STEEL PORTAL FRAME DESIGN FOR DECONSTRUCTION AND REUSE

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This Dissertation Submitted in Partial Fulfillment of the Requirements for the Degree
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DECLARATION

“I declare that this is my own work and this thesis does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other University or institute of higher learning and to the best of our knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

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Dr. (Mrs) MTP Hettiarachchi

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ABSTRACT

Steel Portal Frame Design for Deconstruction and Reuse

Today, the world over, much emphasis is focused on the use of sustainable materials in infrastructure. One of the main attributes that has led to the widespread use of steel in infrastructure is that is considered sustainable. Steel meets the 3R concept of Sustainability, Reduce, Reuse and Recycle. While steel components are 100% recyclable, the manufacture of stronger and better quality steel products has enabled the design and construction of structures using reduced quantities of steel. Due to scarcity of raw material, conservation of energy coupled with escalation of steel prices, it is prudent to design steel structures that can be re-used, thus extending the life cycle of steel. This is an aspect that has hitherto not received sufficient consideration by structural engineers.

The focus of this thesis is on extending the life cycle of steel components, with particular emphasis on the design of single span steel portal framed structures by considering the aspects of deconstruction and re-use. The scope of the study was limited to a span range of 20m to 40m and eaves heights of 4m and 6m and typical vertical action of 10kN/m.

The adoption of haunches hinders the re-use of the rafter. An innovation recently adopted facilitating the re-use of rafters is that of replacing the haunch at the eaves with a steel knee brace pinned at either end to the column and rafter. This concept was investigated and found to be viable within this range of span and eaves heights. Optimum locations for knee brace connections were found to be 10% of span length at the rafter end and 3% of the span length from the rafter axis at the column end.

These initial studies indicate that greater attention should be paid on the aspect of deconstruction and re-use of steel at the preliminary stages of design in order to extend the life cycle of steel components and thereby enhance the sustainability of steel structures.

Key words:

3R concept, deconstruction, re use, sustainability, knee brace

TABLE OF CONTENT

DECLARATION i
ACKNOWLEDGEMENT ii
ABSTRACT iii
TABLE OF CONTENT iv
LIST OF FIGURES vii
LIST OF ABBREVIATION x
LIST OF APPENDICES x
1. INTRODUCTION 1
 1.1. Background 1
 1.2. Problem identification – The need to reuse steel 3
 1.3. Aim 4
 1.4. Objectives 4
 1.5. Scope 4
 1.6. Methodology adopted 5
 1.7. Outline of dissertation 6
2. LITERATURE REVIEW 8
 2.1. Re-use of steel portal frames 8
 2.2. Critical detail/practices hindering re-use of steel portal frames 9
 2.3. Effect of corrosion with time on the reuse of steel members 11
 2.4. Key concepts for steel portal frame re-use 13
  2.4.1. Standardization (building geometry, detailing, etc.) 13
  2.4.2. Reduce number of interfaces (number of building layers) 13
  2.4.3. Reduce number of components (members; connections) 13
  2.4.4. Design for adaptability and relocation 13
  2.4.5................ Design and detailing for construction, deconstruction and transportation 13
 2.5. Design and detailing steel portal frames for deconstruction and re-use ..14

2.5.1. Using standard (expandable) components 14

2.5.2. Using bolted connection in preference to welded joints...............15

2.5.3. Using knee brace in place of haunch 16

3. METHODOLOGY..18

3.1. General 18

3.1.1. Analyzing with haunch 22

3.1.2. Analyzing with knee bracing..23

4. RESULTS AND DISCUSSION..26

4.1. Comparison of Bending Moments 26

4.2. Comparison of in plane stability of portal frames..................................35

4.3. Design resistance of knee brace and cost effectiveness 39

4.4. Connection design of knee brace 44

4.5. Feasibility of end moment connection at eave.......................................46

4.6. Resistance of column for transverse force from knee brace....................48

4.7. Feasibility of using lesser weight sections for 4m eave height................50

5. APPLICATION OF SRI LANKAN PRACTICE...51

5.1. Comparison of bending moment results..51

5.2. Determination of in plane stability..53

6. SUMMARY AND CONCLUSION...54

REFERENCES 56

Appendix – A Analysis and design philosophy of portal frames 58

Appendix – B Specimen calculation for design action of portal frame analysis ..64

Appendix – C Preliminary sizing tables – S355 steel with 6^0 roof......................65

Appendix – D Calculation of buckling amplification factor, $\alpha_{cr,est}$......................67

Appendix – E Specimen calculation for design resistance of knee brace..........116

Appendix – F Specimen calculation for knee brace connection design. 118

Appendix – G Specimen calculation of column resistance to transverse force from knee brace...120

Appendix – H Calculation of buckling amplification factor, $\alpha_{cr,est}$ for portal frames with reduced column section...122

Appendix – I Calculation of buckling amplification factor, $\alpha_{cr,est}$ for portal frames with roof slope of 10 degree 125

LIST OF FIGURES

Figure 1.1 Components of a steel portal frame structure..1
Figure 2.1 Bespoke members shape restricts the reuse on different applications....9
Figure 2.2 Reclaim individual components with a considerable length..................9
Figure 2.3 Large quantity of small elements 10
Figure 2.4 Cladding elements as the critical building layer for deconstruction 10
Figure 2.5 Fabricating each segments separately as reusable components 14
Figure 2.6 Bolted haunch component in a portal frame 15
Figure 2.7 Use of knee brace/ inclined knee brace arrangement 16
Figure 3.1 General portal frame arrangement with loading.................................19
Figure 3.2 Bending Moment diagram of portal frame with haunch......................19
Figure 3.3 Bending Moment diagram of portal frame with knee brace 20
Figure 3.4 Haunch dimensions for 20m span portal frame..................................22
Figure 3.5 Segments of haunch 22
Figure 3.6 Knee brace arrangement..23
Figure 3.7 Knee brace arrangement for 20m span frame with Y= 2% of span 24
Figure 3.8 Knee brace arrangement for 20m span frame with Y= 3% of span 24
Figure 3.9 Knee brace arrangement for 30m span frame with Y= 2% of span 24
Figure 3.10 Knee brace arrangement for 30m span frame with Y= 3% of span 24
Figure 3.11 Knee brace arrangement for 40m span frame with Y= 2% of span 25
Figure 3.12 Knee brace arrangement for 40m span frame with Y= 3% of span 25
Figure 3.13 Concept of knee brace connection 25
Figure 4.1 Bending Moment variation for knee braced portal frames of 4m eave 33
Figure 4.2 Bending Moment variation for knee braced portal frames of 6m eave 34
Figure 4.3 $\alpha_{cr,est}$ variation for portal frames with knee brace – 4 m eave height....37
Figure 4.4 $\alpha_{cr,est}$ variation for portal frames with knee brace – 6 m eave height....38
Figure 4.5 Knee brace configuration for 40m span portal frame 44
Figure 4.6 Connection detail for knee brace of 40m span portal frame 44
Figure 4.7 Standard connection details from (SCI P398, 2013) 46
Figure 4.8 Moment resistance of end plate connection from (SCI P398, 2013)....47
Figure 4.9 Web buckiling due to concentrated force 48
Figure 4.10 Knee bracing with web stiffener at the connection 48

Figure A.1 Asymmetric or sway mode deflection of portal frame 60
Figure A.2 Symmetric mode deflection of portal frame 60

LIST OF TABLES

Table 1.1 U, I or H Sections of steel imports to Sri Lanka 3

Table 2.1 Atmospheric corrosively categories and typical environments............11

Table 4.1 Bending Moment of frame with haunch 27

Table 4.2 Bending Moment of frame without haunch 27

Table 4.3 Bending Moment of frame with knee brace: X= 5%, Y=2% span.......28

Table 4.4 Bending Moment of frame with knee brace: X= 5%, Y=3% span.......28

Table 4.5 Bending Moment of frame with knee brace: X= 10%, Y=2% span.....29

Table 4.6 Bending Moment of frame with knee brace: X= 10%, Y=3% span.....29

Table 4.7 Bending Moment of frame with knee brace: X= 15%, Y=2% span.....30

Table 4.8 Bending Moment of frame with knee brace: X= 15%, Y=3% span.....30

Table 4.9 Buckling Amplification Factor for portal frames.................................36

Table 4.10 Design resistance of knee brace (K.B.) for 20m span 40

Table 4.11 Design resistance of knee brace (K.B.) for 30m span 41

Table 4.12 Design resistance of knee brace (K.B.) for 40m span 43

Table 4.13 Connection design summary of knee brace 45

Table 4.14 M12 results for X=10% and Y=3% of span knee brace condition.......47

Table 4.15 Stability and steel saving with reduced sections 50

Table 5.1 Bending moment of portal frame with different roof slopes................51

Table 5.2 Bending resistance of selected steel sections 51

Table 5.3 Bending moment of 20m span portal frame with 10^0 roof slope 52

Table 5.4 Buckling amplification factor for portal frames with two roof slopes .53

LIST OF ABBREVIATION

Abbreviation	Description
BM	Bending Moment
CHS	Circular Hollow Section
EHF	Equivalent Horizontal Force
GA	General Arrangement
KB	Knee Brace
NHF	Notional Horizontal Force
PEB	Pre Engineered Buildings
SLS	Serviceability Limit State

LIST OF APPENDICES

Appendix – A Analysis and design philosophy of portal frames…………..…58

Appendix – B Specimen calculation for design action of portal frame analysis……………………………………………………64

Appendix – C Preliminary sizing tables…………………………………….65

Appendix – D Calculation of buckling amplification factor, $\alpha_{cr,est}$ ………67

Appendix – E Specimen calculation for design resistance of knee brace….116

Appendix – F Specimen calculation for knee brace connection design……118

Appendix – G Specimen calculation of column resistance to transverse force from knee brace …………………………………………..120

Appendix – H Calculation of buckling amplification factor $\alpha_{cr,est}$ for portal frames with reduced column section …………….………..122

Appendix – I Calculation of buckling amplification factor, $\alpha_{cr,est}$ for portal frames with roof slope of 10 degrees ………………………125

1. INTRODUCTION

1.1. Background

The steel portal frame is the dominant form of structure for single-story industrial and commercial buildings for most of the countries. It is the most popular structural form in pitched roof buildings, as it is economical and versatile for a wide range of spans.

In Sri Lanka too, single story buildings form the largest sector of the steel construction market. These buildings are used mainly for workshops, factories, warehouses, stores and recreation. The size of these structures varies from small workshops to warehouses with more than ten thousand square metres. Pre Engineered Building (PEB) systems are popular in Sri Lanka for wide span portal frames while on site fabrication is done for small spans structures by using the locally available steel sections.

Portal frames are a type of structural frame, that comprise a beam (or rafter) supported by columns at both ends. The joints between the beam and columns are 'rigid' therefore, the bending moment in the beam is transferred to the columns. Figure 1.1 shows the components of a single span steel portal frame.

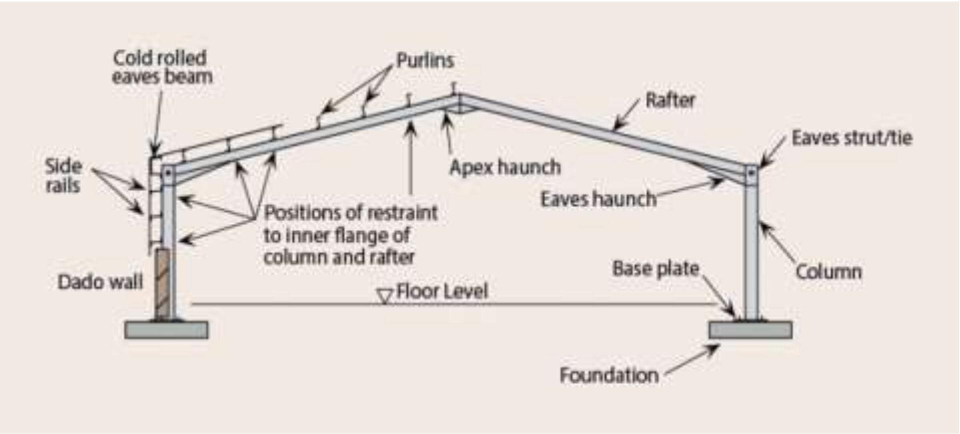


Figure 1.1 Components of a steel portal frame structure

Portal frame has several advantages; it is effective in medium to long span range of 30 to 50 m with a range of member sizes, stable in-plane due to the rigidity of the frame, member sizes and haunches could be further optimized by adopting plastic design.

The eaves haunch of a portal frame plays an important role in terms of strength and stability in the ultimate limit state (ULS) and for deformation in the serviceability limit state (SLS). Generally in portal frames, highest bending moment occurs at the rafter/ column connection, then large steel section would be required to resist this highest bending moment at the rafter. However such a section is not required throughout the entire length. To avoid using larger section for entire length of the rafter, designers either use tapered section or use a haunch. When haunch is used with a standard section frame, lever arm of the rafter near support is increased and therefore, moment resistance at support is also increased. Then reduced depth could be used for rest of the span of the rafter. Eave haunch also increases stiffness of the frame and reduces deflection.

The length of the eaves haunch is generally 10% of the span. This haunch length is adopted as the hogging moment at the end of the haunch is approximately equal to the maximum sagging moment near the apex. The depth from the rafter axis to the underside of the haunch is approximately 2% of the span.

The haunch is an independent triangular element which should not be confused with tapered beam. Tapered beams are made according to the bending moment profile, so required bending resistance is provided by increasing section depth at the locations where high bending moment occurred. Pre Engineered Buildings (PEB) designers commonly use this tapered beam concept for portal frames.

When designing portal frames, the frame behaviour should be checked prior to carrying out any strength check. The global analysis gives a good representation of the behaviour of the actual structure.

1.2. Problem identification – The need to reuse steel

Today the world is more focused on the use of sustainable materials in infrastructure. One of the main attributes that has led to the widespread use of steel in infrastructure is that is considered sustainable. Steel meets the 3R concept of Sustainability, Reduce, Reuse and Recycle.

Steel components are 100% recyclable; also manufacturing of high strength and high quality steel products has enabled the designers and contractors to build structures using reduced quantities of steel. Due to scarcity of raw material, escalation of steel prices, reusing steel become an important topic.

In Sri Lanka, the structural steel sections used are not manufactured locally but imported. According to the Sri Lanka Custom (2022) the price of steel imported over the past few years has gone up by a considerable margin. Steel section import data over the past four years is shown in Table 1.1.

Table 1.1 U, I or H Sections of steel imports to Sri Lanka

Year	Net weight (Ton)	Cost (Rs.)	Cost/ton
2018	13350	1495 Million	0.11 Million
2019	7600	888 Million	0.12 Million
2020	5275	686 Million	0.13 Million
2021	6337	1039 Million	0.16 Million

(Sri Lanka Customs-information department, 2022)

With the escalation of steel prices, it is prudent to design steel structures that can be re-used, thus extending the life cycle of steel. This is an aspect that has hitherto not receive sufficient consideration by structural engineers.

Considering the option of reusing existing steel structures, there are several details/practices that hinder its re-use. When pre-engineered building structures are concerned, tapered sections are used and therefore, not suitable as a reusing option. This issue can be overcome if structure is designed for deconstruction and re-use.

According to Dunant et al. (2018), although steel reuse is most frequently understood as the reuse of steel elements to construct a completely new design, in practice reuse occurs frequently within the context of extending the lives of existing structures in the same form.

Then one of the main alternatives identified as potential for reuse is to use alternatives for haunch rafter – so that the frame can be dismantled from one site, reconstructed at another location with minimum modifications.

In Sri Lanka, the concept of design for deconstruction and re-using of portal frame structures, then extending the life cycle has not being given sufficient consideration. A detailed analysis of the costs and risks of reuse is lacking.

The need to enhance the design life of these steel structures is a matter of immediate concern in the light of the economic crisis that the country is currently faced with. As active members of the industry, structural engineers have greater responsibility on this kind of study then proposing alternatives options too.

1.3. Aim

The aim of this research is to extend the life cycle of single span steel portal framed structures by considering the concepts of design and detailing for de-construction and re-use.

1.4. Objectives

The specific objective of this research is to determine the optimum location of the knee brace considering the multiple design criteria of moment resistance, stability, joint resistance and cost effectiveness.

1.5. Scope

The scope of the study was limited to steel portal frames of span range of 20m to 40m and eaves heights of 4m and 6m, a representative of frame geometry used in this country. A design action of 10kN/m is considered as typical vertical loading.

1.6. Methodology adopted

The following methodology was adopted in order to study the concept of using knee brace in place of haunch of steel portal frames.

Structural analysis was carried out for steel portal frame spans of 20m, 30m and 40m with eave heights of 4m and 6m. Typical design vertical action of 10kN/m on rafter was considered for all portal frames. Roof angle of 6^0 was taken for initial analysis of all portal frames. There are lot of information available for 6 degree portal frames and therefore, using 6 degree roof slope will be useful to cover up wide range of data. On the other hand, 6 degree is generally a moderate slope and will give relatively critical analysis results in terms of in plane stability of portal frame, then anything steeper than this slope will be on favorable side too.

Steel grade of S355 was taken as design strength of steel – higher strength of steel will provide relatively lower size sections therefore, lesser quantity of steel as well. Further the strength class does not make any effect on the stability criteria too. Therefore, S355 was used for initial calculation to find the best configuration of knee brace.

Analysis was done for portal frames with haunch, without haunch and different configuration of knee brace arrangements. Initially, bending moments of frames at different locations were determined to find the best location for knee brace. Calculation was then carried out to determine in plane stability based on the buckling amplification factor. All these results were analyzed to find the best configuration of knee brace.

Once the better location of knee brace is selected based on the above results, the knee brace was designed using circular hollow section. Circular hollow section was selected as the section properties are constant for all principal axis. Knee brace was designed as compression member considering pinned connections at both ends. Connection design for knee brace also carried out based on the guidelines given in EN1993-1-8.

Based on the design results, the cost effectiveness of the knee brace was determined by comparing corresponding weight of haunch.

Then the feasibility of end moment connection of rafter at eave when knee brace is existed was checked. The effect of column due to transverse force from knee brace was also checked to find out the requirement of any web stiffeners for column at knee brace connection.

1.7. Outline of dissertation

This dissertation comprises six chapters. Chapter 1 is introductory chapter on research including background and problem identification. It is also included aim and objective of this research and then the methodology adopted to achieve the specific objective is briefly explained.

Chapter 2 comprises detail literature available in relation to steel portal frame reuse and barriers of reusing in current practice. Effect of corrosion with time on the reuse of steel member and globally recognized standards are given in this chapter. At the later part of this chapter, possible design and detailing practices for steel portal frame reuse are discussed. More details are included in the knee brace concept which is the main topic of this research.

Details of methodology adopted in this research is explained with more details under Chapter 3. At the first sub section, all portal frame options used with loads and other limitations are indicated. Analysis details of portal frames with haunch, without haunch and different configurations of knee brace are well explained under proceeding sub sections.

Chapter 4 comprises results and discussion. All analysis results of bending moments and buckling amplification factors are tabulated under this chapter. Based on the results, detail discussion has been done to find out the best location of knee brace. This chapter included design results of knee brace and cost effectiveness against corresponding haunch as well. The end connection of knee brace considering pinned connection is also being done. Details are included in relation to the feasibility of end moment connection of rafter at eave and resistance of column due to transverse force from knee brace, then the feasibility of using lesser weight sections for 4m eave height is also discussed.

Under Chapter 5, details have been included for 20m span portal frame applying the Sri Lankan practice, i.e. S275 steel and roof slope of 10 degrees. The selected knee brace configuration was checked for this condition in terms of bending moment and in plane stability criteria.

Summary and conclusion are included in the chapter 6. Details of pros and cons of using knee brace and also its benefits in relation to design for deconstruction and reuse of steel portal frames are discussed in details.

2. LITERATURE REVIEW

2.1. Re-use of steel portal frames

As distinct from recycling, reuse of steel products involves their reuse with little or no reprocessing. Reuse offers even greater environmental advantage than recycling since there are no (or very few) environmental impacts associated with reprocessing. For example, reusing a steel beam in its existing form is better than re-melting it and rolling a new steel beam, i.e. the energy used to re-melt and re-roll the beam is saved.

Recycling steel only saves approximately 50% of the energy and carbon over making new steel (Norgate et al, 2007) as the recycling of steel consumes much energy even using the best technology currently available (Milford, 2010). By contrast, steel reuse can play an important part in a global strategy for the efficient use of materials (Allwood, 2012)

The benefits of reuse can be therefore important in two lifecycle stages. First is building with reused products (reuse today) may decrease the cost of the assembled steelwork and its environmental impacts in the product and construction stage. Second is design for deconstruction and reuse (future reuse) increases the residual value of the building and benefits beyond the current building's lifecycle.

The steel portal frames should be designed for flexibility in use and, at the end of its life; its materials must be reusable or recyclable. The overall benefit of reuse of steel portal frames depends on widespread adoption of this approach in design and construction practices. Designers should understand how to optimize their designs for deconstruction and reuse. Therefore designers should be encouraged to think about not only how the steel portal frames can be easily and effectively constructed but also how they can be efficiently deconstructed with a view to preserving the integrity of the reclaimed products for subsequent reuse. This is a new discipline for most designers. (Steel construction.info, 2013)

2.2. Critical detail/practices hindering re-use of steel portal frames

There are several roadblocks in reusing of existing steel portal frame structures over the years. These have led people to oppose going for reusing ideas. Some of them are listed below;

a. In PEB steel structures, the frame is built according to the bending moment profile and then it is very difficult to do modification for re-use. The existing shape of the frame will need more modifications to fit in to new loads and so the required strength. This process could be more expensive as well. Figure 2.1 indicate better example for the situation.

Figure 2.1 Bespoke members shape restricts the reuse on different applications

b. Consisting of individual components in different length will also restrict the reusing of frame and increase the wastage too. Figure 2.2 shows a better detail on this matter.

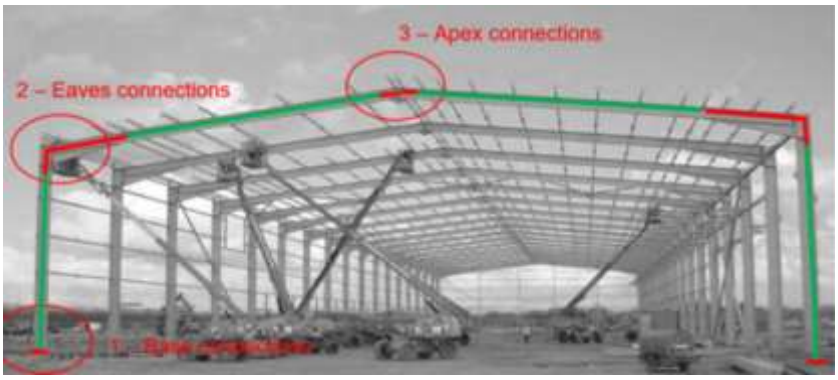


Figure 2.2 Reclaim individual components with a considerable length

c. When the structure comprising large quantity of small elements, deconstruction and reusing activities are difficult and time consuming. This will be comparatively more expensive as well. As shown in Figure 2.3, truss frames are the most common example for this problem.

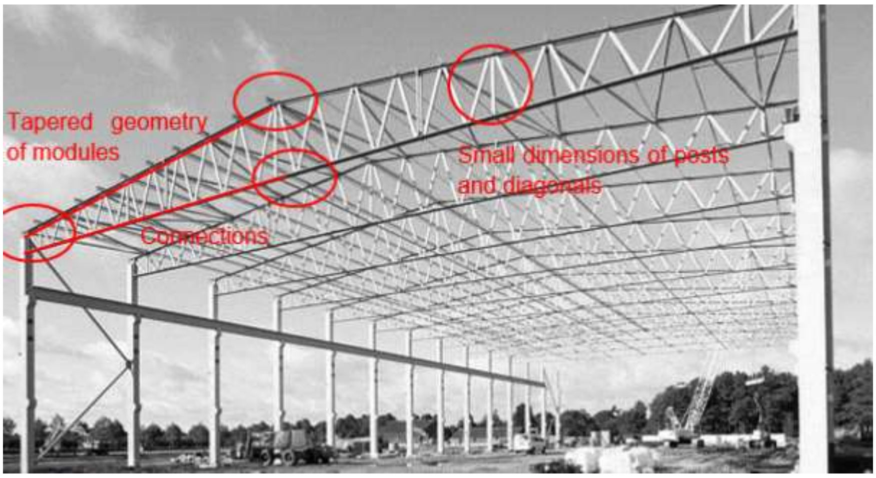


Figure 2.3 Large quantity of small elements

d. Different elements of the structure could lead deconstruction activities harder for reuse. During deconstruction process, these small elements could be damaged and need to be replaced with new materials.

