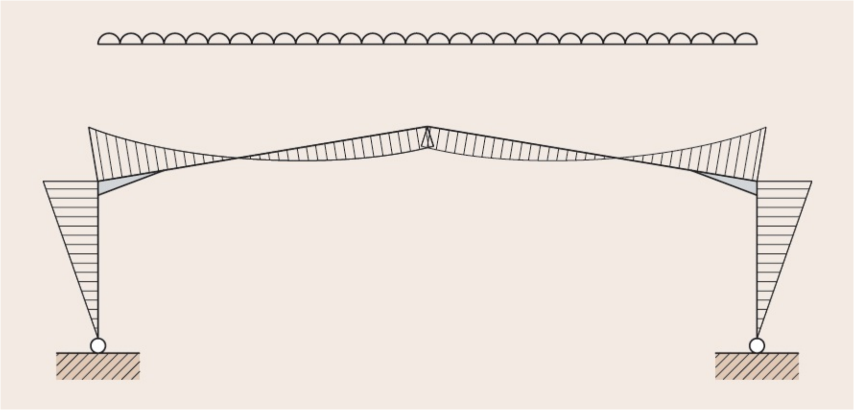


Figure B.1 Typical Bending moment under symmetrical vertical loading arrangement [7]

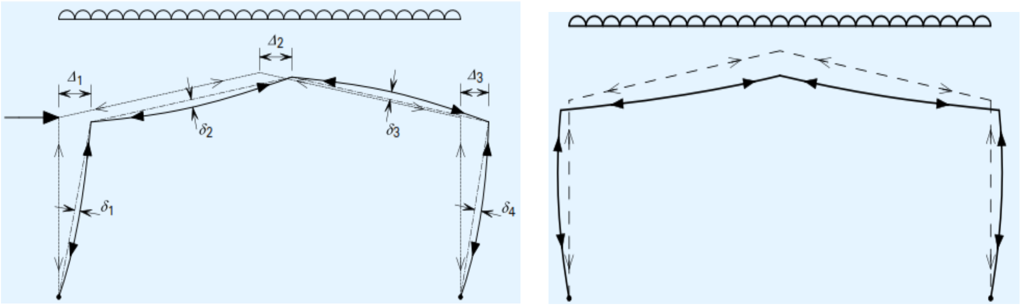
B.3.1 Elastic analysis

Frame analysis for ultimate limit state can be done using two methods; elastic analysis and plastic analysis. Plastic theory is a more economical approach.The main reason for this is redistribution of moments from highly stressed segments to underutilized segments. Portal frames designed to elastic theory are more conservative.

Elastic analysis was used for the parametric study since it is the common analysis method used in Sri Lanka.

B.3.2 First order and second order analysis

Portal frame deflects vertically and horizontally when it is loaded, as shown in figureB.2. The deflection results in additional moments as the axial force acts along a different axis than assumed in the analysis.



Asymmetric sway mode Symmetric sway mode

Figure B.2 Sway mode of portal frame [17]

First order analysis is sufficient if the deflections are small, where the effects of the axial force can be ignored. Second order effects should be considered when the deformed shape is large enough to cause significant additional moments and deflections due to the axial loads acting in the deformed shape. It is determined by evaluating theα_{cr} factor as discussed below.

Second order effects

There are 2 categories of second order effects.

1. P – δ effects - effects of deflections within the length of members
2. P - Δ effects – effects of displacements at the intersections of members

Stiffness of the frame and the individual members are reduced by the P – δ and P - Δ effects. Second order analysis results include the P – δ and P - Δ effects.

In-plane flexibility of frame is used to decide whether to conduct a first order or a second order analysis by calculating α_{cr} factor.[17]

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10$$

Where, Fcr– elastic critical buckling load for global instability mode

F_{Ed} – Design load on the structure

Second order effects can be ignored in a first order analysis if the frame is sufficiently stiff. Limitations to use the first order analysis are defined in BS EN 1993-1-1 as follows.

$\alpha_{cr} \geq 10$ - Elastic analysis shall be carried out

$\alpha_{cr} \geq 15$ - Plastic analysis shall be carried out

When the second order effects are significant, approximate second order analysis method was used. In this method, applied actions are amplified to cater the second order effects for first order calculations.

The α_{cr} is given by[7],

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right) X \left(\frac{h}{\delta_{H,Ed}}\right)$$

H_{Ed}	Algebraic sum of the base shear on the two columns due to the horizontal loads and the EHF
V_{Ed}	Total design vertical load on the frame; the algebraic sum of the two base reactions
$\delta_{H,Ed}$	Maximum horizontal deflection
h	column height

For portal frames, above expression is simplified to

$$\alpha_{cr} = \left(\frac{h}{200\,\delta_{NHF}}\right)$$

Provided that

- Roof slope < 26^{0}
- Axial force in the rafter is not significant.

NHF is taken as 1/200 of the design vertical base reaction and that load should be applied on top of either column in the same direction to obtain the deflection at the top of the column.

Axial compression of the rafter is significant if

$$\lambda' \geq 0.3\sqrt{\frac{Af_y}{N_{Ed}}}$$

N_{Ed} Design compression force in rafter

λ' In-plane non dimensional slenderness

Equivalent equation givenin [17]is as follows.

Axial force in the rafter is significant if,

Where

$$N_{Ed} > 0.09N_{cr}$$

$$N_{cr} = \frac{\pi^2 E I_y}{L_{cr}{}^2}$$

N cr elastic critical buckling load for the complete span of the rafter

L developed length of the rafter from column to column taken as span/cos(θ) (θ is the roof slope)

To calculate α_{cr},when the axial force in rafter is significant,

$$\alpha_{cr,\,est} = min(\alpha_{cr,\,s,\,est}\ ,\ \alpha_{cr,\,r,\,est})$$

Where,

$\alpha_{cr,s,est}$ Estimate of α_{cr} for the sway buckling mode

$\alpha_{cr,r,est}$ Estimate of α_{cr} for the rafter snap- through buckling mode. This mode only needs to be checked when there are three or more spans, or if the rafter is horizontal, or when the columns are not vertical

Calculation of $\alpha_{cr,s,est}$

$$\alpha_{cr,\,s,\,est} = 0.8\left\{1 - \left(\frac{N_{Ed}}{N_{cr,R}}\right)max\right\}\alpha_{cr}$$

Where,

$\left(\frac{N_{Ed}}{N_{cr,R}}\right)max$ Maximum ratio in any of the rafters

N_{Ed} Axial force in rafter at ULS

Modified first order analysis

The 'amplified moment method' was used for the parametric studyto allow for second order effects in a first order elastic analysis.

If the second order effects are significant, all horizontal actions are increased by an amplification factor to allow for the second order effects. The horizontal actions comprise the externally applied actions such as wind load and the equivalent horizontal forces used to allow for frame imperfections; both are amplified. NHF used to calculate α_{cr} are not amplified.

Provided $\alpha_{cr} \geq 3$,

Amplification factor is given by

- When axial load in the rafter is not significant

$$Amplification\ factor = \frac{1}{1-\frac{1}{\alpha_{cr}}}$$

- When axial load in the rafter is significant

$$Amplification\ factor = \frac{1}{1-\frac{1}{\alpha_{cr\ est}}}$$

If $\alpha_{cr} \leq 3$, second order analysis must be used.

B.3.3 Frame imperfection

Equivalent horizontal forces (EHF) were applied to the model to obtain frame imperfections.

The global initial sway imperfection can be modeled by

$$\varphi = \varphi_0\ \alpha_h\ \alpha_m$$

Where

φ_0 is the basic value: $\varphi_0 = 1 / 200$

$\alpha_h = \frac{2}{\sqrt{h}}\left(but\ \frac{2}{3} < \alpha_h < 1.0)\right)$

H is the height of the structure in meters

$$\alpha_m = \sqrt{0.5\,(1 + \frac{1}{m})}$$

m is the number of columns in a row – (for a portal frame, number of columns in a single frame)

EHF = φ x 'vertical reaction at the base of the column'

EHF is applied horizontally in the same direction at the top of each column. Sway imperfection can be disregarded when $H_{Ed} \geq 0.15\ V_{Ed}$.

B.4 Design of members

B.4.1 Cross section verification

Cross sections of column and rafter were classified as given in BS EN 1993-1-1:2005. Eurocode 3 classifies cross sections based onitsdimensions of the flange and web and axial force acting on the section. The structural behaviors of the classes are defined as follows.

Class 1	support a rotating plastic hinge without reducing its resistance from local buckling
Class 2	develops full plastic moment with limited rotation capacity before local buckling reduces resistance
Class 3	develops yield in extreme fibers but local buckling prevents development of plastic moment
Class 4	local buckling will take place at stresses below first yield

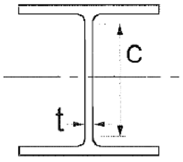
According to the BS EN 1993-1-1:2005, Table 5.2,

- Web classification

$$\text{when } \alpha > 0{,}5:\ c/t \leq \frac{396\varepsilon}{13\alpha - 1}$$

Whe

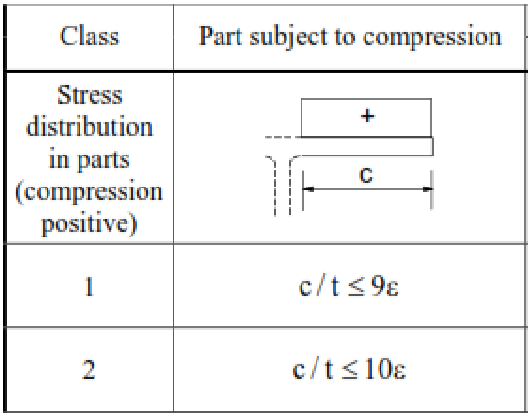
$$\text{when } \alpha \leq 0{,}5:\ c/t \leq \frac{36\varepsilon}{\alpha}$$



$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

$$\alpha = \frac{1}{2}\left(1 + \frac{N_{Ed}}{f_y t_w c}\right) \qquad [18]$$

- Flange classification



Class	Part subject to compression
Stress distribution in parts (compression positive)	
1	c / t ≤ 9ε
2	c / t ≤ 10ε

Figure B.3 Flange classification [5]

The section is considered to be class 1 if both the flange and the web are class 1 separately.

B.4.2 Resistance of the cross section

B.4.2.1 Shear resistance

BS EN 1993-1-1Cl.6.2.6 states that the design plastic resistance is given by the following equation when torsion is absent.

\

$$V_{pl,Rd} = A_v \frac{(f_y/\sqrt{3})}{\gamma_{MO}}$$

When the load is applied parallel to the web of a hot rolled 'I' section,

$Av = A - 2bt_f + (t_w + 2r)t_f$ but not less than $\eta h_w t_w$

$\gamma_{MO} = 1.0$

Shear resistance of a section is reduced in BS EN 1993-1-1:2005by a factor of ($\frac{1}{\sqrt{3}}$=0.577) where as in BS5950-1:2000, the factor is 0.6. Av given in BS5950-1:2000 is equal to tD, which is less than the Av given in Eurocode 3.

B.4.2.2 Bending and shear interaction

According to BS EN 1993-1-1 cl 6.2.8 when shear force is less than half the plastic shear resistance, its effect on the moment resistance may be neglected except where shear buckling reduces the section resistance.

Otherwise design resistance of the cross section should be calculated using the reduced yield strength.

$$(1-\rho)f_y \quad ; \quad \rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

B.4.2.3 Compression resistance

BS EN 1993-1-1 Cl 6.2.4 gives that the design value of compression resistance of the cross section should satisfy

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0$$

Where, $N_{c\,Rd} = \frac{A f_y}{\gamma_{MO}}$ for class 1, 2 or 3 members

Compression resistance of member is reduced by γ_{MO} factor in Eurocode. But in BS5950-1:2000, compressive strength is reduced depending on slenderness ratio of member.

B.4.2.4 Combined bending and axial force

It is not necessary to provide allowance for the effect of the axial force on the plastic resistance moment about the y-y axis when both the following criteria are satisfied as per the BS EN 1993-1-1 Cl 6.2.9.

$$N_{Ed} \leq 0.25\, N_{pl,\,Rd} \qquad \text{and} \qquad N_{Ed} \leq \frac{0.5 h_w t_w f_y}{\gamma_{MO}}$$

B.4.2.5 Bending resistance

BS En 1993-1-1 Cl6.25 states that bending moment of cross section should satisfy

$$\left(\frac{M_{Ed}}{M_{c,Rd}}\right) \leq 1$$

For class 1 and 2 sections,

$$M_{c,Rd} = M_{pl,y,Rd} = \frac{W_{pl} f_y}{\gamma_{MO}}$$

B.4.2.6Buckling verification

The rafters and columns should be verified for buckling between restraints.

According to BS EN 1993-1-1 Cl6.3.3 (4), the members should satisfy both in-plane and out of plane buckling resistance, unless full second order analysis including member imperfections is carried out. The equations are simplified in CTICM & SCI (2008) as follows.

$$\frac{N_{Ed}}{N_{b,y,Rd}} + \frac{k_{yy} M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + \frac{k_{zy} M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

Where,

$N_{b, y, Rd}$	flexural buckling resistance in the major axis
$N_{b, z, Rd}$	flexural buckling resistance in the minor axis
$M_{b, Rd}$	lateral torsional buckling resistance

Values of k_{yy} and k_{zy}can be obtained from annex B of BS EN 1993-1-1.

A non-uniform moment is less critical when calculating the lateral torsional buckling resistance of a member. Annex B of BS EN 1993-1-1 gives the moment gradient, $C_{mi,o}$and C_{mLT}.

B.5Rafter design

The resistance of all critical cross sections was verified as discussed above. (Tension, compression, bending, shear, Bending and shear interaction, bending and axial force interaction, etc)

A typical moment distribution of a rafter is shown in the figureB.4.

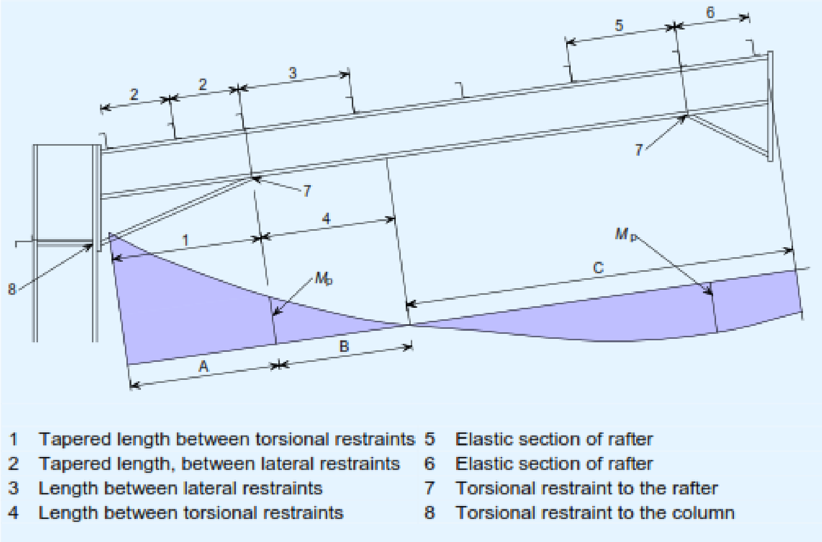


Figure B.4 Typical bending moment of rafter [17]

Both in-plane and out of plane checks are required. Purlins are placed at about 1.2m spacing. The rafter is categorized in to three stability zones as zone A, zone B & zone C as shown in the figureB.4.

Zone A

This includes the haunch length along the rafter. The bottom flange of the haunch is in compression. The stability checks are complicated as the geometry of the section varies along the haunch. Restraints of the haunch region are shown in the figure B.5. Underside of the haunch position at the column face should always be restrained.

Zone A checks were carried out for 5 different cross sections by dividing the zone in to quarter points.

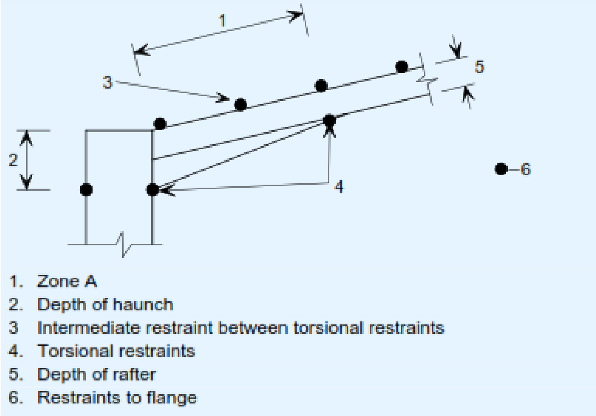


Figure B.5 Restraints of haunch region [17]

Zone B

Zone B consists of the haunch end point at the rafter to the point of contraflexure as shown in the figureB.4. Bottom flange is in compression in zone B.

Zone C

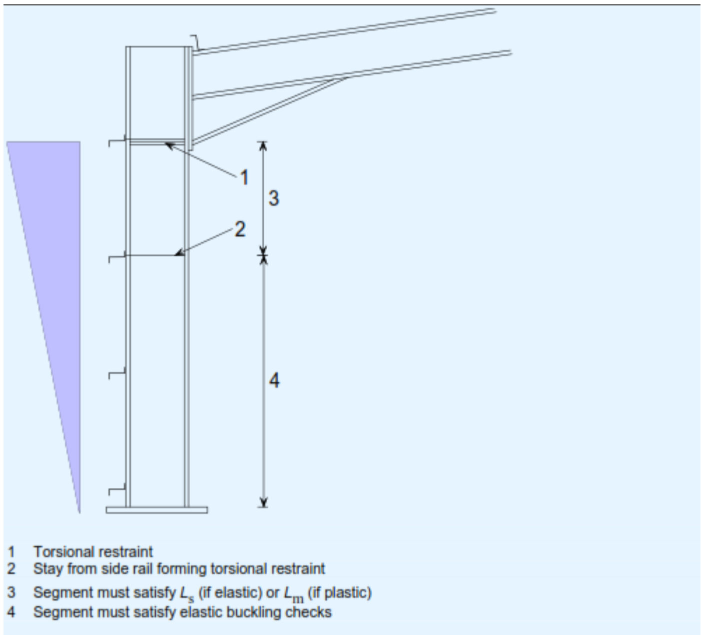
Top flange is in compression in zone C. purlins in regular spacing provides the lateral restraints over this lengthy segment. Hence out-of-plane checks over the rafter between restraints were carried out assuming the maximum bending moment and maximum axial force as stated in [17].

B.6Column Design

The column is subjected to a large bending moment similar to the haunch end. The haunch end moment is resisted by the combination of rafter and the haunch. Hence the column needs to be a larger section than the rafter. The optimum size of the column is generally 1.5 times larger than the rafter size and its plastic section modulus.

Column section should be a class1 or class2 member under the ultimate forces. Full depth web stiffeners are provided at the plastic hinge locations if necessary.

The resistance of all critical cross sections was verified accordingly. (tension, compression, bending, shear, bending and shear interaction, bending and axial force interaction, etc)



1 Torsional restraint
2 Stay from side rail forming torsional restraint
3 Segment must satisfy L_s (if elastic) or L_m (if plastic)
4 Segment must satisfy elastic buckling checks

Figure B.6 Typical Bending moment diagram for a column [17]

Side rails provide the lateral restraint to the tension flange of the column. Hence the compression flange may require torsional restraints. Torsional restraints were provided under the haunch at the column- haunch connection.

First the columns were checked for the minor axis flexural buckling and lateraltorsional buckling between restraints. Then the tension flange restrains were checked to utilize for buckling resistance. The column stability against the major axis was checked at the end for flexural buckling.

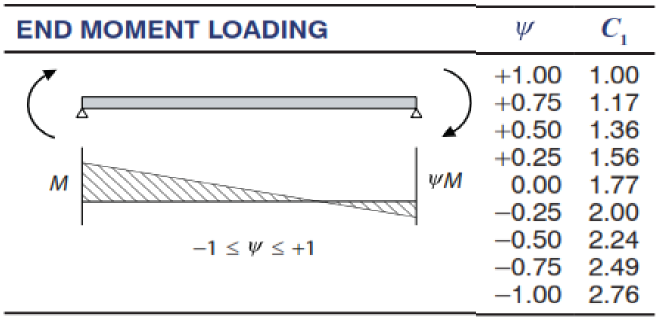
Spacing of restraints to the tension flange

It is assumed that the restraints to the tension flange are effective in increasing the resistance to lateral torsional buckling if their spacing does not exceed L_m

$$L_m = \frac{38\, i_z}{\sqrt{\frac{1}{57.4}\left(\frac{N_{Ed}}{A}\right) + \frac{1}{756\, C_I^{\,2}} \frac{W_{pl,y}^{\,2}}{A\, I'_T} \left(\frac{f_y}{235}\right)^2}}$$

N_{Ed}	design value of the compression force (N) in member
A	cross section area of the member
$W_{pl, y}$	plastic section modulus of the member
I_T	torsional constant of the member
f_y	yield strength (N/mm^2)
C_1	factor depending on the loading and the end conditions

C1 is obtained from figure B.7.



END MOMENT LOADING	ψ	C_1
	+1.00	1.00
	+0.75	1.17
	+0.50	1.36
	+0.25	1.56
	0.00	1.77
	−0.25	2.00
	−0.50	2.24
	−0.75	2.49
	−1.00	2.76

Figure B.7 Table to obtain ‘C1’ factor[7]

First the column is checked with the restraint at the underside of the haunch and the base, assuming no intermediate restraints. If the flexural buckling, lateral torsional buckling and interaction checks are satisfied for this length, no intermediate restraints are required. Otherwise, intermediate torsional restraints need to be introduced to the column or the column size increased.

Appendix C:

Specimen design calculations of a portal frame

Elastic Analysis of Single Bay Portal Frame

1.0 Basic design information

Location = Zone 3
Roofing sheets = Zn/Al sheets
Base type = Pinned base

2.0 Frame geometry

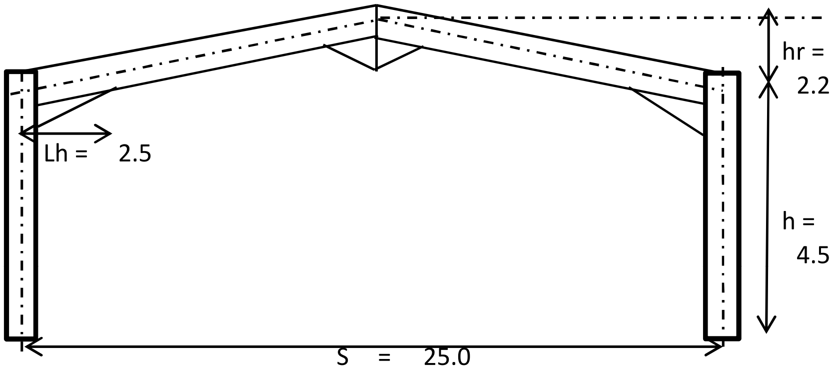


Figure C 1

Span of the building	(S)	=	25.0	m
Height from base to eaves	(h)	=	4.5	m
Height from eaves to apex	(hr)	=	2.2	m
Haunch length (10% of span)	(L_h)	=	2.5	m
Spacing of frames		=	6.0	m
Purling spacing		=	1.3	m
Roof angle		=	10	°
Standard builing length		=	90	m

Rafter	356x127x39	UB section
Column	356x171x51	UB section

Reference	Calculations	Output

3.0 Actions - Roof

The cladding to the roof and walls is supported by purlins and side rails.