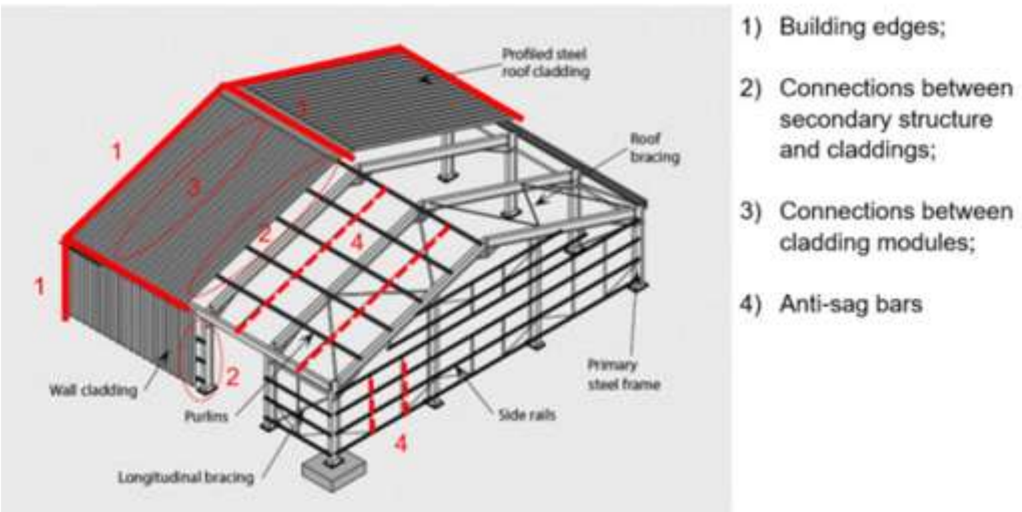


Figure 2.4 Cladding elements as the critical building layer for deconstruction

2.3. Effect of corrosion with time on the reuse of steel members

Corrosion of steel is often investigated in the context of the durability performance of the structure; however, corrosion can equally impact the structural performance. Corrosion with time will reduce cross sectional area of steel members. This will result decreasing their capacities as well. Therefore, it is very important to maintain steel members in their original design capacities with time so the structure performs well during its design life. The best way to protect steel from corrosion is applying comprehensive corrosion resistance coating system.

ISO 12944 is the globally recognized standard that lays out the rules and guidelines for protection of steel elements from corrosion by use of coating system. The principal factors that determine the rate of corrosion of steel are time of wetness and atmospheric condition. Table 2.1 shows the corrosive categories given in ISO12944 based on environmental conditions.

Table 2.1 Atmospheric corrosively categories and typical environments

Corrosivity category	Low-carbon steel Thickness loss (μm)[a]	Examples of typical environments (informative only)	
		Exterior	Interior
C1 very low	≤ 1.3	-	Heated buildings with clean atmospheres, e.g. offices, shops, schools, hotels
C2 low	> 1.3 to 25	Atmospheres with low level of pollution: mostly rural areas	Unheated buildings where condensation can occur, e.g. depots, sports halls
C3 medium	> 25 to 50	Urban and industrial atmospheres, moderate sulphur dioxide pollution; coastal area with low salinity	Production rooms with high humidity and some air pollution, e.g. food-processing plants, laundries, breweries, dairies
C4 high	> 50 to 80	Industrial areas and coastal areas with moderate salinity	Chemical plants, swimming pools, coastal ship and boatyards
C5 very high	> 80 to 200	Industrial areas with high humidity and aggressive atmosphere and coastal areas with high salinity	Buildings or areas with almost permanent condensation and high pollution
CX extreme	> 200 to 700	Offshore areas with high salinity and industrial areas with extreme humidity and aggressive atmosphere and sub-tropical and tropical atmospheres	Industrial areas with extreme humidity and aggressive atmosphere

(BS EN ISO 12944-2, 2017)

The design of structure is also affect the durability of protective coating system. If the structure is designed with many small components and fasteners are more difficult to protect than those with large flat surfaces. Therefore, design and detailing is important to ensure that the protective treatment can be applied to all surfaces, to avoid the creation of water and dirt traps that would accelerate corrosion, and could lead to pitting corrosion.

Detailing is also important to ensure that future inspection and maintenance can be carried out effectively. Guidance for the prevention of corrosion by good design detailing can be found in BS EN ISO 12944-3.

Having considered all above factors, it is clear that designing frames with simplified detailing can be more protective and durable than more complex details. The most common example for this in relation to portal frame is eave connection. Typical eave connection share highest portion of the bending moment and therefore, connection is designed as moment resisting connection with having many number of bolts. Therefore, it is very difficult to apply proper coating and maintain cleaner surface. If this connection can be simplified, then it could be more beneficial in terms of protection against corrosion and then enhance the durability of the frame.

2.4. Key concepts for steel portal frame re-use

Deconstruction and reusing could be made more favorable by practicing several methods. Industry experts have identified following key concepts for steel portal frames reuse.

2.4.1. Standardization (building geometry, detailing, etc.)

Following standard details can make modifications much simpler and easier. This could support to reusing of structure even without modifications. The best example on this is avoiding tapered sections.

2.4.2. Reduce number of interfaces (number of building layers)

Avoiding secondary structures as much as possible could help deconstruction much easier and faster. Most common example for this is using long span cladding for structure, then more secondary elements which requires to fix cladding could be minimized.

2.4.3. Reduce number of components (members; connections)

Making the structure is simpler with fewer type of robust elements and reduce number of different cross sections will be easier for reuse. It is also helpful of reducing number of materials (steel grade, sub grades etc.) as well.

2.4.4. Design for adaptability and relocation

If the structure is designed for adoption of different loads and environmental conditions different locations, would help for reusing. For example, wind loading concerns, assuming average altitude for the zone representing most areas of the country will help for relocation as well. Keeping separate capacity and documenting it will also helpful.

2.4.5. Design and detailing for construction, deconstruction and transportation

In this regard following methods could be applied.

a. Reduce the number of connections and connectors.
b. Use bolt/screw instead of other solutions, reduce welding.
c. Detail for easy access of connections.
d. Repetitive detailing (modular/standard).
e. Avoid permanent attachments (floor system are critical).

2.5. Design and detailing steel portal frames for deconstruction and re-use

It is far more important to plan for deconstruction and reuse of the structure at the design stage. This can avoid more hindering aspects of the structural re-use. In portal frame concerns, several design and detailing methods could be followed at the design stage to make the structure more favorable for deconstruction and reuse. Some of them are described as below;

2.5.1. Using standard (expandable) components

It is preferable to use standard components rather that more complex custom made elements. Standard elements are easy to be replaced, demounted and easy to be reused for different applications in future.

In Figure 2.5, re-usable components are used as connections at the eaves and apex. This permits the re-use of the beam and column components which are of uniform cross-section. If bolted connections and standardized connection details are adopted it would also permit the connection components to be re-used elsewhere. Use long-span beams as they are more likely to allow flexibility of use and to be reusable by cutting the beam to a new length.

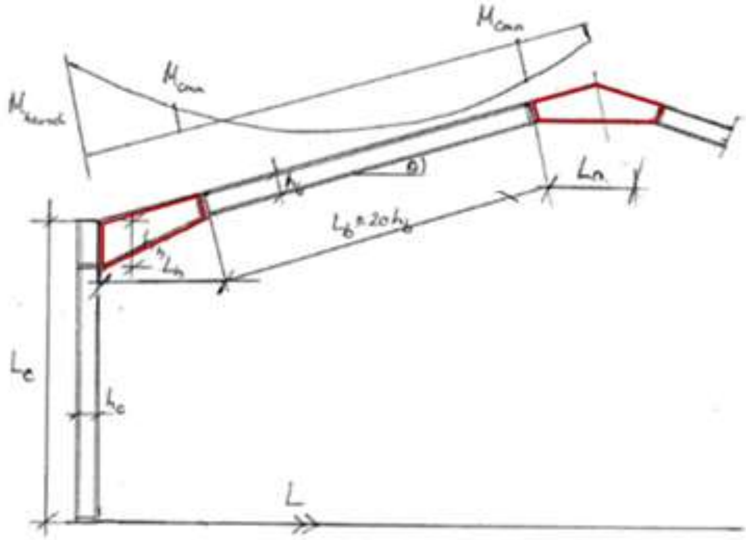


Figure 2.5 Fabricating each segments separately as reusable components

Identify the origin and properties of the component, for example by bar-coding or e-tagging or stamping and keep an inventory of products will be useful.

2.5.2. Using bolted connection in preference to welded joints

The commonly adopted practice in steel portal frames is to weld the haunch to the rafter. This practice hinders the re-use of the rafter section. Hence using a bolted connection as suggested in Figure 2.6 is a feasible alternative. Using standard connection details including bolt sizes and the spacing of holes will facilitate easy dismantling work. It is also important to ensure easy and permanent access to connections.

Although, it is an alternative option, it increases the complexity of verification of member and joint resistance due to possible slip at the bolted joint interface of rafter and haunch.

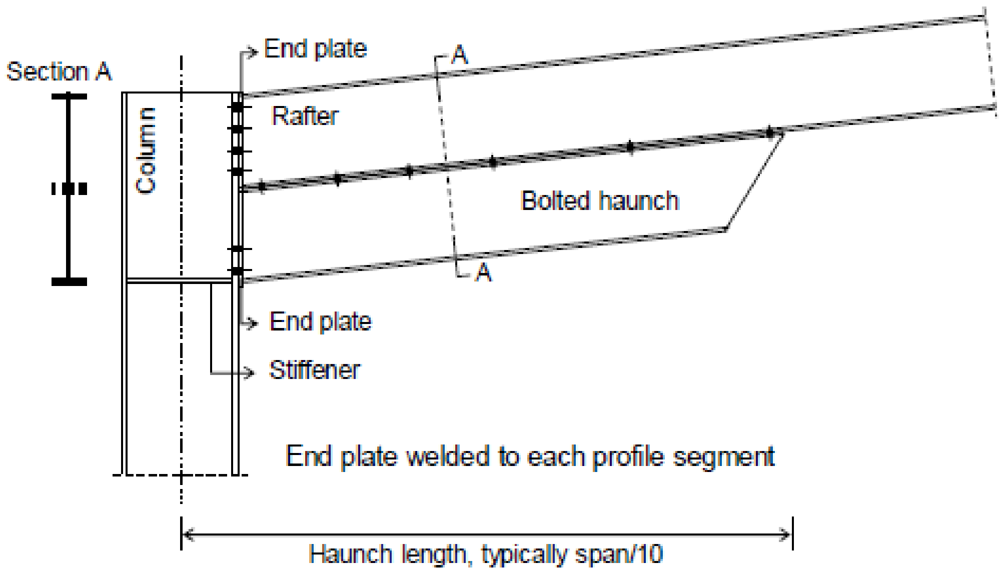


Figure 2.6 Bolted haunch component in a portal frame
(CSI -Steel Knowledge, 2022)

The reuse of members with holes for the structural bolts is permitted if all geometric and design requirements according to EN 1993-1-1 and EN 1993-1-8 are fulfilled. (Eurocode3, 2010)

In this arrangement, holes created at the flange of rafter for bolts could be an issue when reusing.

2.5.3. Using knee brace in place of haunch

Replacing haunch by a knee bracing (pinned connection) using square hollow section or circular hollow section would be an idea to check for small and medium span portal frames. The web of the column and rafter would have to be stiffened locally by a web stiffener. This solution will facilitate easy deconstruction techniques and less time. Figure 2.7 present the idea.

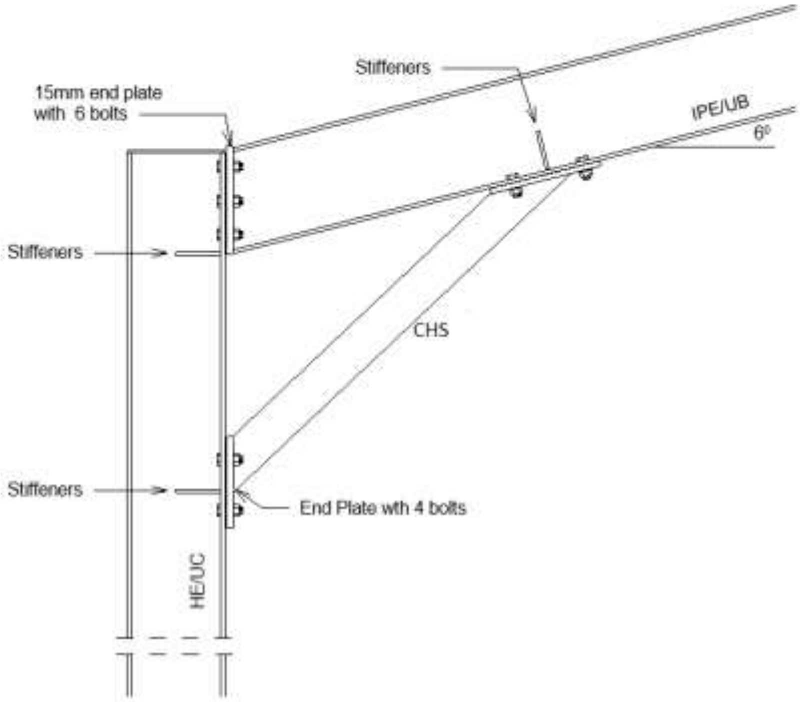


Figure 2.7 Use of knee brace/ inclined knee brace arrangement

With this concept, beam, column and knee brace can be separately dismantled, so there is no wastage as the case of haunch. To be able to reuse the full length of the rafter as there will not be any damage to that, a small segment of the same size as the rafter may be used to which the rafter is bolted by an end plate. This means that the end details of the rafter are compatible with their use in general applications and the fabricated segments can be removed in the second use if the complete portal frame is not reused.

The main concern of this configuration is the connection of the brace. Since the connection being done with 4 bolts, there may be a moment portion shared by the connection and therefore the brace is not in pure axial force. Therefore, analyzing and designing with pinned connection need to be checked, then unsure pure compression in the bracing as well.

The same strut detail may be used at the apex but in this case the strut is horizontal. It is less efficient in resisting bending than the haunch because of the shallow inclination of the rafter

In the aspect of deconstruction and reuse, the concept of using knee brace in the place of haunch could be an attractive idea for the industry. But the detailed study on this is very minimum in a worldwide context. Such discussion in Sri Lanka is hardly found, and no data are available on this kind of research or practical usage.

Sri Lanka as a developing country, need to focus on alternate construction solutions in view of minimizing waste and reusing materials. Therefore, the study of this concept would be a much needed start for the discussion. Especially the investors who willing to reuse the steel frames for their future expansions, will be attracted to the concept.

As there is no detailed literature available on this context, it is vital to study the pros and cons of using this knee bracing concept for the portal frames in terms of strength and stability of it. Therefore, it is necessary to carry out a detailed study to determine:

- The bending moment variation of frame with knee brace compare with haunch.
- The effect of knee bracing for overall in plane stability of the frame.
- The optimum location for knee brace in terms of capacity and the connection.
- The possible cost effectiveness.

The methodology adopted in order to carry out the above study is explained in detail in the next chapter 3.

3. METHODOLOGY

3.1. General

In this study, the main focus was given to check the following criteria in relation to single span portal frame analyzing with knee brace in place of haunch.
The bending moment variation of portal frame with knee brace compare to haunch.
The overall in plane stability of the portal frame with knee brace.
The optimum location of knee brace and possible cost effectiveness.

Analysis was carried out for single span portal frames - span range of 20m to 40m and eave heights of 4m and 6m. A design action of 10kN/m was used for all frames as typical vertical action. Details of design action are provided in Appendix B.

Following data were used for analysis of portal frames:

a. The roof pitch is 6^0
b. The steel grade is S355
c. The Design load is design value of permanent action and variable action. The wind loading has not been included as the presumption is that gravity load is dominating the choice of member sizes.
d. The haunch length was taken as 10% of the span and depth from the rafter axis to the bottom of haunch was 2% of span.
e. Knee brace was suggested for length equal to 5%, 10% and 15% of the span. Two options were considered for the depth from the rafter axis to the knee brace connection to column which are 2% and 3% of span.
f. Pinned connection was considered for knee bracing

The general arrangement of portal frame is shown in Figure 3.1.

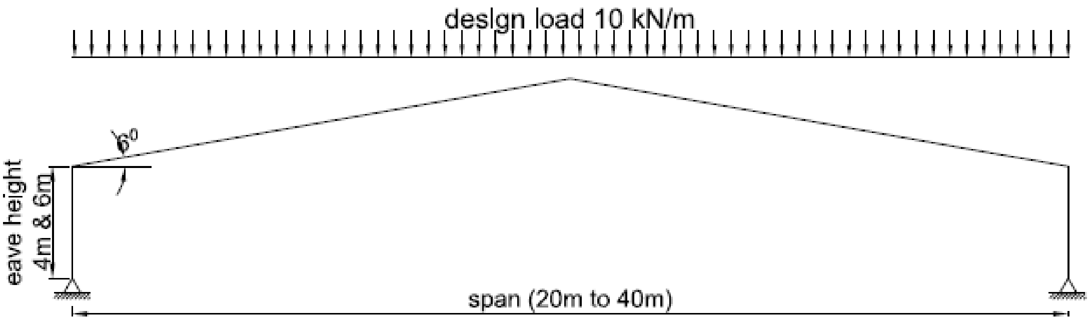


Figure 3.1 General portal frame arrangement with loading

For the preliminary analysis, column and rafter sections were selected from Table A-1 of SCI Publication P399 (Table A-1 of SCI P399 is provided in Appendix C) In the portal frame for 20m span with eave height of 4m steel sections selected are:

Rafter = 356x127x33 kg/m UB

Column = 406x178x60 kg/m UB

In the first step of analysis, the bending moment (BM) at column top and different locations of the rafter; the eaves connection, the knee brace connection and the apex connection are determined.

Typical bending moment diagrams with notation adopted for the bending moments at critical locations are shown in Figure 3.2 and Figure 3.3

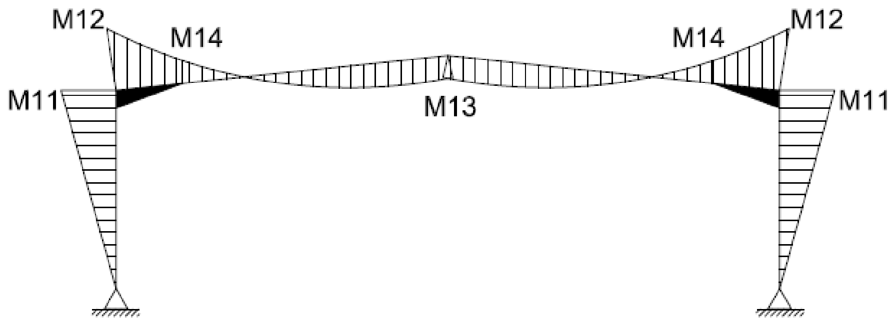


Figure 3.2 Bending Moment diagram of portal frame with haunch

M11 = Maximum Bending Moment at top of the column (at rafter connection))

M12 = Bending Moment of rafter at column connection

M13 = Maximum Bending Moment of rafter near apex

M14 = Bending Moment of rafter at sharp end of haunch

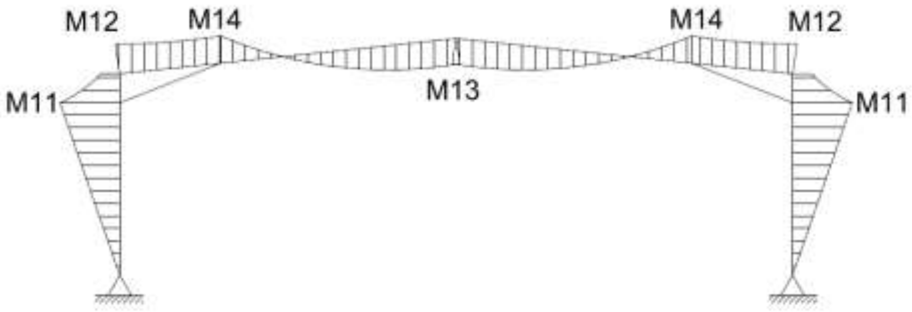


Figure 3.3 Bending Moment diagram of portal frame with knee brace

M11 = Maximum Bending Moment of column (at knee bracing)

M12 = Bending Moment of rafter at column connection

M13 = Maximum Bending Moment of rafter near apex

M14 = Maximum Bending Moment of rafter at knee bracing connection

Then the in plane stability of the portal frames was checked by the determination of the buckling amplification factor (α_{cr}) for all of the frames described corresponding to the different knee brace locations. The amplification factor was determined according to clause 5.2.1 of EN 1993-1-1. Results were compared with, without haunch condition as well.
Detailed calculation of α_{cr} is presented in Appendix D

All output results of above analysis were considered to select the most suitable arrangement of knee bracing. It was advisable to maintain α_{cr} value above 10, so there won't be any second order effect, and then amplification of results is not necessary.

Once the better location of knee brace is selected, calculations were done to check the required capacity of knee brace with suitable Circular Hollow Section (CHS). CHS was selected, as the even distribution of material around the center of cross section results in a radius of gyration (r), moment of inertia (I_{xx}) and section modulus (Z) which is constant for all principal axis of CHS members. Therefore, CHS is more effective as compression member compared to rectangular hollow section and angular sections.

As in the design check of a haunched portal frame, following criteria were also checked to validate knee bracing concept.

a. Design capacity and cost effectiveness of knee brace for all spans

   Structural design of bracing with selected CHS was carried out based on the guidelines given in clause 6.2.4 and clause 6.3 of EN 1993-1-1. The cost effectiveness of the knee brace connection was determined by comparing its weight to that of the haunch of the corresponding portal frame.

b. Feasibility of end moment connection at eave and resistance of column for transverse force from knee brace.

   Based on the optimum location of knee brace and available spacing between rafter and bracing, the recommendation of (SCI P398, 2013) were adopted to determine the adequacy of the flush end moment connection at the column to rafter connection.

   And the effect of the column due to transverse force was checked to find out the requirement of any web stiffener for column web at knee bracing connection.

3.1.1. Analyzing with haunch

The haunch length was taken as 10% of span and depth from rafter axis to the haunch bottom was taken as 2% of span. Haunch is generally fabricated from cutting of rafter section.

In the portal frame for 20m span with 4m eave height the haunch length is taken as 2m (i.e.. 20m x 10% = 2m.) and depth is equal to 0.4m (i.e.. 20m x 2% = 0.4m). General arrangement of haunch is shown in Figure 3.4

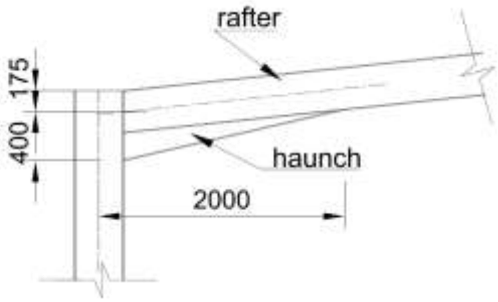


Figure 3.4 Haunch dimensions for 20m span portal frame

Analysis was carried out using finite element software (STAAD Pro V8i). Modeling of haunch done by dividing length in to four segments, section properties of each segment were calculated based on the average depth. The middle flange effect was also considered. Details are shown in Figure 3.5

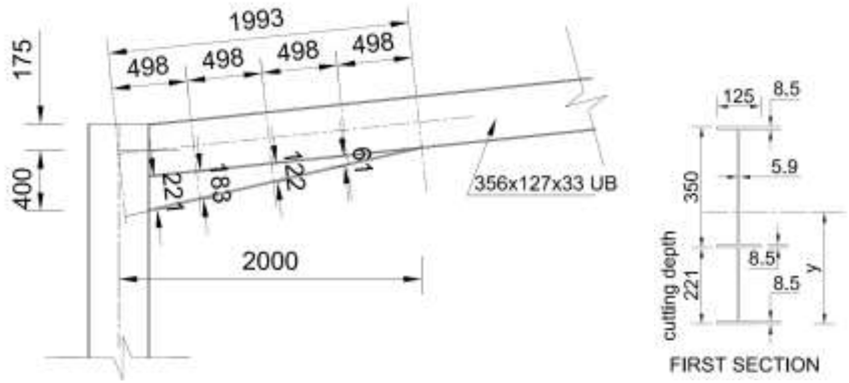


Figure 3.5 Segments of haunch

Analysis was carried out to find the bending moment at critical locations of portal frames. Analysis was also carried out for the frames for without haunch condition to compare results.

3.1.2. Analyzing with knee bracing

Steel Circular Hollow Section (CHS) was proposed for knee bracing instead of haunch. Rafter and column sections remain unchanged, same as for haunch situation.

General arrangement is shown in Figure 3.6

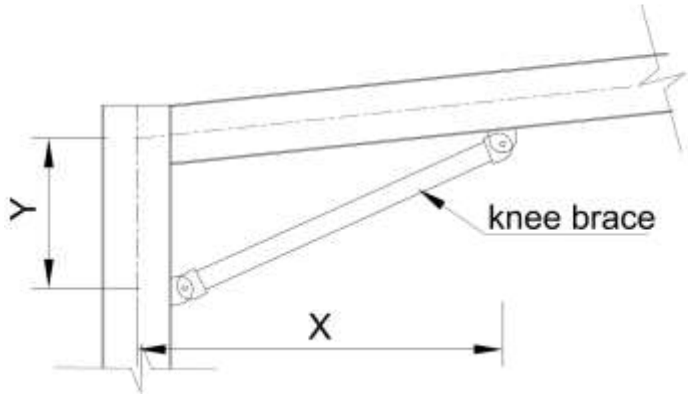


Figure 3.6 Knee brace arrangement

Different options for dimensions X and Y (as shown in figure 3.6) were selected for analysis of portal frame with knee brace.