3.1 Permanent actions

Dead load consists of

1. $g_{\text{self weight}}$ - Self weight of the rafter
   (self weight of the rafter is increased by 5% to include the weight of the haunches and connections
2. g_{roof} - Roofing sheets with purlins
3. g_{other} - Services (lighting & Ac ducts)

g_{roof} = 0.30 kN/m^2

g_{other} = 0.05 kN/m^2

$g_k = g_{\text{self Weight}} + g_{\text{roof}} + G_{\text{other}}$

For an internal frame

$g_{\text{self weight}}$ = $39.1 \times 9.81 \times 10^{-3} \times 1.05$kN/m
= 0.40 kN/m

g_{roof} = 0.3kN/m2 x6m
= 1.8 kN/m

g_{other} = 0.05kN/m2 x6m
= 0.3 kN/m

g_k = 0.40 + 1.8 + 0.3 kN/m
= 2.50 kN/m

Output: g_k = 2.5 kN/m

g_k = 2.50 kN/m

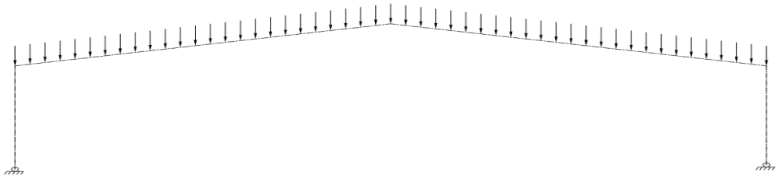


Figure C 2

3.2 Imposed load on roof

Reference: EN 1991- 1-1 Table 6.10

The characteristic value for imposed load on roof,
for Type H roofing with no access

q_k = 0.4 kN/m^2

For an internal frame

q_k = 0.4kN/m2 x6m
= 2.40 kN/m

Output: q_k = 2.40 kN/m

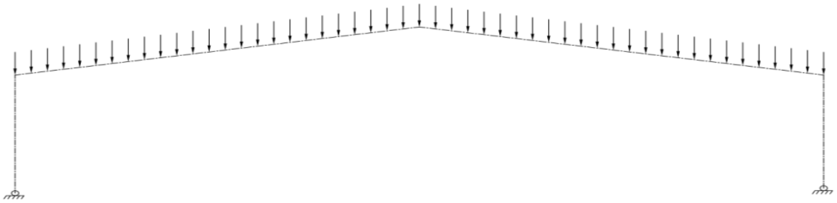


Figure C 3

Reference	Calculations	Output

3.3 Wind load calculation

Wind actions were calculated based on ,

ref 1 BS EN 1991-1-4 [13]
ref 2 Draft National Annex [14]
ref 3 Lewangamage et al. [15]

Following assumptions were made for the wind calculation

1 Site altitude above the mean sea level is 40 m
2 The distance upwind to shore line is 5 km
3 Terrain category is taken as country terrain.
4 Average slope of upwind terrain is less than 3^0
5 Structures are situated in zone 3

[15]

3.3.1 Basic wind velocity

V_b = $V_{b,o} . C_{dir}. C_{season}$
$V_{b,o}$ = $V_{b,zone}$ x C_{alt}

V_b = basic wind speed
$V_{b,zone}$ = fundamental value of basic wind speed before altitude correction is applied(10 min mean speed)
C_{dir} = directional factor1
C_{season} = season factor

[15]
Table 1

$V_{b,zone}$ for 50 years return period,

$V_{b,zone}$ = 22.0 m/s
$V_{b,o}$ = 22.0 x 1.04 = 22.88 m/s

Recommended values
C_{dir} = 1
C_{season} = 1

V_b = 1 x 1 x 22.88m/s = 22.9 m/s

3.3.2 Peak velocity pressure calculation

[12]
Table 3

q_p = $C_{e(z)} . q_b$
q_b = 0.613 V_b^2

[12]
Figure 7

$C_{e(z)}$ = 2.05

q_b = 0.613 x 23^2
= 320.9 N/mm^2

q_p = $C_{e(z)} . q_b$
= 2.05 x 320.9020672 / 1000 kN/m^2
= 0.66 kN/m^2

Reference | Calculations | Output

EN 1991-1-4 :2005

3.3.3 Wind forces

5.3 (2)

$$F_w = C_s\ C_{d.}\ C_{f.}\ q_{p\,(ze)}\,.\,A_{ref}$$

$C_s\ C_d$ = structural factor

C_f = force coefficient for the structure

$q_{p(ze)}$ = peak velocity pressure at reference height Z_e

A_{ref} = reference area of the structure

$C_f = C_{pe,10} - C_{pi}$

6.2 (1) a)

For buildings with a height less than 15m,

$C_s\ C_d = 1$

Frame spacing = 6 m

For a middle frame,

$$F_w = 1 \times Cf \times 0.66 \times 6$$

$$= 3.95 \times C_f \quad \text{kN/m}$$

Pressure coefficients

For walls

Cl 7.2.9 (6) Note 2

Internal pressure coefficients

C_{pi} should be taken as the more onerous of

$C_{pi} = 0.2$ or

$C_{pi} = -0.3$

External pressure coefficients

Table 7.1

Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings

d = 25 m

b = 90 m

h = 4.5 m

e = b or 2h, whichever is smaller

= 9 m

h/d = 0.18

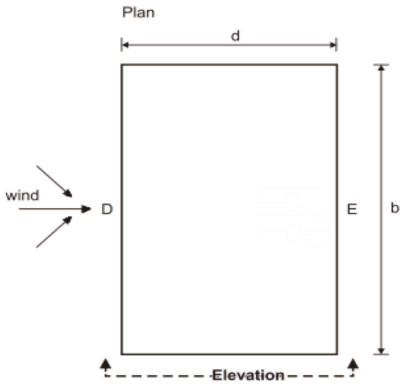


Table 7.1

Table C 1

	$C_{pe,10}$	
h/d	D	E
1	0.8	-0.5
0.2	0.7	-0.3
0.25	0.7	-0.3

	D		E	
$C_{pe,10}$	0.7		-0.3	
C_{pi}	0.2	-0.3	0.2	-0.3
C_f	0.5	1.0	-0.5	0.0
F_w	2.0	4.0	-2.0	0.0

Table C 2 - wind forces

EN 1991-1-4
:2005

For roof

Figure 7.8(b)

Roof angle = 10 ^{0}

Θ = 0 ^{0}

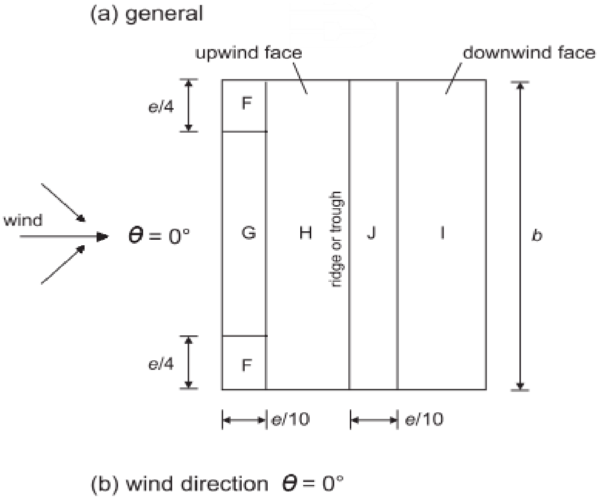


Figure C 4 - Key for duopitch roof - figure 7.8(a) -EN 1991-1-4:2005 [13]

Table 7.4a

	$C_{pe,10}$ (negative)			
pitch angle	G	H	I	J
5	-1.2	-0.6	-0.6	0.2
10	-1.00	-0.45	-0.50	-0.40
15	-0.8	-0.3	-0.4	-1.0

Table C 3 $C_{pe,10}$ (Negative)

	$C_{pe,10}$ (positive)			
pitch angle	G	H	I	J
5	0.0	0	-0.6	-0.6
10	0.10	0.10	-0.30	-0.30
15	0.2	0.2	0.0	0.0

Table C 4 $C_{pe,10}$ (positive)

	$C_{pe,10}$ (negative) - (C_{pi} = +0.2)				C pe ,10 (negative) - (Cpi = -0.3)			
	G	H	I	J	G	H	I	J
C_{pe}	-1.0	-0.45	-0.50	-0.40	-1.00	-0.45	-0.50	-0.4
C_{pi}	0.2	0.2	0.2	0.2	-0.3	-0.3	-0.3	-0.3
C_f	-1.20	-0.65	-0.70	-0.60	-0.70	-0.15	-0.20	-0.10
F_w	-4.74	-2.57	-2.77	-2.37	-2.77	-0.59	-0.79	-0.40

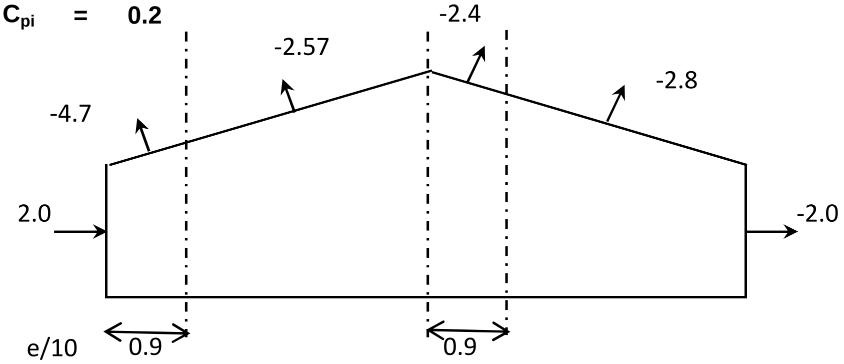
Table C 5 Forces on roof - $C_{pe,10}$ (negative)

	$C_{pe,10}$ (positive)- (C_{pi} = +0.2)				$C_{pe,10}$ (positive) - (Cpi = -0.3)			
	G	H	I	J	G	H	I	J
C_{pe}	0.1	0.10	-0.30	-0.30	0.10	0.1	-0.3	-0.3
C_{pi}	0.2	0.2	0.2	0.2	-0.3	-0.3	-0.3	-0.3
C_f	-0.10	-0.10	-0.50	-0.50	0.40	0.40	0.00	0.00
F_w	-0.40	-0.40	-1.98	-1.98	1.58	1.58	0.00	0.00

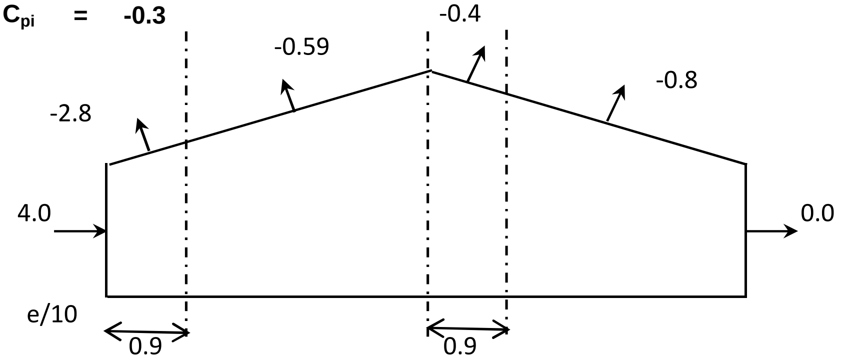
Table C 6 Forces on roof - $C_{pe,10}$ (positive)

For negative pressure

C_{pi} = 0.2

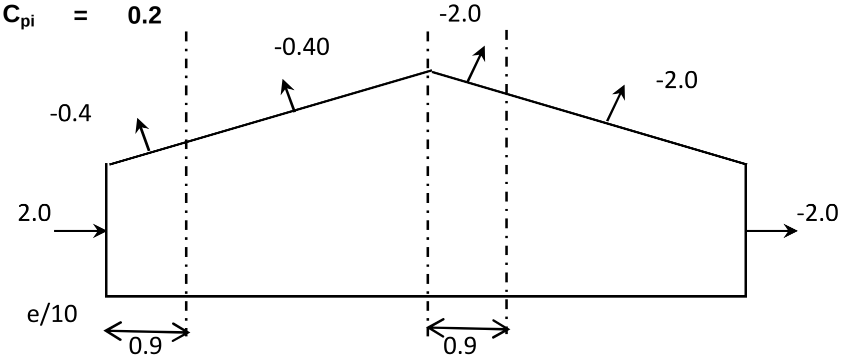


C_{pi} = -0.3



For positive pressure

C_{pi} = 0.2



C_{pi} = -0.3

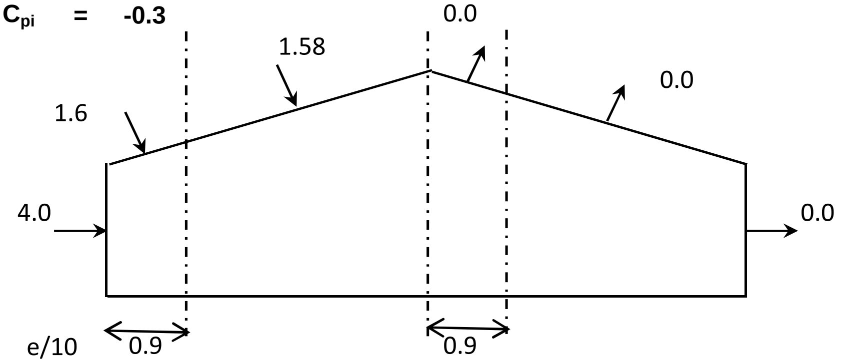


Figure C 5 - forces acting on an individual frame

3.4 Combinations of actions

EN 1991 -1-1
Cl 3.3.2 (1)

Imposed actions on a roof are not considered with the wind load.

Hence critical combinations of actions for preliminary sizing are,

1. Permanent actions with imposed roof load
2. Permanent actions with wind action

For preliminary sizing, it is assumed that the combination considering the wind action is not critical.
Hence the following combination was considered for preliminary sizing.

$\gamma_G\, g + \gamma_Q\, q$

γ_G = 1.35

γ_Q = 1.5

Ultimate Design Load = 6.98 kN/m

4.0 Prelimiary sizing

- Initial member sizing was done referreing the table given in Appendix E of the "SCI P 399" Publication. [11]
- Frame sensitivity for the second order effects shall be checked prior to analysis
- Selected sizes will be reviewed against the design effects.

Following sections were selected and reviewed.

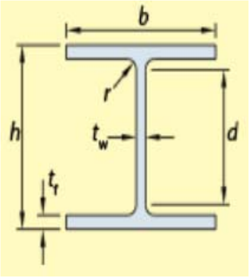
Steel grade - S 355

Rafter 356x127x39

Column 356x171x51

4.1 Section properties

Column			Rafter		
356x171x51			356x127x39		
Mass	51	kg/m	*Mass*	39.1	kg/m
h	355	mm	h	353	mm
b	172	mm	b	126	mm
t_w	7.4	mm	t_w	6.6	mm
t_f	11.5	mm	t_f	10.7	mm
r	10.2	mm	r	10.2	mm
h_w	332	mm	h_w	332	mm
d	311.6	mm	d	312	mm
A	6.49E+03	mm^2	A	5.0E+03	mm^2
$W_{el,y}$	7.96E+05	mm^3	$W_{el,y}$	5.8E+05	mm^3
$W_{pl,y}$	8.96.E+05	mm^3	$W_{pl,y}$	6.6E+05	mm^3
I_y	1.41E+08	mm^4	I_y	1.02E+08	mm^4
I_z	9.68E+06	mm^4	I_z	3.58E+06	mm^4
I_T	2.380E+05	mm^4	I_T	1.51E+05	mm^4
I_w	2.86E+11	mm^6	I_w	1.05E+11	mm^6
i_y	148.0	mm	i_y	143.0	mm
i_z	38.6	mm	i_z	26.8	mm



BS EN 10025-2

Because T < 40 mm

Design strength = 355 N/mm^2

E = 2.10E+05 N/mm^2

Because T < 40 mm

= 355 N/mm^2

G = 81000 N/mm^2

5.0 Initial Analysis

Initial analysis was carried out using the above selected sections to determine the sensitivity of the frame to second order effects and to determine the necessity for an allowance for frame imperfections.

Combinations of actions are shown in the figure.

Combination	Permanent	variable	
	Partial factor	Action	Partial factor
	γ_G		γ_Q
C1 - Permanent & imposed	1.35	Imposed	1.5
C2 - Permanent & wind	1.35	Wind	1.5
C3 - Permanent & wind	1	Wind	1.5

Table C 7 -Combination of actions

Nodes

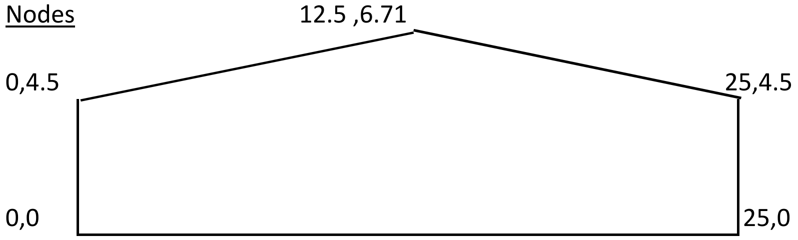


Figure C 6 - Nodes

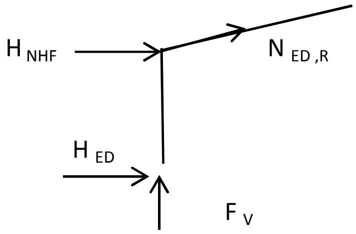
Initial Analysis Results

Combination	Base reactions (kN)				Axial force in rafter
	Left Column		Right Column		
	F_V	F_H	F_V	F_H	$N_{ED,R}$
C1	87	70	87	-70	83
C2	62	24	51	-51	60
C3	51	15	41	-42	49

Table C 8 -Initial analysis results

Combination	H_{NHF} (kN) Left	H_{NHF} (kN) Right	δ_{NHF} (mm)
C1	0.435	0.44	1.41
C2	0.31	0.255	0.91
C3	0.255	0.21	0.74

Table C 9 - Notional horizontal forces and deflections



$$H_{NHF} = 1/200\, F_v$$

Initial analysis was carried out using computer software and the results obtained are shown in the Table A8 and table A9 .

6.0 Sensitivity to second order effects

BS EN 1993 -1-1 Cl 5.2.1

Sensitivity of the frame to the second order effects is evaluated by calculating α_{cr} factor.

If

α_{cr} > 10

second order effects are small enough to be ignored.

BS EN 1993 -1-1 Cl 5.2.1 (4)B eq 5.2

When the roof slope is less than 26 ^{0} and the axial force in the rafter is not significant, α_{cr} can be calculated using the following equation

$$\alpha_{cr}=((H_{Ed)}/(V_{Ed)})X(h/(\delta_{H,Ed)})$$

H_{Ed} = algebraic sum of the base shear on the two columns - due to the horizontal loads and the EHF

V_{Ed} = total design vertical load on the frame - the algebraic sum of the two base reactions

$\delta_{H,Ed}$ = maximum horizontal deflection at the top of either column, relative to the base, when the frame is loaded with horizontal loads

h = column height

Reference | Calculations | Output

SCI P397
6.5

For portal frames, the above expression can be simplified as

$$\alpha_{cr}=(h/(200\ \delta_{NHF)}\ )$$

h = height to the eave

δ_{NHF} = lateral deflection at the top of the column due to the NHF

For combination 1,

$$\alpha_{cr} = \frac{4.5\text{x } 1000}{200 \text{ x } 1.41}$$

= 16.0

Combination	α_{cr}
C1	16.0
C2	24.7
C3	30.4

Table A 10 - α_{cr} factor

<u>Axial compression in rafter</u>

BS EN 1993
-1-1
Cl 5.2.1 (4)B
Note 2B

Axial compression is significant if

$$\lambda'\geq 0.3\sqrt{(Af_{y/}N_{Ed)}}$$

N_{Ed} = Design compression force in rafter

λ' = in-plane non dimensional slenderness calculated for the beam or rafters considered as hinged at its end of the system length measured along the beams of rafters

This can be rearranged to show that the compression is significant if,

$$N_{Ed}\ >0.09\ N_{cr}$$

where,

$$N_{cr}=(\pi\text{^}(2\ )\ E\ I_{\ y})/〖L_{cr}$$

L_{cr} = developed length of the rafter pair between columns

I_y = in-plane second moment of area of rafter

L_{cr} = 25 / Cos(10)

= 25.39 m

$$N_{cr}=(\pi\text{^}(2\ )\ E\ I_{\ y})/〖L_{cr}$$

$$N_{cr} = \frac{\pi^2 \text{ x } 2.10E+05 \text{ x } 1.02E+08}{25385.67^2 \text{ x } 1000}$$

N_{cr} = 328.1 kN

0.09 N_{cr} = 29.5 kN

Combination	$N_{Ed,R}$	Axial compression in rafter
C1	83.0	$N_{Ed,R}$ > 0.09 N_{cr} Axial compression in rafter is significant BS EN 1993-1-1,Cl 5.2.1 (4)B is not applicable
C2	60.0	$N_{Ed,R}$ > 0.09 N_{cr} Axial compression in rafter is significant BS EN 1993-1-1,Cl 5.2.1 (4)B is not applicable
C3	49.0	$N_{Ed,R}$ > 0.09 N_{cr} Axial compression in rafter is significant BS EN 1993-1-1,Cl 5.2.1 (4)B is not applicable

Table C 11- Significance of axial compression in rafter

Calculation of α_{cr} when axial compression in rafter is significant

A conservative measure of frame stability defined as $\alpha_{cr,\ est}$ shall be calculated,

SCI P397 6.6

$$\alpha_{cr,\ est} = min(\alpha_{cr,\ s,\ est}\ ,\alpha_{cr,\ r,est})$$

Where,

$\alpha_{cr,s,est}$ = estimate of α_{cr} for the sway buckling mode

$\alpha_{cr,r,est}$ = estimate of α_{cr} for the rafter snap- through buckling mode. This mode only needs to be checked when there are three or more spans, or if the rafter is horizontal, or when the columns are not vertical.

Calculation of $\alpha_{cr,s,est}$

6.6.1

$$\alpha_{cr,\ s,est} = 0.8\{1-((N_{\ Ed)/(}N_{\ cr,R))max}\}\alpha_{cr}$$

Where,

$((N_{\ Ed)/(}N_{\ cr,\ R))max}$ = maximum ratio in any of the rafters

N_{Ed} = axial force in rafter at ULS

N_{cr} =(π^(2) $E\ I_{\ y}$

Combination 1

$$\alpha_{cr,s,est} = 0.8 \quad x \quad \left\{ 1 - \left(\frac{83}{328.1} \right) \right\} \quad x \quad 16.0$$

$$= 9.54$$

Calculation of $\alpha_{cr,r,est}$

[7] 6.6

This calculation should be carried out if the frame has three or more spans, or if the rafter is horizontal or when the columns are not vertical.