The dimension X was varied from 5% to 10% to 15% of the span, while the dimension Y was varied from 2% to 3% of the span. The range of X values was selected based on the typical haunch length of 10% of span. On the same basis a value of 2% of the span typical of the haunch depth was selected as the dimension Y. A second option of 3% permitting a greater vertical distance was considered to facilitate the connection of the knee brace. The knee brace comprised a Circular Hollow Section (CHS) of the same steel grade.

The general arrangements (G.A.) of knee bracing for each case are shown in Figure 3.7 to Figure 3.12

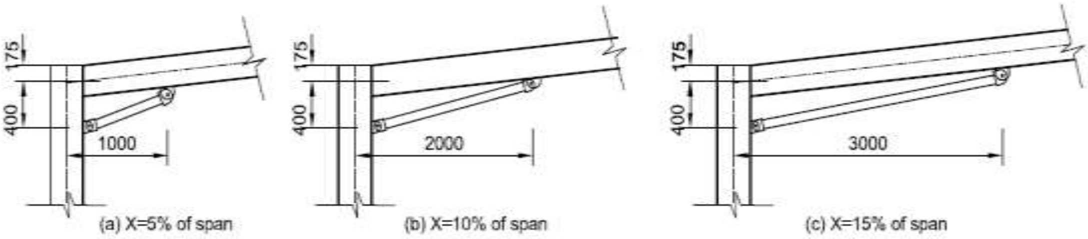


Figure 3.7 Knee brace arrangement for 20m span frame with Y= 2% of span

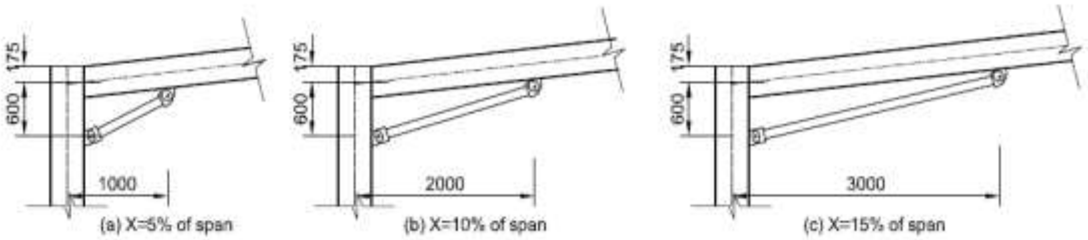


Figure 3.8 Knee brace arrangement for 20m span frame with Y= 3% of span

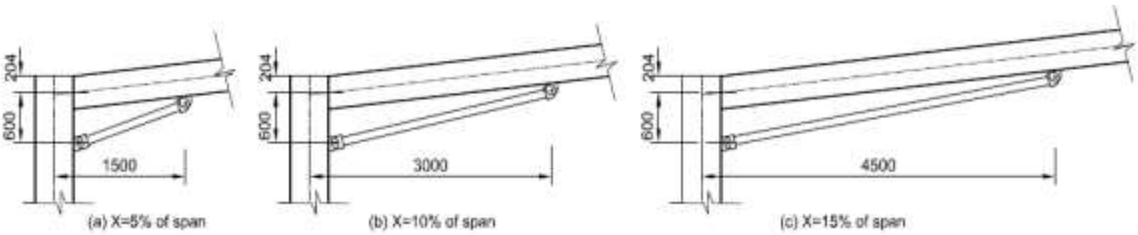


Figure 3.9 Knee brace arrangement for 30m span frame with Y= 2% of span

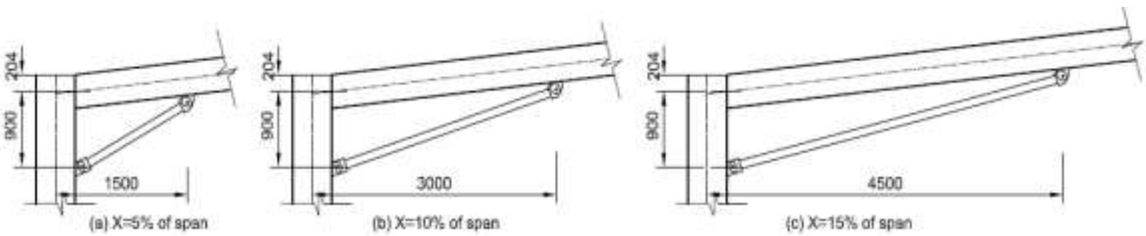


Figure 3.10 Knee brace arrangement for 30m span frame with Y= 3% of span

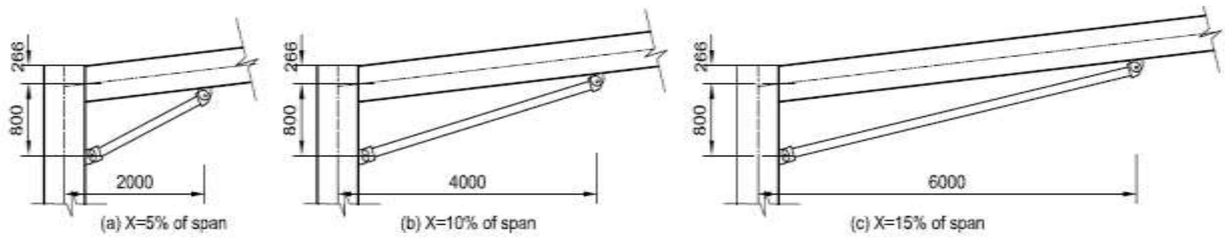


Figure 3.11 Knee brace arrangement for 40m span frame with Y= 2% of span

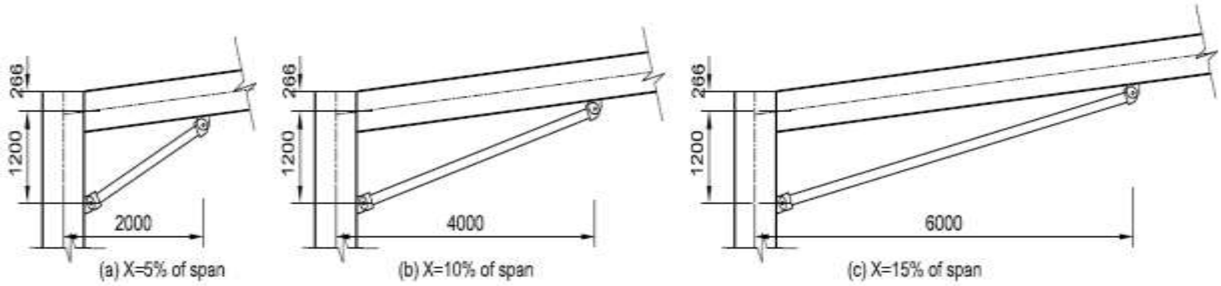


Figure 3.12 Knee brace arrangement for 40m span frame with Y= 3% of span

Analysis was carried out using finite element software (STAAD Pro) to find bending moment of portal frames and compare relevant values for portal frame with haunch and without haunch situation.

The connections of knee brace at column and rafter were modeled as pinned connections, then there is no moment at the connection and knee brace is purely acting as compression member. The concept of the connection is shown in Figure 3.13.

Figure 3.13 Concept of knee brace connection

All analysis results of bending moments and buckling amplification factors are tabulated to compare each other. The detail discussion of results to find out best option of knee brace and its advantages are presented in proceeding chapter 4.

4. RESULTS AND DISCUSSION

In this chapter, all the results received from the analysis and design methodology explained in Chapter 3 are discussed.

In the first step, bending moment (BM) diagrams received for different configuration of knee brace (i.e. X= 5%, 10%, 15% of span and Y=2%, 3% of span) are reviewed and compare with the BM diagram for with haunch and without haunch situations. The percentage variation of BM corresponding to the value of the haunched portal frame, for different configurations of the knee brace were determined to select the optimum location of knee brace.

Then the relevant values for buckling amplification factor are presented to validate optimum location of knee brace selected.

At the later part of the chapter, design resistance of knee brace, feasibility of rafter connection and column resistance to transverse force are discussed in detail.

4.1. Comparison of Bending Moments

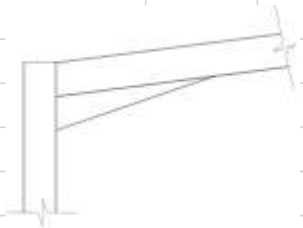
The Bending Moments (BM) at the column top and different locations of the rafter; the eaves connection, the knee brace connection and the apex connection are presented in Table 4.3 to Table 4.8. Each table represent identified knee brace configuration like X=5% and Y=2%, X=10%, Y=2% etc… up to X=15%, Y=3%.

Corresponding BM values for portal frame with haunch and without haunch conditions are shown in Table 4.1 and Table 4.2 respectively.

The typical BM diagram with notations and general arrangement of knee brace are also shown at the bottom of relevant tables.

Table 4.1 Bending Moment of frame with haunch

Span (m)	eave height (m)	Column section	Rafter section	Moment resistance $M_{c,Rd}$ (kNm) Column	Moment resistance $M_{c,Rd}$ (kNm) Rafter	With haunch M11 (kNm)	With haunch M12 (kNm)	With haunch M13 (kNm)	With haunch M14 (kNm)
20	4	406x178x60 UB	356x127x33 UB	376	165	311	311	113	147
20	6	406x178x67 UB	356x127x33 UB	422	165	318	318	131	148
30	4	533x210x82 UB	406x178x60 UB	630	376	670	670	213	315
30	6	533x210x92 UB	406x178x60 UB	740	376	700	700	254	330
40	4	686x254x125 UB	533x210x82 UB	1235	630	1141	1141	315	537
40	6	686x254x140 UB	533x210x82 UB	1400	630	1218	1218	388	579



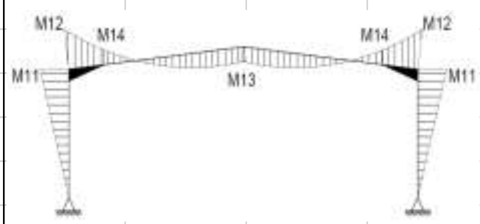


Table 4.2 Bending Moment of frame without haunch

Span (m)	eave height (m)	Column section	Rafter section	Moment resistance $M_{c,Rd}$ (kNm) Column	Moment resistance $M_{c,Rd}$ (kNm) Rafter	Without haunch M11 (kNm)	Without haunch M12 (kNm)	Without haunch M13 (kNm)	Without haunch M14 (kNm)
20	4	406x178x60 UB	356x127x33 UB	376	165	291	291	138	126
20	6	406x178x67 UB	356x127x33 UB	422	165	296	296	155	127
30	4	533x210x82 UB	406x178x60 UB	630	376	631	631	264	275
30	6	533x210x92 UB	406x178x60 UB	740	376	656	656	310	285
40	4	686x254x125 UB	533x210x82 UB	1235	630	1080	1080	402	473
40	6	686x254x140 UB	533x210x82 UB	1400	630	1144	1144	485	504

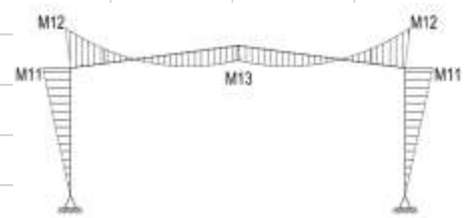
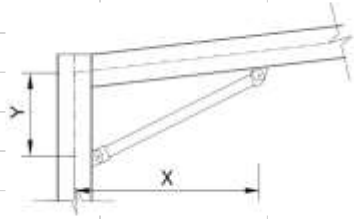
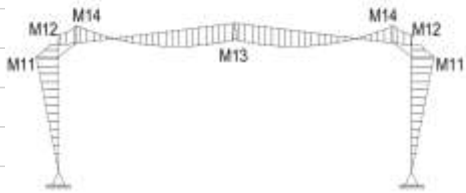


Table 4.3 Bending Moment of frame with knee brace: X= 5%, Y=2% span

Span (m)	eave height (m)	Column section	Rafter section	Moment resistance $M_{c,Rd}$ (kNm) Column	Moment resistance $M_{c,Rd}$ (kNm) Rafter	with knee brace, X= 5% of span, Y= 2% of span M11 (kNm)	M12 (kNm)	M13 (kNm)	M14 (kNm)
20	4	406x178x60 UB	356x127x33 UB	376	165	269	136	129	211
20	6	406x178x67 UB	356x127x33 UB	422	165	283	158	148	213
30	4	533x210x82 UB	406x178x60 UB	630	376	550	252	241	458
30	6	533x210x92 UB	406x178x60 UB	740	376	606	270	288	476
40	4	686x254x125 UB	533x210x82 UB	1235	630	886	373	364	784
40	6	686x254x140 UB	533x210x82 UB	1400	630	1019	412	443	835

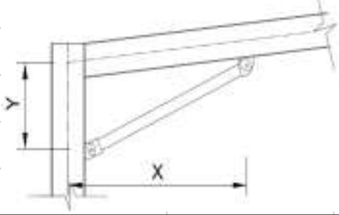


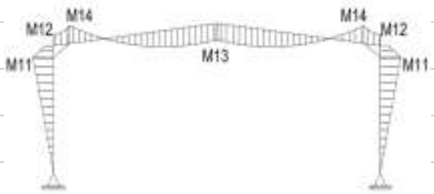


Note: Bending moment, M14 exceeds the bending resistance of selected section for rafter

Table 4.4 Bending Moment of frame with knee brace: X= 5%, Y=3% span

Span (m)	eave height (m)	Column section	Rafter section	Moment resistance $M_{c,Rd}$ (kNm) Column	Moment resistance $M_{c,Rd}$ (kNm) Rafter	with knee brace, X= 5% of span, Y= 3% of span M11 (kNm)	M12 (kNm)	M13 (kNm)	M14 (kNm)
20	4	406x178x60 UB	356x127x33 UB	376	165	257	84	125	214
20	6	406x178x67 UB	356x127x33 UB	422	165	276	106	144	216
30	4	533x210x82 UB	406x178x60 UB	630	376	507	137	233	465
30	6	533x210x92 UB	406x178x60 UB	740	376	578	151	279	483
40	4	686x254x125 UB	533x210x82 UB	1235	630	782	178	350	794
40	6	686x254x140 UB	533x210x82 UB	1400	630	950	203	428	847

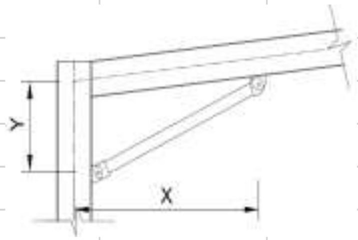




Note: Bending moment, M14 exceeds the bending resistance of selected section for rafter

Table 4.5 Bending Moment of frame with knee brace: X= 10%, Y=2% span

Span (m)	eave height (m)	Column section	Rafter section	Moment resistance $M_{c,Rd}$ (kNm) Column	Moment resistance $M_{c,Rd}$ (kNm) Rafter	with knee brace, X= 10% of span, Y= 2% of span M11 (kNm)	M12 (kNm)	M13 (kNm)	M14 (kNm)
20	4	406x178x60 UB	356x127x33 UB	376	165	276	129	119	142
20	6	406x178x67 UB	356x127x33 UB	422	165	291	133	137	142
30	4	533x210x82 UB	406x178x60 UB	630	376	564	242	221	309
30	6	533x210x92 UB	406x178x60 UB	740	376	622	260	265	321
40	4	686x254x125 UB	533x210x82 UB	1235	630	908	362	324	530
40	6	686x254x140 UB	533x210x82 UB	1400	630	1047	398	401	568



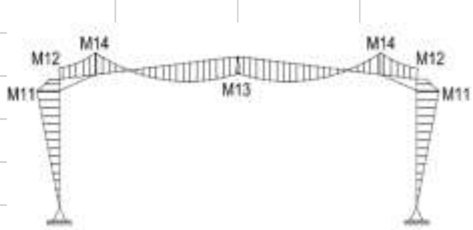
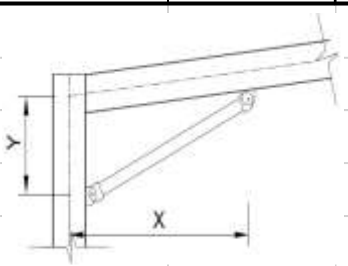


Table 4.6 Bending Moment of frame with knee brace: X= 10%, Y=3% span

Span (m)	eave height (m)	Column section	Rafter section	Moment resistance $M_{c,Rd}$ (kNm) Column	Moment resistance $M_{c,Rd}$ (kNm) Rafter	with knee brace, X= 10% of span, Y= 3% of span M11 (kNm)	M12 (kNm)	M13 (kNm)	M14 (kNm)
20	4	406x178x60 UB	356x127x33 UB	376	165	266	66	111	148
20	6	406x178x67 UB	356x127x33 UB	422	165	287	69	129	149
30	4	533x210x82 UB	406x178x60 UB	630	376	524	105	203	323
30	6	533x210x92 UB	406x178x60 UB	740	376	600	116	247	336
40	4	686x254x125 UB	533x210x82 UB	1235	630	807	133	297	551
40	6	686x254x140 UB	533x210x82 UB	1400	630	985	153	371	593



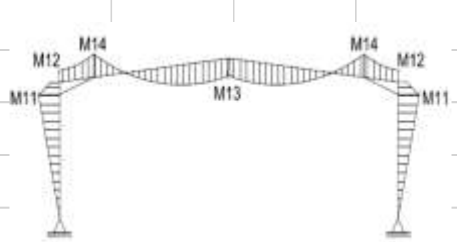
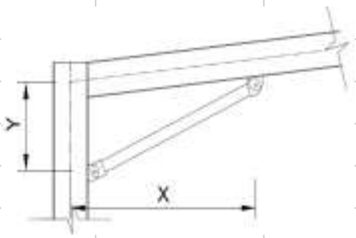


Table 4.7 Bending Moment of frame with knee brace: X= 15%, Y=2% span

Span (m)	eave height (m)	Column section	Rafter section	Moment resistance $M_{c,Rd}$ (kNm) Column	Moment resistance $M_{c,Rd}$ (kNm) Rafter	with knee brace, X= 15% of span, Y= 2% of span M11 (kNm)	M12 (kNm)	M13 (kNm)	M14 (kNm)
20	4	406x178x60 UB	356x127x33 UB	376	165	281	141	111	81
20	6	406x178x67 UB	356x127x33 UB	422	165	298	147	130	79
30	4	533x210x82 UB	406x178x60 UB	630	376	577	258	200	182
30	6	533x210x92 UB	406x178x60 UB	740	376	638	277	243	188
40	4	686x254x125 UB	533x210x82 UB	1235	630	926	419	291	314
40	6	686x254x140 UB	533x210x82 UB	1400	630	1074	421	361	344



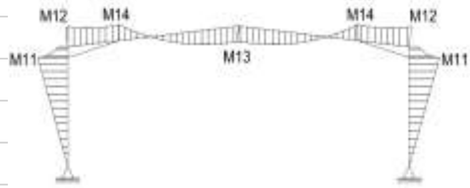
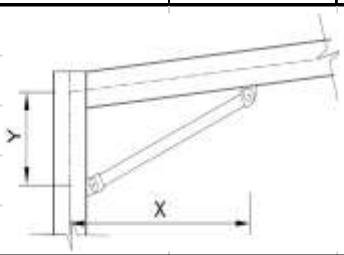
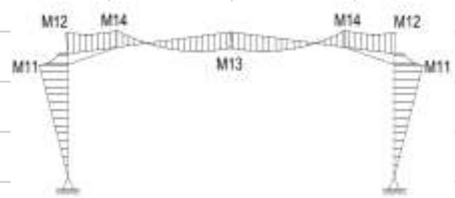


Table 4.8 Bending Moment of frame with knee brace: X= 15%, Y=3% span

Span (m)	eave height (m)	Column section	Rafter section	Moment resistance $M_{c,Rd}$ (kNm) Column	Moment resistance $M_{c,Rd}$ (kNm) Rafter	with knee brace, X= 15% of span, Y= 3% of span M11 (kNm)	M12 (kNm)	M13 (kNm)	M14 (kNm)
20	4	406x178x60 UB	356x127x33 UB	376	165	274	79	99	91
20	6	406x178x67 UB	356x127x33 UB	422	165	296	83	118	90
30	4	533x210x82 UB	406x178x60 UB	630	376	540	125	176	203
30	6	533x210x92 UB	406x178x60 UB	740	376	620	137	217	210
40	4	686x254x125 UB	533x210x82 UB	1235	630	829	193	252	345
40	6	686x254x140 UB	533x210x82 UB	1400	630	1014	217	324	375





Bending moment (BM) analysis results could be summarized with salient points as indicated below;

In all portal frames with the haunch, column top bending moment (M11) show the largest value compare to all other locations of portal frame. This is obvious as haunch enhances the stiffness of the connection and therefore it will share the largest portion of BM of frame. Then bending moment (M11) without haunch is reduced and about 93% of the corresponding value of with haunch condition. When the knee brace is used in place of haunch in the portal frame, the bending moment of column (M11) is decreased by 11% to 17%. The values are increased when the length of knee brace is extended from 5% to 15% of the span. On the other hand, the knee brace arrangement of 3% of span depth (i.e. Y=3% of span) gives lower bending moment values than corresponding values for Y-2% of span. This indicates when the slop of the knee brace to horizontal is increased, the column bending moment is decreased. Therefore, knee brace arrangement with Y=3% of span is preferable than Y=2% of span by considering bending moment at column. Bending moment of column is also increased by about 15% when eave height is increased from 4m to 6m.

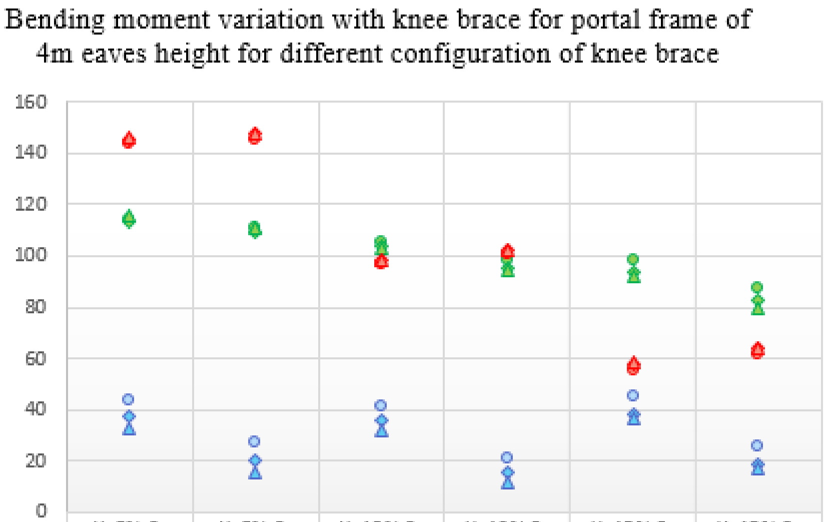
Considering BM of rafter at eave connection (i.e. M12), again high values are shown with the haunch due to relatively high stiffness of haunch. When the knee brace is used, bending moment of rafter at eave connection is lower than corresponding value of both with haunch and without haunch condition. By looking at the results, optimum value is shown for X=10% span and Y=3% of span configuration of knee brace for both 4m eave height and 6m eave height. The bending moment of rafter at eave connection is increased by a range of 5% to 10% when eave height is increased from 4m to 6m.

The bending moment at the apex (i.e.. M13), is highest in portal frame without a haunch. Portal frame with knee brace for X=5% of span shows 10% increment of M13 from the corresponding value for with haunch condition. But M13 results for X=10% condition of knee brace are close to corresponding results with haunch. It is also clear that apex bending moment is decreased with length of bracing extending from 5% to 15% of span. Portal frame with knee brace configuration of Y=3% span gives M13 values about 93% corresponding M13 values for Y=2% span configuration. It is also observed that apex bending moment is increased to a range of 15% to 20% when eave height is increased from 4m to 6m.

Bending moment of rafter at knee brace connection (i.e. M14) is increased by about 45% when knee brace is place at X=5% of span, and it exceeds the bending resistance of selected section as well. Having considered this factor, knee brace arrangement of X=5% span is unsatisfactory and not a suitable option. When knee brace is placed at X=10% span, the values of M14 shows almost equal values corresponding to the frame with the haunch. The values are also decreased with increasing brace length from 10% to 15% of span, but it was decided the optimum vale is at 10% span knee brace. When the eave height is increased from 4m to 6m, portal frame of 20m span shows almost same results for M14, while 30m and 40m span portal frames show about 5% increment of M14 when increasing eave height.

For clear analysis, the bending moments at different locations of the rafter; the eaves connection, the knee brace connection and the apex connection are determined as percentage values of the corresponding value of the haunched portal frame, for different configurations of the knee brace.

Figure 4.1 and Figure 4.2 shows the corresponding percentage variation of bending moment for different configuration of the knee brace.

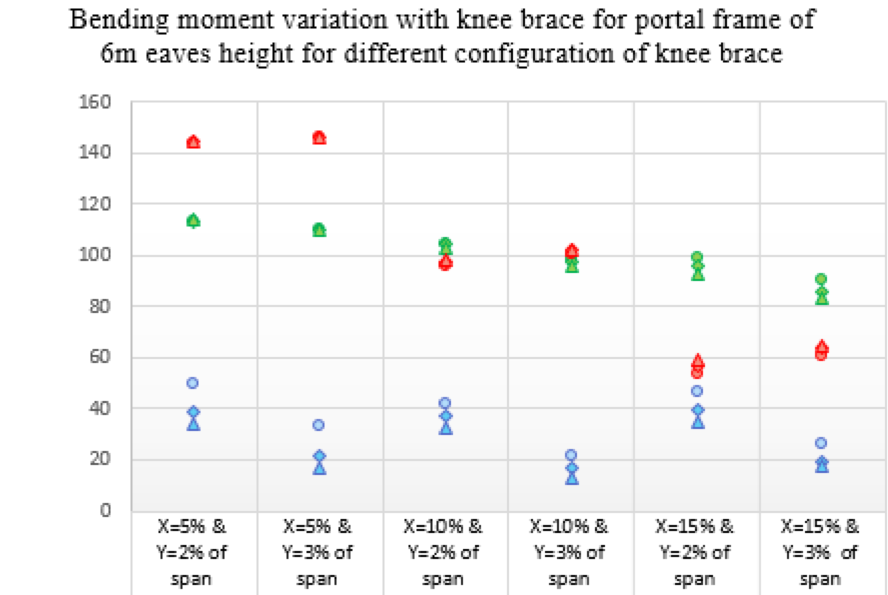


	X=5% & Y=2% of span	X=5% & Y=3% of span	X=10% & Y=2% of span	X=10% & Y=3% of span	X=15% & Y=2% of span	X=15% & Y=3% of span
M12 - 20m	44	27	41	21	45	25
M12 - 30m	38	20	36	16	39	19
M12 - 40m	33	16	32	12	37	17
M13 - 20m	114	111	105	98	98	88
M13 - 30m	113	109	104	95	94	83
M13 - 40m	116	111	103	94	92	80
M14 - 20m	144	146	97	101	55	62
M14 - 30m	145	148	98	103	58	64
M14 - 40m	146	148	99	103	58	64

Figure 4.1 Bending Moment variation for knee braced portal frames of 4m eave

Having considered the Figure 4.1, It is observed that the hogging bending moment at the eaves reduces to a range of 12% to 50% that of the corresponding haunched portal frame. The hogging bending moment at the knee brace connection varies between 55% to 145% that of the bending moment at the end of the haunch of the corresponding haunched portal frame. The maximum sagging bending moment near the apex of the knee braced portal frame ranges between 80% to 110% that of the corresponding haunched portal frame.

It is evident from the trend in variation that the optimum configuration for the knee brace connections is X value of 10% of span and Y value of 3% of span. A similar conclusion can be made pertaining to the 6m eaves height portal frames and the results are sown in Figure 4.2



	X=5% & Y=2% of span	X=5% & Y=3% of span	X=10% & Y=2% of span	X=10% & Y=3% of span	X=15% & Y=2% of span	X=15% & Y=3% of span
M12 - 20m	50	33	42	22	46	26
M12 - 30m	39	22	37	17	40	20
M12 - 40m	34	17	33	13	35	18
M13 - 20m	113	110	105	98	99	90
M13 - 30m	113	110	104	97	96	85
M13 - 40m	114	110	103	96	93	84
M14 - 20m	144	146	96	101	53	61
M14 - 30m	144	146	97	102	57	64
M14 - 40m	144	146	98	102	59	65

Figure 4.2 Bending Moment variation for knee braced portal frames of 6m eave

Considering all above factors, it was decided that the knee bracing is feasible for portal frame in terms of bending moments and the optimum location for knee brace connection to rafter is the 10% of span and to the column at a distance of 2% of the span from the rafter axis.

As the rafter bending moment at column connection is reduced, the connection capacity required to be checked. In the absence of the haunch, the depth of connection is only the depth of rafter.

Since the column bending moment is reduced to a range of 15% to 20% that of the corresponding haunched portal frame, using a reduced section for column is possible, however it need to be checked for stability criteria.

4.2. Comparison of in plane stability of portal frames

The in plane stability of the portal frames was checked by the determination of the buckling amplification factor (α_{cr}) for all of the frames described corresponding to the different knee brace locations.

The α_{cr} value has to be calculated according to the guidelines given in clause 5.2.1 of EN 1993-1-1.

When the axial force in the rafter ($N_{R,Ed}$) is significant, α_{cr} from EN 1993-1-1 is not applicable. Adopting the guidelines in Lim and King (2005) frame stability is assessed based on $\alpha_{cr,est.}$. Detailed calculations are provided in Appendix B.

The axial force is significant if $N_{R,Ed} > 0.09\ N_{cr}$

$$N_{cr,R} = \frac{\pi^2 EI_y}{L_{cr}^2}$$

Where,

L_{cr} is the developed length of the rafter pair from column to column

E = Elastic modulus

I_y = Second moment of area

Detailed explanation of in plane stability of portal frames are provided in Appendix A.

According to the calculations done, it was observed that the axial force in the rafter for all of the portal frames are significant and therefore, buckling amplification factor was calculated based on the values obtained for $\alpha_{cr,est.}$

Detailed calculations of in plane stability of portal frames (i.e. $\alpha_{cr,est}$) for all portal frames are provided in Appendix D.

The $\alpha_{cr,est}$ values for all of the portal frames are shown in Table 4.9

Table 4.9 Buckling Amplification Factor for portal frames

Span (m)	eave height (m)	Column section	Rafter section	with haunch	without haunch	With KB, X=5%,Y=2%	With KB, X=5%,Y=3%	With KB, X=10%,Y=2%	With KB, X=10%,Y=3%	With KB, X=15%,Y=2%	With KB, X=15%,Y=3%
20	4	406x178x60 UB	356x127x33 UB	15.92	13.66	13.90	13.98	14.20	14.43	14.52	14.89
30	4	533x210x82 UB	406x178x60 UB	16.42	14.35	14.54	14.55	14.71	14.81	14.94	15.15
40	4	686x254x125 UB	533x210x82 UB	20.19	18.22	18.27	18.14	18.26	18.13	18.33	18.24
20	6	406x178x67 UB	356x127x33 UB	10.65	9.03	9.93	9.98	10.19	10.47	10.42	10.69
30	6	533x210x92 UB	406x178x60 UB	11.37	9.80	10.05	10.11	10.28	10.39	10.53	10.74
40	6	686x254x140 UB	533x210x82 UB	14.31	13.31	13.32	13.14	13.50	13.36	13.72	13.63

The results in Table 4.9 indicate that the knee braced portal frames have lower $\alpha_{cr,est}$ values than the corresponding values for haunched portal frames. This means the knee brace portal frames are less stable in plane compared with the corresponding haunched portal frame and taller knee braced frames are less stable. The in plane stability of X=5% span knee braced portal frames are similar to portal frame without haunch. The value of $\alpha_{cr,est}$ for portal frames with knee brace configuration of X=5% and 10% is higher than the corresponding value of frame without the haunch. Although using knee brace is not good as haunch in terms of stability, it is certainly better than without haunch.

The variation of $\alpha_{cr,est}$ values of knee braced portal frames with different configuration of knee brace are shown in Figure 4.3 and Figure 4.4. The results show that the configuration of the knee brace has only a marginal effect on the buckling amplification factor.

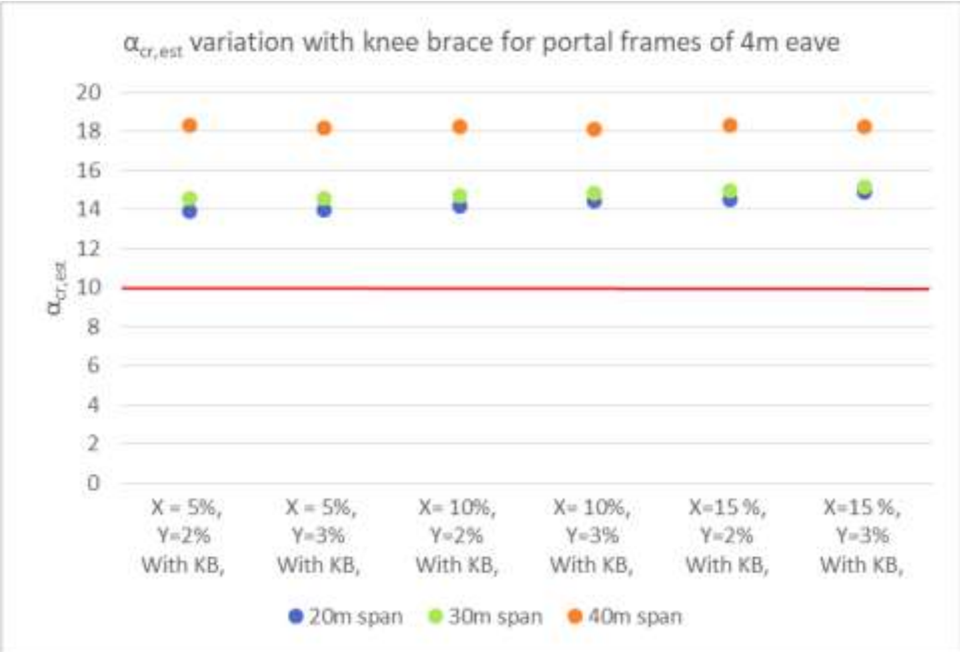


Figure 4.3 $\alpha_{cr,est}$ variation for portal frames with knee brace – 4 m eave height

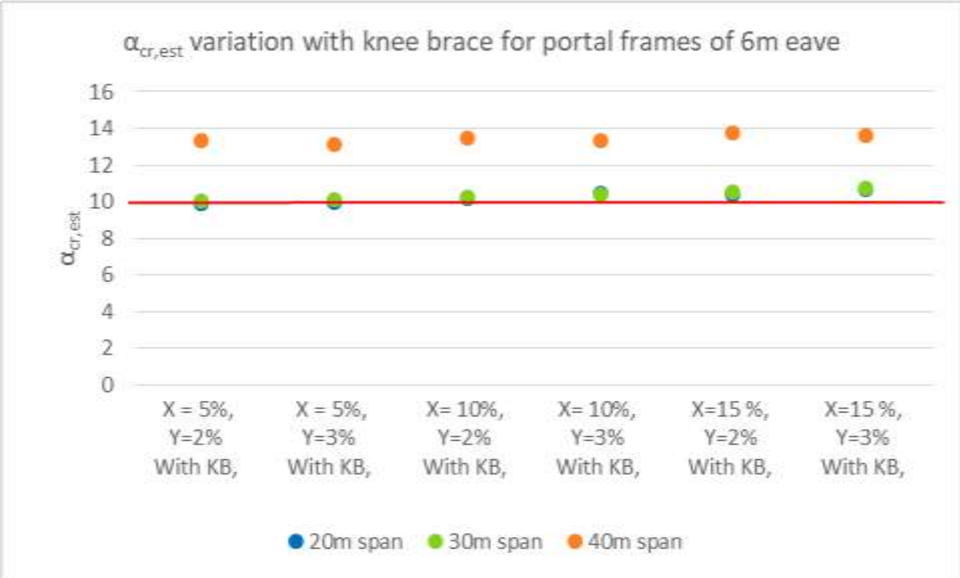


Figure 4.4 $\alpha_{cr,est}$ variation for portal frames with knee brace – 6 m eave height

All of the 4m eaves height knee braced portal frames could be designed accounting only for first order effects since the buckling amplification factor exceeds a value of 10. In the 6m eaves height knee braced frames, the optimized knee brace configuration based on the bending moments variation just meets the requirement of a value of 10 for the buckling amplification factor. Thus the knee braced portal frames have an added advantage that second order effects are not of significance.

Overall, $\alpha_{cr,est}$ has higher values for 4m eave height portal frames than corresponding values of 6m height portal frames. Hence there is a possibility of reducing section sizes for 4m eave height portal fames. That cannot be done for 6m eave height portal frames as the $\alpha_{cr.est}$ values are closer to 10.

4.3. Design resistance of knee brace and cost effectiveness

In order to validate the concept of knee bracing, it is required to design the knee bracing for its capacity.

Design calculation for knee brace resistance was carried out based on the guidelines given in Clause 6.2.4 and clause 6.3.1 of EN 1993-1-1.

Circular Hollow Sections (CHS) of size ranging from 114 to 219 mm were used for knee brace of portal frames considered.

The specimen design calculation for 20m span and eave height of 4m with knee bracing arrangement of X=10% and Y=3% of span is shown in Appendix E.

Tables 4.10, 4.11 and 4.12 provide details of the knee brace both in relation to its structural performance and cost saving when used as an alternative connection to haunches in steel portal frames. In portal frames of 30m and 40m spans the axial force in knee brace decreased when its inclination to the horizontal decreased. However, since its length increases the weight of steel increases.

A knee brace of Circular Hollow Section of 114 mm outer diameter and 5 mm wall thickness was found to have sufficient compression resistance for the 20m span portal frame of both 4 m and 6 m eaves heights when the knee brace is connected at either 5% or 10% of the span length. However, the wall thickness required to be increased to 6.3 mm for the longer knee brace when connected at 15% of the span length.

A similar observation is made in Table 4.11 for the portal frames of 30m span. A knee brace of Circular Hollow Section of 168mm outer diameter and 6.3mm wall thickness was found to have sufficient compression resistance for the 30m span portal frame of both 4 m and 6 m eaves heights when the knee brace is connected at either 5% or 10% of the span length. However, the wall thickness required to be increased to 8 mm for the longer knee brace when connected at 15% of the span length.

Table 4.10 Design resistance of knee brace (K.B.) for 20m span

		20m span with eave height of 4m						20m span with eave height of 6m					
		With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.
		x=5%,y=2%	x=5%,y=3%	x=10%,y=2%	x=10%,y=3%	x=15%,y=2%	x=15%,y=3%	x=5%,y=2%	x=5%,y=3%	x=10%,y=2%	x=10%,y=3%	x=15%,y=2%	x=15%,y=3%
	N_{Ed} (kN)	457	422	464	406	440	412	406	394	467	412	442	390
	Section	CHS 114,5	CHS 114,5	CHS 114,5	CHS 114,5	CHS 114,6.3	CHS 114,6.3	CHS 114,5	CHS 114,5	CHS 114,5	CHS 114,5	CHS 114,6.3	CHS 114,6.3
	L (m)	1.12	1.35	2.09	2.24	3.08	3.2	1.12	1.35	2.09	2.24	3.08	3.2
	mass (kg/m)	13.5	13.5	13.5	13.5	16.8	16.8	13.5	13.5	13.5	13.5	16.8	16.8
	Sec. class	class-1	class-1	class-1	class-1	class-2	class-1	class-1	class-1	class-1	class-1	class-2	class-1
cross section	$N_{c,Rd}$ (kN)	610	610	610	610	759	759	610	610	610	610	759	759
check	$N_{E,d}$ / $N_{c,Rd}$	0.75	0.69	0.76	0.67	0.58	0.54	0.67	0.65	0.77	0.68	0.58	0.51
Buckling	$N_{b,Rd}$ (kN)	580	574	507	495	478	456	580	574	507	495	478	456
resistance	$N_{E,d}$ /$N_{b,Rd}$	0.79	0.74	0.92	0.82	0.92	0.90	0.70	0.69	0.92	0.83	0.92	0.86
Haunch section used		356x127x33 kg/m UB						356x127x33 kg/m UB					
Weight of haunch (kg)		69.30	69.30	69.30	69.30	69.30	69.30	69.30	69.30	69.30	69.30	69.30	69.30
Weight of k.brace (kg)		30.24	36.45	56.43	60.48	103.48	107.52	30.24	36.45	56.43	60.48	103.48	107.52
Steel saving / frame (kg)		39.06	32.85	12.87	8.82	-34.18	-38.22	39.06	32.85	12.87	8.82	-34.18	-38.22
percentage of saving		56.36%	47.40%	18.57%	12.73%	-49.32%	-55.15%	56.36%	47.40%	18.57%	12.73%	-49.32%	-55.15%

CHS = Circular Hollow Section

L = Length of knee brace

Steel grade of S355 used in this calculation

N_{Ed} = Axial compression force (kN)

$N_{c,Rd}$ = Axial resistance of section (kN)

$N_{b,Rd}$ = Buckling resistance of section (kN)

Table 4.11 Design resistance of knee brace (K.B.) for 30m span

		30m span with eave height of 4m						**30m span with eave height of 6m**					
		With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.
		x=5%,y=2%	x=5%,y=3%	x=10%,y=2%	x=10%,y=3%	x=15%,y=2%	x=15%,y=3%	x=5%,y=2%	x=5%,y=3%	x=10%,y=2%	x=10%,y=3%	x=15%,y=2%	x=15%,y=3%
	N_{Ed} (kN)	738	703	734	686	720	665	752	720	752	707	740	688
	Section	CHS 168,6.3	CHS 168,6.3	CHS 168,6.3	CHS 168,6.3	CHS 168,8	CHS 168,8	CHS 168,6.3	CHS 168,6.3	CHS 168,6.3	CHS 168,6.3	CHS 168,8	CHS 168,8
	L (m)	1.68	1.84	3.14	3.24	4.65	4.70	1.68	1.84	3.14	3.24	4.65	4.70
	mass (kg/m)	25.2	25.2	25.2	25.2	31.60	31.60	25.2	25.2	25.2	25.2	31.60	31.60
	Sec. class	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1
cross section	$N_{c,Rd}$ (kN)	1139	1139	1139	1139	1430	1430	1139	1139	1139	1139	1430	1430
check	$N_{E,d}$ / $N_{c,Rd}$	0.65	0.62	0.64	0.60	0.50	0.47	0.66	0.63	0.66	0.62	0.52	0.48
Buckling	$N_{b,Rd}$ (kN)	1105	1083	968	946	887	858	1105	1083	968	946	887	858
resistance	$N_{E,d}$ /$N_{b,Rd}$	0.67	0.65	0.76	0.73	0.81	0.78	0.68	0.66	0.78	0.75	0.83	0.80
Haunch section used		406x178x60kg/m UB						406x178x60kg/m UB					
Weight of haunch (kg)		186.00	186.00	186.00	186.00	186.00	186.00	186.00	186.00	186.00	186.00	186.00	186.00
Weight of k.brace (kg)		84.67	92.74	158.26	163.29	293.88	297.04	84.67	92.74	158.26	163.29	293.88	297.04
Steel saving / frame (kg)		101.33	93.26	27.74	22.71	-107.88	-111.04	101.33	93.26	27.74	22.71	-107.88	-111.04
percentage of saving		54.48%	50.14%	14.91%	12.21%	-58.00%	-59.70%	54.48%	50.14%	14.91%	12.21%	-58.00%	-59.70%

CHS = Circular Hollow Section N_{Ed} = Axial compression force (kN)

L = Length of knee brace $N_{c,Rd}$ = Axial resistance of section (kN)

Steel grade of S355 used in this calculation $N_{b,Rd}$ = Buckling resistance of section (kN)

It is further observed that when the knee brace connection is at 5% of the span, the weight of knee brace reduces to approximately 50% of the weight of the haunch of the corresponding steel frame, when the knee brace connections is at 10% of the span, its weight is approximately 10% to 15% of the haunch of the corresponding steel frame. The weight of knee brace with the least inclination to the horizontal, that is when connected at 15% of the span length, exceeds that of the corresponding haunch. Thus the cost effective configuration for the knee brace is that of the connection at 10% of the span.

Table 4.12 indicates that knee brace of Circular Hollow Section of 193 mm outer diameter and 8 mm wall thickness was found to have sufficient compression resistance for the 40 m span portal frame of both 4 m and 6 m eaves heights when the knee brace is connected at either 5% or 10% of the span length. However, the outer diameter is increased to 8 mm for the longer knee brace when connected at 15% of the span length. Here too, the cost effective knee brace configuration is that when connected at 10% of the span length.

In all of the frames considered, the vertical connection at 3% of the span also proves to be more economical.

Table 4.12 Design resistance of knee brace (K.B.) for 40m span

		40m span with eave height of 4m						**40m span with eave height of 6m**					
		With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.	With K.B.
		x=5%,y=2%	x=5%,y=3%	x=10%,y=2%	x=10%,y=3%	x=15%,y=2%	x=15%,y=3%	x=5%,y=2%	x=5%,y=3%	x=10%,y=2%	x=10%,y=3%	x=15%,y=2%	x=15%,y=3%
	N_{Ed} (kN)	1029	958	1010	917	948	863	1070	1003	1058	969	1051	946
	Section	CHS 193, 8	CHS 193, 8	CHS 193, 8	CHS 193, 8	CHS 219, 8	CHS 219, 8	CHS 193, 8	CHS 193, 8	CHS 193, 8	CHS 193, 8	CHS 219, 8	CHS 219, 8
	L (m)	2.24	2.45	4.18	4.32	6.17	6.27	2.24	2.45	4.18	4.32	6.17	6.27
	mass (kg/m)	36.6	36.6	36.6	36.6	41.6	41.6	36.6	36.6	36.6	36.6	41.6	41.6
	Sec. class	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1	class-1
cross section	$N_{c,Rd}$ (kN)	1657	1657	1657	1657	1885	1885	1657	1657	1657	1657	1885	1885
check	$N_{E,d}$ / $N_{c,Rd}$	0.62	0.58	0.61	0.55	0.50	0.46	0.65	0.61	0.64	0.58	0.56	0.50
Buckling	$N_{b,Rd}$ (kN)	1542	1541	1276	1260	1131	1131	1542	1541	1276	1260	1131	1131
resistance	$N_{E,d}$ /$N_{b,Rd}$	0.67	0.62	0.79	0.73	0.84	0.76	0.69	0.65	0.83	0.77	0.93	0.84
Haunch section used		533x210x82 kg/m UB						533x210x82 kg/m UB					
Weight of haunch (kg)		336.20	336.20	336.20	336.20	336.20	336.20	336.20	336.20	336.20	336.20	336.20	336.20
Weight of k.brace (kg)		163.97	179.34	305.98	316.22	513.34	521.66	163.97	179.34	305.98	316.22	513.34	521.66
Steel saving / frame (kg)		172.23	156.86	30.22	19.98	-177.14	-185.46	172.23	156.86	30.22	19.98	-177.14	-185.46
percentage of saving		51.23%	46.66%	8.99%	5.94%	-52.69%	-55.16%	51.23%	46.66%	8.99%	5.94%	-52.69%	-55.16%

CHS = Circular Hollow Section

L = Length of knee brace

Steel grade of S355 used in this calculation

N_{Ed} = Axial compression force (kN)

$N_{c,Rd}$ = Axial resistance of section (kN)

$N_{b,Rd}$ = Buckling resistance of section (kN)

4.4. Connection design of knee brace

In the analysis, connection of knee brace at column and rafter were considered as pinned connection. There is no moment at the connection and knee brace was designed as a member having axial compression only.

Design calculation for knee brace connection was carried out based on the guidelines given in Clause 3 of EN 1993-1-8. Connection design was done for the selected knee brace configuration of X=10% and Y=3% of span only.

The corresponding arrangement for 40m span frame is shown in Figure 4.5 and the details of connection are shown in figure 4.6.

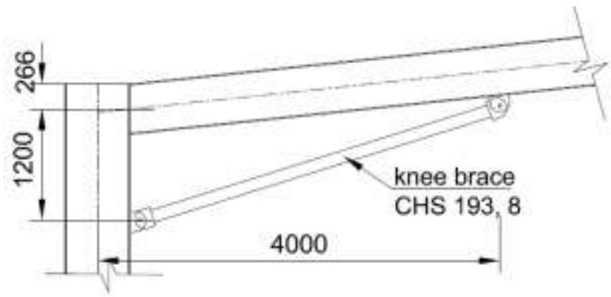


Figure 4.5 Knee brace configuration for 40m span portal frame

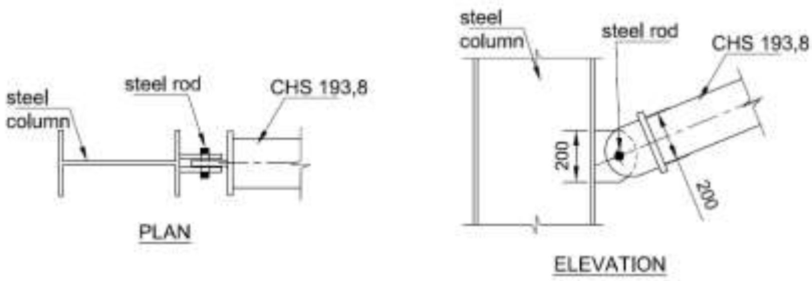


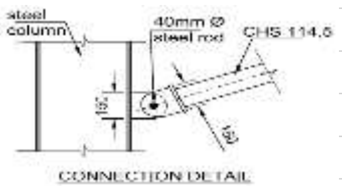
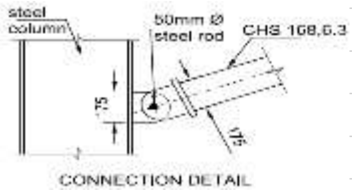
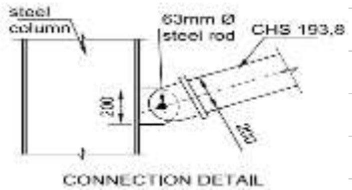
Figure 4.6 Connection detail for knee brace of 40m span portal frame

The specimen design calculation for connection of knee brace for 40m span portal frame is shown in Appendix F.