Since single bay protal frames are considered, this calculation is avoided.
Hence,

$$\alpha_{cr,\ est} = min(\alpha_{cr,\ s,\ est}\ ,\alpha_{cr,\ r,est})$$

$$= \alpha_{cr,s,est}$$

$$= 9.54$$

Combination	α_{cr} / $\alpha_{cr,est}$		Second order effects
C1	α cr,est	$N_{Ed,R}$ > 0.09 N_{cr} α cr,est = 9.54 < 10	second order effects cannot be ignored
C2	α cr,est	$N_{Ed,R}$ > 0.09 N_{cr} α cr,est = 16.2 > 10	second order effects can be ignored
C3	α cr,est	$N_{Ed,R}$ > 0.09 N_{cr} α cr,est = 20.7 > 10	second order effects can be ignored.

Table C 12 - second order effects

<u>Modified first order analysis</u>

BS EN 1993
-1-1
Cl 5.2.2(5B)

The 'amplified moment method' is the simplest method to allow for second order effects in a first order elastic analysis.

If second order effects are significant, all horizontal actions (externally applied actions such as wind load and the equivalent horizontal forces used to allow for frame imperfections) are increased by an amplification factor to allow for the second order effects. NHF used to calculate α_{cr} are not amplified.

Provided $\alpha_{cr} \geq 3$ amplification factor is given by

When axial load in the rafter is not significant

$$Amplification\ factor=1/(1-1/\alpha_{cr})$$

When axial load in the rafter is significant

$$Amplification\ factor=1/(1-1/(\alpha_{cr,est}))$$

If $\alpha_{cr} \leq 3$, second order analysis must be carried out.

Combination	α_{cr} / $\alpha_{cr,est}$		Amp. Factor
C1	α cr,est	$N_{Ed,R}$ > 0.09 N_{cr} α cr,est = 9.54	1.12
C2	α cr,est	$N_{Ed,R}$ > 0.09 N_{cr} α cr,est = 16.2	does not apply
C3	α cr,est	$N_{Ed,R}$ > 0.09 N_{cr} α cr,est = 20.7	does not apply

Table C 13 - Amplification factors

7.0 Frame imperfections

BS EN 1993-1-1 cl 5.3.2 eq 5.5

The global initial sway imperfection may be determined from:

$\Phi = \Phi_0 \alpha_h \alpha_m$

$\Phi_0 = 1/200$

h = 4.5 m (height to eaves)

m = 2 (number of columns)

$\alpha_h = 2/\sqrt{h}$ ($2/3 < \alpha_h < 1.0$)

= 0.94

$2/3 < 0.94 < 1.0$ Hence, $\alpha_h = 0.94$

$\alpha_m = \sqrt{(0.5\ (1+1/m))}$

= 0.87

$\Phi = 1/200 \times 0.94 \times 0.87$

= 0.0041

Initial sway imperfections are included in the analysis by applying equivalent horizontal forces(EHF).

Sway imperfections can be ignored when,

$H_{Ed} \geq 0.15\ V_{Ed}$

Com	Column reactions (kN)				Total reactions (kN)		0.15 V_{Ed} (kN)
	Left Column		Right Column				
	F_V	F_H	F_V	F_H	H_{Ed}	V_{Ed}	
C1	87	70	87	-70	0	174	26.10
C2	62	24	51	-51	-27	113	16.95
C3	51	15	41	-42	-27	92	13.80

Table C 14 - Sway imperfection calculation

Com	$\lvert H_{ed} \rvert$	$0.15V_{Ed}$ (kN)	Sway imperfections
C1	0	26.10	Sway imperfections should be considered.
C2	27	16.95	sway imperfections can be disregarded
C3	27	13.80	sway imperfections can be disregarded

Table C 15 - Significanse of sway imperfections

The equivalent horizontal forces are taken as a propotion of the design base vertical reactions. Amplification factor appies or EHF loads.

For combination 1

$H_{EHF} = \Phi \; V_{Ed}$

$= 0.20$ kN

Com	V_{Ed}	$\Phi V_{Ed} = H_{EHF}$
C1	194.4	0.40
C2	113	ignore
C3	92	ignore

Table A 16 - Equivalent horizontal forces

EHF is applied horizontally at the top of each column, in the same direction, in combination with the permanent and variable actions.

8.0 Analysis results

The final analysis has been carried out using the amplified moment method and equivalent horizontal forces

Software Analysis results

	Column			Rafter			
Com	$N_{Ed, C}$ (kN)	F_H (kN)	M column (kNm)	$N_{Ed, R}$ (kN)	F_v (kN)	M (end of haunch) (kNm)	Amplification factor
C1	87	70	313	83	74	150	1.12
C2	62	24	169	60	52	57	does not apply
C3	51	15	130	50	42	38	does not apply

Table C 17 - Analysis results

9.0 Cross section verification

Section classification

Section classification for combination 1 is shown below.

$\gamma_{MO} = 1$

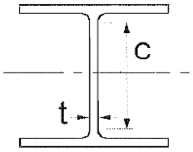
BS EN 1993-1-1
Section 5.5

9.1.1 Column classification

Web classification

BS EN 1993-1-1
Table 5.2

$$\frac{C}{t_w} = \frac{311.6}{7.4} = 42.1$$



$\varepsilon = \sqrt(235/(f_{y}))$

$= 0.81$

$\alpha = 1/2(\ 1+(N_{Ed)/(}f$

$$= \frac{1}{2}\left(1 + \frac{87 \quad x \quad 1000}{355 \quad x \quad 7.4 \quad x \quad 311.6}\right)$$

$= 0.55 > 0.50$

when $\alpha > 0,5 : \ c/t \leq \frac{396\varepsilon}{13\alpha - 1}$ C/t ≤ 52.04

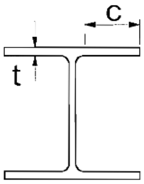
when $\alpha \leq 0,5 : \ c/t \leq \frac{36\varepsilon}{\alpha}$ C/t ≤ 52.95

the limit for Class 1 is 52.04

$42.1 < 52.04$

Therefore the web is class 1

Flange classification



C = 71.85 mm

$$C/T = \frac{71.85}{11.5} = 6.25$$

The limit for class 1 is $9\varepsilon = 9 \ x \ 0.81 = 7.32$

$6.25 < 7.32$

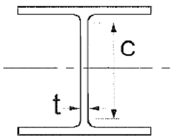
Therefore the flange is class 1

Section Classification

Because both the web and the flanges are class 1, the column section is class one.

9.1.2 Rafter classification

Web classification



$$\frac{C}{t_w} = \frac{311.6}{6.6} = 47.2$$

$\varepsilon = \sqrt{(235/(f_{y))}}$

= 0.81

$\alpha = 1/2(\ 1+(N_{Ed)/(}f$

$$= \frac{1}{2}\left(1 + \frac{83 \times 1000}{355 \times 6.6 \times 311.6}\right)$$

= 0.56 > 0.5

$$\text{when } \alpha > 0{,}5: \ c/t \le \frac{396\varepsilon}{13\alpha - 1} \qquad C/t \le 51.64$$

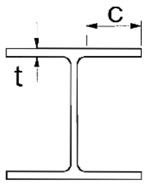
$$\text{when } \alpha \le 0{,}5: \ c/t \le \frac{36\varepsilon}{\alpha} \qquad C/t \le 52.60$$

the limit for Class 1 is 51.64

47.2 < 51.64

Therefore the web is class 1

Flange classification



C = 49.50 mm

$$C/T = \frac{49.50}{10.7} = 4.63$$

The limit for class 1 is 9 ε = 9 x 0.81 = 7.32

4.63 < 7.32

Therefore the flange is class 1

Section classification

Because both the web and the flanges are class 1, the rafter section is class one.

9.2 Resistance of the cross section

9.2.1 Column

9.2.1.1 Shear resistance

Reference	Calculations	Output
BS EN 1993-1-1 Cl 6.2.6(3)(a)	Shear area A_v = A - 2b t_f + (t_w +2 r) t_f but not less than $\eta h_w t_w$ A_v = 6490 -2 x 172 x 11.5 + (7.4 + 2 x 10.2) x 11.5 A_v = 2865.2 mm^2 conservatively η = 1.0 $\eta h_w t_w$ = 1.0 x 332 x 7.4 = 2456.8 A_v = 2865.2 mm^2	
BS EN 1993-1-1 Cl 6.2.6(2) eq 6.18	$V_{pl,Rd}$= A v $((f_y/\sqrt{(3)}))/(\gamma_{MO})$ = $\frac{2865.2 \text{ x } (355 / \sqrt{3})}{1 \text{ x } 1000}$ = 587.25 kN V_{Ed} = 70 kN 70 kN < 587.25 kN section is adequate for shear	
BS EN 1993-1-1 Cl 6.2.8(2)	9.2.1.2 Bending and shear interaction When shear force is less than half the plastic shear resistance its effect on the moment resistance may be neglected except where shear bucking reduces the section resistance. 0.5 $V_{pl, Rd}$ = 0.5 x 587.25 = 293.62 kN 70 kN < 293.62 kN Therefore the effect of the shear force on the moment resistance may be neglected.	
BS EN 1993-1-1 Cl 6.2.4(2) eq 6.10 & eq 6.9	9.2.1.3Compression resistance $N_{cRd} = (A f_{y})/(\gamma_{MO})$ = $\frac{6490 \text{ x } 355}{1 \text{ x } 1000}$ = 2303.95 kN N_{Ed} = 87 kN 87 kN < 2303.95 kN compression resistance is adequate	
BS EN 1993-1-1 Cl 6.2.9(4)	9.2.1.4 Combined bending and axial force It is not necessary to provide allowance for the effect of axial force on plastic resistance moment about y-y axis when both the following criteria are satisfied. $N_{Ed} \leq 0.25 N_{pl,Rd}$ and $N_{Ed} \leq (0.5 h_w t_w f_y)/(\gamma_{MO})$ 0.25 $N_{pl,Rd}$ = 0.25 x 2304.0 kN = 575.99 kN	

Reference	Calculations	Output
	$\frac{0.5 h_w t_w f_y}{\gamma_{MO}} = \frac{0.5 \times 332 \times 7.4 \times 355}{1 \times 1000}$ $= 436.08$ kN 87 < 575.99 kN and 87 < 436.08 kN therefore the effect of the axial force on the moment resistance may be neglected. 9.2.1.5 Bending resistance	
BS EN 1993-1-1 Cl 6.2.5 (2) eq 6.13 & eq 6.12	$M_{pl,y,Rd} = (W_{pl} f_y = \frac{896000 \times 355}{1 \times 1000000}$ $= 318.1$ kNm Taking haunch depth as 343 mm, (from the centerline intersections of rafter and column) the bending moment at the underside of the haunch is $M_{y,Ed} = 313 \times \frac{(4.5 - 0.343)}{4.5}$ $= 289.1$ kNm $M_{y,Ed} = 289.1$ kNm < 318.1 kNm Bending resistance is adequate 9.2.2 Rafter 9.2.2.1 Shear resistance	
BS EN 1993-1-1 Cl 6.2.6(3)(a)	Shear area $A_v = A - 2b t_f + (t_w + 2 r) t_f$ but not less than $\eta h_w t_w$ $A_v = 4980 - 2 \times 126 \times 10.7 + (6.6 + 2 \times 10.2) \times 10.7$ $A_v = 2572.5$ mm^2	
BS EN 1993-1-1 Cl 6.2.6(3)	conservatively $\eta = 1.0$ $\eta h_w t_w = 1.0 \times 332 \times 6.6$ $= 2191.2$ $A_v = 2572.5$ mm^2	
BS EN 1993-1-1 Cl 6.2.6(2) eq 6.18	$V_{pl,Rd} = A_v((f_y /\sqrt{(3)}))/(\gamma_{MO)} = \frac{2572.5 \times (355 / \sqrt{3})}{1 \times 1000}$ $= 527.26$ kN $V_{Ed} = 74$ kN 74 kN < 527.26 kN section is adequate for shear 9.2.2.2 Bending and shear interaction	
BS EN 1993-1-1 Cl 6.2.8(2)	When shear force is less than half the plastic shear resistance its effect on the moment resistance may be neglected except where shear bucking reduces the section resistance.	

Reference	Calculations	Output
	$0.5\ V_{pl,\ Rd}$ = 0.5 x 527.258 = 263.63 kN 74 kN < 263.63 kN Therefore the effect of the shear force on the moment resistance may be neglected.	
	<u>9.2.2.3 Compression resistance</u>	
BS EN 1993-1-1 Cl 6.2.4(2) eq 6.10 & eq 6.9	$N_{c\,Rd}= (A\,f_{\,y)/(}\gamma_{MO)}$ = $\frac{4980 \text{ x } 355}{1 \text{ x } 1000}$ = 1767.9 kN N_{Ed} = 83 kN 83 kN < 1767.9 kN Compression resistance is adequate	
	<u>9.2.2.4 Combined bending and axial force</u>	
BS EN 1993-1-1 Cl 6.2.9(4)	It is not necessary to provide allowance for the effect of axial force on plastic resistance moment about y-y axis when both the following criteria are satisfied. $N_{Ed} \leq 0.25\,N_{pl,Rd}$ and $N_{Ed} \leq (0.5 h_w t_w f_{\,y)/(}\gamma_{MO})$ $0.25\ N_{pl,Rd}$ = 0.25 x 1767.9 kN = 441.98 kN $\frac{0.5 \text{ hw tw fy}}{\gamma_{MO}}$ = $\frac{0.5 \text{ x } 332 \text{ x } 6.6 \text{ x } 355}{1 \text{ x } 1000}$ = 388.94 kN 83 < 441.98 kN and 83 < 388.94 kN Therefore the effect of the axial force on the moment resistance may be neglected.	
	<u>9.2.2.5 Bending resistance</u>	
BS EN 1993-1-1 Cl 6.2.5 (2) eq 6.13 & eq 6.12	$M_{pl,y,Rd}=(W_{pl}\,f_{\,y)/(}\gamma$ = $\frac{659000 \text{ x } 355}{1 \text{ x } 1000000}$ = 233.9 kNm The maximum bending moment in the rafter is 150 kNm $M_{y,Ed}$ = 150 kNm < 233.9 kNm	Bending resistance is adequate
	<u>10.0 Buckling verification</u> The rafters and columns should be verified for buckling between restraints.	
BS EN 1993-1-1 Cl 6.3.3 (4)	$M_{z,\ Ed}$ = 0 ; no minor axis bending Equation 6.61 & 6.62 can be reduced to	

Single storey - part 4 [17] 6.3.3

$$(N_{Ed)/(}N_{b,y,Rd)}+k_{yy}(M_{y,Ed}\,)_{/(}M_{b,Rd)} \leq 1.0$$

$$(N_{Ed)/(}N_{b,z,Rd)}+k_{zy}(M_{y,Ed)/(}M_{b,Rd)} \leq 1.0$$

where'

$N_{b, y, Rd}$ = flexural buckling resistance in the major axis

$N_{b, z, Rd}$ = flexural buckling resistance in the minor axis

$M_{b, Rd}$ = lateral torsional buckling resistance

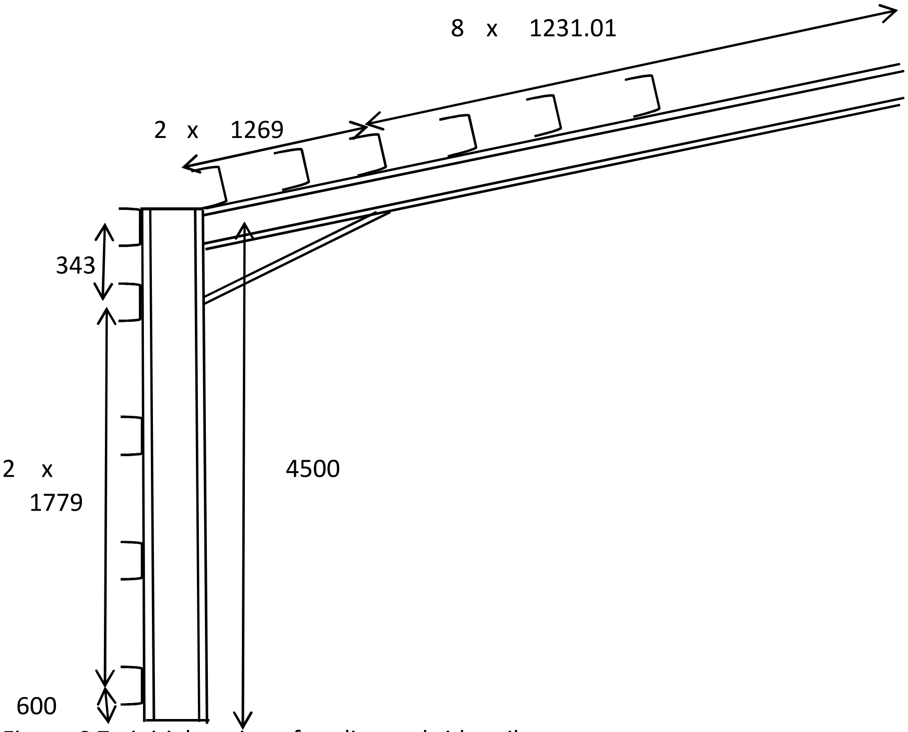


Figure C 7 - initial setting of purlins and side rails

Initial setting of the purlins and side rails are shown above.
At some purlin and side rail positions, stays to the inner flange will be used to provide a torsional restraint at that location.
Intermediate restraints to the tension flange shall increase the buckling resistance, provided that the spacing of such tension flange restraints is within the limiting distance.

10.1 Column verification

First the column shall be checked for the minor axis flexural buckling and lateral torsional buckling between restraints. Then the tension flange restrains shall be checked to utilize for buckling resistance. The column stability against the major axis shall be checked at the end for flexural buckling.

10.1.1 Spacing of restraints to the tension flange

It is assumed that the restraints to the tension flange are effective in increasing the resistance to lateral torsional buckling if their spacing does not exceed L $_m$

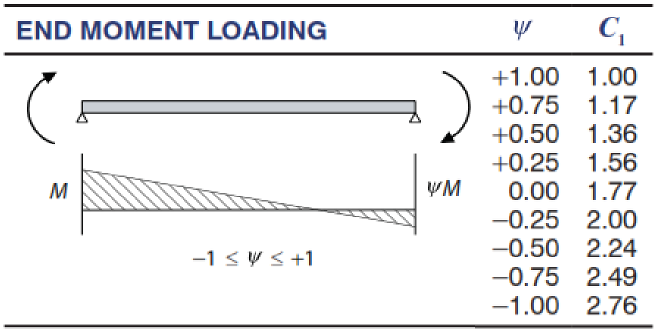
BS EN 1993-1-1
BB 3.1.1
eq BB.5

where L $_m$

$L_m = (38\, i_{z)/\sqrt{(}} 1/57.4\,(N_{Ed/}A)+1/(756\, C_I{}^{2)}\,{}^{(}W_{pl,y}{}^{2)/(}A\, I_{T)\,((}f_{y)/}235)^{2)}$

N $_{Ed}$ = Design value of the compression force (N) in member
A = cross section area of the member
W $_{pl,y}$ = plastic section modulus of the member
I $_T$ = Torsional constant of the member
f_y = Yield strength (N/mm˜)
C_1 = Factor depending on the loading and the end conditions

[7]
Table B.1



END MOMENT LOADING ψ	C_1
+1.00	1.00
+0.75	1.17
+0.50	1.36
+0.25	1.56
0.00	1.77
−0.25	2.00
−0.50	2.24
−0.75	2.49
−1.00	2.76

Figure C 8 - C1 factor - SCI P 397 Table B.1

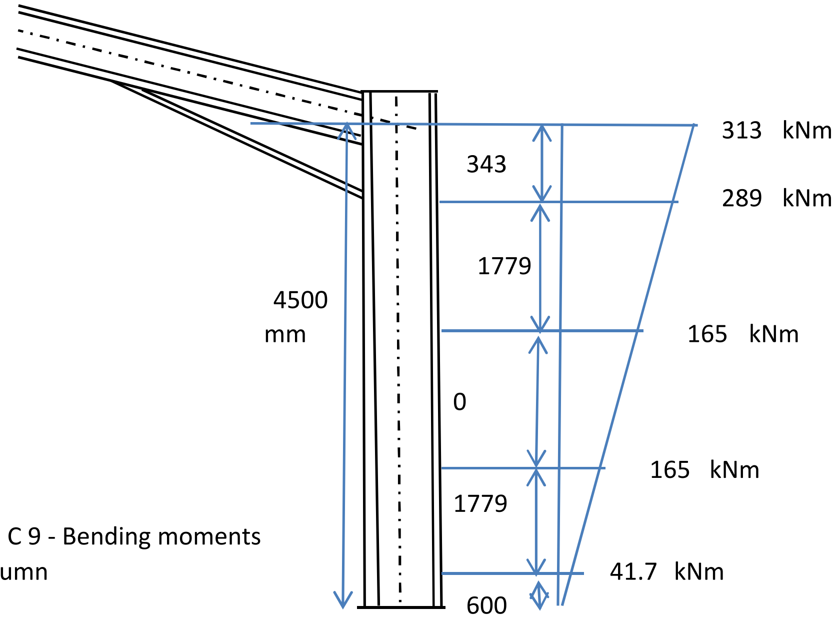


Figure C 9 - Bending moments on column

The ratios of bending moments for the column segments, from the top of the column, are as follows.

ψ	C 1
$\frac{165}{289}$ = 0.57	1.31
$\frac{165}{289}$ = 0.57	1.31
$\frac{41.7}{289}$ = 0.14	1.76

Table C 18 - C1 factor

The most onerous value of C_1 is 1.31 ; this case will be assessed.

$$L_m = \frac{38 \times 38.6}{\left\{\frac{1}{57.4} \times \left(\frac{87000}{6.49E+03}\right) + \frac{1}{756 \times 1.31^2} \times \frac{8.96.E+05^2}{6.49E+03 \times 238000} \times \left(\frac{355}{235}\right)^2\right\}^{0.5}}$$

$$= 1365.1 \text{ mm}$$

Side rail spacing is 1779 mm which exceeds this limiting value.

Therefore the restraints to the tension flange are not close enough to be used to enhance the resistance to lateral-torsional buckling.

10.1.2 Verification with no intermediate restraints

First the column is checked with the restraint at the underside of the haunch and the base, assuming no intermediate restraints. If the flexural buckling, lateral torsional buckling and interaction checks are satisfied for this length, no intermediate restraints are required. otherwise, intermediate torsional restraints need to be introduced to the column or the column size increased.