Based on the calculation it is prudent that pinned connections can be done by using 25mm thick, grade S355 plates and 63mm diameter steel rod (double shear) as per arrangement shown in Figure 4.6. This is a practical solution.

Summary of design calculation for knee brace connection for all three spans are shown in Table 4.13

Table 4.13 Connection design summary of knee brace

			20m span portal frame	30m span portal frame	40m span portal frame
		N_{Ed} (kN)	**412**	**707**	**969**
		knee brace section	CHS 114,5	CHS 168,6.3	CHS 193, 8
		steel rod diameter (mm)	40	50	63
		plate width (mm)	150	175	200
		plate thickness (mm)	20	25	25
Steel plate		gross cross sec. area (mm^2)	3000	4375	5000
		N_{Rd} (kN)	1065	1553	1775
		net cross sec. area (mm^2)	2160	3125	3375
		$N_{U,Rd}$ (kN)	**793**	**1147**	**1239**
Rod	Shear resistance	rod diameter (mm)	40	50	63
		area of rod, A_s (mm^2)	1256	1963	3117
		$F_{v,Rd}$ single shear (kN)	307	480	763
		$F_{v,Rd}$ double shear (kN)	**614**	**960**	**1526**
		bearing resistance $F_{b,Rd}$(kN)	**490**	**765**	**995**
			proposed pinned connection is satisfied	**proposed pinned connection is satisfied**	**proposed pinned connection is satisfied**
		connection detail			

4.5. Feasibility of end moment connection at eave

The knee brace with configuration of X=10% & Y=3% of span was identified as the optimum location. Therefore, feasibility of rafter/column connection for said knee brace configuration was carried out.

The standard details for bolted end moment connections are provided in (SCI P398, 2013). The connection details are reproduced in Figure 4.6. The use of M24 bolts with a 25 mm thick end plate is recommended for beam sizes of 533 Universal Beam Sections and above while the use of M20 bolts with a 20mm thick end plate is recommended for beams sizes of 457 Universal Beam Sections and below.

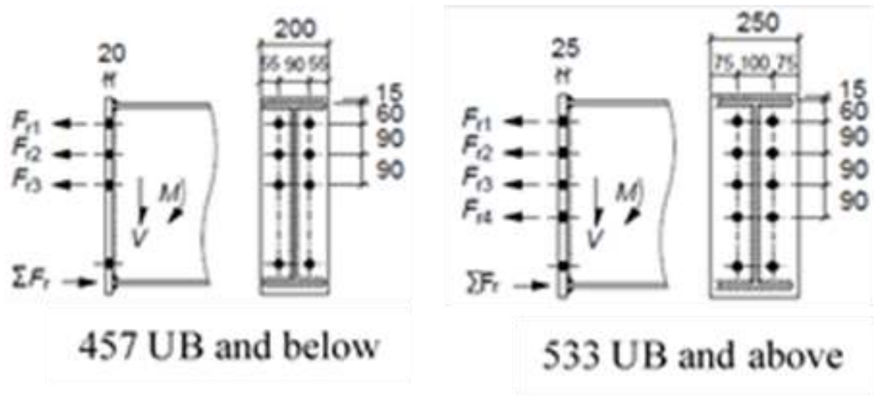


Figure 4.7 Standard connection details from (SCI P398, 2013)

Base on the standard details shown in Figure 4.7, indicative connection resistance are shown in (SCI P398, 2013) which is reproduced as Figure 4.8.

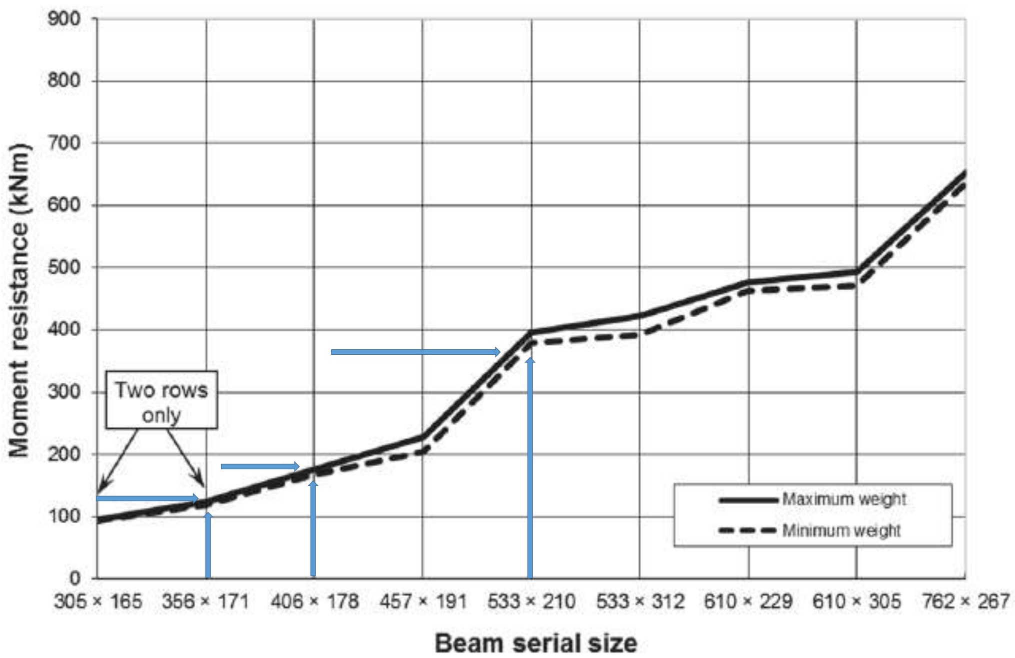


Figure 4.8 Moment resistance of end plate connection from (SCI P398, 2013)

Table 4.14 shows the bending moment results at column/rafter connection (M12) for frames with knee brace configuration of X=10% & Y=3 of span. Table 4.14 also shows the capacity of the moment connection obtained from Figure 4.8

Table 4.14 M12 results for X=10% and Y=3% of span knee brace condition

Span (m)	eave height (m)	Column section	Rafter section	M12 (kNm)	Resistance of connection (kNm)
20	4	406x178x60 UB	356x127x33 UB	66	130
20	6	406x178x67 UB	356x127x33 UB	69	130
30	4	533x210x82 UB	406x178x60 UB	105	175
30	6	533x210x92 UB	406x178x60 UB	116	175
40	4	686x254x125 UB	533x210x82 UB	133	360
40	6	686x254x140 UB	533x210x82 UB	153	360

The moment resistance of bolted end moment connection details recommended in Figure 4.8 exceed the requirements for the design bending moment at the eaves connection for all of the frames considered. Hence the bolted end moment connections is adequate and can be accommodated within the depth of the rafter itself.

4.6. Resistance of column for transverse force from knee brace

It is noted that there is a concentrated load applied on the column from the knee bracing and possibility of failing column web due to this transverse force has to be checked.

Figure 4.9 shows the possible web bucking of column due to the transverse force.

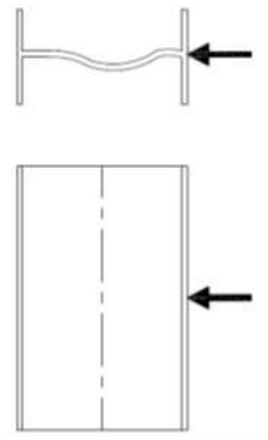


Figure 4.9 Web buckiling due to concentrated force

If the force is larger than the web resistance of column, introduction of web stiffener as shown in Figure 4.10 will be required.

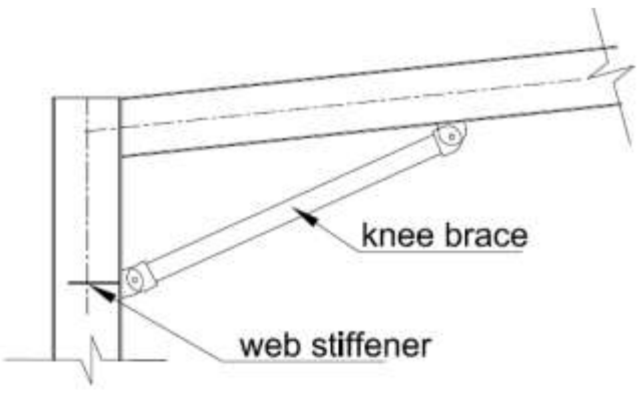


Figure 4.10 Knee bracing with web stiffener at the connection

Checking web of column at knee brace connection is vital, as web can buckle due to concentrated load from knee brace.

In the portal frame of 30m span and 6m eave height with 10% span brace, transverse force from brace is 707kN. According to the calculations carried out for web resistance base on the guidelines given in clause 6.4 of EN 1993-1-5, web resistance of selected column section, 533x210x92kg/m UB is 1009kN which is greater than 707kN. Therefore web of column can take the transverse force from brace without having any web stiffeners.

Specimen calculation is shown in Appendix G.

Since force of all other cases are also less than 1009kN and web of 40m span is thicker than this, web resistance of all other cases are assumed to be adequate.

4.7. Feasibility of using lesser weight sections for 4m eave height

As the column moments are reduced with 10% span brace for 4m eave height, looking at the aspect of using reduced section size is useful. In this regard, the checking of stability criteria is vital.

Calculation was carried out for 20m span with 4m eave height to determine buckling amplification factor, $\alpha_{cr,est}$ using reduced sections for column.

Detailed calculation of buckling amplification factor is shown in Appendix H. Results for all three cases are tabulated in Table 4.15

Table 4.15 Stability and steel saving with reduced sections

		with original section sizes			with reduced section sizes				
Span	eave hei.	Column	Rafter	$\alpha_{cr,s,est}$	Column	Rafter	$\alpha_{cr,s,est}$	steel saving per frame	
(m)	(m)	section	section		section	section		for 2 columns	% saving
20	4	406x178x60 UB	356x127x33 UB	14.72	406x140X46 UB	356x127x33 UB	12.00	112 kg	23.00%
30	4	533x210x82 UB	406x178x60 UB	14.81	457x191x74 UB	406x178x60 UB	13.02	64 kg	9.75%
40	4	686x254x125 UB	533x210x82 UB	18.13	610X229X113 UB	533x210x82 UB	15.37	96 kg	9.60%

$\alpha_{cr,est}$ values are still greater than 10 for all three spans with 4m eave height. Therefore, second order effects can be neglected for designs. It is very clear that there is a possibility of reduce column section with using 10% span knee brace for 4m eave height.

Considerable saving of steel (i.e. 23%) from column reduction can be achieved for 20m span while about 10% saving can be achieved for 30m and 40m spans. Since the rafter moment at brace has no significant change, we will need to use same section for rafter and reduction for rafter is impossible.

This calculation does not apply for 6m eave height as the $\alpha_{cr,est}$ values are much closer to 10 with present sections used.

As the all above calculations in this chapter 4 are done for portal frames with 6 degree roof slope and steel grade of S355, it is also vital to check this knee brace concept for Sri Lankan common practice as well. This has been explained under proceeding chapter 5 in detail.

5. APPLICATION OF SRI LANKAN PRACTICE

In Sri Lankan practice, normally steel grade of S275 is used instead of S355 and typical roof slope in most of the portal frames are 10^0. Therefore, the calculation was carried out to check the feasibility of selected knee brace arrangement of X=10% and Y=3% of span portal frames. This was done for 20m span portal frame only as a comparison with original results.

5.1. Comparison of bending moment results

The original bending moment results with 6^0 roof slope and revised analysis results with 10^0 roof slope are shown in Table 5.1. Same 10kN/m of design action and typical column and beam sections were used for analysis.

Table 5.1 Bending moment of portal frame with different roof slopes

	eave			with roof slope of 6 degrees				with roof slope of 10 degrees			
Span	height	Column	Rafter	M11	M12	M13	M14	M11	M12	M13	M14
(m)	(m)	section	section	(kNm)	(kNm)	(kNm)	(kNm)	(kNm)	(kNm)	(kNm)	(kNm)
20	4	406x178x60 UB	356x127x33 UB	266	66	111	148	250	66	92	137
20	6	406x178x67 UB	356x127x33 UB	287	69	129	149	277	71	114	143

It is clear that when roof slope is increased, the bending moment values of column and rafter are decreased except bending moment at eave (M12).
The moment resistance of selected steel sections with steel grade of S355 and S275 are shown in Table 5.2.

Table 5.2 Bending resistance of selected steel sections

	eave			Moment resistance $M_{c,Rd}$ (kNm)			
Span	height	Column	Rafter	with S355 steel		with S275 steel	
(m)	(m)	section	section	Column	Rafter	Column	Rafter
20	4	406x178x60 UB	356x127x33 UB	376	165	290	129
20	6	406x178x67 UB	356x127x33 UB	422	165	327	129

When consider steel grade of S275, the bending resistance of rafter is 129 kNm. From Table 5.1 rafter bending moment at knee brace connection (ie. M14) exceeds this value and therefore, higher steel section need to be used for rafter. Steel section of 356x127x39 UB was proposed to use as rafter. Analysis was carried out using revised steel section and updated bending moment values with section resistance (S275) are shown in Table 5.3.

Table 5.3 Bending moment of 20m span portal frame with 10^0 roof slope

Span	eave height	Column	Rafter	Moment resistance $M_{c,Rd}$ (kNm)		with roof slope of 10 degrees with knee brace, X= 10% of span, Y= 3% of span			
						M11	M12	M13	M14
(m)	(m)	section	section	Column	Rafter	(kNm)	(kNm)	(kNm)	(kNm)
20	4	406x178x60 UB	356x127x39 UB	290	155	250	78	97	135
20	6	406x178x67 UB	356x127x39 UB	327	155	279	71	115	144

It is clear that relatively high quantity of steel has to be utilized when lower strength steel is used.

Calculation of steel quantity increment due to using of lower strength steel:
Rafter length = 2 x 10 / cos 10^0 = 20.31 m
Total weight increment = (39-33) kg x 20.31 = 121.86 kg
Percentage increment = (6/33) x100% = 18%
This means 18% extra cost has to be spent when using S275 steel.

Although column moments are less than section resistance, the values are very closer to the section resistance. Therefore, previously had benefit of getting saving by using lesser weight column sections, will not be having when steel grade of S275 is used.

Furthermore, in modern world much concerns are given in sustainability and encourage utilizing less material/energy for construction. Having considered all these factors, using higher grade of steel (ie. S355) is preferred than using S275 grade steel for portal frames.

5.2. Determination of in plane stability

It is also necessary to calculate the buckling amplification factor when roof slope is increased from 6 degree to 10 degree. Calculations were carried out to find buckling amplification factor $\alpha_{cr.}$

According to the calculations done, it was observed that the axial force in the rafter for all of the portal frames are significant and therefore, buckling amplification factor was calculated based on the values obtained for $\alpha_{cr,est.}$

All results are shown in Table 5.4 and detail calculations with roof slope of 10 degree are shown in Appendix I.

Table 5.4 Buckling amplification factor for portal frames with two roof slopes

with roof slope of 6^0				
Span (m)	eave height (m)	Column section	Rafter section	With KB, X=10%,Y=3%
20	4	406x178x60 UB	356x127x33 UB	14.43
20	6	406x178x67 UB	356x127x33 UB	10.47
with roof slope of 10^0				
Span (m)	eave height (m)	Column section	Rafter section	With KB, X=10%,Y=3%
20	4	406x178x60 UB	356x127x39 UB	16.32
20	6	406x178x67 UB	356x127x39 UB	11.54

The buckling amplification factor depends only on the frame geometry and does not depend on the steel grade.

It is clear, when the roof slope is increased from 6^0 to 10^0, the buckling amplification factor is also increased by about 10%.

This indicates that 10 degree roof slope is more stable than 6 degree slope in terms of in plane stability. Therefore, using 10 degree slope is more preferable than 6 degree roof slope.

6. SUMMARY AND CONCLUSION

The end-of-life of construction products play an important role, considering the circular economy, as buildings have a long life-span and require a significant amount of material resources. In order to maintain circularity, preserve resources and minimize environmental impact, these resources should be kept in-use through re-use, reclaim or recycling.

Developing countries such as Sri Lanka, are in need of economic reforms. At the initial planning stage, it is necessary to think in terms of minimizing cost and future re use so the waste is minimized. Therefore, alternative solution in construction industry is a must.

The focus of this paper is on extending the life cycle of steel components, with particular emphasis on the design of single span steel portal framed structures by considering the aspects of deconstruction and re-use.

An innovation recently adopted facilitating the re-use of rafters is that of replacing the haunch at the eaves with a steel knee brace pinned at either end to the column and rafter. This concept was investigated in this study. The scope of the study was limited to a span range of 20m to 40m and eaves heights of 4m and 6m and typical vertical action of 10kN/m.

Main attention was given to review the Bending Moment variation on column and rafter when knee brace is used. Stability criteria was also checked by calculating buckling amplification factor for all portal frames.

It was observed that the concept of using knee brace is viable for this range of spans and eave heights. Although using knee brace is not good as haunch in terms of stability, it is certainly better than without haunch. The optimum locations for knee brace connections was found to be 10% of span length at the rafter end and 3% of the span length from the rafter axis at the column end. As the buckling amplification factor $\alpha_{cr,est}$ values are greater than 10, second order effect are not required to consider for the above range. Using 10% of span knee brace gives about 12% saving of steel quantity compared to haunch.

The bending moment at rafter/column connection reduces drastically when using knee brace, and therefore, simple flush end plate connection is possible. This simplified connection also provides advantage of applying proper corrosion resisting coating system as it reduces number of bolts compare to conventional haunched connection. Further, it also facilitate easy access to the connection and subsequent maintenance is easy.

The potential for using lower weight column section with using 10% knee brace for 4m eave height was found to be possible. It was also observed that considerable saving of steel (i.e. 23%) from column reduction can be achieved for 20m span while about 10% saving can be achieved for 30m and 40m spans. But for 6m eave height $\alpha_{cr,est}$ values are much closer to 10, there is not much scope for reducing section sizes for 6m eave height portal frames.

From all above results, the optimum location for the knee brace connection is 10% of span length at the rafter end and 3% of the span length from the axis of the rafter to the column end. This justification was based on multiple criteria; a comparison of bending moments at critical locations on the rafter, the in-plane stability of the portal frame, the moment resistance of the bolted end moment connection, sufficient access for the knee brace connection to column, and the cost effectiveness of the knee brace.

Results indicate that the knee braced steel portal frames are viable within the span range of 20m to 40m and eave heights of 4m to 6m. The optimum location for knee brace is 10% of span length at the rafter end and 3% of the span length from the axis of the rafter to the column end.

This study indicates that greater attention should be paid on the aspect of deconstruction and re-use of steel at the preliminary stages of design in order to extend the life cycle of steel components and thereby enhance the sustainability of steel structures.

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Appendix – A Analysis and design philosophy of portal frames

Frame behavior is paramount important when designing portal frames. The strength checks for any structure are valid only if the global analysis gives a good representation of the behavior of the actual structure.

<u>Global Imperfections</u>

Global imperfections are assumed to follow the shape of the elastic buckling mode of the structure and for erection out-of-plumb; the relevant buckling mode is in sway. The effect of such a global imperfection is most simply modeled by assuming an initial sway which can be achieved by applying equivalent horizontal forces (EHF) applied in the same direction at the top of each column in the frame, in addition to other actions.

The equivalent horizontal force at the top of each column can be determined as

$\phi\ N_{Ed}$,

ϕ is the initial sway imperfection

N_{Ed} is the design value of the compression force in the column; for a portal frame it is equal to the design value of the vertical reaction at the column base.

The magnitudes of the equivalent horizontal forces (EHF) are based on the initial sway imperfection ϕ, given by Equation 5.5 of BS EN 1993-1-1 as:

$\phi = \phi_0\ \alpha_h\ \alpha_m$

Where:

$\phi_0 = 1/200$

α_h is the reduction factor for height h applicable to columns:

$\alpha_h = 2/\sqrt{h}$, but $2/3 \leq \alpha_h \leq 1$

h is the height of the structure in metres. For a portal frame, h should be taken as the height of the columns. For columns 9.0 m high or greater,

$\phi_0\ \alpha_h = 1/300$

α_m is the reduction factor for the number of columns in a row = $\sqrt{0.5(1+1/m)}$

m is the number of vertical members contributing to the horizontal force on the lateral stability system. For single span portal frames m = 2.

The initial sway imperfections may be disregarded if: $H_{Ed} \geq 0.15\ V_{Ed}$

Where:

H_{Ed} is the design value of the horizontal loads

V_{Ed} is the design value of the vertical loads.

Local Imperfections

When any frame is loaded, it deflects and its shape under load is different from the un-deformed shape. The deflection causes the axial loads in the members to act along different lines from those assumed in the analysis, as shown diagrammatically in Figure A.1 and Figure A.2. If the deflections are small, the consequences are very small and a first-order analysis (neglecting the effect of the deflected shape) is sufficiently accurate. However, if the deflections are such that the effects of the axial load on the deflected shape are large enough to cause significant additional moments and further deflection, the frame is said to be sensitive to second order effects. These second order effects, or P-delta effects, can be sufficient to reduce the resistance of the frame.

These second order effects are geometrical effects and should not be confused with non-linear behavior of materials.

As shown in Figure A.1, there are two categories of second order effects:

Effects of deflections within the length of members, usually called P-δ (P-little delta) effects.

Effects of displacements of the intersections of members, usually called P-Δ (P-big delta) effects.

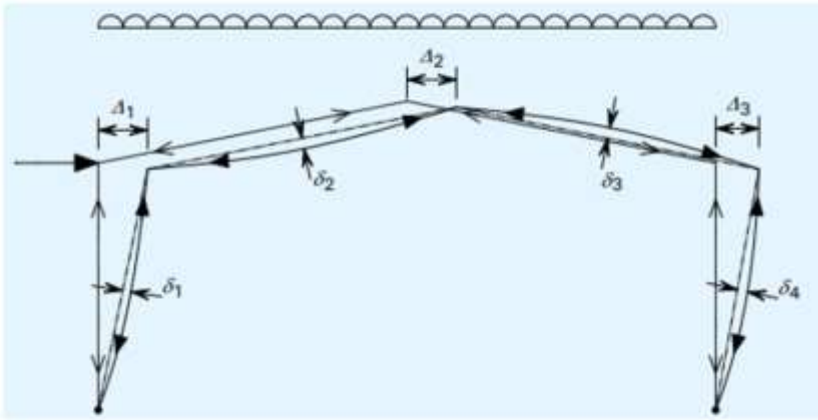


Figure A.1 Asymmetric or sway mode deflection of portal frame

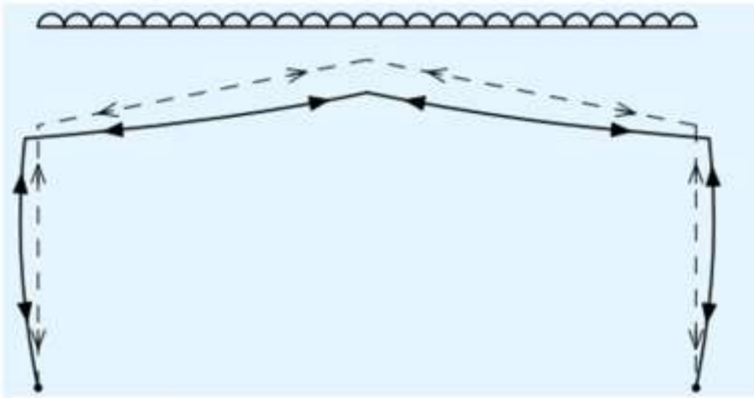


Figure A.2 Symmetric mode deflection of portal frame

The practical consequence of P-δ and P-Δ effects is to reduce the stiffness of the frames and its elements below that calculated by first-order analysis. Single-story portals are sensitive to the effects of the axial compression forces in the rafters and columns. These axial forces are commonly of the order of 10% of the elastic critical buckling loads of the rafters and columns, around which level the reduction in effective stiffness becomes important.

Second order effects may be significant in practical portal frames. Second order effects increase not only the deflections but also the moments and forces beyond those calculated by first-order analysis. Second order analysis is the term used to describe analysis methods in which the effects of increasing deflection under increasing load are considered explicitly in the solution, so that the results include the P-Δ and P-δ effects described above. The results will differ from the results of first-order analysis by an amount dependent on the magnitude of the P-Δ and P-δ effects.