Flexural buckling resistance about the minor axis, $N_{b,z,Rd}$

$\frac{h}{b} = \frac{355}{172} = 2.07$

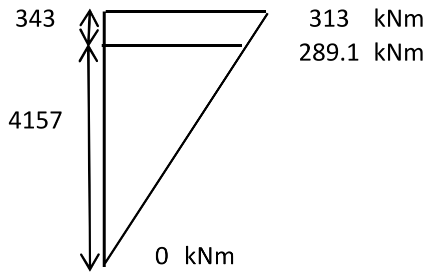
t_f = 11.5 mm

f_y = 355 N/mm^2

γ_{M1} = 1.0

BS EN 1993-1-1 Cl 6.3.1.3 (1)

$$\lambda_1 = \pi\sqrt{(E/f_y)} = \pi \left(\frac{2.10E+05}{355}\right)^{0.5} = 76.41$$

Reference	Calculations	Output
eq 6.50	$(\lambda)^{-}_{z} = (L_{c} \quad = \frac{4157}{38.6} \times \frac{1}{76.41} = 1.41$	
BS EN 1993-1-1 Table 6.2 Table 6.1	For buckling about z-z axis, Buckling curve = curve b Imperfection factor, α = 0.34	
Cl 6.3.1.2 (1)	$\Phi = 0.5\ [1 + \alpha_{z}\ ((\lambda_{z})^{-} - 0.2) + (\lambda_{z})^{-\ 2]}$ $= 0.5 \times \left(1 + 0.34 \times \left(1.41 - 0.2 \right) + 1.41^{2} \right)$ $= 1.70$	
eq 6.49	$\chi_{z} = 1/(\Phi + \sqrt{(\Phi^{2} - \lambda^{-\ 2))}}$ but < 1 $= \frac{1}{1.70 + \left(1.70^{2} - 1.41^{2}\right)^{0.5}}$ $= 0.38$	
BS EN 1993-1-1 6.3.1 (3) eq 6.47	$N_{b,z,Rd} = (\chi_{z}\ A\ f_{y)/(}\gamma_{M1)}$ $= \frac{0.38 \times 6490 \times 355}{1.0 \times 1000}$ $= 870.28$ kN	
	N_{Ed} = 87 kN < 870.28 kN	Flexural buckling resistance about minor axis is ok
	Lateral- torsional buckling resistance, $M_{b,Rd}$  C 1 is calculated based on the bending moment diagram over the column length between the base and the underside of the haunch.	
[7] Table B.1	$\psi = \frac{0}{289.1} = 0$ hence C 1 = 1.77	
[17] Appendix C C.1.1	$M_{cr} = c_{1} (\pi^{2}\ E\ I_{z})/(L^{2})\sqrt{((I_{w)/(}I_{z}) + (L^{2}\ G\ I_{T)/(}\pi^{2}\ E\ I_{z}))}$	

Reference	Calculations	Output
	E = Modulus of elasticity	
	G = Shear Modulus	
	I z = Second moment of area about the minor axis	
	I_T = Torsional constant of the member	
	I_w = warping constant of the member	
	L = beam length between points of lateral restraint	
	C1 = factor that counts for the shape of the bending moment diagram	
	$M_{cr}=c_1(\pi^2 E I_z)/(L^2)\sqrt{((I_{w)/(}I_z\ _)+(L^2 G I_{T)/(}\pi^2 E I_{z))}}$	
	$= \left(\frac{1.77 \text{ x } \pi^2 \text{ x } 210000 \text{ x } 9.68E+06}{4157^2} \right)$	
	$\text{x} \left(\frac{2.9E+11}{9680000} + \frac{4157^2 \text{ x } 81000 \text{ x } 238000}{\pi^2 \text{ x } 210000 \text{ x } 9.68E+06} \right)^{0.5}$	
	= 2054980.07 x 214.825567	
	= 4.41E+08 Nmm	
	The non-dimensional slenderness, $\overline{\lambda}_{LT}$ is given by	
BS EN 1993-1-1 Cl 6.3.2.2(1)	$\bar{\lambda}_{LT}=\sqrt{((W_y f_{y)/(}M_{cr))}}$	
	W_y = Appropriate section modulus as follows	
	W pl,y - for Class 1 or 2 cross sections	
	W el, y - for Class 3 cross sections	
	W eff, y - for Class 4 cross sections	
	$\bar{\lambda}_{LT}=\sqrt{((W_y f_{y)/(}M_{cr))}}$	
	$= \left(\frac{8.96E+05 \text{ x } 355}{4.41E+08} \right)^{0.5}$	
	= 0.85	
BS EN1993-1-1 Cl 6.3.2.3 (1)	to calculate the reduction factor, χ_{LT}	
	$\Phi_{LT} = 0.5[1+\alpha_{LT}(\bar{\lambda}_{LT}-\bar{\lambda}_{LT,0)}+\beta\bar{\lambda}_{LT}^{2]}$	
BS EN1993-1-1 NA	$\overline{\lambda}_{LT,0}$ = 0.40 (maximum value)	
	β = 0.75 (minimum value)	
BS EN1993	(h/b) = 2.07	
Table 6.5	Lateral-torsional buckling curve = c	
Table 6.3	Imperfection factor, α_{LT} = 0.49	
	$\Phi_{LT} = 0.5[1+\alpha_{LT}(\bar{\lambda}_{LT}-\bar{\lambda}_{LT,0)}+\beta\bar{\lambda}_{LT}^{2]}$	
	$= 0.50\left(1 + 0.49\left(0.85 - 0.40 \right) + 0.75 \text{ x } 0.85^2 \right)$	
	= 0.88	

Reference	Calculations	Output

BS EN1993
-1-1
Cl 6.3.2.3(1)

eq 6.57

$$\chi_{LT} = 1/(\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^{2))}}$$

$$= \frac{1}{0.88 + [\,0.88^2 - 0.75 \times 0.85^2\,]}$$

$$= 0.73$$

BS EN1993
-1-1
Cl 6.3.2.1
(3)
eq 6.55

$$M_{b,Rd} = (\chi_{LT} W_{pl,y} f_{y)/(}\gamma$$

$$= \frac{0.73 \times 8.96.E{+}05 \times 355}{1.0 \times 1.E{+}06}$$

$$= 233 \text{ kNm}$$

M_{Ed} = 289 kNm > 233 kNm Unsatisfactory

Intermediate restraints are required
10.1.3 calculation is required

10.1.3 Revised restraint arrangement

Intermediate restraints must be at a side rails position, since bracing from the side rail to the inner flange is used to provide the torsional restraint.

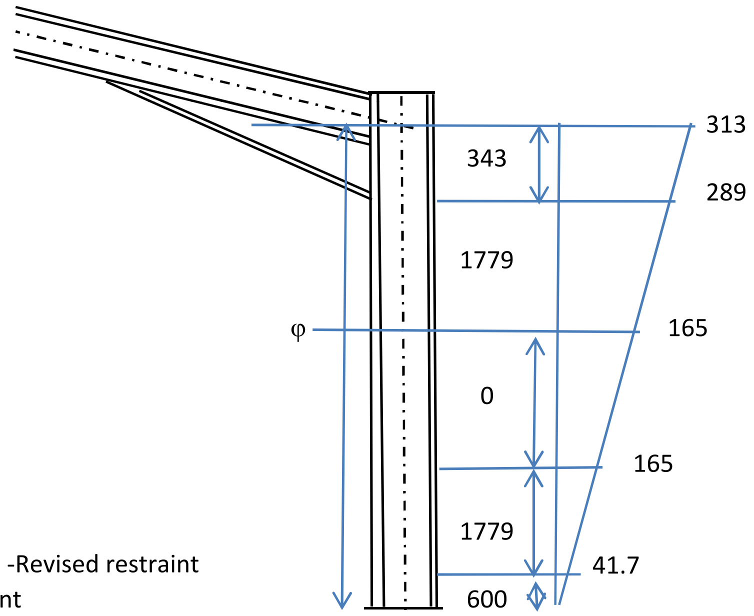


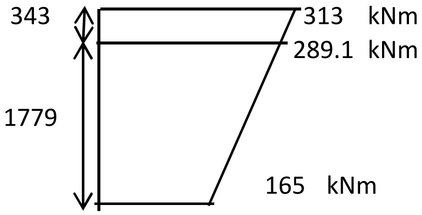
Figure C 10 -Revised restraint arrangement

10.1.4 Verification of revised restraint arrangement - upper segment

Flexural buckling and lateral torsional buckling verifications are carried out independently before proceeding to verify the interaction between the two.

Flexural buckling resistance about the minor axis, $N_{b,z,Rd}$

$\frac{h}{b}$ = 2.07 and λ_1 = 76.41 as calculated before

Reference	Calculations	Output
eq 6.50	$(\lambda)^{-}_{z} = (L_{c} \quad = \frac{1779}{38.6} \times \frac{1}{76.41} = 0.6$	
BS EN 1993-1-1 table 6.2 Table 6.1	For buckling about z-z axis, Buckling curve = curve b Imperfection factor, α = 0.34	
Cl 6.3.1.2 (1)	$\Phi=0.5\,[1+\alpha_z\,((\lambda_{z)^{-}}-0.2)+(\lambda_z)^{-\ 2]}$ $= 0.5 \times \left[1 + 0.34 \times \left(0.60 - 0.2 \right) + 0.6^{2} \right]$ $= 0.75$	
eq 6.49	$\chi_z = 1/(\Phi+\sqrt{(\Phi^2-\lambda^{-\ 2}))}$ but < 1 $= \frac{1}{0.75 + \left(0.75^{2} - 0.60^{2} \right)^{0.5}}$ $= 0.84$	
BS EN 1993-1-1 6.3.1 (3) eq 6.47	$N_{b,z,Rd} = (\chi_z A f_{y)/(}\gamma_{M1)}$ $= \frac{0.84 \times 6490 \times 355}{1.0 \times 1000}$ $= 1925$ kN	
	N_{Ed} = 87 kN < 1925 kN With restraints, flexural buckling resistance about the minor axis is ok	
	<u>Lateral- torsional buckling resistance, $M_{b,Rd}$</u>  	
[7] Table B.1	$\psi = \frac{165.4}{289.1} = 0.57$ hence C 1 = 1.31	
[17] Appendix C C.1.1	$M_{cr}=c_1(\pi^2 E I_z)/(L^2)\sqrt{((I_{w)/(}I_z)+(L^2 G I_{T)/(}\pi^2 E I_{z))}}$	

$$= \left[\frac{1.31 \quad x \quad \pi^{2} \quad x \quad 210000 \quad x \quad 9.68E+06}{1779^{2}}\right]$$

$$x \left[\frac{2.86.E+11}{9680000} + \frac{1779^{2} \quad x \quad 81000 \quad x \quad 238000}{\pi^{2} \quad x \quad 210000 \quad x \quad 9.68E+06}\right]^{0.5}$$

$= \quad 8.29E+06 \quad x \quad 180.51$

$= \quad 1.50E+09 \quad$ Nmm

The non-dimensional slenderness, $\overline{\lambda}_{LT}$ is given by

BS EN 1993-1-1
Cl 6.3.2.2(1)

$\lambda\bar{}_{LT} = \sqrt{((W_{y} f_{y)/(} M_{cr))}}$

$$= \left[\frac{8.96E+05 \quad x \quad 355}{1.50E+09}\right]^{0.5}$$

$= \quad 0.46$

BS EN1993 -1-1
Cl 6.3.2.3 (1)

to calculate the reduction factor, χ_{LT}

$\Phi_{LT} = 0.5[1+\alpha_{LT} (\lambda\bar{}_{LT} - \lambda\bar{}_{LT,0)} + \beta\lambda\bar{}_{LT}{}^{2]}$

BS EN1993 -1-1
NA

$\overline{\lambda}_{LT,0} \quad = \quad 0.40 \quad$ (maximum value)

$\beta \quad = \quad 0.75 \quad$ (minimum value)

BS EN1993
Table 6.5
Table 6.3

(h/b) = 2.07

Lateral-torsional buckling curve = c

Imperfection factor, α_{LT} = 0.49

$\Phi_{LT} = 0.5[1+\alpha_{LT} (\lambda\bar{}_{LT} - \lambda\bar{}_{LT,0)} + \beta\lambda\bar{}_{LT}{}^{2]}$

$= \quad 0.50\left[1 + \quad 0.49\left(0.46 \quad - \quad 0.40\right) + \quad 0.75 \quad x \quad 0.46^{2}\right]$

$= \quad 0.59$

BS EN1993 -1-1
Cl 6.3.2.3(1)
eq 6.57

$\chi_{LT} = 1/(\Phi_{LT} + \sqrt{(\Phi_{LT}{}^{2} - \beta\lambda\bar{}_{LT}{}^{2))}}$

$$= \frac{1}{0.59 \quad + \left(0.59^{2} \quad - \quad 0.75 \quad x \quad 0.46^{2}\right)^{0.5}}$$

$= \quad 0.97$

BS EN1993 -1-1
Cl 6.3.2.1 (3)
eq 6.55

$M_{b,Rd} = (\chi_{LT} W_{pl,y} f_{y)/(}\gamma$

$$= \frac{0.97 \quad x \quad 8.96.E+05 \quad x \quad 355}{1.0 \quad x \quad 1.E+06}$$

$= \quad 307 \quad$ kNm

$M_{Ed} \quad = \quad 289$ kNm < 307 kNm   With restraints, lateral torsionl buckling resistance is ok

BS EN1993 -1-1
Cl 6.3.3(4)

Interaction of axial force and bending moment

Equation 6.62 is reduced to

Reference | Calculations | Output

[17]
6.3.3

$(N_{Ed)/(}N_{b,z,Rd)}+k_{zy}(M_{y,Ed)/(}$

$\bar{\lambda}_z = 0.6 > 0.4$

BS EN1993-1-1
Table B.2

$k_{zy}=max[1-(0.1\bar{\lambda}_{z)/((}C_{mLT}-0.25))\ (N_{Ed};[1-0.1/((C_{mLT}-0.25))\ (N_{Ed)/(}N$
$)/(N_{b,Rd,z)}\ ]$ $\quad {}_{b,Rd,z)]}$

BS EN1993-1-1
Table B.3

$C_{mLT} = 0.6 + 0.4\,\psi$
$= 0.83 > 0.4$

Hence $C_{mLT} = 0.83$

$$k_{zy} = \max\left[\left(1 - \frac{0.1 \times 0.60}{(0.83 - 0.25)} \times \frac{87.0}{1925}\right);\ \left(1 - \frac{0.1}{(0.83 - 0.25)} \times \frac{87.0}{1925}\right)\right]$$

$k_{zy} = \max\{0.995\ ;\ 0.992\} = 0.995$

$(N_{Ed)/(}N_{b,z,Rd)}+k$ $= \frac{87}{1925} + 0.995 \times \frac{289}{307}$
$= 0.98$
< 1

Interaction axial force and bending moment is ok

10.1.5 Verification of revised restraint arrangement - intermediate segment

Since the moments acting on the intermediate segment is lesser and the length is similar to the upper segment, it satisfies the requirements.

10.1.6 Verification of revised restraint arrangement - major axis

Flexural buckling resistance about the majour axis, $N_{b,y,Rd}$

$\frac{h}{b} = 2.07$ and $\lambda_1 = 76.41$ as calculated before

BS EN 1993-1-1

For hot rolled I sections
For buckling about y-y axis,

table 6.2
Table 6.1

Buckling curve = curve a
Imperfection factor, $\alpha = 0.21$

The buckliing length is taken as the total leng of the column nodes.

L = 4500 mm

Reference	Calculations	Output
eq 6.50	$(\lambda)^{-}_{y} = (L_{c}$ $= \frac{4500}{148.0} \times \frac{1}{76.41} = 0.4$	
Cl 6.3.1.2 (1)	$\Phi_y = 0.5\ [1+\alpha_y\ ((\lambda_{y})^{-} - 0.2) + (\lambda_y)^{-\ 2}]$	
	$= 0.5 \times \left(1 + 0.21 \times \left(0.40 - 0.2 \right) + 0.4^{2} \right)$	
	$= 0.60$	
eq 6.49	$\chi_y = 1/(\Phi + \sqrt{(\Phi^2 - \lambda^{-\ 2}))}$ but < 1	
	$= \frac{1}{0.60 + \left(0.60^{2} - 0.40^{2} \right)^{0.5}}$	
	$= 0.95$	
BS EN 1993-1-1 6.3.1 (3) eq 6.47	$N_{b,y,Rd} = (\chi_y A f_y)/(\gamma_{M1})$ $= \frac{0.95 \times 6490 \times 355}{1.0 \times 1000}$ $= 2196$ kN	
	$N_{Ed} = 87$ kN $<$ 2196 kN Flexural buckling resistance about the major axis is ok	
BS EN1993-1-1 6.3.3(4)	<u>Interaction of axial force and bending moment</u> Equation 6.61 is reduced to	
[17]	$(N_{Ed})/(N_{b,y,Rd}) + k_{yy}(M_{y,Ed}$	
	Most onerous ratio is considered to be in the upper segment	
	$(M_{y,Ed})/(M_{b,Rd}) = \frac{313}{307} = 1.02$	
	The interaction factor kyy is given by $k_{yy} = min[C_{my}(1+(\bar{\lambda}_{y} - 0.2)\ (N_{Ed})/(N_{b,Rd,y})\ ;[C_{my}\ (1+0.8\ (N_{Ed})/(N_{b,Rd,y})]$	
	For C_{my}, the relevant braced points are the torsional restraints at the ends of the member,	
	$\psi = \frac{0}{313} = 0$	
BS EN1993-1-1 Table B.3	$C_{my} = 0.6 + 0.4\ \psi$ $= 0.6 > 0.4$	
	Hence $C_{my} = 0.6$	

Reference	Calculations	Output

$$k_{yy} = \min \left\{ \left(0.6 \left(1 + \left(0.4 - 0.2 \right) \frac{87}{2196} \right) ; \right. \left. 0.6 \left(1 + 0.8 \left(\frac{87}{2196} \right) \right) \right\}$$

$$= \min (0.60 ; 0.62)$$

$$= 0.60$$

$$(N_{Ed)/(}N_{b,y,Rd)}+k_{yy}(= \frac{87}{2196} + 0.60 \times 1.02$$

$$= 0.66 < 1$$

Interaction of axial force and bendind moment is ok

10.1.8 Summary: Adequacy of the column section

The cross sectional resistance, flexural buckling resistance and lateral torsional buckling resistance have been demonstrated to be adequate. The interaction of flexural and lateral torsional buckling has been verified using expressions 6.61 and 6.62.

Therefore it is concluded that a 356x171x51 ,Grade S 355 is adequate for use as the column in this portal frame, considering load combination 1.

10.2 Rafter Verification

From the analysis

V_{Ed} (maximum value) = 74 kN

N_{Ed} (maximum value) = 83 kN

M_{Ed} (at the end of the haunch) = 150 kNm

M_{Ed} (Adjacent to the apex) = 85 kNm

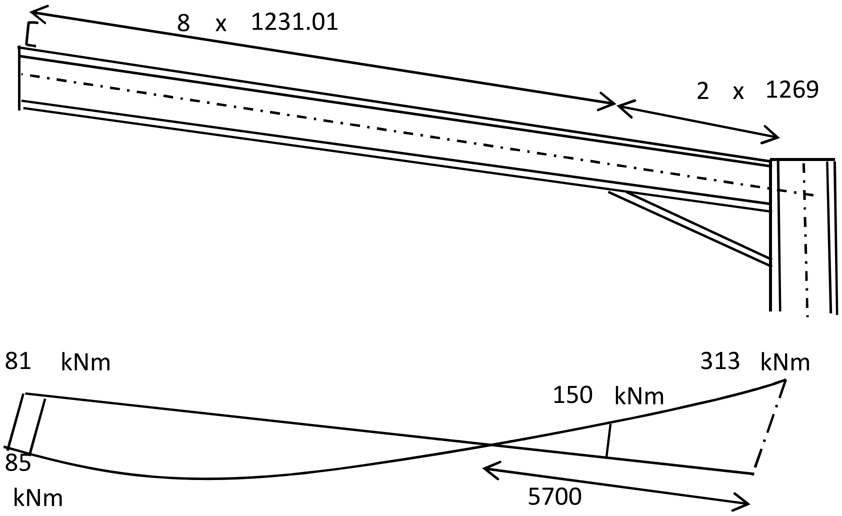


Figure C 11 -Bending moment diagram over the length of the rafter

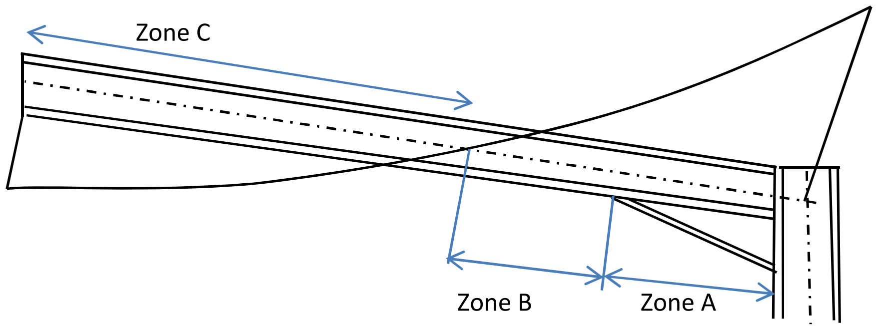
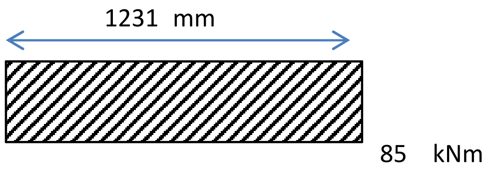


Figure C 12 -Definition of Zones

10.2.1 Zone C - sagging region

It is assumed that the maximum moment is uniform between the purlins.



Flexural buckling resistance about the minor axis, $N_{b,\ z,\ Rd}$

$$\frac{h}{b} = \frac{353}{126} = 2.80$$

t_f = 10.7 mm

f_y = 355 N/mm^2

BS EN1993 -1-1 cl 6.3.1.3 (1) eq 6.50

as before λ_1 = 76.41

$$\bar{\lambda}_z = (L_{cr})/(i_z) = \frac{1231}{26.8} \times \frac{1}{76.41} = 0.6$$

BS EN1993 -1-1 Table 6.1 Table 6.2 Cl 6.3.1.2(1)

For buckling about z-z axis,

curve b ; α_z = 0.34

$$\Phi_z=0.5\ [1+\alpha_z\ ((\bar{\lambda}_z)-0.2)+(\bar{\lambda}_z)^{2}]$$

$= 0.5\ [1 + 0.34\ (0.60 - 0.2) + 0.6^2]$

$= 0.75$

eq 6.49

$$\chi_z= 1/(\Phi_z+\sqrt{(\Phi_z^2-\bar{\lambda}_z^{2})}) = \frac{1}{0.749 + [0.75^2 - 0.6^2]^{0.5}}$$

$= 0.84$

BS EN 1993-1-1 6.3.1 (3) eq 6.47

$$N_{b,z,Rd} =(\chi_z\ A\ f_y)/(\gamma_{M1}) = \frac{0.84 \times 4980 \times 355}{1.0 \times 1000}$$

$= 1478.82$ kN

N_{Ed} = 83 kN < 1478.82 kN

Flexural buckling resistance to the minor axis is ok

Reference	Calculations	Output
	Lateral -torsional buckling resistance, $M_{b,Rd}$	
[7] Appendix B B.2.2	It is assumed that the bending moment diagram to be constant along the segment in consideration. So ψ = 1.0 Therefore C1 = 1.0	
Appendix B B.2.1	$M_{cr}=c_1(\pi^2 E I_z)/(L^2)\surd((I_{w)/(}I_{z)}+(L^2 G I_{T)/(}\pi^2 E I_{z))}$ $= \left(\frac{1.0 \times \pi^2 \times 210000 \times 3.58E+06}{1231.01^2}\right) \times \left(\frac{1.05E+11}{3.58E+06}+\frac{1231^2 \times 81000 \times 1.51E+05}{\pi^2 \times 210000 \times 3.58E+06}\right)^{0.5} \times 10^6$ = 873.54 kNm	
BS EN 1993-1-1 Cl 6.3.2.2(1)	$\bar{\lambda}_{LT}=\surd((W_{Pl,y} f_{y)/(}M_{cr))}$ $= \left(\frac{659000 \times 355}{873.54 \times 10^6}\right)^{0.5}$ = 0.52	
BS EN1993 -1-1 Table 6.5 Table 6.3	$\frac{h}{b}$ = 2.80 curve c ; α_{lT} = 0.49	
BS EN1993 -1-1 NA	$\bar{\lambda}_{LT,0}$ = 0.40 (maximum value) β = 0.75 (minimum value)	
BS EN1993 -1-1 Cl 6.3.2.3	$\Phi_{LT} = 0.5[1+\alpha_{LT}(\bar{\lambda}_{LT}-\bar{\lambda}_{LT,0)}+\beta\bar{\lambda}_{LT}^{2]}$ $= 0.5\left(1 + 0.49\left(0.52 - 0.40\right) + 0.75 \times 0.52^2\right)$ = 0.63	
BS EN1993 -1-1 Cl 6.3.2.3(1) eq 6.57	$\chi_{LT}= 1/(\Phi_{LT}+\surd(\Phi_{LT}^2-\beta\bar{\lambda}_{LT}^{2))}$ $= \frac{1}{0.63 + \left(0.63^2 - 0.75 \times 0.52^2\right)^{0.5}}$ = 0.93 < 1 χ_{LT} = 0.93	
BS EN1993 -1-1 Cl 6.3.2.1(3) eq 6.55	$M_{b,Rd} =(\chi_{LT} W_{pl,y} f_{y)/(}\gamma$ $= \frac{1 \times 659000 \times 355}{1.0 \times 10^6}$ = 218.5 kNm	
	$M_{v,Ed}$ = 85 kNm < 218.5 kNm Lateral torsional buckling resistance is ok	

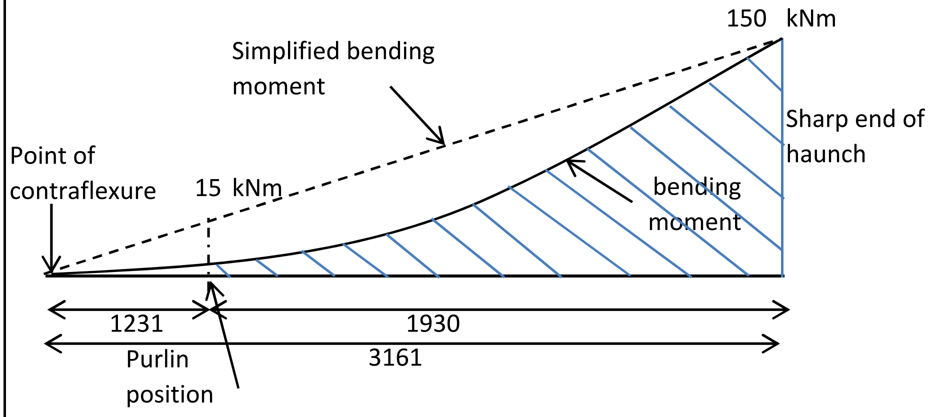
Reference	Calculations	Output
BS EN1993 -1-1 Cl 6.3.3(4) [17]	Interaction of axial force and bending moment in accordance with Expression 6.62 $(N_{Ed})/(N_{b,z,Rd})+k_{zy}(M_{y,Ed})/(M_{b,Rd})< 1.0$ for $\overline{\lambda}_z \geq 0.4$ k_{zv} is calculated as $k_{zy}=max[1-(0.1\bar{\lambda}_z)/((C_{mLT}-0.25))(N_{Ed})/(N_{b,Rd,z})];[1-0.1/((C_{mLT}-0.25))(N_{Ed})/(N_{b,Rd,z})]$	
BS EN1993 -1-1 Table B.3	The bending moment is assumed to be uniform. Therefore C_{mLT} = 1 $k_{zy} = \max\left[\left(1 - \frac{0.1 \times 0.60}{(1 - 0.25)} \times \frac{83.0}{1479}\right); \left(1 - \frac{0.1}{(1 - 0.25)} \times \frac{83.0}{1479}\right)\right]$ $k_{zy} = \max\{0.996 ; 0.993\} = 0.996$ $(N_{Ed})/(N_{b,z,Rd})+k = \frac{83.0}{1479} + 0.996 \times \frac{85}{218}$ $= 0.44$ < 1 Interaction of axial forces wit bending moment is ok	
	10.2.2 Zone B - hogging region In Combination 1, the bottom flange is in compression. Torsional restraints are provided at certain locations by stays from the purlins to the inside flange. The buckling length is taken from the torsional restraint at the sharp end of the haunch to the 'virtual' restraint at the point of contraflexure of the bending moment diagram. If the rafter cannot be verified over this length, additional restraints to the inside flange will be required.	
[7] Sec 7.2.3	A virtual restraint maybe assumed at the point of contraflexure, as the rafter is a UB section, the depth of the purlins is not less than 0.25 times the depth of the rafter and the purlin-to-rafter connection comprices atleast two bolts. For the cases when the above conditions are not satisfied, the buckling length should be taken to the next purlin past the point of contraflexure. (i.e. the first restraint to the compression flange).	

Reference	Calculations	Output

From the analysis,

Distance from haunch to the point of contraflexure
= 3161 mm

Figure C 13 - Zone B bending moments



Spacing of restraints to the tension flange

The limiting spacing is given by

BS EN 1993-1-1
BB 3.1.1

$L_m = (38\ i_{z)/\surd(}1/57.4\ (N_{Ed/}A)+1/(756\ C_I{}^{2)}\ {}^{(}W_{pl,y}{}^{2)/(}A\ I_{T)\ ((}f_{\ y)/}235)^{2)}$

[7]
Apendix B
Table B1

$\psi = \frac{15}{150} = 0.1$

$C_1 = 1.69$

$$L_m = \frac{38 \times 26.8}{\left\{\frac{1}{57.4} \times \left(\frac{83000}{4.98E+03}\right) + \frac{1}{756 \times 1.69^2} \times \frac{6.59.E+05^2}{4.98E+03 \times 1.5E+05}\left(\frac{355}{235}\right)^2\right\}^{0.5}}$$

= 1071.3 mm

Restraint to the tension flange is provided by purlins spaced at 1231 m

1231 mm > 1071.3 mm

Purlin spacing exceeds the limiting spacing
Advantage cannot be taken of the restraints to the tension flange.

Initially, the hogging region is verified assuming no intermediate torsional restraints.

If the flexural buckling, lateral torsional buckling and interaction checks are satisfied for the length of the whole hogging region, no further torsional restraints are required. Otherwise intermediate restraints will need to be introduced to the rafter in the hogging zone or the rafter size increased.

Reference	Calculations	Output
	Flexural buckling resistance about the minor axis, $N_{b,z,Rd}$ The distance 'a' between the restrained longitudinal axis and the shear center of the rafter taking 200 mm deep purlins is given by a = 0.5 x 353 + 0.5 x 200 = 276.7 mm	
BS EN 1993-1-1 BB 3.3.1	The elastic critical torsional buckling force between torsional restraints is given by: $N_{crT}=1/(i_s^2)\ ((\pi^2 E\, I_z\, a^2)/(L_t^2)\ +(\pi^2 E\, I_w)/(L_t^2)+ GI_{T)}$ in which $i_s^2 = i_y^2 + i_z^2 + a^2$ $= 143^2 + 26.8^2 + 276.7^2$ $= 97730\ \text{mm}^2$ $L_t = 3161$ mm $N_{cr\,T} = \frac{10^{-3}}{97730.1}\left[\frac{\pi^2 \times 2.10E+05 \times 3.58E+06 \times 276.7^2}{3161^2} + \frac{\pi^2 \times 2.10E+05 \times 1.05E+11}{3161^2} + 81000 \times 1.5E+05\right]$ = 929.55 kN	
BS EN 1993-1-1 cl 6.3.1.3(1) eq 6.50	$\bar{\lambda}_z=\sqrt{((A\ f_y)/(N_{cr}))}$ $= \left(\frac{4980.0 \times 355}{929.55 \times 1000}\right)^{0.5}$ = 1.38	
BS EN 1993-1-1 Table 6.2 Table 6.3	As before $\frac{h}{b} = 2.80$ curve b ; $\alpha_z = 0.34$	
BS EN 1993-1-1 cl 6.3.1.2(1)	$\Phi=0.5\ [1+\alpha_z\ ((\bar{\lambda}_z) -0.2)+(\bar{\lambda}_z)^2]$ $= 0.5 \times \left(1 + 0.34 \times (1.38 - 0.2) + 1.4^2\right)$ = 1.65	
eq 6.49	$\chi_z = 1/(\Phi+\sqrt{(\Phi^2-\bar{\lambda}^2)})$ $= \frac{1}{1.65 + (1.65^2 - 1.38^2)^{0.5}}$ = 0.39	

Reference	Calculations	Output
BS EN 1993-1-1 cl 6.3.1.1(3)	$N_{b,z,Rd} = (\chi_z A f_{y)/(}\gamma_{M1)}$ $= \frac{0.39 \times 4980 \times 355}{1.0 \times 1000}$ $= 690.641$ kN $N_{Ed} = 83$ kN $<$ 690.641 kN OK	
	Lateral torsional buckling resistance, $M_{b,Rd}$ To determine the non-dimensional slenderness for lateral torsional buckling, the value of M_{cr0} must first be calculated for a member with intermediate restraints subject to a uniform moment.	
[7] Section C 2.2	$M_{cr0}=(i_s^{2)/(}2\,a)$ $= \frac{97730}{2 \times 276.7} \times \frac{929.55}{1000}$ $= 164.2$ kNm Then, for a linear bending moment, M_{cr} is given by $M_{cr} = c^2 C_m M_{cr0}$	
Section C.2.1	To calculate C_m, $N_{cr,E}$ must be calculated. $N_{cr,E} = (\pi^2 E\, I_{z)/(} L_t^{2)}$ $= \frac{\pi^2 \times 2.10E+05 \times 3.58E+06}{3161^2 \times 1000}$ $= 742.39$ kN $\eta = \frac{N_{cr\,E}}{N_{crT}} = \frac{742.39}{929.55} = 0.80$ $B_0 = (1+10\eta)/(1+20\,\eta)$ $= \frac{1 + 10 \times 0.80}{1 + 20 \times 0.80} = 0.53$ $B_1 = (5\sqrt{\eta})/(\pi+10\sqrt{\eta})$ $= \frac{5 \times (0.80)^{0.5}}{\pi + 10 \times (0.80)^{0.5}} = 0.37$ $B_2 = (0.5)/(1+\pi\sqrt{\eta}) - 0.5/(1+20\,\eta)$ $= \frac{0.5}{1 + \pi \times (0.80)^{0.5}} - \frac{0.5}{1 + 20 \times 0.80}$ $= 0.10$ $\beta_t = (M_{min)/(}$ $= \frac{0}{150} = 0$ $C_m = 1/(B_0+B_1\beta_t+B_2\beta_t^{2)}$ $= \frac{1}{0.53 + 0.37 \times 0 + 0.10 \times 0^2}$ $= 1.89$	

Reference	Calculations	Output
	Beacause the member is uniform, the taper factor "c" is taken as 1	
	$M_{cr} = c^2 C_m M_{cr0}$ $= 1^2 \times 1.89 \times 164.2$ $= 310.0$ kNm	
	The non-dimensional slenderness, $\overline{\lambda}_{LT}$ is given by,	
BS EN1993 -1-1 Cl 6.3.2.2	$\bar{\lambda}_{LT}= \sqrt{((W_{Pl,y}\, f_{y})/(M_{cr}))}$ $= \left(\frac{659000 \times 355}{310.05 \times 10^6} \right)^{0.5}$ $= 0.87$	
BS EN1993 -1-1 Table 6.3 Table 6.5	$\frac{h}{b} = 2.80$ curve c ; $\alpha_{LT} = 0.49$	
BS EN1993 -1-1 NA	$\overline{\lambda}_{LT,0} = 0.40$ (maximum value) $\beta = 0.75$ (minimum value)	
BS EN1993 -1-1 Cl 6.3.2.3	$\Phi_{LT} = 0.5[1+\alpha_{LT}\,(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0})+\beta\bar{\lambda}_{LT}^2]$ $= 0.5 \left[1 + 0.49 \left[0.87 - 0.40 \right] + 0.75 \times 0.87^2 \right]$ $= 0.9$	
BS EN1993 -1-1 Cl 6.3.2.3	$\chi_{LT}= 1/(\Phi_{LT}+ \sqrt{(\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^{2})})$ $= \frac{1}{0.9 + \left[0.9^2 - 0.75 \times 0.87^2 \right]^{0.5}}$ $= 0.72 < 1$ $\frac{1}{\lambda_{LT.0}^2} = \frac{1}{0.40^2} = 6.25$ $\chi_{LT} = 0.72$	
	To calculate the modification factor, f $k_c= 1/\sqrt{C_1}$ $C_1 = 1.77$ $kc = \frac{1}{(1.77)^{0.5}} = 0.752$ $f=1-0.5(1-k_c)[1-2((\lambda_{LT})^{-} - 0.8)^{2}]$ $f = 1 - 0.5 (1 - 0.75)\{1 - 2(0.87 - 0.8)^2\}$ $= 0.88$	
	The modified reduction factor is given by,	

$\chi_{LT,mod} = (\chi_{LT)/} f \leq \quad = \dfrac{0.72}{0.88} \quad = \quad 0.82 \quad < \quad 1$

$\chi_{LT,mod} \quad = \quad 0.82$

The buckling resistance moment is given by

$M_{b,Rd} = (\chi_{LT,mod} W_{pl,y} f_{y)/(} \gamma_{M1)}$

$= \dfrac{0.82 \times 659000 \times 355}{1 \times 10^{6}}$

$= \quad 192.2 \quad \text{kNm}$

$M_{Ed} \quad = \quad 150 \quad \text{kNm} \quad < \quad 192.2 \quad \text{kNm}$

ok

BS EN1993
-1-1
Cl 6.3.3(4)

Interaction of axial force and bending moment in accordance with Expression 6.62

As noted earlier, in this situation, Expression 6.62 reduces to,

$(N_{Ed)/(} N_{b,z,Rd)} + k_{zy} (M_{y,Ed)/(} M_{b,Rd)} \leq 1.0$

BS EN1993
-1-1
Table B.2

$\bar{\lambda}_{z} \quad = \quad 1.4 \quad > \quad 0.4$

$k_{zy} = max[1-(0.1\,\bar{\lambda}_{z)/((}\, C_{mLT} - 0.25)\,)\ (N_{Ed\,)/(} N_{b,Rd,z)}\ ] ;[1-0.1/((C_{mLT} - 0.25)\,)\ (N_{Ed)/(} N_{b,Rd,z)}\ ]$

BS EN1993
-1-1
Table B.3

$C_{mLT} \quad = \quad 0.6 + 0.4\,\psi$

$\psi \quad = \dfrac{0}{150} \quad = \quad 0$

$= \quad 0.6 \quad > \quad 0.4$

Hence $C_{mLT} \quad = \quad 0.6$

$$k_{zy} = \max\left[\left(1 - \frac{0.1 \times 1.38}{(0.6 - 0.25)} \times \frac{83.0}{691}\right) ; \left(1 - \frac{0.1}{(0.6 - 0.25)} \times \frac{83.0}{691}\right)\right]$$

$k_{zy} \quad = \quad \max \{ \quad 0.953 \quad ; \quad 0.966 \quad \} \quad = \quad 0.966$

$(N_{Ed)/(} N_{b,z,Rd)} + k_{zy} (M_{y,Ed)/(} M_{b,Rd)} \quad = \dfrac{83.0}{691} + 0.966 \times \dfrac{150}{192}$

$= \quad 0.87$

$< \quad 1$

OK

<u>10.2.3 Resistance to in-plane buckling and bending</u>

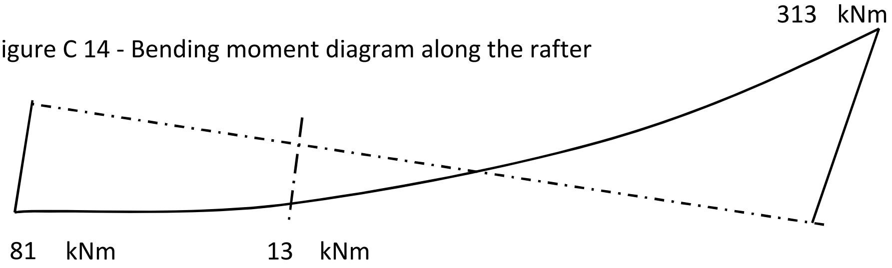
<u>Flexural buckling resistance about the major axis, $N_{b,y,Rd}$</u>

Reference	Calculations	Output
	$\frac{h}{b} = \frac{353}{126} = 2.80$	
	t_f = 10.7 mm	
	f_y = 355 N/mm^2	
	as before λ_1 = 76.41	
BS EN1993 -1-1 Table 6.2	For buckling about the y-y axis, curve **a** is used with α_y = 0.21	
Table 6.3	For a symmetric, single span, elastically designed portal frame of orthodox geometry,a reasonable approximation is to assume that the buckling length is the developed length from eaves to apex. Hence,	
Sec 7.4.4	$L_{cr} = \frac{12500}{\cos\ 10^0} = 12692.83$ mm	
	$(\lambda_{y)}^{-} = (L_{cr)/(}i_{y)}$ $1/(\lambda_{1)}$ $= \frac{12692.83}{143} \times \frac{1}{76.41}$	
	= 1.16	
BS EN1993 -1-1 Cl 6.3.1.2	$\Phi_y = 0.5\ [1+\alpha_y\ ((\lambda_{y)}^{-} - 0.2)+(\lambda_y)^{-\ 2]}$	
	$= 0.5\ \left(1 + 0.21\left(1.16 - 0.2\right) + 1.16^2\right)$	
	= 1.28	
	$\chi_y = 1/(\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y}$ $^{2))}$ $= \frac{1}{1.276 + \left(1.28^2 - 1.16^2\right)^{0.5}}$	
	= 0.55	
	$N_{b,y,Rd} = (\chi_y A f_{y)/(}\gamma_{M1)}$ $= \frac{0.55 \times 4980 \times 355}{1.0 \times 1000}$	
	= 980.582 kN	
	N_{Ed} = 83 kN < 980.582 kN OK	
	<u>Interaction of axial force and bending moment in accordance with Expression 6.61</u>	
	As noted earlier, in this situation, Expression 6.61 reduces to:	
Section 7.2	$(N_{Ed)/(}N_{b,y,Rd)} + k_{yy}(M_{y,Ed)/(}M_{b,Rd)} \leq 1.0$	
	The most onerous ratio of $(M_{y,Ed}$ from Zone B and C will be considered	
	in combination with the major axis flexural buckling.	
	Adjacent to the haunch $(M_{y,Ed} = \frac{150}{192.2} = 0.78$	
	Adjacent to the apex $(M_{y,Ed} = \frac{85}{218.5} = 0.39$	

Reference | Calculations | Output

The bending moment diagram along the entire length of the rafter is considered when determining C_{mv}

Figure C 14 - Bending moment diagram along the rafter



BS EN1993 -1-1 Table B.2

The interaction factor, k_{yy}, is calculated as follows:

$k_{yy}=\min [C_{my} (1+ (\bar{\lambda}_y -0.2)\ (N_{Ed)/(}N_{b,y,Rd))};C_{my} (1+0.8\ (N_{Ed)/(}N_{b,y,Rd))}]$

BS EN1993 -1-1 Table B.3

The expression for C_{my} depends on the value of α_s (the ratio of the midspan moment to the larger end moment) and ψ (the ratio of the end moments).

$$\psi = \frac{-81}{313} = -0.26$$

The midspan moment (determined from the analysis) = 13 kNm

$$\alpha_s = (M_{s)/(} = \frac{-13}{313} = -0.04$$

Because $-1 \leq \alpha_s < 0$ and $-1 \leq \psi < 0$, C_{my} is calculated as:

$C_{my} = 0.1\,(1-\psi) - 0.8\,\alpha_s$ but ≥ 0.4

$C_{my} = 0.1\ (\ 1 -(\ -0.26\ )\ ) - 0.8\ (\ -0.04\ )$

$= 0.16$

< 0.4

so $C_{my} = 0.4$

BS EN1993 -1-1 Table B.2 Table B.1

$$k_{yy} = \min \left\{ 0.4 \left(1 + \left(1.16 - 0.2\right) \frac{83}{980.582} \right); \quad 0.4 \left(1 + 0.8 \times \frac{83}{980.582} \right) \right\}$$

$= \min\ (\ 0.43\ ;\ 0.43\ )$

$= 0.43$

Using the most onerous $(M_{y,Ed}$ ratio

$$(N_{Ed)/(}N_{b,y,Rd)}+k = \frac{83}{980.582} + 0.43 \times 0.78$$

$= 0.42 < 1$

hence ok

10.2.4 Adequacy of the rafter section

The cross sectional resistance, flexural buckling resistance and lateral torsional buckling resistance have been demonstrated to be adequate. The interaction of lateral torsional buckling with both in-plane and out-of-plane flexural buckling has been verified using Expression 6.61 &6.62.

Therefore it is concluded that a 356x127x39 , S 355 is adequate for use as the rafter in this portal frame.

11.0 Verification of haunched length

The haunch is fabricated from a cutting of 356x127x39 ,S 355 section.

Checks are carried out at the end and the quarter points as shown.