The effects of the deformed geometry are assessed in EN 1993-1-1 by calculating the factor α_{cr}, defined as:

$\alpha_{cr} = F_{cr} / F_{Ed}$

Where:

F_{cr} is the elastic critical buckling load for global instability, based on initial elastic stiffness.

F_{Ed} is the design load on the structure

According to BS EN 1993-1.1 clause 5.2.1(4) B, for the sway mode the value of α_{cr} is calculated as bellow;

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

Where,

H_{Ed} = Algebraic sum of the base shear on the two columns – due to the horizontal load and Equivalent Horizontal Force (EHF)

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

h = Column height

$\delta_{H,Ed}$ = Maximum lateral deflection of at the top of either column, relative to base, when the frame is loaded with horizontal force (i.e. Wind) and the EHF

Since loads on a pitched roof contribute to the lateral deflection at the top of the portal column, it is recommended that α_{cr}, is calculated based on H_{Ed} and $\delta_{H,Ed}$ due only to notional horizontal force (NHF). NHF should be calculated as 1/200 of the design vertical base reaction. They should be applied to each column at eave level on the same direction.

Then,

$$\alpha_{cr} = \left\{\frac{1}{200}\;\frac{h}{\delta_{NHF}}\right\}$$

h = Column height

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

The above expression is applied only when the axial compression in the rafter (N_{Ed}) is not significant.

The axial compression of the rafter is significant if $N_{ed} > 0.09N_{cr,\,R}$

$N_{cr,R}$ is the elastic critical buckling load about the major axis for complete span of the rafter pair,

$$N_{cr.R} = \frac{\pi^2 EI_y}{L_{cr}^2}$$

L_{cr} is the developed horizontal length of the rafter pair from column to column

I_y is the second moment of area rafter about major axis

E is the modulus of elasticity (210000 N/mm^2)

If the axial compression of the rafter is significant, the above expression for α_{cr} cannot be used and $\alpha_{cr,est}$ should be used to assed the effect of deformed geometry.

For pitched roof frame, $\alpha_{cr,est}$= min ($\alpha_{cr,s,est}$; $\alpha_{cr,r,est}$)

$\alpha_{cr,r,est}$ only need to be checked for portal frames of 3 or more spans.

$$\alpha_{cr,s,est} = 0.8\left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\}\left\{\frac{1}{200}\frac{h}{\delta_{NHF}}\right\}$$

$N_{R,Ed}$ = Axial compression in the rafter

$N_{cr,R}$, δ_{NHF} and h are as defined previously.

Second order effects can be ignored in a first order analysis when the frame is sufficiently stiff. Second order effects may be ignored when:

For elastic analysis: $\alpha_{cr} \geq 10$

For plastic analysis: $\alpha_{cr} \geq 15$

When second order effects are significant, two options are possible:

a. Rigorous 2nd order analysis (i.e. in practice, using an appropriate second order software)
b. Approximate 2nd order analysis (i.e. hand calculations using first-order analysis with appropriate allowance for second order effects).

In the second method, also known as ‘modified first order analysis’, all horizontal actions are increased by an amplification factor, to allow for second order effects while using first order calculations.

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

$$\left(\frac{1}{1-1/\alpha_{cr}}\right)$$

If the axial load in the rafter is significant, then $\alpha_{cr,s,est}$ to be calculated and amplification factor is given by;

$$\left(\frac{1}{1-1/\alpha_{cr,est}}\right)$$

Appendix – B Specimen calculation for design action of portal frame analysis

For portal frame of 20m span with 4m eave height and roof angle of 10^0

Design load on the rafter

Permanent actions:

Roof sheeting = 0.05 kN/m^2

Insulation = 0.10 kN/m^2

Purlin = 0.05 kN/m^2

Rafter = 0.15 kN/m^2

Services = 0.15 kN/m^2

Total = 0.50 kN/m^2 on slope

Then, gk = 0.5/cos10^0 = 0.508 kN/m^2 on plan

Imposed load for slope < 30^0 = 0.6 kN/m^2

Assume bay spacing of 6m,

Then, gk = 0.508 x 6 = 3.048 kN/m

qk = 0.6 x 6 = 3.6 kN/m

Design load on rafter (ULS) = (1.35x3.048) + (1.5x3.6) = 9.51 kN/m ≈ 10 kN/m

Appendix – C Preliminary sizing tables – S355 steel with 6^0 roof

	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	762×267×147 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

SCI P399. (2015).

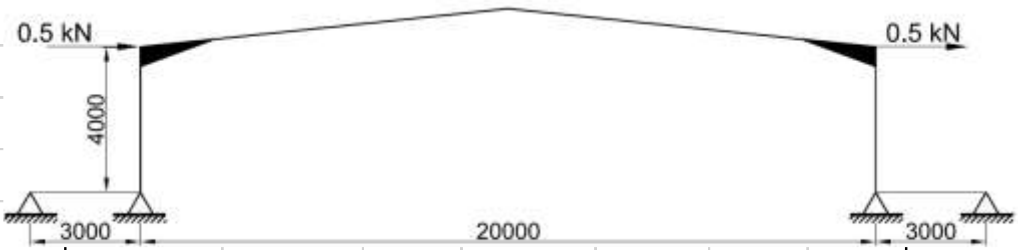
	DESIGN LOAD ON RAFTER (kN/m)	EAVES HEIGHT (m)	SPAN OF FRAME (m)					
			15	20	25	30	35	40
Rafter	14	6	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	457×191×67 UKB	457×191×82 UKB	533×210×92 UKB
	14	8	254×146×31 UKB	356×171×45 UKB	356×171×57 UKB	457×191×674 UKB	533×210×82 UKB	533×210×92 UKB
	14	10	254×146×31 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	356×171×67 UKB	457×191×82 UKB	533×210×82 UKB	610×229×101 UKB
	16	8	254×146×31 UKB	356×171×45 UKB	356×171×67 UKB	457×191×82 UKB	533×210×92 UKB	610×229×101 UKB
	16	10	254×146×31 UKB	356×171×45 UKB	356×171×67 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

SCI P399. (2015).

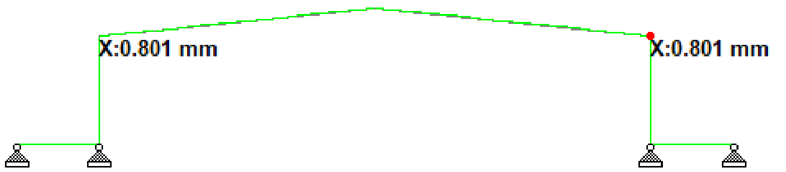
Appendix – D Calculation of buckling amplification factor, $\alpha_{cr,est}$

Referance	Calculations	Output
	<u>For 20m span with eave height of 4m</u>	
	<u>with haunch</u>	
	The vertical reaction at each base ; V_{Ed} = 101 kN	
	The horizontal reaction at each base ; H_{Ed} = 78 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 86 kN	
EN 1993-1-1	The axial compression is significant if $N_{ed} > 0.09N_{cr}$	
Cl.5.2.1	$N_{cr,R} = \dfrac{\pi^2 EI_y}{L_{cr}^2}$	
	Lcr is the developed length of the rafter pair from column to column	
EN 1993-1-1	Lcr = 20/cos 6 = 20.11 m	
Cl.3.2.6	E = 210000 N/mm^2	
	Iy = 8280 cm^4 (Rafter = 356x127x33 kg/m UB)	
	$N_{cr} = \dfrac{\pi^2 \times 210000 \times 8280 \times 10^4 \times 10^{-3}}{(20.11 \times 10^3)^2}$ = 424 kN	
	0.09 Ncr = 38.16 kN	
	$N_{R,Ed}$ = 86 kN > 38.16 kN	
	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
	Following the guidelines from appendix B (developed by J. Lim and C. King)	
	frame stability is assessed base on $\alpha_{cr,est}$	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	For pitched roof frame, $\alpha_{cr,est} = \min(\alpha_{cr,s,est} ; \alpha_{cr,r,est})$	
	$\alpha_{cr,r,est}$ only need to be checked for portal frames of 3 or more spans.	
	To calculate α_{cr}, a notional horizontal force is applied to the frame and	
	the horizontal deflection of the top of the columns is determined under this load.	
	The notional horizontal force is ;	
	$H_{NHF} = \dfrac{1}{200} V_{Ed} = \dfrac{1 \times 101}{200}$ = 0.505 kN	

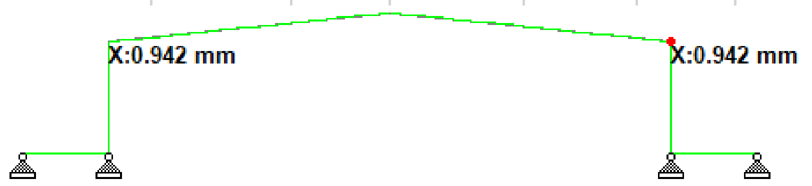
Referance	Calculations	Output
	When assessing frame stability, allowance can be made for the base stiffness.	
	Here a base stiffness equal to 10% of the column stiffness has been	
	assumed to allow for nominally pinned bases.	
	In this regard, modeling is done with dummy member,	
	the length of the dummy member L= 0.75 L_{column}	
	0.75 x 4000 = 3000	
	$Iy = 0.1\ I_{y,column} = 0.1 \times 21600 = 2160\ cm^4$	

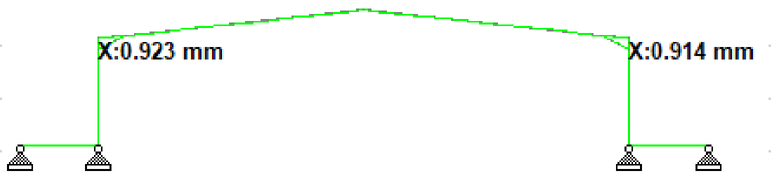


Referance	Calculations	Output
	Frame was modeled as above and analyzed in STAAD Pro V8i software	
	and got results as bellow;	

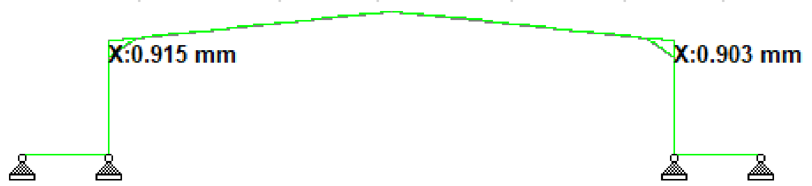


Referance	Calculations	Output
	The horizontal deflection of the top of the column under H_{NHF} is obtained from	
	the elastic analysis as 0.793mm	
	ie; $\delta_{NHF} = 0.801$ mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[86/424]} {(1/200)/(4000/0.801)} = 15.92	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

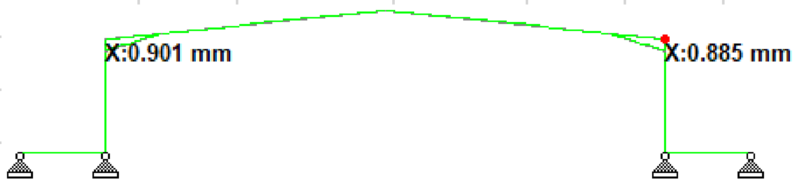
Referance	Calculations	Output
	<u>For 20m span with eave height of 4m</u>	
	<u>Without haunch</u>	
	The vertical reaction at each base ; V_{Ed} = 101 kN	
	The horizontal reaction at each base ; H_{Ed} = 73 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 83 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 83 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix –	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 100}{200}$ = 0.505 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.942 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8\left\{1-\left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\}\left\{\frac{1}{200}\,\frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[83/424]} {(1/200)/(4000/0.942)} = 13.66	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

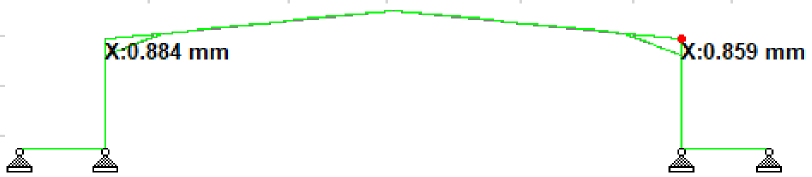
Referance	Calculations	Output
	<u>For 20m span with eave height of 4m</u>	
	<u>with knee brace (X=5% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 101 kN	
	The horizontal reaction at each base ; H_{Ed} = 75 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 84 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 84 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed}$ $\frac{1 \times 101}{200}$ = 0.505 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.923 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[84/424]} {(1/200)/(4000/0.923)} = 13.90	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 20m span with eave height of 4m</u>	
	<u>with knee brace (X=5% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 101 kN	
	The horizontal reaction at each base ; H_{Ed} = 75 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 85 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 85 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 101}{200}$ = 0.505 kN	
	modeling is done with dummy member, as previous and got following results	

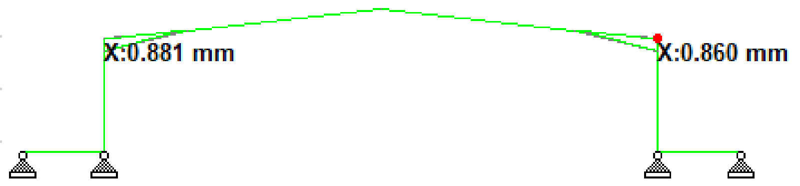


	δ_{NHF} = 0.915 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8\left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\}\left\{\frac{1}{200}\frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[85/424]} {(1/200)/(4000/0.915)} = 13.98	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

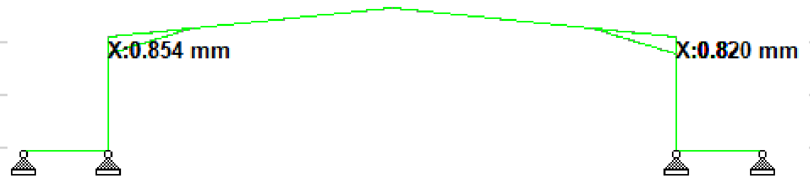
Referance	Calculations	Output
	<u>For 20m span with eave height of 4m</u>	
	<u>with knee brace (X=10% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 101 kN	
	The horizontal reaction at each base ; H_{Ed} = 77 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 85 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 85 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed}$ $\frac{1 \times 101}{200}$ = 0.505 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.901 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[85/424]} {(1/200)/(4000/0.901)} = 14.20	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

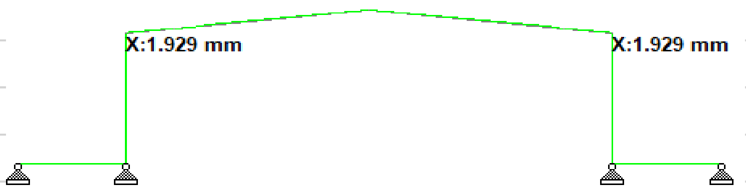
Referance	Calculations	Output
	For 20m span with eave height of 4m	
	with knee brace (X=10% of span and Y=3% of span)	
	The vertical reaction at each base ; V_{Ed} = 101 kN	
	The horizontal reaction at each base ; H_{Ed} = 78 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 86 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 86 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 101}{200} = 0.505$ kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.884 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[86/424]} {(1/200)/(4000/0.884)} = 14.43	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

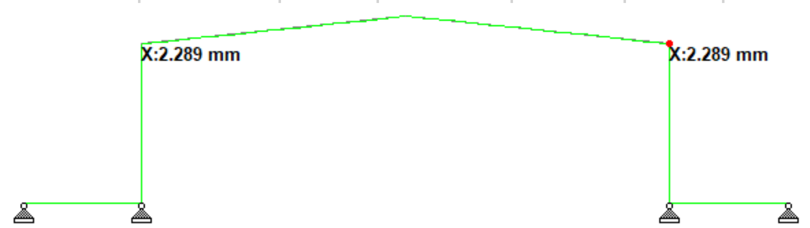
Referance	Calculations	Output
	For 20m span with eave height of 4m	
	with knee brace (X=15% of span and Y=2% of span)	
	The vertical reaction at each base ; V_{Ed} = 101 kN	
	The horizontal reaction at each base ; H_{Ed} = 78 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 85 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 85 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix –B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed}$ $\frac{1 \times 101}{200}$ = 0.505 kN	
	modeling is done with dummy member, as previous and got following results	

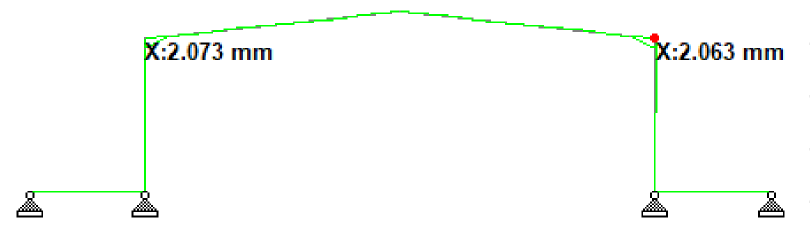


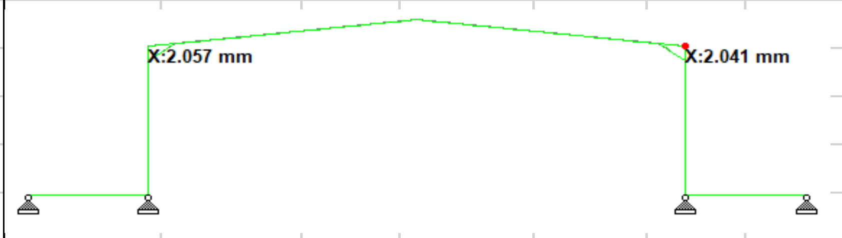
	δ_{NHF} = 0.881 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[85/424]} {(1/200)/(4000/0.881)} = 14.52	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

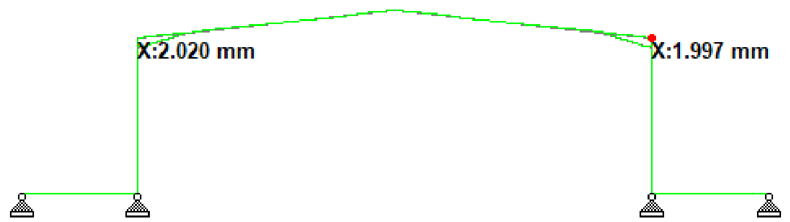
Referance	Calculations	Output
	<u>For 20m span with eave height of 4m</u>	
	<u>with knee brace (X=15% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 101 kN	
	The horizontal reaction at each base ; H_{Ed} = 81 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 87 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 87 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 101}{200}$ = 0.505 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.854 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[87/424]} {(1/200)/(4000/0.854)} = 14.89	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

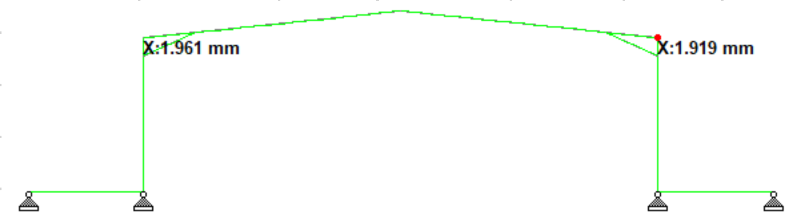
Referance	Calculations	Output
	<u>For 20m span with eave height of 6m</u>	
	<u>with haunch</u>	
	The vertical reaction at each base ; V_{Ed} = 102.6 kN	
	The horizontal reaction at each base ; H_{Ed} = 53 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 61 kN	
EN 1993-1-1	Lcr = 20/cos 6 = 20.11 m	
Cl.3.2.6	E = 210000 N/mm^2	
	Iy = 8280 cm^4 (Rafter = 356x127x33 kg/m UB)	
	$N_{cr} = \dfrac{\pi^2 \times 210000 \times 8280 \times 10^4 \times 10^{-3}}{(20.11 \times 10^3)^2}$ = 424 kN	
EN 1993-1-1	0.09 Ncr = 38.16 kN	
Cl.5.2.1	$N_{R,Ed}$ = 61 kN > 38.16 kN	
	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \dfrac{1}{200} V_{Ed} = \dfrac{1 \times 102}{200}$ = 0.513 kN	
	modeling is done with dummy member,	
	the length of the dummy member L= 0.75 L_{column}	
	0.75 x 6000 = 4500	
	Iy = 0.1 $I_{y,column}$ = 0.1 x 24300 = 2430 cm^4	
	Modeling was done with dummy member and got following results	
		
	δ_{NHF} = 1.929 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \cdot \frac{h}{\delta_{NHF}} \right\}$$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[61/424]} {(1/200)/(6000/1.929)} = 10.65	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	For 20m span with eave height of 6m	
	Without haunch	
	The vertical reaction at each base ; V_{Ed} = 102.6 kN	
	The horizontal reaction at each base ; H_{Ed} = 49 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 59 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 59 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \dfrac{1}{200} V_{Ed} = \dfrac{1 \times 102}{200}$ = 0.513 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} =2.289 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\dfrac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \dfrac{1}{200} \dfrac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[59/424]} {(1/200)/(6000/2.289)} = 9.03	< 10
EN 1993-1-1	Since α_{cr} is less than 10, first order elastic analysis is not enough and	
Cl.5.2	second order effects need to be considered	

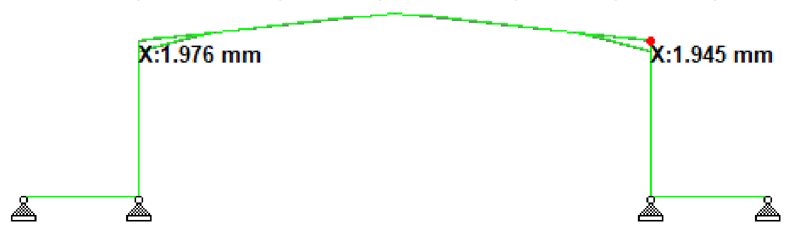
Referance	Calculations	Output
	<u>For 20m span with eave height of 6m</u>	
	<u>with knee brace (X=5% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 102.6 kN	
	The horizontal reaction at each base ; H_{Ed} = 51 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 60 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 60 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed}$ = $\frac{1 \text{ x } 102}{200}$ = 0.513 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 2.073 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[60/424]} {(1/200)/(6000/2.073)} = 9.93	< 10
EN 1993-1-1	Since α_{cr} is less than 10, first order elastic analysis is not enough and	
Cl.5.2	second order effects need to be considered	
Cl.5.2.2 (5)	Amplification factor	
	$\left(\frac{1}{1-1/\alpha_{cr,est}} \right)$ = $\frac{1}{(1- (1/9.93))}$ = 1.11	

Referance	Calculations	Output
	<u>For 20m span with eave height of 6m</u>	
	<u>with knee brace (X=5% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 102.6 kN	
	The horizontal reaction at each base ; H_{Ed} = 51 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 61 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 61 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 102}{200}$ = 0.513 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 2.057 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[61/424]} {(1/200)/(6000/2.057)} = 9.98	< 10
EN 1993-1-1	Since α_{cr} is less than 10, first order elastic analysis is not enough and	
Cl.5.2	second order effects need to be considered	
Cl.5.2.2 (5)	Amplification factor	
	$\left(\frac{1}{1 - 1/\alpha_{cr,est}} \right) = \frac{1}{(1- (1/9.98))\backslash}$ = 1.11	

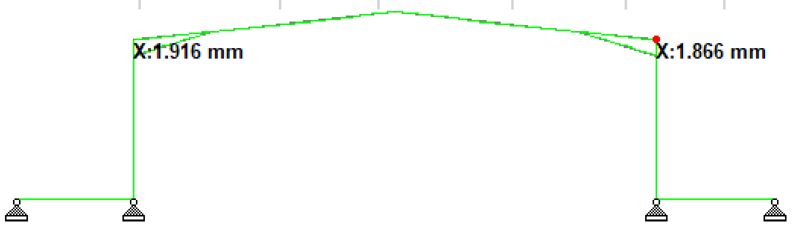
Referance	Calculations	Output
	<u>For 20m span with eave height of 6m</u>	
	<u>with knee brace (X=10% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 102.6 kN	
	The horizontal reaction at each base ; H_{Ed} = 52 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 60 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 60 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 102}{200}$ = 0.513 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 2.02 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[60/424]} {(1/200)/(6000/2.02)} = 10.19	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

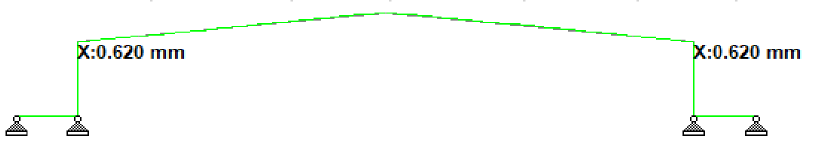
Referance	Calculations	Output
	<u>For 20m span with eave height of 6m</u>	
	<u>with knee brace (X=10% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 102.6 kN	
	The horizontal reaction at each base ; H_{Ed} = 53 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 61 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 61 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 102}{200} = 0.513$ kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.961 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[61/424]} {(1/200)/(6000/1.961)} = 10.47	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