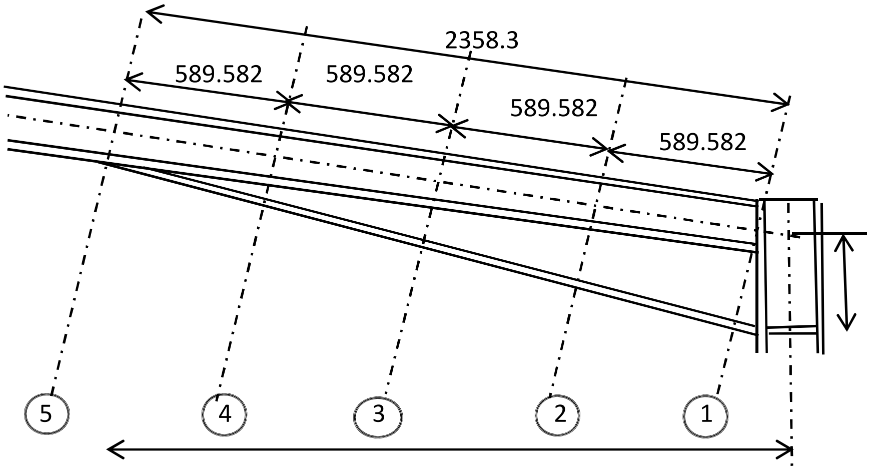


Figure C 15 - Zone A

sec	cutting depth	total depth	gross area	I $_y$	W $_{el,min}$	N $_{Ed}$	M $_{Ed}$
	mm	mm	mm ^{2}	mm ^{4}	cm ^{3}	kN	kNm
1	342.7	696.1	8.5E+03	4.91E+08	1.4E+06	83	313
2	257.025	610.425	8.0E+03	2.46E+08	7.9E+05	83	272.3
3	171.35	524.75	7.4E+03	1.88E+08	6.8E+05	83	231.5
4	85.675	439.075	6.8E+03	1.76E+08	7.3E+05	83	190.8
5	0	353.4	5.0E+03	1.08E+08	5.1E+05	83	150

Tabl C 19 - Section properties of Zone A

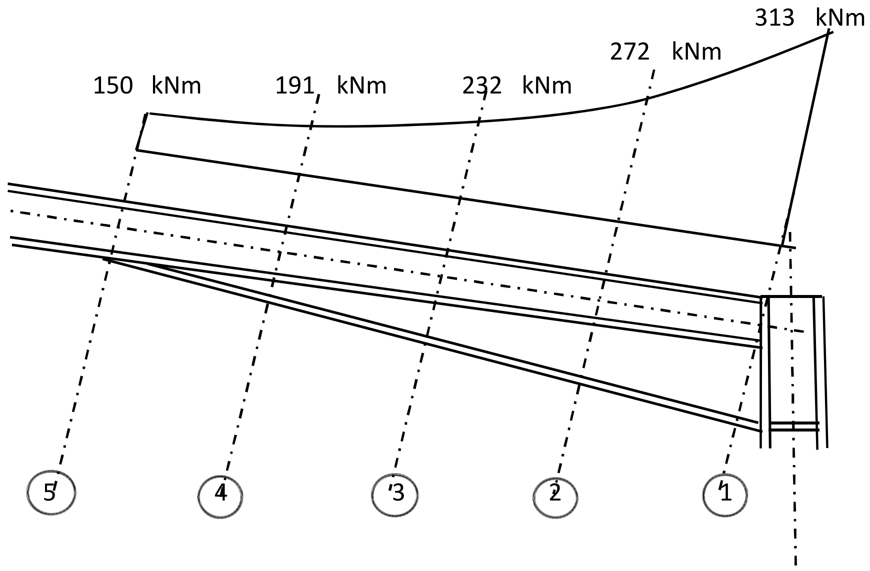
sectional properties are calculated normal to the longitudinal axis of the rafter section.

11.1 Cross section classification

The elements are classified assuming a conservative stress distribution, simply to identify whether they are class 2 or better. If the cross section is atleast class 2, the bending resistance will be calculated based on the plastic properties

In the gravity load combination, the web of the haunch is likely to be the critical element, especially at deeper cross sections.

Figure C 16- Moments in Zone A



If flanges are class 1 or 2, but the web is not, effective plastic properties are calculated.

As the axial compression in the rafter is small, most elements of the rafter section will be in tension under the gravity load combination(due to the large bending moment). It is therefore not necessary to classify the elements of the rafter section.

11.1.1 Calculation for the cross section 1

Haunch web

BS EN1993
-1-1
Table 5.2

Assuming the haunch web is subject only to compression(the most onerous condition).

t_w = 6.6 mm f_y = 355 N/mm

$\varepsilon = \sqrt{(235/(f_y))}$ = 0.814

Class 2 limit = 38 ε = 38 x 0.814 = 30.92

$\frac{C}{t_w} = \frac{321.8}{6.6}$ = 48.76 > 30.9

Haunch web is not class 2

Haunch flange

C = 49.50 mm t_f = 10.7 mm

$$C/T = \frac{49.50}{10.7} = 4.63$$

The limit for class 2 is 10 ε = 10 x 0.81 = 8.14

4.63 < 8.14

Therefore the flange is atleast class 2

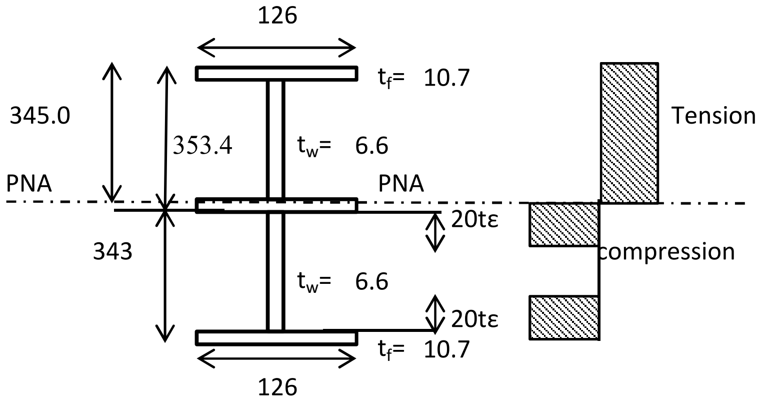
Effective plastic modulus -cross section 1

Sec 7.1.3

Because the haunch web is not class 2, effective properties will be calculated assuming the haunch web is only effective over a distance of '20 t ε' from the flanges. Conservatively ,the fillets have been ignored, and the '20 t ε' has been taken from the face of the flange, not from the end of the fillet.

20 t ε = 20 x 6.6 x 0.814
= 107.40 mm

Area of the haunch components are



Top flange	=	126	x	10.7	=	1348	mm^2
Rafter web	=	332	x	6.6	=	2191	mm^2
middle flange					=	1348	mm^2
20tε(upper) x t	=	107.40	x	6.6	=	709	mm^2
20tε(lower) x t					=	709	mm^2
haunch flange	=	126	x	10.7	=	1348	mm^2
					Σ =	7653	mm^2

If the position of the plastic neutral axis is 'x' mm down from the top of the middle flange, then

1348 + 2191 + 126 x = 1348 + 2 x 709 + (10.7 - x) x 126

then x = 2.28 mm

The plastic neutral axis is 345.0 mm from top of the compound section.

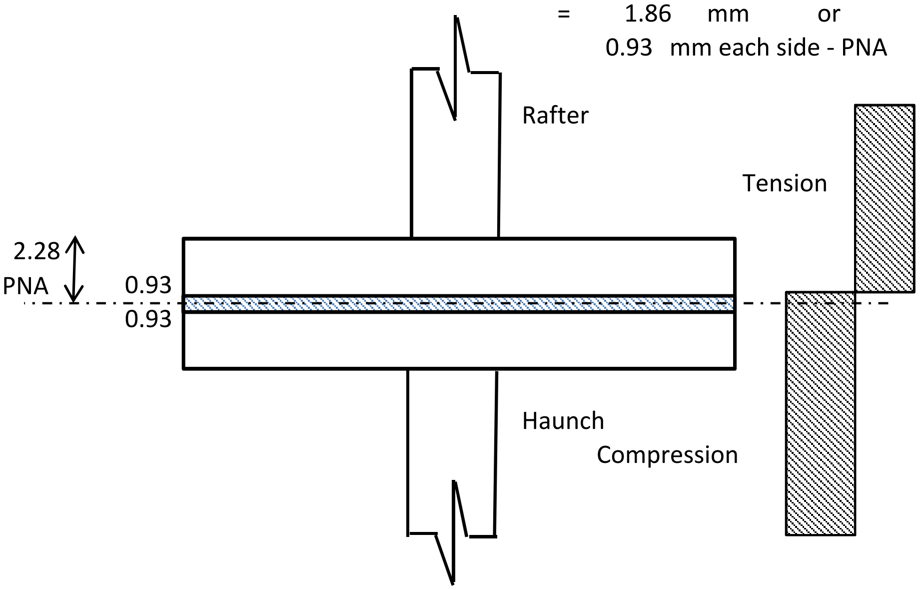
Part of the cross section, distributed equally about the plastic neutral axis, is utilised in carrying the axial compression.

For cross section 1, axial compression = 83 kN

Total depth of section carrying compression $= \dfrac{83 \times 10^{6}}{355 \times 126}$

$= 1.86$ mm or

0.93 mm each side - PNA



Bending resistance- cross section 1

The moment resistance of the effective section is the summation of the various areas multiplied by the lever arm from the plastic neutral axis, multiplied by the design strength. The areas allocated to carrying the axial compression are excluded from the calculation of moment resistance.

	area	Lever arm	Resistance
	mm2	mm	kNm
Top flange	1348.2	339.63	162.55
Rafter web	2191.2	0	0.00
Top of the middle flange	287.3	1.14	0.12
Bottom of the middle flange	1060.9	4.21	1.59
20tε(upper)	709	62.12	15.63
20tε(lower)	709	286.72	72.15
Bottom flange	1348	345.77	165.49
Σ	6305.2		
Area for axial compression (above PNA)	0.93 x 126	0.93 / 2	-0.02
Area for axial compression (below PNA)	0.93 x 126	0.93 / 2	-0.02
		Σ	417.48

Table C 20 - Bending resistance of cross section 1

Thuss $M_{c,Rd}$ = 417.48 kNm

M_{ed} = 313 kNm

313 kNm < 417.48 kNm ok

11.1.2 Calculation for the cross section 2

Haunch web

BS EN1993 -1-1 Table 5.2

Assuming the haunch web is subject only to compression(the most onerous condition).

t_w = 6.6 mm f_y = 355 N/mm

$\varepsilon = \sqrt{(235/(f_y))}$ = 0.814

Class 2 limit = 38 ε = 38 x 0.814 = 30.92

$\frac{C}{t_w} = \frac{236.1}{6.6}$ = 35.78 > 30.9

Haunch web is not class 2

Haunch flange

C = 49.50 mm t_f = 10.7 mm

C/T = $\frac{49.50}{10.7}$ = 4.63

The limit for class 2 is 10 ε = 10 x 0.81 = 8.14

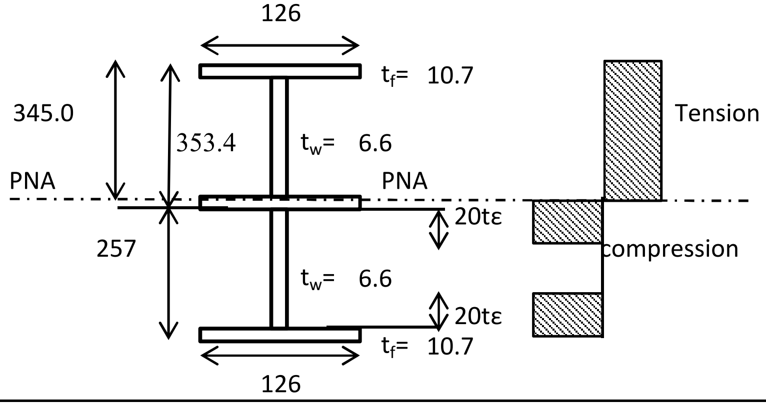
4.63 < 8.14

Therefore the flange is atleast class 2

Effective plastic modulus - cross section 2

20 t ε = 20 x 6.6 x 0.814
= 107.40 mm

Area of the haunch components are



Top flange = 126 x 10.7 = 1348 mm^2
Rafter web = 332 x 6.6 = 2191 mm^2
middle flange = 1348 mm^2
20tε(upper) x t = 107.40 x 6.6 = 709 mm^2
20tε(lower) x t = 709 mm^2
haunch flange = 126 x 10.7 = 1348 mm^2

Σ = 7653 mm^2

If the position of the plastic neutral axis is 'x' mm down from the top of the middle flange, then

1348 + 2191 + 126 x = 1348 + 2 x 709 + (10.7 - x) x 126
then x = 2.28 mm

The plastic neutral axis is 345.0 mm from top of the compound section.

Part of the cross section, distributed equally about the plastic neutral axis, is utilised in carrying the axial compression.
For cross section 2, axial compression = 83 kN

Total depth of section carrying compression $= \frac{83 \times 10^{6}}{355 \times 126}$
= 1.86 mm or
0.93 mm each side - PNA

<u>Bending resistance- cross section 2</u>

	area mm2	Lever arm mm	Resistance kNm
Top flange	1348	339.63	162.55
Rafter web	2191	168	130.90
Top of the middle flange	287.3	1.14	0.12
Bottom of the middle flange	1060.9	4.21	1.59
20tε(upper)	709	62.12	15.63
20tε(lower)	709	201.05	50.59
Bottom flange	1348	10.70	5.12
Σ	6305.2		
Area for axial compression (above PNA)	0.93 x 126	0.93 / 2	-0.02
Area for axial compression (below PNA)	0.93 x 126	0.93 / 2	-0.02
		Σ	366.46

Table C 21 - bending resistance of cross section 2

Thuss $M_{c,Rd}$ = 366.46 kNm

M_{ed} = 272 kNm

272 kNm < 366.46 kNm ok

Reference	Calculations	Output

11.1.3 Calculation for the cross section 3

Haunch web

BS EN1993 -1-1 Table 5.2

Assuming the haunch web is subject only to compression(the most onerous condition).

t_w = 6.6 mm f_y = 355 N/mm

$\varepsilon = \sqrt{(235/(f_y))}$ = 0.814

Class 2 limit = 38 ε = 38 x 0.814 = 30.92

$\frac{C}{t_w} = \frac{150.5}{6.6}$ = 22.80 < 30.9

hence haunch web is class 2

Haunch flange

C = 49.50 mm t_f = 10.7 mm

C/T = $\frac{49.50}{10.7}$ = 4.63

The limit for class 2 is 10 ε = 10 x 0.81 = 8.14

4.63 < 8.14

Therefore the flange is atleast class 2

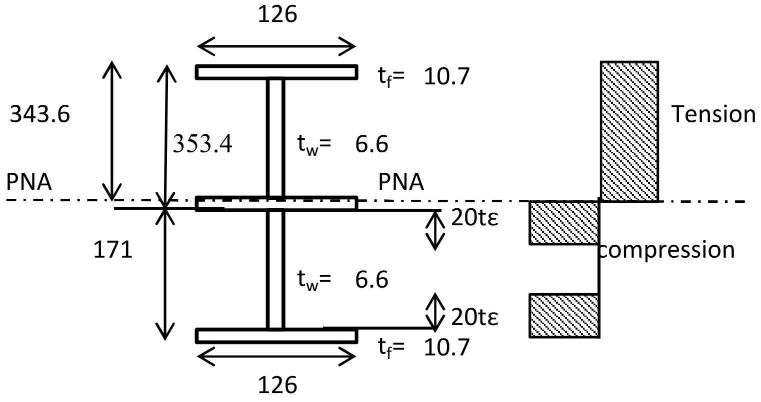
Effective plastic modulus - cross section 3

20 t ε = 20 x 6.6 x 0.814

= 107.40 mm

Since the area overlaps, total length of the web is effective.

Area of the haunch components are



Top flange = 126 x 10.7 = 1348 mm^2
Rafter web = 332 x 6.6 = 2191 mm^2
middle flange = 1348 mm^2
area of the web = 160.65 x 6.6 = 1060 mm^2
haunch flange = 126 x 10.7 = 1348 mm^2

Σ = 7296 mm^2

If the position of the plastic neutral axis is 'x' mm down from the top of the middle flange, then

1348 + 2191 + 126 x = 1348 + 1060.3 + (10.7 - x) x 126
then x = 0.86 mm

The plastic neutral axis is 343.6 mm from top of the compound section.

Part of the cross section, distributed equally about the plastic neutral axis, is utilised in carrying the axial compression.

For cross section 3, axial compression = 83 kN

Total depth of section carrying compression $= \dfrac{83 \times 10^6}{355 \times 126}$

= 1.86 mm or
0.93 mm each side - PNA

<u>Bending resistance- cross section 3</u>

	area	Lever arm	Resistance
	mm2	mm	kNm
Top flange	1348	338.21	161.87
Rafter web	2191	167	129.80
Top of the middle flange	108.6	0.43	0.02
Bottom of the middle flange	1239.6	4.92	2.16
Web of the haunch	1060	90.16	33.94
Bottom flange	1348	10.70	5.12
Σ	5947.9		
Area for axial compression (above PNA)	0.93 x 126	0.93 / 2	-0.02
Area for axial compression (below PNA)	0.93 x 126	0.93 / 2	-0.02
		Σ	332.87

Table C 22 - bending resistance of cross section 3

Thuss $M_{c,Rd}$ = 332.87 kNm

M_{ed} = 232 kNm

232 kNm < 332.87 kNm ok

11.1.4 Calculation for the cross section 4

Haunch web

BS EN1993 -1-1 Table 5.2

Assuming the haunch web is subject only to compression(the most onerous condition).

t_w = 6.6 mm f_y = 355 N/mm

$\varepsilon = \sqrt{(235/(f_y))}$ = 0.814

Class 2 limit = 38 ε = 38 x 0.814 = 30.92

$\frac{C}{t_w} = \frac{64.8}{6.6}$ = 9.81 < 30.9

hence haunch web is class 2

Haunch flange

C = 59.70 mm t_f = 10.7 mm

C/T = $\frac{59.70}{10.7}$ = 5.58

The limit for class 2 is 10 ε = 10 x 0.81 = 8.14

5.58 < 8.14

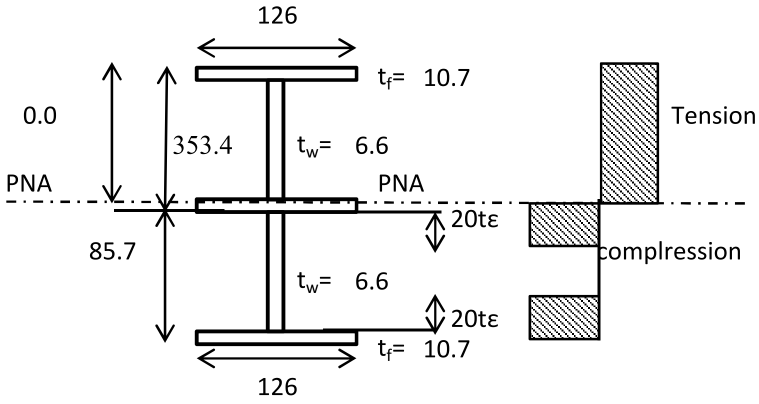
Therefore the flange is atleast class 2

Effective plastic modulus - cross section 4

20 t ε = 20 x 6.6 x 0.814
= 107.40 mm

Since the area overlaps, total length of the web is effective.

Area of the haunch components are



Top flange = 126 x 10.7 = 1348 mm^2
Rafter web = 332 x 6.6 = 2191 mm^2
middle flange = 1348 mm^2
area of the web = 74.98 x 6.6 = 495 mm^2
haunch flange = 126 x 10.7 = 1348 mm^2

Σ = 6731 mm^2

If the position of the plastic neutral axis is 'x' mm down from the top of the middle flange, then
1348 + 2191 + 126 x = 1348 + 494.8 + (10.7 - x) x 126
then x = -1.38 mm

The plastic neutral axis is 341.3 mm from top of the compound section.

Part of the cross section, distributed equally about the plastic neutral axis, is utilised in carrying the axial compression.
For cross section 4, axial compression = 83 kN

$$\text{Total depth of section carrying compression} = \frac{83 \times 10^{6}}{355 \times 6.6}$$

= 35.42 mm or
17.7 mm each side - PNA

<u>Bending resistance- cross section 4</u>

	area mm2	Lever arm mm	Resistance kNm
Top flange	1348	335.97	160.80
Rafter web	2191	165	128.05
Top of the middle flange	-174.1	-0.69	0.04
Bottom of the middle flange	1522.3	6.04	3.26
Web of the haunch	495	49.57	8.71
Bottom flange	1348	10.70	5.12
Σ	5382.4		
Area for axial compression (above PNA)	17.7 x 126	17.7 / 2	-7.02
Area for axial compression (below PNA)	17.7 x 126	17.7 / 2	-7.02
		Σ	291.95

Table C 23 - Bending resistance of cross section 4

Thuss $M_{c,Rd}$ = 291.95 kNm

M_{ed} = 191 kNm

191 kNm < 291.95 kNm ok

Table C 24-Summary of classification and bending resistance of cross sections

Cross sec no	Classification	Modulus	Bending resistance (kNm)
1	not class 2	Effective plastic modulus	417.48
2	not class 2	Effective plastic modulus	366.46
3	atleast class 2	Plastic	332.87
4	atleast class 2	Plastic	291.95
5	Class 1	Rafter alone	233.9

Shear resistance

The shear area of cross section 1 can be conservatively taken as:

$A_v = h_w t_{w,min}$

where h $_w$ is taken as the depth between the top and bottom flanges of the compound section.

$A_v = (353.4 + 342.70 - 10.7 \times 2) \times 6.6$

$= 4453.02 \text{ nm}^2$

BS EN1993 -1-1 Cl 6.2.6

$V_{pl,Rd} = (A_v f_{y/\sqrt{}}3)/(\gamma_{M0})$

$$= \frac{4453.02 \times (355 / 3^{0.5})}{1.0}$$

$= 913 \text{ kN}$

$V_{Ed} = 91 < 912.69$ ok

BS EN1993 -1-1 Cl 6.2.8

Bending and shear interaction

When shear force and bending moment act simultaneously on a cross section, the effect of the shear foce can be ignored if it is less than 50 % of the plastic shear resistance.

$0.5 V_{pl,Rd} = 0.5 \times 912.69$

$= 456.3 \text{ kN}$

$V_{Ed} = 91 < 456.34$ ok

Therefore the effect of the shear force on the moment resistance may be neglected.

Calculation for all the cross sections are summarised below.

Reference	Calculations	Output

Cross sec	V_{Ed} (kN)	A_v (mm ^{2})	$V_{pl,Rd}$ (kN)	$V_{Ed} \leq V_{Rd}$	0.5 V_{Rd} (kN)	Bending and shear interaction
1	91	4453.02	913	yes	456.344	no
2	86	3887.57	797	yes	398.396	no
3	81	3322.11	681	yes	340.449	no
4	76	2756.66	565	yes	282.501	no
5	70	2191.20	449	yes	224.553	no

Table C 25 - Calculation of bending and shear interaction

Compression resistance

The compression resistance of cross section 1, using the effective area calculated above is given by:

$$N_{c,Rd}=(A\ f_{y)/(}\gamma_{M0)} \quad = \frac{7653.45 \ \text{x} \ 355}{1.0 \ \text{x} \ 1000} \quad = \quad 2716.97 \ \text{kN}$$

N_{Ed} = 83 kN < 2716.97 kN ok

Bending and axial force interaction

BS EN1993
-1-1
Cl 6.2.1(7)

Following conservative criterian may be used.

$$(N_{Ed)/(}N_{Rd)}+M_{Ed}$$

For cross section 1,

$$(N_{Ed)/(}N_{Rd)}+M \quad = \frac{83}{2716.97} + \frac{313}{417.48} \quad = \quad 0.78 \ < \ 1 \quad \text{ok}$$

Calculation for all the cross sections are summarised below.

Cross sec	N_{Ed} (kN)	$N_{c,Rd}$ (kN)	M_{Ed} (kNm)	$M_{c,Rd}$ (kNm)	$(N_{Ed)/(}N_{Rd}$	
1	83	2716.97	313	417.48	0.78	ok
2	83	2716.97	272	366.46	0.77	ok
3	83	2590.11	232	332.87	0.73	ok
4	83	6730.64	191	291.95	0.67	ok
5	83	1767.90	150	233.95	0.69	ok

Table C 26 - Adequacy of cross sections

Because neither shear nor compression reduces the bending resistance of the cross section, bending resisttances remain as calculated above.

11.2 Bucklig resistance

Section 7.4

The elastic verification of a haunched member is not covered explicitly in the Eurocode.
Hence the stability of the haunch is verified by considering an equivalent compression flange, following the principle is illustrated in Clause 6.3.2.4 of BS EN 1993-1-1.

The equivalent compression flange is verified, composed of the compression flange plus 1/3 the compressed part of the web. The resistance of this Tee- shaped equivalent flange is compared to the force in the Tee arising from the axial compression and the bending moment.

There are restraints to the compression flange adjacent to the column, and at the sharp end of the haunch as shown below.

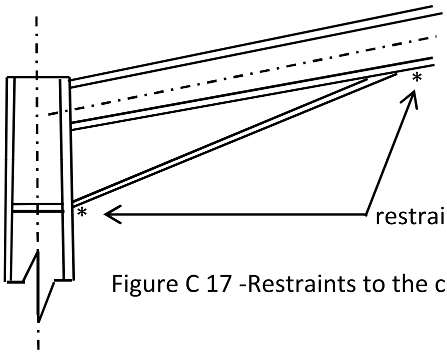


Figure C 17 -Restraints to the compression flange

The dimensions of the equivalent compression flange are determined at a point 1/3 of the haunch length from the deepest section (adjacent to the column).

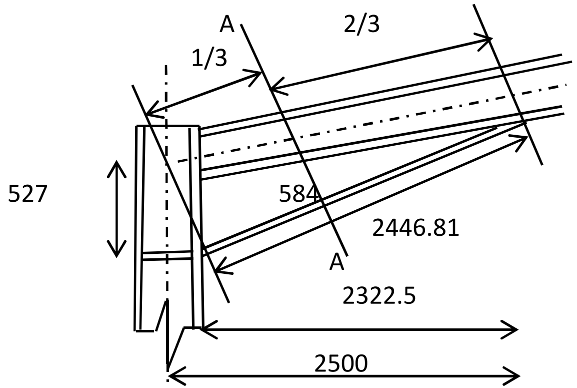


Figure C 18 -Cross section used to determine equivalent compression flange

At the cross section A-A , the overall depth is 584 mm.
The depth of the web between flanges is therefore = 584 - 2x 11.5
= 570.5 mm

Reference	Calculations	Output

Assuming that half the web is in compression,

1/3 of the compressed part of the web = 570.5 / 6 = 95.1 mm.

The dimensions of the equivalent compression flange, ignoring the root radius , are shown bellow.

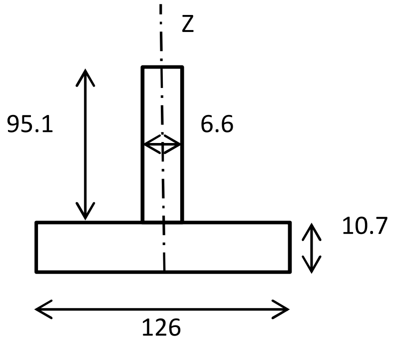


Figure C 19- Equivalent compression flange

A = 6.6x95.0833333333333+1 = 1975.75 mm ^{2}

$$I_z = \frac{95.1\ \text{x}\ 6.6^{\,3}}{12} + \frac{10.7\ \text{x}\ 126^{\,3}}{12}$$

= 1.79E+06 mm ^{2}

$$i_z = \left(\frac{1.79\text{E+06}}{1975.75}\right)^{0.5} = 30.1\ \text{mm}$$

To calculate the flexural buckling resistance

BS EN1993 -1-1 Cl 6.3.1.3

$(\lambda)^{-}= (L_{\,cr)/(}i\,)$ where , as before, λ_1 = 76.41

$$\bar{\lambda} = \frac{2447}{30\ \text{x}\ 76.41} = 1.07$$

BS EN1993 -1-1 Cl 6.3.1.2 Table 6.2

Curve c

Table 6.1

α = 0.49

BS EN1993 -1-1 Cl 6.3.1.2

$\Phi_y=0.5\ [1+\alpha_y\ ((\lambda_{y)^{-}}-0.2)+(\lambda_y)^{-\ 2]}$

= 0.5 (1 + 0.49 (1.07 - 0.2) + 1.07 ^{2})

= 1.28

$\chi_y = 1/(\Phi_y + \sqrt{}(\Phi_y^{2} - \bar{\lambda}_y^{\ 2))}$

$$= \frac{1}{1.279 + \left(1.28^{2} - 1.07^{2}\right)^{0.5}}$$

= 0.50

$N_{b,y,Rd} = (\chi_y\, A\, f_{\,y)/(}\gamma_{M1)}$

$$= \frac{0.50\ \text{x}\ 1976\ \text{x}\ 355}{1.0\ \text{x}\ 1000}$$

= 352.891 kN

Reference | Calculations | Output

From calculations

The maximum compressive stress $= \frac{83000}{4980.0} = 16.7 \text{ N/mm}^2$

The maximum bending stress is at cross section 5

$$= \frac{150000000}{512291603.0} = 0.29 \text{ N/mm}^2$$

The force in the equivalent compression flange (assuming uniform, maximum stress)

$$= \frac{1975.75 \times (16.7 + 0.29)}{1000}$$

$$= 33.5 \text{ kN}$$

33.5 kN < 352.89

The haunch is stable between restraints to the compression flange.

12.0 Deflection

The deflections of the portal frame subject to characteristic load values of action are shown below.

Action	horizontal		vertical	
	δ	h/100 =45	δ	b/200=125
Imposed	11	ok	52	ok
Wind 1	24	ok	57	ok
Wind 2	26	ok	13	ok

Table C 27 - Deflection

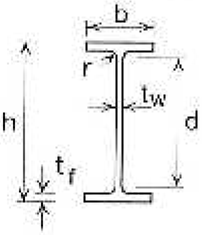
Appendix D:

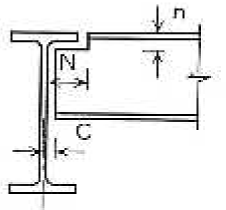
Steel universal beams - property table

BS EN 1993-1-1:2005
BS 4-1:2005

UNIVERSAL BEAMS

Advance UKB





Dimensions

Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area	
				Web	Flange			Flange	Web	End Clearance	Notch		For Metre	Per Tonne
		h	b	t_w	t_f	r	d	c_f/t_f	c_w/t_w	C	N	n		
	kg/m	mm	mm	mm	mm	mm	mm			mm	mm	mm	m^2	m^2
1016x305x487 +	485.7	1036.3	308.5	30.0	54.1	30.0	868.1	2.02	28.9	17	150	86	3.20	6.58
1016x305x437 +	437.0	1026.1	305.4	26.9	49.0	30.0	868.1	2.23	32.3	15	150	80	3.17	7.25
1016x305x393 +	392.7	1015.9	303.0	24.4	43.9	30.0	868.1	2.49	35.6	14	150	74	3.14	8.00
1016x305x349 +	349.4	1008.1	302.0	21.1	40.0	30.0	868.1	2.76	41.1	13	152	70	3.13	8.96
1016x305x314 +	314.3	999.9	300.0	19.1	35.9	30.0	868.1	3.08	45.5	12	152	66	3.11	9.89
1016x305x272 +	272.3	990.1	300.0	16.5	31.0	30.0	868.1	3.60	52.6	10	152	62	3.10	11.4
1016x305x249 +	248.7	980.1	300.0	16.5	26.0	30.0	868.1	4.30	52.6	10	152	56	3.08	12.4
1016x305x222 +	222.0	970.3	300.0	16.0	21.1	30.0	858.1	5.31	54.3	10	152	52	3.06	13.8
914x419x388	388.0	921.0	420.5	21.4	36.6	24.1	799.6	4.78	37.4	13	210	62	3.44	8.87
914x419x343	343.3	911.8	418.5	19.4	32.0	24.1	799.6	5.48	41.2	12	210	58	3.42	9.96
914x305x289	289.1	926.6	307.7	19.5	32.0	19.1	824.4	3.91	42.3	12	156	62	3.01	10.4
914x305x253	253.4	918.4	305.5	17.3	27.9	19.1	824.4	4.45	47.7	11	156	48	2.99	11.8
914x305x224	224.2	910.4	304.1	15.9	23.9	19.1	824.4	5.23	51.8	10	156	44	2.97	13.2
914x305x201	200.9	903.0	303.3	15.1	20.2	19.1	824.4	6.19	54.6	10	156	40	2.96	14.7
838x292x226	226.5	850.9	293.8	16.1	26.8	17.8	761.7	4.52	47.3	10	150	46	2.81	12.4
838x292x194	193.8	840.7	292.4	14.7	21.7	17.8	761.7	5.58	51.8	9	150	40	2.79	14.4
838x292x176	175.9	834.9	291.7	14.0	18.8	17.8	761.7	6.44	54.4	9	150	38	2.78	15.8
762x267x197	196.8	769.8	268.0	15.6	25.4	16.5	686.0	4.32	44.0	10	138	42	2.55	13.0
762x267x173	173.0	762.2	266.7	14.3	21.6	16.5	686.0	5.08	48.0	9	138	40	2.53	14.6
762x267x147	146.9	754.0	265.2	12.8	17.5	16.5	686.0	6.27	53.6	8	138	34	2.51	17.1
762x267x134	133.9	750.0	264.4	12.0	15.5	16.5	686.0	7.08	57.2	8	138	32	2.51	18.7
686x254x170	170.2	692.9	255.8	14.5	23.7	15.2	615.1	4.45	42.4	9	132	40	2.35	13.8
686x254x152	152.4	687.5	254.5	13.2	21.0	15.2	615.1	5.02	46.6	9	132	38	2.34	15.4
686x254x140	140.1	683.5	253.7	12.4	19.0	15.2	615.1	5.55	49.6	8	132	36	2.33	16.6
686x254x125	125.2	677.9	253.0	11.7	16.2	15.2	615.1	6.51	52.6	8	132	32	2.32	18.5
610x305x238	238.1	635.8	311.4	18.4	31.4	16.5	540.0	4.14	29.3	11	158	48	2.45	10.3
610x305x179	179.0	620.2	307.1	14.1	23.6	16.5	540.0	5.51	38.3	9	158	42	2.41	13.5
610x305x149	149.2	612.4	304.8	11.8	19.7	16.5	540.0	6.60	45.8	8	158	30	2.39	16.0
610x229x140	139.9	617.2	230.2	13.1	22.1	12.7	547.6	4.34	41.8	9	120	36	2.11	15.1
610x229x125	125.1	612.2	229.0	11.9	19.6	12.7	547.6	4.89	46.0	8	120	34	2.09	16.7
610x229x113	113.0	607.6	228.2	11.1	17.3	12.7	547.6	5.54	49.3	8	120	30	2.08	18.4
610x229x101	101.2	602.6	227.6	10.5	14.8	12.7	547.6	6.48	52.2	7	120	28	2.07	20.5
610x178x100 +	100.3	607.4	179.2	11.3	17.2	12.7	547.6	4.14	48.5	8	94	30	1.89	18.8
610x178x92 +	92.2	603.0	178.8	10.9	15.0	12.7	547.6	4.75	50.2	7	94	28	1.88	20.4
610x178x82 +	81.8	598.6	177.9	10.0	12.8	12.7	547.6	5.57	54.8	7	94	26	1.87	22.9
533x312x273 +	273.3	577.1	320.2	21.1	37.6	12.7	476.5	3.64	22.6	13	160	52	2.37	8.67
533x312x219 +	218.8	560.3	317.4	18.3	29.2	12.7	476.5	4.69	26.0	11	160	42	2.33	10.7
533x312x182 +	181.5	550.7	314.5	15.2	24.4	12.7	476.5	5.61	31.3	10	160	38	2.31	12.7
533x312x151 +	150.6	542.5	312.0	12.7	20.3	12.7	476.5	6.75	37.5	8	160	34	2.29	15.2

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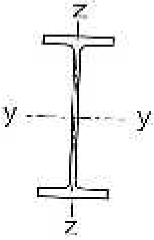
+ These sections are in addition to the range of BS 4 sections.

FOR EXPLANATION OF TABLES SEE NOTE 2

BS EN 1993-1-1:2005
BS 4-1:2005

UNIVERSAL BEAMS

Advance UKB



Properties

Section Designation	Second Moment of Area		Radius of Gyration		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
	Axis y-y	Axis z-z	Axis y-y	Axis z-z	Axis y-y	Axis z-z	Axis y-y	Axis z-z	U	X	I_w	I_T	A
	cm^4	cm^4	cm	cm	cm^3	cm^3	cm^3	cm^3			dm^6	cm^4	cm^2
1016x305x487 +	1022000	26700	40.6	6.57	19700	1730	23200	2800	0.867	21.1	64.4	4300	620
1016x305x437 +	910000	23400	40.4	6.49	17700	1540	20500	2470	0.868	23.1	56.0	3190	557
1016x305x393 +	808000	20500	40.2	6.40	15900	1350	18500	2170	0.868	25.5	48.4	2330	500
1016x305x349 +	723000	18500	40.3	6.44	14300	1220	16600	1940	0.872	27.9	43.3	1720	445
1016x305x314 +	644000	16200	40.1	6.37	12900	1060	14800	1710	0.872	30.7	37.7	1260	400
1016x305x272 +	554000	14000	40.0	6.35	11200	934	12800	1470	0.872	35.0	32.2	835	347
1016x305x249 +	481000	11800	38.9	6.09	9820	701	11000	1240	0.861	39.9	26.8	582	317
1016x305x222 +	408000	9550	38.0	5.81	8410	636	9810	1020	0.850	45.7	21.5	390	283
914x419x388	720000	45400	38.2	9.59	15600	2160	17700	3340	0.885	26.7	88.9	1730	494
914x419x343	626000	39200	37.8	9.46	13700	1870	15500	2890	0.883	30.1	75.8	1190	437
914x305x289	504000	15600	37.0	6.51	10900	1010	12600	1600	0.867	31.9	31.2	926	368
914x305x253	436000	13300	36.8	6.42	9500	871	10900	1370	0.865	36.2	26.4	626	323
914x305x224	376000	11200	36.3	6.27	8270	739	9530	1150	0.860	41.3	22.1	422	285
914x305x201	325000	9420	35.7	6.07	7200	621	8350	952	0.853	46.9	18.4	291	256
838x292x226	340000	11400	34.3	6.27	7980	770	9160	1210	0.869	35.0	19.3	514	289
838x292x194	279000	9070	33.6	6.06	6640	620	7640	974	0.862	41.6	16.2	306	247
838x292x176	246000	7800	33.1	5.90	5890	535	6810	842	0.856	46.5	13.0	221	224
762x267x197	240000	8170	30.9	5.71	6230	610	7170	958	0.869	33.1	11.3	404	251
762x267x173	205000	6850	30.5	5.58	5390	514	6200	807	0.865	38.0	9.39	267	220
762x267x147	169000	5460	30.0	5.40	4470	411	5160	647	0.858	45.2	7.40	159	187
762x267x134	151000	4790	29.7	5.30	4020	362	4640	570	0.853	49.8	6.46	119	171
686x254x170	170000	6630	28.0	5.53	4920	518	5630	811	0.872	31.8	7.42	308	217
686x254x152	150000	5780	27.8	5.46	4370	455	5000	710	0.871	35.4	6.42	220	194
686x254x140	136000	5180	27.6	5.39	3990	409	4560	638	0.870	40.0	5.72	169	178
686x254x125	118000	4380	27.2	5.24	3480	346	3990	542	0.863	43.8	4.80	116	159
610x305x238	209000	15800	26.3	7.23	6590	1020	7490	1570	0.886	21.3	14.5	785	303
610x305x179	153000	11400	25.9	7.07	4930	743	5550	1140	0.886	27.7	10.2	340	228
610x305x149	126000	9310	25.7	7.00	4110	611	4590	937	0.886	32.7	8.17	200	190
610x229x140	112000	4510	25.0	5.03	3620	391	4140	611	0.875	30.6	3.99	216	178
610x229x125	98600	3930	24.9	4.97	3220	343	3680	535	0.875	34.0	3.45	154	159
610x229x113	87300	3430	24.6	4.88	2870	301	3280	469	0.870	38.0	2.99	111	144
610x229x101	75800	2910	24.2	4.75	2520	256	2880	400	0.863	43.0	2.52	77.0	129
610x178x100 +	72500	1660	23.6	3.60	2390	185	2790	296	0.854	38.7	1.44	95.0	128
610x178x92 +	64600	1440	23.4	3.50	2140	161	2510	258	0.850	42.7	1.24	71.0	117
610x178x82 +	55900	1210	23.2	3.40	1870	136	2190	218	0.843	48.5	1.04	48.8	104
533x312x273 +	199000	20600	23.9	7.69	6890	1290	7870	1990	0.891	15.9	15.0	1290	348
533x312x219 +	151000	15600	23.3	7.48	5400	982	6120	1510	0.884	19.8	11.0	642	279
533x312x182 +	123000	12700	23.1	7.40	4480	806	5040	1240	0.885	23.4	8.77	373	231
533x312x151 +	101000	10300	22.9	7.32	3710	659	4150	1010	0.885	27.8	7.01	216	192

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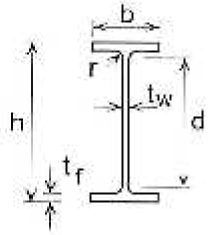
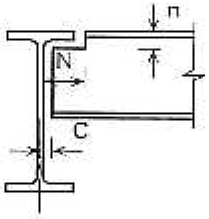
FOR EXPLANATION OF TABLES SEE NOTE 3

BS EN 1993-1-1:2005
BS 4-1:2005

UNIVERSAL BEAMS

Advance UKB





Dimensions

Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area	
				Web	Flange			Flange	Web	End Clearance	Notch		Per Metre	Per Tonne
		h	b	t_w	t_f	r	d	c_f/t_f	c_w/t_w	C	N	n		
	kg/m	mm	mm	mm	mm	mm	mm			mm	mm	mm	m^2	m^2
533x210x138 +	138.3	549.1	213.9	14.7	23.6	12.7	476.5	3.69	32.4	9	110	38	1.90	13.7
533x210x122	122.0	544.5	211.9	12.7	21.3	12.7	476.5	4.08	37.5	8	110	34	1.89	15.5
533x210x109	109.0	539.5	210.8	11.6	18.8	12.7	476.5	4.62	41.1	8	110	32	1.88	17.2
533x210x101	101.0	536.7	210.0	10.8	17.4	12.7	476.5	4.99	44.1	7	110	32	1.87	18.5
533x210x92	92.1	533.1	209.3	10.1	15.6	12.7	476.5	5.57	47.2	7	110	30	1.86	20.2
533x210x82	82.2	528.3	208.8	9.6	13.2	12.7	476.5	6.58	49.6	7	110	26	1.85	22.5
533x165x85 +	84.8	534.9	166.5	10.3	16.5	12.7	476.5	3.96	46.3	7	90	30	1.69	19.9
533x165x75 +	74.7	529.1	165.9	9.7	13.6	12.7	476.5	4.81	49.1	7	90	28	1.68	22.5
533x165x66 +	65.7	524.7	165.1	8.9	11.4	12.7	476.5	5.74	53.5	6	90	26	1.67	25.4
457x191x161 +	161.4	492.0	199.4	18.0	32.0	10.2	407.6	2.52	22.6	11	102	44	1.73	10.7
457x191x133 +	133.3	480.6	196.7	15.3	26.3	10.2	407.6	3.06	26.6	10	102	38	1.70	12.8
457x191x106 +	105.8	469.2	194.0	12.6	20.6	10.2	407.6	3.91	32.3	8	102	32	1.67	15.8
457x191x98	98.3	467.2	192.8	11.4	19.6	10.2	407.6	4.11	35.8	8	102	30	1.67	17.0
457x191x89	89.3	463.4	191.9	10.5	17.7	10.2	407.6	4.55	38.8	7	102	28	1.66	18.6
457x191x82	82.0	460.0	191.3	9.9	16.0	10.2	407.6	5.03	41.2	7	102	28	1.65	20.1
457x191x74	74.3	457.0	190.4	9.0	14.5	10.2	407.6	5.56	45.3	7	102	26	1.64	22.1
457x191x67	67.1	453.4	189.9	8.5	12.7	10.2	407.6	6.34	48.0	6	102	24	1.63	24.3
457x152x82	82.1	465.8	155.3	10.5	18.9	10.2	407.6	3.29	38.8	7	84	30	1.51	18.4
457x152x74	74.2	462.0	154.4	9.6	17.0	10.2	407.6	3.66	42.5	7	84	28	1.50	20.2
457x152x67	67.2	458.0	153.8	9.0	15.0	10.2	407.6	4.15	45.3	7	84	26	1.50	22.3
457x152x60	59.8	454.6	152.9	8.1	13.3	10.2	407.6	4.68	50.3	6	84	24	1.49	24.9
457x152x52	52.3	449.8	152.4	7.6	10.9	10.2	407.6	5.71	53.6	6	84	22	1.48	28.3
406x178x85 +	85.3	417.2	181.9	10.9	18.2	10.2	360.4	4.14	33.1	7	96	30	1.52	17.8
406x178x74	74.2	412.8	179.5	9.5	16.0	10.2	360.4	4.68	37.9	7	96	28	1.51	20.4
406x178x67	67.1	409.4	178.8	8.8	14.3	10.2	360.4	5.23	41.0	6	96	26	1.50	22.3
406x178x60	60.1	406.4	177.9	7.9	12.8	10.2	360.4	5.84	45.6	6	96	24	1.49	24.8
406x178x54	54.1	402.6	177.7	7.7	10.9	10.2	360.4	6.86	46.8	6	96	22	1.48	27.3
406x140x53 +	53.3	406.6	143.3	7.9	12.9	10.2	360.4	4.46	45.6	6	78	24	1.35	25.3
406x140x46	46.0	403.2	142.2	6.8	11.2	10.2	360.4	5.13	53.0	5	78	22	1.34	29.1
406x140x39	39.0	398.0	141.8	6.4	8.6	10.2	360.4	6.69	56.3	5	78	20	1.33	34.1
356x171x67	67.1	363.4	173.2	9.1	15.7	10.2	311.6	4.58	34.2	7	94	26	1.38	20.6
356x171x57	57.0	358.0	172.2	8.1	13.0	10.2	311.6	5.53	38.5	6	94	24	1.37	24.1
356x171x51	51.0	355.0	171.5	7.4	11.5	10.2	311.6	6.25	42.1	6	94	22	1.36	26.7
356x171x45	45.0	351.4	171.1	7.0	9.7	10.2	311.6	7.41	44.5	6	94	20	1.36	30.2
356x127x39	39.1	353.4	126.0	6.6	10.7	10.2	311.6	4.63	47.2	5	70	22	1.18	30.2
356x127x33	33.1	349.0	125.4	6.0	8.5	10.2	311.6	5.82	51.9	5	70	20	1.17	35.4
305x165x54	54.0	310.4	166.9	7.9	13.7	8.9	265.2	5.15	33.6	6	90	24	1.26	23.3
305x165x46	46.1	306.6	165.7	6.7	11.8	8.9	265.2	5.98	39.6	5	90	22	1.25	27.1
305x165x40	40.3	303.4	165.0	6.0	10.2	8.9	265.2	6.92	44.2	5	90	20	1.24	30.8

Advance and UKB are trademarks of Corus. A fuller description of the relationship between Universal Beams (UB) and the Advance range of sections manufactured by Corus is given in section 12.