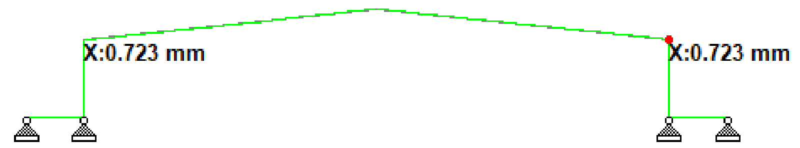
Referance	Calculations	Output
	<u>For 20m span with eave height of 6m</u>	
	<u>with knee brace (X=15% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 102.6 kN	
	The horizontal reaction at each base ; H_{Ed} = 53 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 60 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 60 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 102}{200}$ = 0.513 kN	
	modeling is done with dummy member, as previous and got following results	



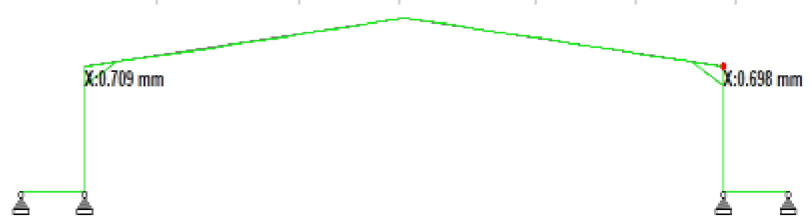
	δ_{NHF} = 1.976 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\}\left\{\frac{1}{200}\frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[60/424]} {(1/200)/(6000/1.976)} = 10.42	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 20m span with eave height of 6m</u>	
	<u>with knee brace (X=15% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 102.6 kN	
	The horizontal reaction at each base ; H_{Ed} = 55 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 62 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 62 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed}$ = $\frac{1 \times 102}{200}$ = 0.513 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.916 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[62/424]} {(1/200)/(6000/1.916)} = 10.69	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	For 30m span with eave height of 4m	
	with haunch	
	The vertical reaction at each base ; V_{Ed} = 151 kN	
	The horizontal reaction at each base ; H_{Ed} = 167 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 179 kN	
EN 1993-1-1	Lcr = 30/cos 6 = 30.16 m	
Cl.3.2.6	E = 210000 N/mm^2	
	Iy = 21600 cm^4 (Rafter = 406x178x60 kg/m UB)	
	$N_{cr} = \dfrac{\pi^2 \times 210000 \times 21600 \times 10^4 \times 10^{-3}}{(30.16 \times 10^3)^2}$ = 492 kN	
EN 1993-1-1	0.09 Ncr = 44.28 kN	
Cl.5.2.1	$N_{R,Ed}$ = 179 kN > 44.28 kN	
	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \dfrac{1}{200} V_{Ed} = \dfrac{1 \times 151}{200}$ = 0.755 kN	
	modeling is done with dummy member,	
	the length of the dummy member L= 0.75 L_{column}	
	0.75 x 4000 = 3000	
	Iy = 0.1 $I_{y,column}$ = 0.1 x 47490 = 4749 cm^4	
	Modeling was done with dummy member and got following results	
		
	δ_{NHF} = 0.62 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\dfrac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \dfrac{1}{200} \dfrac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[179/492]} {(1/200)/(4000/0.62)} = 16.42	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

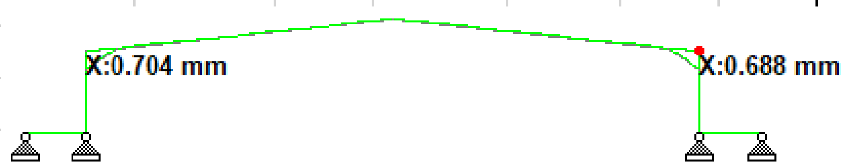
Referance	Calculations	Output
	For 30m span with eave height of 4m	
	Wthout haunch	
	The vertical reaction at each base ; V_{Ed} = 151 kN	
	The horizontal reaction at each base ; H_{Ed} = 158 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 173 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 173 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed}$ = $\frac{1 \times 151}{200}$ = 0.755 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.723 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8\left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\}\left\{\frac{1}{200}\,\frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[173/492]} {(1/200)/(4000/0.723)} = 14.35	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 30m span with eave height of 4m</u>	
	<u>with knee brace (X=5% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 151 kN	
	The horizontal reaction at each base ; H_{Ed} = 162 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 175 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 175 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 151}{200}$ = 0.755 kN	
	modeling is done with dummy member, as previous and got following results	

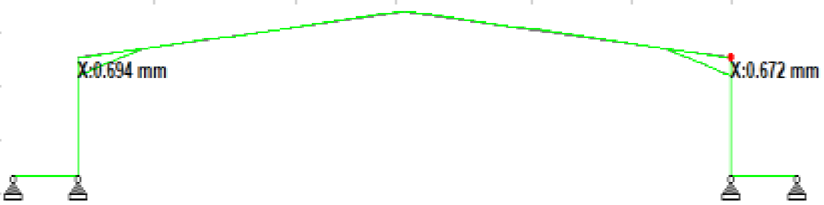


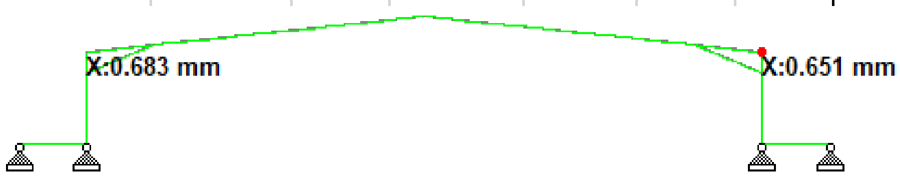
Referance	Calculations	Output
	δ_{NHF} = 0.709 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[175/492]} {(1/200)/(4000/0.709)} = 14.54	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	For 30m span with eave height of 4m	
	with knee brace (X=5% of span and Y=3% of span)	
	The vertical reaction at each base ; V_{Ed} = 151 kN	
	The horizontal reaction at each base ; H_{Ed} = 163 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 177 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 177 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 151}{200} =$ 0.755 kN	
	modeling is done with dummy member, as previous and got following results	

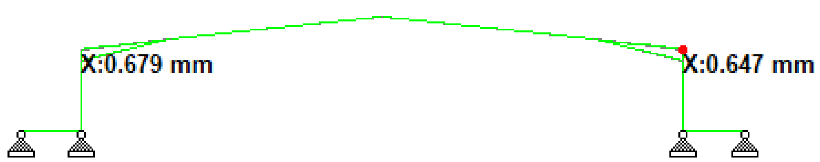


Referance	Calculations	Output
	δ_{NHF} = 0.704 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[177/492]} {(1/200)/(4000/0.704)} = 14.55	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

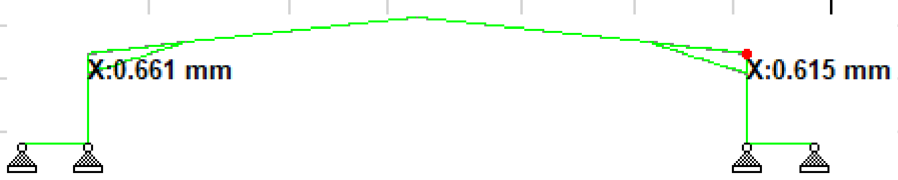
Referance	Calculations	Output
	<u>For 30m span with eave height of 4m</u>	
	<u>with knee brace (X=10% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 151 kN	
	The horizontal reaction at each base ; H_{Ed} = 166 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 178 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 178 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 151}{200}$ = 0.755 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.694 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[178/492]} {(1/200)/(4000/0.694)} = 14.71	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

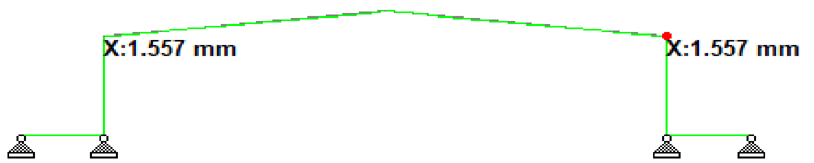
Referance	Calculations	Output
	<u>For 30m span with eave height of 4m</u>	
	<u>with knee brace (X=10% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 151 kN	
	The horizontal reaction at each base ; H_{Ed} = 169 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 181 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 181 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 151}{200}$ = 0.755 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.683 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\frac{1}{200} \frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[181/492]} {(1/200)/(4000/0.683)} = 14.81	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

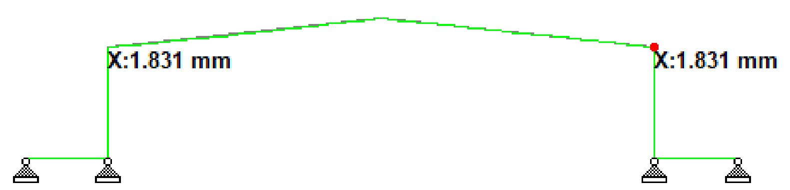
Referance	Calculations	Output
	<u>For 30m span with eave height of 4m</u>	
	<u>with knee brace (X=15% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 151 kN	
	The horizontal reaction at each base ; H_{Ed} = 170 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 180 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 180 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed}$ = $\frac{1 \times 151}{200}$ = 0.755 kN	
	modeling is done with dummy member, as previous and got following results	

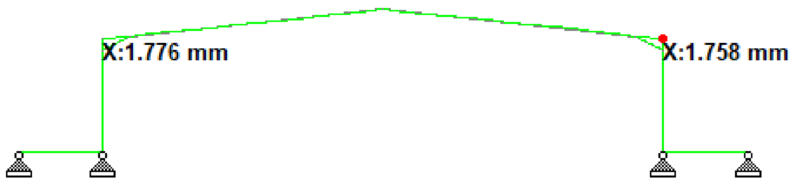


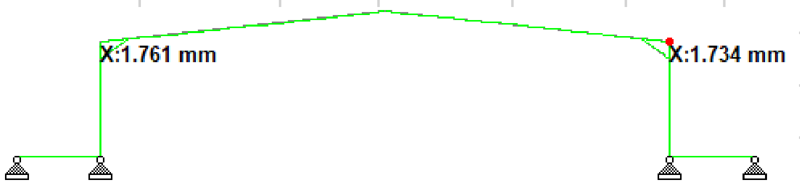
	δ_{NHF} = 0.679 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[180/492]} {(1/200)/(4000/0.679)} = 14.94	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

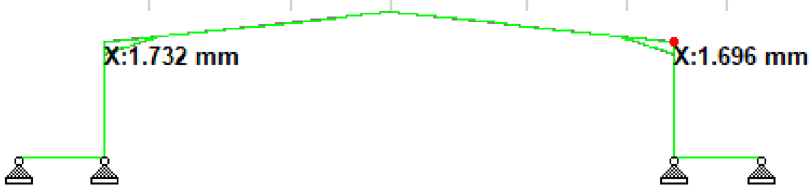
Referance	Calculations	Output
	<u>For 30m span with eave height of 4m</u>	
	<u>with knee brace (X=15% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 151 kN	
	The horizontal reaction at each base ; H_{Ed} = 174 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 184 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 184 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 151}{200}$ = 0.755 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.661 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8\left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\}\left\{\frac{1}{200}\frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[184/492]} {(1/200)/(4000/0.661)} = 15.15	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	For 30m span with eave height of 6m	
	with haunch	
	The vertical reaction at each base ; V_{Ed} = 153 kN	
	The horizontal reaction at each base ; H_{Ed} = 117 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 129 kN	
EN 1993-1-1	Lcr = 30/cos 6 = 30.16 m	
Cl.3.2.6	E = 210000 N/mm^2	
	Iy = 21600 cm^4 (Rafter = 406x178x60 kg/m UB)	
	$N_{cr} = \dfrac{\pi^2 \times 210000 \times 21600 \times 10^4 \times 10^{-3}}{(30.16 \times 10^3)^2}$ = 492 kN	
EN 1993-1-1	0.09 Ncr = 44.28 kN	
Cl.5.2.1	$N_{R,Ed}$ = 129 kN > 44.28 kN	
	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \dfrac{1}{200} V_{Ed} = \dfrac{1 \times 153}{200}$ = 0.765 kN	
	modeling is done with dummy member,	
	the length of the dummy member L= 0.75 L_{column}	
	0.75 x 6000 = 4500	
	Iy = 0.1 $I_{y,column}$ = 0.1 x 55200 = 5520 cm^4	
	Modeling was done with dummy member and got following results	
		
	δ_{NHF} = 1.557 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8\left\{1 - \left[\dfrac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\}\left\{\dfrac{1}{200}\dfrac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[129/492]} {(1/200)/(6000/1.557)} = 11.37	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

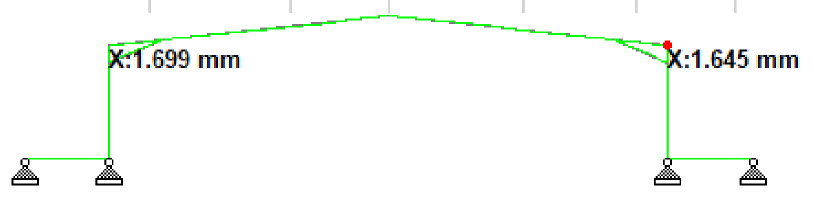
Referance	Calculations	Output
	For 30m span with eave height of 6m	
	Without haunch	
	The vertical reaction at each base ; V_{Ed} = 153 kN	
	The horizontal reaction at each base ; H_{Ed} = 109 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 124 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 124 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 153}{200}$ = 0.765 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.831 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[124/492]} {(1/200)/(6000/1.831)} = 9.80	< 10
EN 1993-1-1	Since α_{cr} is less than 10, first order elastic analysis is not enough and	
Cl.5.2	second order effects need to be considered	

Referance	Calculations	Output
	For 30m span with eave height of 6m	
	with knee brace (X=5% of span and Y=2% of span)	
	The vertical reaction at each base ; V_{Ed} = 153 kN	
	The horizontal reaction at each base ; H_{Ed} = 112 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 126 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 126 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 153}{200} =$ 0.765 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.776 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[126/492]} {(1/200)/(6000/1.776)} = 10.05	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

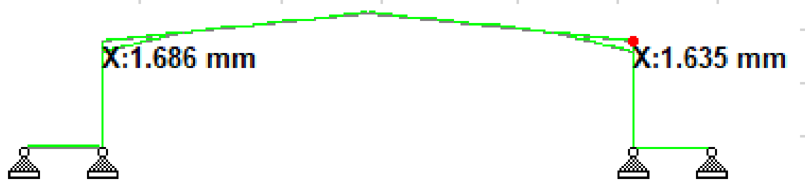
Referance	Calculations	Output
	<u>For 30m span with eave height of 6m</u>	
	<u>with knee brace (X=5% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 153 kN	
	The horizontal reaction at each base ; H_{Ed} = 113 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 127 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 127 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 153}{200}$ = 0.765 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.761 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\frac{1}{200} \frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[127/492]} {(1/200)/(6000/1.761)} = 10.11	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

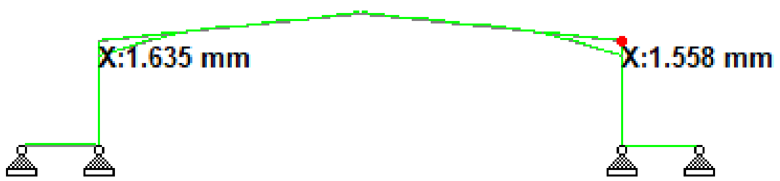
Referance	Calculations	Output
	<u>For 30m span with eave height of 6m</u>	
	<u>with knee brace (X=10% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 153 kN	
	The horizontal reaction at each base ; H_{Ed} = 115 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 127 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 127 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 153}{200}$ = 0.765 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.732 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[127/492]} {(1/200)/(6000/1.732)} = 10.28	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 30m span with eave height of 6m</u>	
	<u>with knee brace (X=10% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 153 kN	
	The horizontal reaction at each base ; H_{Ed} = 118 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 130 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 130 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 153}{200} = 0.765$ kN	
	modeling is done with dummy member, as previous and got following results	

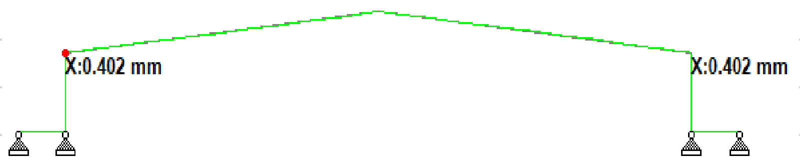


	δ_{NHF} = 1.699 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[130/492]} {(1/200)/(6000/1.699)} = 10.39	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

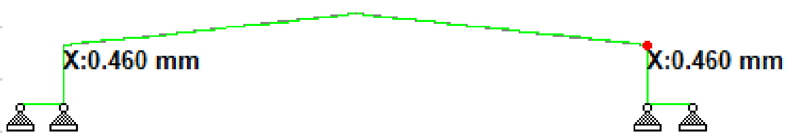
Referance	Calculations	Output
	<u>For 30m span with eave height of 6m</u>	
	<u>with knee brace (X=15% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 153 kN	
	The horizontal reaction at each base ; H_{Ed} = 118 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 128 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 128 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 153}{200} = 0.765$ kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.686 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[128/492]} {(1/200)/(6000/1.686)} = 10.53	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

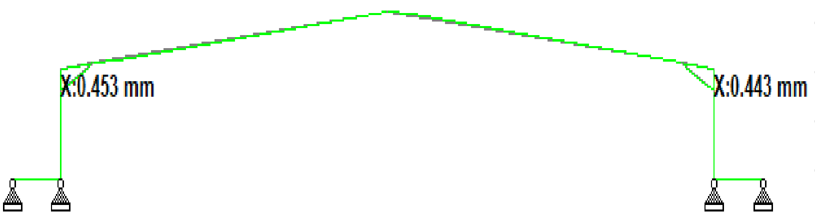
Referance	Calculations	Output
	For 30m span with eave height of 6m	
	with knee brace (X=15% of span and Y=3% of span)	
	The vertical reaction at each base ; V_{Ed} = 153 kN	
	The horizontal reaction at each base ; H_{Ed} = 122 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 132 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 132 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 153}{200}$ = 0.765 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.635 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\frac{1}{200} \frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[132/492]} {(1/200)/(6000/1.635)} = 10.74	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

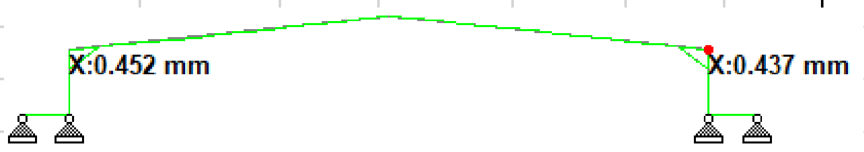
Referance	Calculations	Output
	For 40m span with eave height of 4m	
	with haunch	
	The vertical reaction at each base ; V_{Ed} = 201 kN	
	The horizontal reaction at each base ; H_{Ed} = 285 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 300 kN	
EN 1993-1-1	Lcr = 40/cos 6 = 40.22 m	
Cl.3.2.6	E = 210000 N/mm^2	
	Iy = 47500 cm^4 (Rafter = 533x210x82 kg/m UB)	
	$N_{cr} = \dfrac{\pi^2 \times 210000 \times 47500 \times 10^4 \times 10^{-3}}{(40.22 \times 10^3)^2}$ = 609 kN	
EN 1993-1-1	0.09 Ncr = 54.81 kN	
Cl.5.2.1	$N_{R,Ed}$ = 300 kN > 54.81 kN	
	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \dfrac{1}{200} V_{Ed} = \dfrac{1 \times 201}{200}$ = 1.005 kN	
	modeling is done with dummy member,	
	the length of the dummy member L= 0.75 L_{column}	
	0.75 x 4000 = 3000	
	Iy = 0.1 $I_{y,column}$ = 0.1 x 118000 = 11800 cm^4	
	Modeling was done with dummy member and got following results	

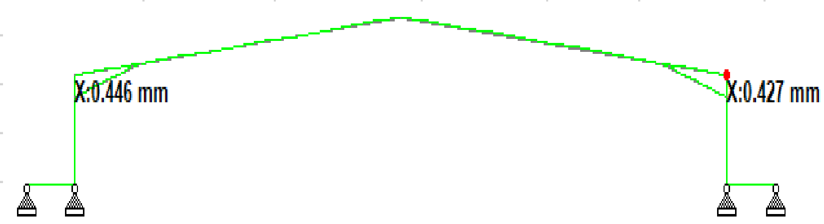


	δ_{NHF} = 0.402 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\dfrac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\dfrac{1}{200}\dfrac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[300/609]} {(1/200)/(4000/0.402)} = 20.19	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

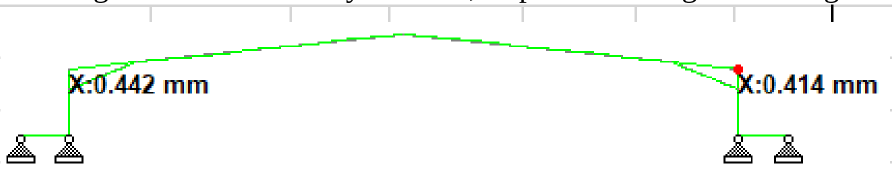
Referance	Calculations	Output
	<u>For 40m span with eave height of 4m</u>	
	<u>Without haunch</u>	
	The vertical reaction at each base ; V_{Ed} = 201 kN	
	The horizontal reaction at each base ; H_{Ed} = 270 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 290 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 290 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 201}{200}$ = 1.005 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.460 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[290/609]} {(1/200)/(4000/0.460)} = 18.22	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	For 40m span with eave height of 4m	
	with knee brace (X=5% of span and Y=2% of span)	
	The vertical reaction at each base ; V_{Ed} = 201 kN	
	The horizontal reaction at each base ; H_{Ed} = 277 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 294 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 294 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 201}{200}$ = 1.005 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.453 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[294/609]} {(1/200)/(4000/0.453)} = 18.27	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

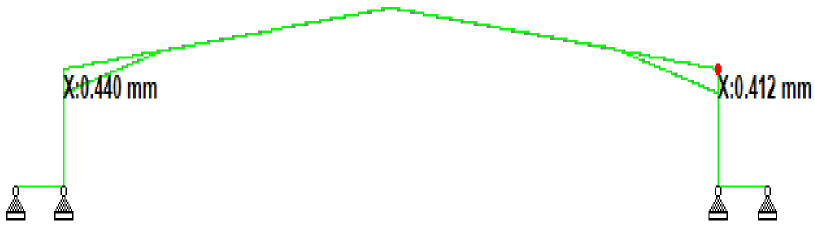
Referance	Calculations	Output
	<u>For 40m span with eave height of 4m</u>	
	<u>with knee brace (X=5% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 201 kN	
	The horizontal reaction at each base ; H_{Ed} = 279 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 297 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 297 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 201}{200}$ = 1.005 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.452 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[297/609]} {(1/200)/(4000/0.452)} = 18.14	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

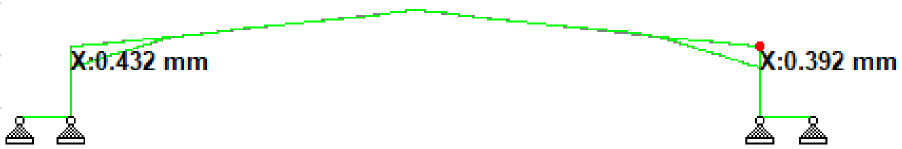
Referance	Calculations	Output
	<u>For 40m span with eave height of 4m</u>	
	<u>with knee brace (X=10% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 201 kN	
	The horizontal reaction at each base ; H_{Ed} = 284 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 299 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 299 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 201}{200} = 1.005$ kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.446 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[299/609]} {(1/200)/(4000/0.446)} = 18.26	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 40m span with eave height of 4m</u>	
	<u>with knee brace (X=10% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 201 kN	
	The horizontal reaction at each base ; H_{Ed} = 288 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 304 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 304 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 201}{200}$ = 1.005 kN	
	modeling is done with dummy member, as previous and got following results	

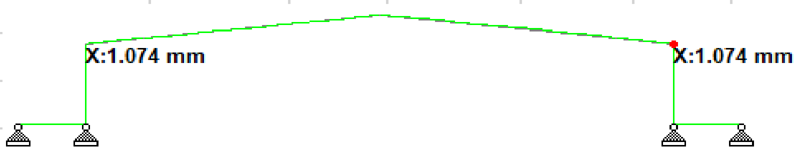


Referance	Calculations	Output
	δ_{NHF} = 0.442 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8\left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\}\left\{\frac{1}{200}\frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[304/609]} {(1/200)/(4000/0.442)} = 18.13	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	For 40m span with eave height of 4m	
	with knee brace (X=15% of span and Y=2% of span)	
	The vertical reaction at each base ; V_{Ed} = 201 kN	
	The horizontal reaction at each base ; H_{Ed} = 289 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 302 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 302 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 201}{200} = 1.005$ kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.44 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\frac{1}{200} \frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[302/609]} {(1/200)/(4000/0.44)} = 18.33	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