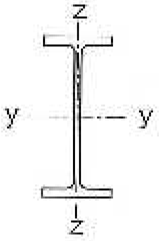
+ These sections are in addition to the range of BS 4 sections.

FOR EXPLANATION OF TABLES SEE NOTE 2

BS EN 1993-1-1:2005
BS 4-1:2005

UNIVERSAL BEAMS

Advance UKB



Properties

Section Designation	Second Moment of Area		Radius of Gyration		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
	Axis y-y	Axis z-z	Axis y-y	Axis z-z	Axis y-y	Axis z-z	Axis y-y	Axis z-z	U	X	I_w	I_T	A
	cm⁴	cm⁴	cm	cm	cm³	cm³	cm³	cm³			dm⁶	cm⁴	cm²
533x210x138 +	86100	3860	22.1	4.68	3140	361	3610	568	0.874	24.9	2.67	250	176
533x210x122	76000	3390	22.1	4.67	2790	320	3200	500	0.878	27.6	2.32	178	155
533x210x109	66800	2940	21.9	4.60	2480	279	2830	436	0.875	30.0	1.99	120	139
533x210x101	61500	2690	21.9	4.57	2290	256	2610	399	0.874	33.1	1.81	101	129
533x210x92	55200	2390	21.7	4.51	2070	228	2360	355	0.873	36.4	1.60	75.7	117
533x210x82	47500	2010	21.3	4.38	1800	192	2060	300	0.863	41.6	1.33	51.5	105
533x165x85 +	48500	1270	21.2	3.44	1820	153	2100	243	0.861	35.5	0.857	73.8	108
533x165x75 +	41100	1040	20.8	3.30	1550	125	1810	200	0.853	41.1	0.691	47.9	95.2
533x165x66 +	35000	859	20.5	3.20	1340	104	1560	166	0.847	47.0	0.566	32.0	83.7
457x191x161 +	79800	4250	19.7	4.55	3240	426	3780	672	0.881	16.5	2.25	515	206
457x191x133 +	63800	3350	19.4	4.44	2660	341	3070	535	0.879	19.6	1.73	292	170
457x191x106 +	48900	2510	19.0	4.32	2080	259	2390	405	0.876	24.4	1.27	146	135
457x191x98	45700	2350	19.1	4.33	1960	243	2230	379	0.881	25.8	1.18	121	125
457x191x89	41000	2090	19.0	4.29	1770	218	2010	338	0.878	28.3	1.04	90.7	114
457x191x82	37100	1870	18.8	4.23	1610	196	1830	304	0.879	30.8	0.922	69.2	104
457x191x74	33300	1670	18.8	4.20	1460	176	1650	272	0.877	33.8	0.818	51.8	94.6
457x191x67	29400	1450	18.5	4.12	1300	153	1470	237	0.873	37.8	0.705	37.1	85.5
457x152x82	36600	1180	18.7	3.37	1570	153	1810	240	0.872	27.4	0.591	89.2	105
457x152x74	32700	1050	18.6	3.33	1410	136	1630	213	0.872	30.1	0.518	65.9	94.5
457x152x67	28900	913	18.4	3.27	1260	119	1450	187	0.868	33.6	0.448	47.7	85.6
457x152x60	25500	795	18.3	3.23	1120	104	1290	163	0.868	37.5	0.387	33.8	76.2
457x152x52	21400	645	17.9	3.11	950	84.6	1100	133	0.859	43.8	0.311	21.4	66.6
406x178x85 +	31700	1830	17.1	4.11	1520	201	1730	313	0.880	24.4	0.728	93.0	109
406x178x74	27300	1550	17.0	4.04	1320	172	1500	267	0.882	27.5	0.608	62.8	94.5
406x178x67	24300	1360	16.9	3.99	1190	153	1350	237	0.880	30.4	0.533	46.1	85.5
406x178x60	21600	1200	16.8	3.97	1060	135	1200	209	0.880	33.7	0.466	33.3	76.5
406x178x54	18700	1020	16.5	3.85	930	115	1060	178	0.871	38.3	0.392	23.1	69.0
406x140x53 +	18300	635	16.4	3.06	899	88.6	1030	139	0.870	34.1	0.246	29.0	67.9
406x140x46	15700	538	16.4	3.03	778	75.7	888	118	0.871	39.0	0.207	19.0	58.6
406x140x39	12500	410	15.9	2.87	629	57.8	724	90.8	0.858	47.4	0.155	10.7	49.7
356x171x67	19500	1360	15.1	3.99	1070	157	1210	243	0.886	24.4	0.412	55.7	85.5
356x171x57	16000	1110	14.9	3.91	896	129	1010	199	0.882	28.8	0.330	33.4	72.6
356x171x51	14100	968	14.8	3.86	796	113	896	174	0.881	32.1	0.286	23.8	64.9
356x171x45	12100	811	14.5	3.76	687	94.8	775	147	0.874	36.8	0.237	15.8	57.3
356x127x39	10200	358	14.3	2.68	576	56.8	659	89.0	0.871	35.2	0.105	15.1	49.8
356x127x33	8250	280	14.0	2.58	473	44.7	543	70.2	0.863	42.1	0.081	8.79	42.1
305x165x54	11700	1060	13.0	3.93	754	127	846	196	0.889	23.6	0.234	34.8	68.8
305x165x46	9900	896	13.0	3.90	646	108	720	166	0.890	27.1	0.195	22.2	58.7
305x165x40	8500	764	12.9	3.86	560	92.6	623	142	0.889	31.0	0.164	14.7	51.3

Advance and UKB are trademarks of Corus. A fuller description of the relationship between Universal Beams (UB) and the Advance range of sections manufactured by Corus is given in section 12.

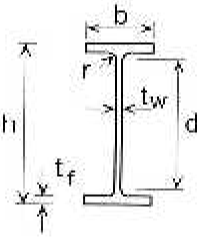
+ These sections are in addition to the range of BS 4 sections.

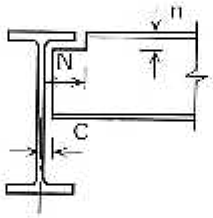
FOR EXPLANATION OF TABLES SEE NOTE 3

BS EN 1993-1-1:2005
BS 4-1:2005

UNIVERSAL BEAMS

Advance UKB





Dimensions

Section Designation	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Dimensions for Detailing			Surface Area	
				Web	Flange			Flange	Web	End Clearance	Notch		Per Metre	Per Tonne
		h	b	t_w	t_f	r	d	c_f / t_f	c_w / t_w	C	N	n		
	kg/m	mm	mm	mm	mm	mm	mm			mm	mm	mm	m^2	m^2
305x127x48	48.1	311.0	125.3	9.0	14.0	8.9	265.2	3.52	29.5	7	70	24	1.09	22.7
305x127x42	41.8	307.2	124.3	8.0	12.1	8.9	265.2	4.07	33.2	6	70	22	1.08	25.8
305x127x37	37.0	304.4	123.4	7.1	10.7	8.9	265.2	4.60	37.4	6	70	20	1.07	28.9
305x102x33	32.8	312.7	102.4	6.6	10.8	7.6	275.9	3.73	41.8	5	58	20	1.01	30.8
305x102x28	28.2	308.7	101.8	6.0	8.8	7.6	275.9	4.58	46.0	5	58	18	1.00	35.5
305x102x25	24.8	305.1	101.6	5.8	7.0	7.6	275.9	5.76	47.6	5	58	16	0.992	40.0
254x146x43	43.0	259.6	147.3	7.2	12.7	7.6	219.0	4.92	30.4	6	82	22	1.08	25.1
254x146x37	37.0	256.0	146.4	6.3	10.9	7.6	219.0	5.73	34.0	5	82	20	1.07	28.9
254x146x31	31.1	251.4	146.1	6.0	8.6	7.6	219.0	7.26	36.5	5	82	18	1.06	34.0
254x102x28	28.3	260.4	102.2	6.3	10.0	7.6	225.2	4.04	35.7	5	58	18	0.904	31.9
254x102x25	25.2	257.2	101.9	6.0	8.4	7.6	225.2	4.80	37.5	5	58	16	0.897	35.7
254x102x22	22.0	254.0	101.6	5.7	6.8	7.6	225.2	5.93	39.5	5	58	16	0.890	40.5
203x133x30	30.0	206.8	133.9	6.4	9.6	7.6	172.4	5.85	26.9	5	74	18	0.923	30.8
203x133x25	25.1	203.2	133.2	5.7	7.8	7.6	172.4	7.20	30.2	5	74	16	0.915	36.5
203x102x23	23.1	203.2	101.8	5.4	9.3	7.6	169.4	4.37	31.4	5	60	18	0.790	34.2
178x102x19	19.0	177.8	101.2	4.8	7.9	7.6	146.8	5.14	30.6	4	60	16	0.738	38.7
152x89x16	16.0	152.4	88.7	4.5	7.7	7.6	121.8	4.48	27.1	4	54	16	0.638	40.0
127x76x13	13.0	127.0	76.0	4.0	7.6	7.6	96.6	3.74	24.2	4	46	16	0.537	41.4

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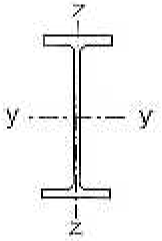
FOR EXPLANATION OF TABLES

BS EN 1993-1-1:2005
BS 4-1:2005

UNIVERSAL BEAMS

Advance UKB



Properties

Section Designation	Second Moment of Area		Radius of Gyration		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
	Axis y-y	Axis z-z	Axis y-y	Axis z-z	Axis y-y	Axis z-z	Axis y-y	Axis z-z	U	X	I_w	I_T	A
	cm^4	cm^4	cm	cm	cm^3	cm^3	cm^3	cm^3			dm^6	cm^4	cm^2
305x127x48	9570	461	12.5	2.74	616	73.6	711	116	0.873	23.3	0.102	31.8	61.2
305x127x42	8200	389	12.4	2.70	534	62.6	614	98.4	0.872	26.5	0.0846	21.1	53.4
305x127x37	7170	336	12.3	2.67	471	54.5	539	85.4	0.872	29.7	0.0725	14.8	47.2
305x102x33	6500	194	12.5	2.15	416	37.9	481	60.0	0.867	31.6	0.0442	12.2	41.8
305x102x28	5370	155	12.2	2.08	348	30.6	403	48.4	0.859	37.3	0.0349	7.40	35.9
305x102x25	4460	123	11.9	1.97	292	24.2	342	38.8	0.846	43.4	0.027	4.77	31.6
254x146x43	6540	677	10.9	3.52	504	92.0	566	141	0.891	21.1	0.103	23.9	54.8
254x146x37	5540	571	10.8	3.48	433	78.0	483	119	0.890	24.3	0.0857	15.3	47.2
254x146x31	4410	448	10.5	3.35	351	61.3	393	94.1	0.879	29.6	0.0660	8.55	39.7
254x102x28	4000	179	10.5	2.22	308	34.9	353	54.8	0.873	27.5	0.0280	9.57	36.1
254x102x25	3410	149	10.3	2.15	266	29.2	306	46.0	0.866	31.4	0.0230	6.42	32.0
254x102x22	2840	119	10.1	2.06	224	23.5	259	37.3	0.856	36.3	0.0182	4.15	28.0
203x133x30	2900	385	8.71	3.17	280	57.5	314	88.2	0.882	21.5	0.0374	10.3	38.2
203x133x25	2340	308	8.56	3.10	230	46.2	258	70.9	0.876	25.6	0.0294	5.96	32.0
203x102x23	2100	164	8.46	2.36	207	32.2	234	49.7	0.888	22.4	0.0154	7.02	29.4
178x102x19	1360	137	7.48	2.37	153	27.0	171	41.6	0.886	22.6	0.0099	4.41	24.3
152x89x16	834	89.8	6.41	2.10	109	20.2	123	31.2	0.890	19.6	0.00470	3.56	20.3
127x76x13	473	55.7	5.35	1.84	74.6	14.7	84.2	22.6	0.894	16.3	0.00200	2.85	16.5

Advance and UKB are trademarks of Corus. A fuller description of the relationship between Universal Beams (UB) and the Advance range of sections manufactured by Corus is given in section 12.

FOR EXPLANATION OF TABLES

Appendix E

Initial design table

Table A.1 Preliminary sizes of columns and rafters for symmetrical single-span portal frame with 6° roof pitch (S355 steel)

	DESIGN LOAD ON RAFTER (kN/m)	EAVES HEIGHT (m)	SPAN OF FRAME (m)					
			15	20	25	30	35	40
Rafter	8	6	254×102×22 UKB	254×146×31 UKB	356×127×39 UKB	356×171×45 UKB	356×171×57 UKB	356×171×67 UKB
	8	8	254×102×22 UKB	254×146×31 UKB	356×127×39 UKB	356×171×51 UKB	356×171×67 UKB	457×191×67 UKB
	8	10	254×102×22 UKB	254×146×31 UKB	356×127×39 UKB	356×171×51 UKB	356×171×67 UKB	457×191×67 UKB
	8	12	*	254×146×31 UKB	356×127×39 UKB	356×171×51 UKB	356×171×67 UKB	457×191×67 UKB
Restrained column	8	6	305×165×40 UKB	305×165×46 UKB	406×178×67 UKB	457×191×82 UKB	533×210×92 UKB	610×229×113 UKB
	8	8	305×165×40 UKB	305×165×46 UKB	406×178×67 UKB	457×191×82 UKB	533×210×101 UKB	610×229×113 UKB
	8	10	305×165×40 UKB	305×165×54 UKB	406×178×67 UKB	457×191×82 UKB	533×210×101 UKB	610×229×125 UKB
	8	12	*	305×165×54 UKB	406×178×67 UKB	457×191×82 UKB	533×210×101 UKB	610×229×125 UKB
Unrestrained column	8	6	305x165x46 UKB	457×191×67 UKB	533×210×82 UKB	533×210×92 UKB	610×229×113 UKB	610×229×125 UKB
	8	8	406×178×60 UKB	457×191×82 UKB	610×229×101 UKB	610×229×113 UKB	686×254×125 UKB	732×267×147 UKB
	8	10	406×178×67 UKB	533×210×92 UKB	610×229×113 UKB	610×229×125 UKB	762×267×147 UKB	610×305×149 UKB
	8	12	*	610×229×101 UKB	610×229×125 UKB	762×267×147 UKB	610×305×149 UKB	838×292×194 UKB
Rafter	10	6	254×102×25 UKB	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	356×171×67 UKB	457×191×82 UKB
	10	8	254×102×25 UKB	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	457×191×67 UKB	457×191×82 UKB
	10	10	254×102×25 UKB	356×127×39 UKB	356×171×45 UKB	356×171×57 UKB	457×191×67 UKB	457×191×82 UKB
	10	12	*	356×127×39 UKB	356×171×45 UKB	356×171×67 UKB	457×191×67 UKB	533×210×82 UKB
Restrained column	10	6	305×165×40 UKB	406×178×60 UKB	406×178×74 UKB	533×210×92 UKB	610×229×113 UKB	610×229×125 UKB
	10	8	305×165×40 UKB	406×178×60 UKB	457×191×82 UKB	533×210×92 UKB	610×229×113 UKB	610×229×125 UKB
	10	10	305×165×40 UKB	406×178×60 UKB	457×191×82 UKB	533×210×101 UKB	610×229×113 UKB	610×229×140 UKB
	10	12	*	406×178×60 UKB	457×191×82 UKB	533×210×101 UKB	610×229×125 UKB	610×229×140 UKB
Unrestrained column	10	6	406×178×54 UKB	457×191×74 UKB	533×210×92 UKB	610×229×101 UKB	610×229×125 UKB	610×229×125 UKB
	10	8	457×191×67 UKB	533×210×92 UKB	610×229×113 UKB	686×254×125 UKB	686×254×140 UKB	610×305×149 UKB
	10	10	457×191×74 UKB	610×229×101 UKB	610×229×125 UKB	762×267×147 UKB	610×305×149 UKB	838×292×194 UKB
	10	12	*	610×229×113 UKB	686×254×140 UKB	610×305×149 UKB	838×292×194 UKB	914×305×224 UKB
Rafter	12	6	254×102×28 UKB	356×127×39 UKB	356×171×51 UKB	356×171×67 UKB	457×191×82 UKB	533×210×82 UKB
	12	8	254×102×28 UKB	356×127×39 UKB	356×171×51 UKB	356×171×67 UKB	457×191×82 UKB	533×210×82 UKB
	12	10	254×102×28 UKB	356×127×39 UKB	356×171×51 UKB	356×171×67 UKB	457×191×82 UKB	533×210×92 UKB
	12	12	*	356×127×39 UKB	356×171×51 UKB	356×171×67 UKB	457×191×82 UKB	533×210×92 UKB
Restrained column	12	6	305×165×40 UKB	406×178×67 UKB	457×191×82 UKB	533×210×101 UKB	610×229×125 UKB	610×229×140 UKB
	12	8	305×165×40 UKB	406×178×67 UKB	457×191×82 UKB	533×210×101 UKB	610×229×125 UKB	686×254×140 UKB
	12	10	305×165×46 UKB	406×178×67 UKB	533×210×92 UKB	610×229×113 UKB	610×229×125 UKB	686×254×140 UKB
	12	12	*	406×178×67 UKB	533×210×92 UKB	610×229×113 UKB	610×229×125 UKB	686×254×140 UKB
Unrestrained column	12	6	406×178×60 UKB	533×210×82 UKB	610×229×101 UKB	610×229×113 UKB	610×229×125 UKB	686×254×140 UKB
	12	8	457×191×74 UKB	610×229×101 UKB	610×229×113 UKB	686×254×140 UKB	610×305×149 UKB	838×292×176 UKB
	12	10	457×191×82 UKB	610×229×113 UKB	686×254×140 UKB	610×305×149 UKB	838×292×194 UKB	914×305×224 UKB
	12	12	*	610×229×125 UKB	610×305×149 UKB	838×292×176 UKB	914×305×224 UKB	914×305×253 UKB

Table A.1 Preliminary sizes of columns and rafters for symmetrical single-span portal frame with 6° roof pitch (S355 steel) (continued)

	DESIGN LOAD ON RAFTER (kN/m)	EAVES HEIGHT (m)	SPAN OF FRAME (m)					
			15	20	25	30	35	40
Rafter	14	6	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	457×191×67 UKB	457×191×82 UKB	533×210×92 UKB
	14	8	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	457×191×674 UKB	533×210×82 UKB	533×210×92 UKB
	14	10	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	457×191×67 UKB	533×210×82 UKB	533×210×92 UKB
	14	12	*	356×171×45 UKB	356×171×57 UKB	457×191×82 UKB	533×210×82 UKB	610×229×101 UKB
Restrained column	14	6	305×165×46 UKB	406×178×74 UKB	533×210×92 UKB	610×229×113 UKB	610×229×140 UKB	686×254×140 UKB
	14	8	305×165×54 UKB	406×178×74 UKB	533×210×92 UKB	610×229×113 UKB	610×229×140 UKB	686×254×170 UKB
	14	10	305×165×54 UKB	406×178×74 UKB	533×210×101 UKB	610×229×125 UKB	610×229×140 UKB	686×254×170 UKB
	14	12	*	406×178×74 UKB	533×210×101 UKB	610×229×125 UKB	686×254×140 UKB	686×254×170 UKB
Unrestrained column	14	6	457×191×67 UKB	533×210×82 UKB	610×229×101 UKB	610×229×125 UKB	610×229×140 UKB	610×305×149 UKB
	14	8	457×191×82 UKB	610×229×101 UKB	686×254×125 UKB	762×267×147 UKB	610×305×149 UKB	838×292×176 UKB
	14	10	533×210×92 UKB	610×229×125 UKB	762×267×147 UKB	610×305×149 UKB	838×292×194 UKB	914×305×224 UKB
	14	12	*	686×254×140 UKB	610×305×149 UKB	838×292×194 UKB	914×305×224 UKB	914×305×289 UKB
Rafter	16	6	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	457×191×82 UKB	533×210×82 UKB	610×229×101 UKB
	16	8	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	457×191×82 UKB	533×210×92 UKB	610×229×101 UKB
	16	10	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	457×191×82 UKB	533×210×92 UKB	610×229×101 UKB
	16	12	*	356×171×45 UKB	356×171×67 UKB	457×191×82 UKB	533×210×92 UKB	610×229×113 UKB
Restrained column	16	6	305×165×54 UKB	457×191×82 UKB	533×210×101 UKB	610×229×125 UKB	686×254×140 UKB	686×254×170 UKB
	16	8	305×165×54 UKB	457×191×82 UKB	533×210×101 UKB	610×229×125 UKB	686×254×140 UKB	762×267×173 UKB
	16	10	406×178×60 UKB	457×191×82 UKB	533×210×101 UKB	610×229×125 UKB	686×254×170 UKB	762×267×173 UKB
	16	12	*	457×191×82 UKB	533×210×101 UKB	610×229×125 UKB	686×254×170 UKB	838×292×176 UKB
Unrestrained column	16	6	457×191×67 UKB	533×210×92 UKB	610×229×113 UKB	610×229×125 UKB	686×254×140 UKB	610×305×149 UKB
	16	8	457×191×82 UKB	610×229×113 UKB	686×254×140 UKB	610×305×149 UKB	838×292×176 UKB	838×292×194 UKB
	16	10	533×210×101 UKB	610×229×125 UKB	610×305×149 UKB	838×292×194 UKB	914×305×224 UKB	914×305×253 UKB
	16	12	*	686×254×140 UKB	838×292×176 UKB	914×305×224 UKB	914×305×253 UKB	914×305×289 UKB