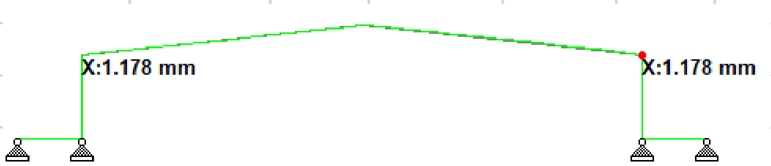
Referance	Calculations	Output
	<u>For 40m span with eave height of 4m</u>	
	<u>with knee brace (X=15% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 201 kN	
	The horizontal reaction at each base ; H_{Ed} = 296 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 309 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 309 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 201}{200}$ = 1.005 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.432 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\frac{1}{200} \frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[309/609]} {(1/200)/(4000/0.432)} = 18.24	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>**For 40m span with eave height of 6m with haunch**</u>	
	The vertical reaction at each base ; V_{Ed} = 204 kN	
	The horizontal reaction at each base ; H_{Ed} = 203 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 219 kN	
EN 1993-1-1	Lcr = 40/cos 6 = 40.22 m	
Cl.3.2.6	E = 210000 N/mm^2	
	Iy = 47500 cm^4 (Rafter = 533x210x82 kg/m UB)	
	$N_{cr} = \dfrac{\pi^2 \times 210000 \times 47500 \times 10^4 \times 10^{-3}}{(40.22 \times 10^3)^2}$ = 609 kN	
EN 1993-1-1	0.09 Ncr = 54.81 kN	
Cl.5.2.1	$N_{R,Ed}$ = 219 kN > 54.81 kN	
	Therefore, the axial compression in the rafter is significant and α_{cr} from EN 1993-1-1 is not applicable.	
Appendix –B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \dfrac{1}{200} V_{Ed} = \dfrac{1 \times 204}{200}$ = 1.02 kN	
	modeling is done with dummy member,	
	the length of the dummy member L= 0.75 L_{column}	
	0.75 x 6000 = 4500	
	Iy = 0.1 $I_{y,column}$ = 0.1 x 136000 = 13600 cm^4	
	Modeling was done with dummy member and got following results	

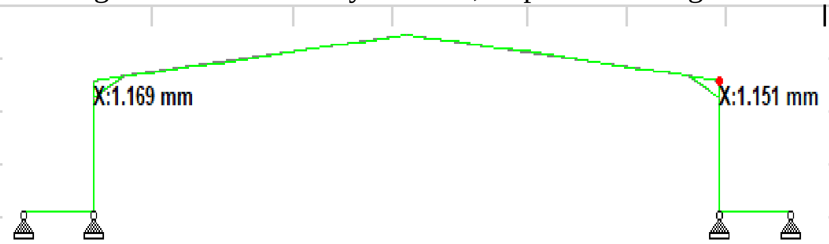


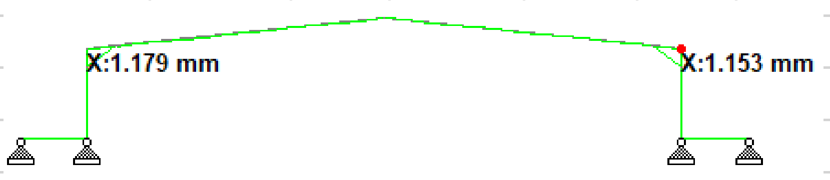
	δ_{NHF} = 1.074 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\dfrac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \dfrac{1}{200} \dfrac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[219/609]} {(1/200)/(6000/1.074)} = 14.31	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

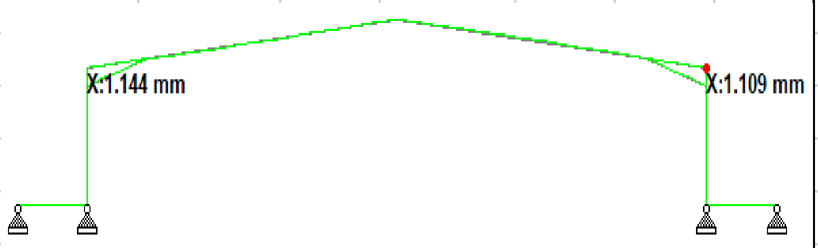
Referance	Calculations	Output
	<u>For 40m span with eave height of 6m</u>	
	<u>Without haunch</u>	
	The vertical reaction at each base ; V_{Ed} = 204 kN	
	The horizontal reaction at each base ; H_{Ed} = 191 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 211 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 211 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 204}{200}$ = 1.02 kN	
	modeling is done with dummy member, as previous and got following results	

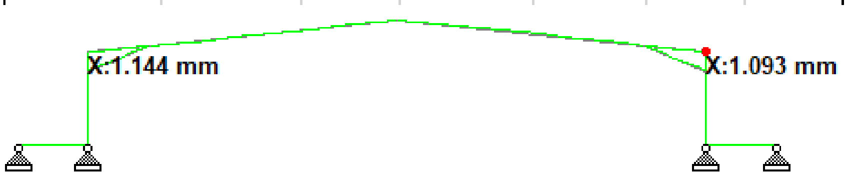


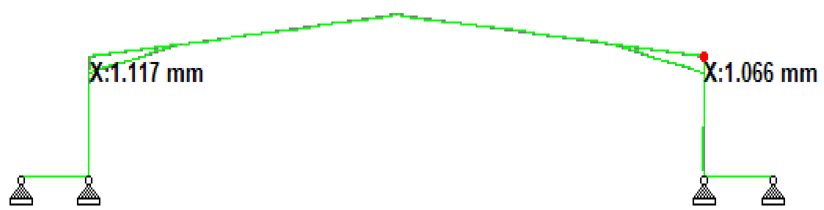
	δ_{NHF} = 1.178 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[211/609]} {(1/200)/(6000/1.178)} = 13.31	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

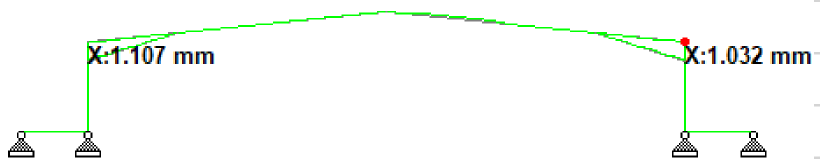
Referance	Calculations	Output
	<u>For 40m span with eave height of 6m</u>	
	<u>with knee brace (X=5% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 204 kN	
	The horizontal reaction at each base ; H_{Ed} = 196 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 214 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 214 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 204}{200}$ = 1.02 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.169 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[214/609]} {(1/200)/(6000/1.169)} = 13.32	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 40m span with eave height of 6m</u>	
	<u>with knee brace (X=5% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 204 kN	
	The horizontal reaction at each base ; H_{Ed} = 198 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 216 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 216 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – I	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed}$ = $\frac{1 \times 204}{200}$ = 1.02 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.179 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[216/609]} {(1/200)/(6000/1.179)} = 13.14	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	For 40m span with eave height of 6m	
	with knee brace (X=10% of span and Y=2% of span)	
	The vertical reaction at each base ; V_{Ed} = 204 kN	
	The horizontal reaction at each base ; H_{Ed} = 201 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 217 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 217 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 204}{200} =$ 1.02 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.144 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8\left\{1-\left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\}\left\{\frac{1}{200}\frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[217/609]} {(1/200)/(6000/1.144)} = 13.50	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 40m span with eave height of 6m</u>	
	<u>with knee brace (X=10% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 204 kN	
	The horizontal reaction at each base ; H_{Ed} = 205 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 221 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 221 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 204}{200}$ = 1.02 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.144 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[221/609]} {(1/200)/(6000/1.144)} = 13.36	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 40m span with eave height of 6m</u>	
	<u>with knee brace (X=15% of span and Y=2% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 204 kN	
	The horizontal reaction at each base ; H_{Ed} = 207 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 220 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 220 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \text{ x } 201}{200} = 1.02$ kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.117 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[220/609]} {(1/200)/(6000/1.117)} = 13.72	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 40m span with eave height of 6m</u>	
	<u>with knee brace (X=15% of span and Y=3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 204 kN	
	The horizontal reaction at each base ; H_{Ed} = 212 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 226 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 226 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 204}{200}$ = 1.02 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 1.107 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[226/609]} {(1/200)/(6000/1.107)} = 13.63	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Appendix – E Specimen calculation for design resistance of knee brace

Reference	Calculations	Output
	This sample calculation is done for portal frame with 20m span and eave height and knee brace arrangement of X=10% of span, Y=3% of span	
Design data	Axial compression force = $N_{E,d}$ = 406kN	
	Clear length = 2.24m	
	Steel grade = S355	
	Select Circular Hollow Section (CHS) of 114 mm diameter with 5mm thickness	
	Section properties from properties table;	
	Weight = 13.5 kg/m	
	A = 17.2 cm^2	
	t = 5.0 mm	
	i = 3.87 cm	
EN 1993-1-1	<u>Section classification:</u>	
Table 3.1	S355, steel and t_w= 5.0 mm <40	
	f_y = 355 N/mm2	
	$\varepsilon = \sqrt{235/355} = 0.81$	
	$\varepsilon^2 = 0.66$	
	d/t = 114/5 = 22.8	
Table 5.2	$50\,\varepsilon^2 = 50 \times 0.66 = 33 > d/t$ → class 1	
Cl. 6.2.4	<u>Cross section capacity check</u>	
	A = 17.2 cm^2= 1720 mm^2	
Cl.6.1(1)	$\gamma_{M0} = 1.0$	Cross
Eq. (6.10)	$N_{c,Rd} = Af_y/\gamma_{M0}$ = 1720 x 355 / 1 = 610kN	section
Eq. (6.9)	$N_{E,d} / N_{c,Rd}$ = 406/610 = 0.67 <1 → Satisfied	OK

Cl 6.3.1 Cl 6.3.1.3(1) Table 6.2 Table 6.1 Cl 6.3.1.2 Cl 6.1(1) Eq. 6.47 Eq. 6.46	<u>Buckling Resistance check</u> $\lambda = \frac{L_{cr}}{i} \frac{1}{\lambda_1}$ both ends free to rotate and restrained to lateral movement in both directions L_{cr}= 1.0 x L = 2.24m λ_1= 93.9 Ɛ = 93.9x0.81 = 76.06 $\lambda = \frac{2.24 \times 10^3 \times 1}{3.87 \times 10 \times 76.06} = 0.76$ Buckling curve for hot finished, any axis; For S355 → curve "a" Imperfection factor, $\alpha = 0.21$ $\chi = 1 / (\phi + \sqrt{\phi^2 - \lambda^2})$; but <1 $\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$ $\phi = 0.5[1 + 0.21(0.76-0.2) + 0.76^2] = 0.85$ $\chi = 1 / (0.85 + \sqrt{0.85^2 - 0.76^2}) = 0.81$ $\gamma_{M1} = 1.00$ $N_{b,Rd} = \chi A f_y / \gamma_{M1} = (0.81 \times 1720 \times 355 \times 10^{-3}) / 1.0 = 495$ kN $N_{E,d} / N_{b,Rd} = 406/495 = 0.82 < 1$ → Satisfied	 Buckling resistance OK

Therefore CHS 114,5mm can be used as knee brace.

Weight of knee brace for the portal frame on both sides = 2x 2.24mx13.5kg/m = 60.48kg

Considering haunch is done using half of rafter section (356x127x33kg/m)

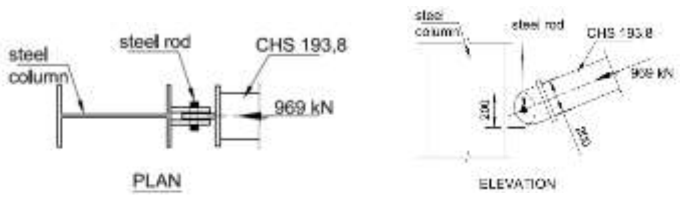
Then, weight of haunch for both sides = 2 (0.5x 2.1m x33kg/m) = 69.3kg

Total steel saving = 69.3-60.48 = 8.82 kg per frame

Percentage of saving in relation to haunch = (8.82/69.3)x100 = 12.73%

This saving is only for single portal frame and need to be multiplied by number of frame to calculate saving for total structure.

Appendix – F Specimen calculation for knee brace connection design.

reference	Calculations	output
	<u>Specimen calculation for 40m span portal frame with 6m eave height with knee brace arrangement of X=10% span and Y=3% of span</u>	
	Knee brace was analyzed with pinned connection, no moment is transferred.	
	For 40m span portal frame,	
	Knee brace size = CHS 193, 8	
	Compression force from knee brace = 969kN	
		
EN1993-1-8 Table 3.3	Consider 200mm wide, 25mm thick plate of grade S355 and 1 no. 63mm dia. steel rod is same steel grade.	
	Arrangement has fulfilled the edge distance (50mm) > 1.2d.	
	<u>Check for steel plate</u>	
	Gross cross sectional area = 200x25 = 5000 mm^2	
EN1993-1-1 Table 3.1	Yield strength = f_y = 355 N/mm^2	
	Design resistance = N_{Rd} = $5000x355x10^{-3}$ = 1775 kN	
	Net cross sectional area of plate,	
EN1993-1-1 Table 3.1	A = (200 – 1x65)x25 = 3375 mm^2	
	Nominal ultimate strength of S355 steel,	
	f_u = 510 N/mm^2	
	γ_{m2} = 1.25	
	Design ultimate resistance,	
	$N_{U,Rd} = (0.9x3375x510)10^{-3}/1.25$ = 1239 kN > 969 kN	

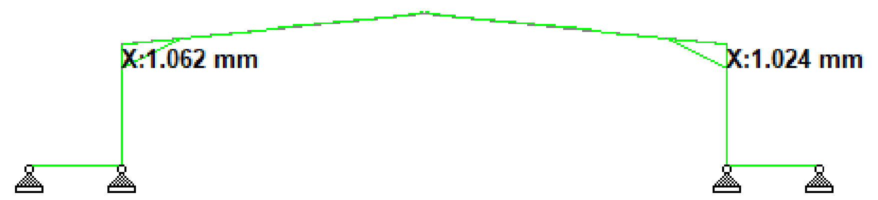
	<u>Bolt resistance</u>	
	<u>Shear resistance:</u>	
	Area of 63mm dia. Rod = 3117 mm^2	
EN1993-1-8	Nominal ultimate strength of S355 steel , f_u = 510 N/mm^2	
Table 3.1	Partial factor for resistance of bolts = γ_{m2} = 1.25	
	Design shear resistance of 63mm dia. steel rod,	
Table 3.4	$F_{v,Rd} = \alpha_v f_{ub} A_s/\gamma_{m2} = (0.6x510x3117)10^{-3}/1.25 = 763kN$	
	According to the connection, the axial force is taken by double shear,	
	Thus shear resistance = 2x763 =1526 kN > 969 kN	
	<u>Bearing resistance:</u>	
	Nominal ultimate strength of S355 steel, f_u = 510 N/mm^2	
	Partial factor for resistance of bolts = γ_{m2} = 1.25	
	Diameter of rod = 63 mm	
EN1993-1-8	Thickness of plate = 25mm	
Table 3.4	Design bearing resistance of rod, $F_{bRd} = 1.5\ f_u\ dt/\gamma_{m2}$	
Cl 3.6.1 (10)	$F_{b,Rd} = 1.5x510x65x25x10^{-3} / 1.25 = 995kN > 969\ kN$	
	Therefore, proposed pin connection of 25mm tk. Grade S355 plate with 63mm diameter rod is satisfied.	
	As 4m eave height portal frame has the compressive force of knee brace 917 kN which is very close to value corresponding to 6m eave height frame, same connection detail can be followed for 40m span with 4m eave height portal frame as well.	

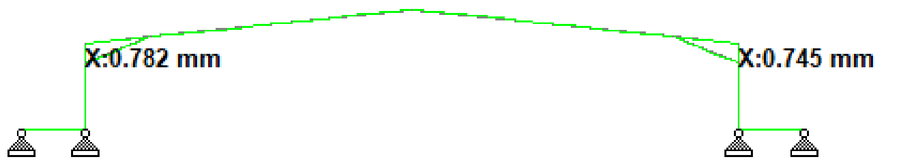
Appendix – G Specimen calculation of column resistance to transverse force from knee brace

reference	Calculations	output
	<u>Specimen calculation for 30m span portal frame with 6m eave height with knee brace arrangement of X=10% span and Y=3% of span</u> Transverse force from knee brace = 707kN Column section 533x210x92kg/m UB Section properties are as below; Overall depth h = 533 mm Flange width b = 209.3 mm Web thickness t_w = 10.2 mm Flange thickness t_f = 15.6 mm root radius r = 12.7 mm depth between fillet C_w = 476.5 mm	
EN1993-1-5 Figure 6.11	For web without longitudinal stiffness $k_F = 2 + 6\,[(s_s+c)/h_w]^2 \leq 6$ $k_F = 2 + 6\,[(50+900)/(476.5-15.6-15.6)]^2 \leq 6$ take $k_F = 6$ L_y is the smaller of ;	
Eq 6.11	$L_y = L_e + t_f \sqrt{(m_1/2) + (L_e/t_f)^2 + m_2}$ and	
Eq 6.12	$L_y = L_e + t_f \sqrt{(m_1+m_2)}$	
Cl 6.5.3	$L_e = k_F E t_w^2 / 2 f_{yw} h_w \leq s_s+c$ $= (6\text{x}210000\text{x}10.2^2)/(2\text{x}355\text{x}445.3) \leq 50+900$ $= 414.62$	
Cl 6.5 (1)	$m_1 = f_y b_f / f_{yw} t_w = (355\text{x}209.3)/(355\text{x}10.2) = 20.52$ assume $\lambda_F > 0.5$ $m_2 = 0.02((h_w/t_f)^2 = 0.02(445.3/15.6)^2 = 16.29$	

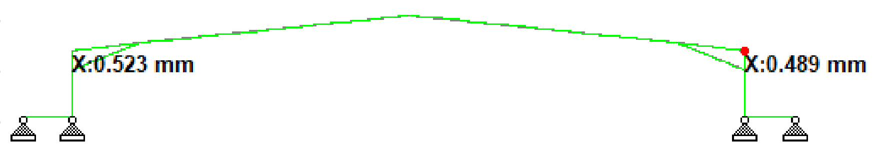
	then L_y is the smaller of; L_y = 414.62+15.6√(20.52/2) +(414.62/15.6)^2 +16.29 836.96mm And L_y = 414.62+15.6√(20.52+16.29) = 509.26 mm Hence L_y = 509.26 mm	
Eq 6.5	$F_{cr} = 0.9k_F E t_w^2/h_w$ = 0.9x6x210000x10.2^3/445.3 = 2702 kN	
Eq 6.4	$\lambda_F = \sqrt{(L_y t_w f_{yw})/F_{cr}}$ = √(509.26x10.2x355)/2702x10^3 = 0.83 > 0.5 Therefore assumption of $\lambda_F > 0.5$ is valid	
Eq 6.3	$\chi_f = 0.5/\lambda_F \leq 1$ = 0.5/0.83 = 0.602	
Eq 6.2	$L_{eff} = \chi_f L_y$ = 0.602 x 509.26 = 306.57 mm	
Eq 6.1	$F_{Rd} = f_{yw} L_{eff} t_w / \gamma_{m1}$ =355 x 306.57 x 10.2 x 10^{-3}/ 1.1 = 1009 kN Since F_{Ed} = 707 kN No bearing stiffeners are required.	Web resistance is ok

Appendix – H Calculation of buckling amplification factor, $\alpha_{cr,est}$ for portal frames with reduced column section

Referance	Calculations	Output
	<u>For 20m span with eave height of 4m</u>	
	<u>with prop (10% of span and depth 3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 99.4 kN	
	The horizontal reaction at each base ; H_{Ed} = 78 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 86 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 424 kN	
	0.09 Ncr = 38.16 kN	
EN 1993-1-1	$N_{R,Ed}$ = 86 kN > 38.16 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 99.4}{200}$ = 0.497 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\frac{1}{200} \frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[86/424]} {(1/200)/(4000/1.062)} = 12.00	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	For 30m span with eave height of 4m	
	with prop (10% of span and depth 3% of span)	
	The vertical reaction at each base ; V_{Ed} = 150 kN	
	The horizontal reaction at each base ; H_{Ed} = 168 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 179 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 492 kN	
	0.09 Ncr = 44.28 kN	
EN 1993-1-1	$N_{R,Ed}$ = 179 kN > 44.28 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 150}{200}$ = 0.75 kN	
	modeling is done with dummy member, as previous and got following results	
		
	δ_{NHF} = 0.782 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\frac{1}{200} \frac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[179/492]} {(1/200)/(4000/0.782)} = 13.02	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Referance	Calculations	Output
	<u>For 40m span with eave height of 4m</u>	
	<u>with prop (10% of span and depth 3% of span)</u>	
	The vertical reaction at each base ; V_{Ed} = 200.5 kN	
	The horizontal reaction at each base ; H_{Ed} = 287 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 303 kN	
EN 1993-1-1	Since same sections are used, N_{cr} is same as previous	
Cl.5.2.1	N_{cr} = 609 kN	
	0.09 Ncr = 54.81 kN	
EN 1993-1-1	$N_{R,Ed}$ = 303 kN > 54.81 kN	
Cl.5.2.1	Therefore, the axial compression in the rafter is significant and	
	α_{cr} from EN 1993-1-1 is not applicable.	
Appendix–B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \frac{1}{200} V_{Ed} = \frac{1 \times 200.5}{200}$ = 1.0025 kN	
	modeling is done with dummy member, as previous and got following results	

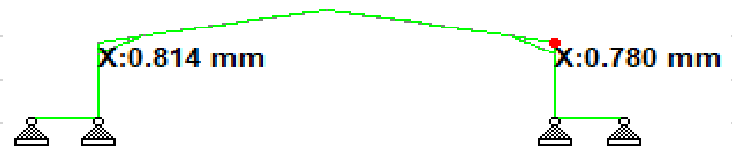


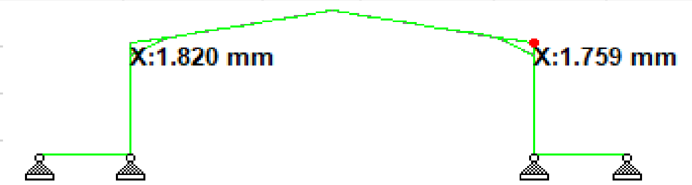
Referance	Calculations	Output
	δ_{NHF} = 0.523 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	

$$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left[\frac{N_{R,Ed}}{N_{R,cr}} \right]_{max} \right\} \left\{ \frac{1}{200} \frac{h}{\delta_{NHF}} \right\}$$

Referance	Calculations	Output
	$\alpha_{cr,s,est}$ = 0.8 {1 -[303/609]} {(1/200)/(4000/0.523)} = 15.37	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	

Appendix – I Calculation of buckling amplification factor, $\alpha_{cr,est}$ for portal frames with roof slope of 10 degree

Referance	Calculations	Output
	<u>**For 20m span with eave height of 4m - roof slope 10 degree with knee brace (X=10% of span and Y=3% of span)**</u>	
	The vertical reaction at each base ; V_{Ed} = 102 kN	
	The horizontal reaction at each base ; H_{Ed} = 73 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 87 kN	
EN 1993-1-1 Cl.5.2.1	The axial compression is significant if $N_{ed} > 0.09N_{cr}$	
	$N_{cr,R} = \dfrac{\pi^2 EI_y}{L_{cr}^2}$	
	Lcr is the developed length of the rafter pair from column to column	
EN 1993-1-1 Cl.3.2.6	Lcr = 20/cos 10 = 20.31 m	
	E = 210000 N/mm^2	
	Iy = 10200 cm^4 (Rafter = 356x127x39 kg/m UB)	
	$N_{cr} = \dfrac{\pi^2 \times 210000 \times 10200 \times 10^4 \times 10^{-3}}{(20.31 \times 10^3)^2}$ = 512 kN	
	0.09 Ncr = 46.08 kN	
	$N_{R,Ed}$ = 87 kN > 46.08 kN	
	Therefore, the axial compression in the rafter is significant and α_{cr} from EN 1993-1-1 is not applicable.	
Appendix –B	<u>Calculation of $\alpha_{cr,est}$</u>	
	The notional horizontal force is ;	
	$H_{NHF} = \dfrac{1}{200} V_{Ed} = \dfrac{1 \times 102}{200}$ = 0.51 kN	
	modeling is done with dummy member, and got following results	
		
	δ_{NHF} = 0.814 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\dfrac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\dfrac{1}{200} \dfrac{h}{\delta_{NHF}}\right\}$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[87/512]} {(1/200)/(4000/0.814)} = 16.32	> 10
EN 1993-1-1 Cl.5.2	Since α_{cr} is greater than 10, first order elastic analysis can be used and second order effects can be neglected	

Referance	Calculations	Output
	For 20m span with eave height of 6m - roof slope 10 degree with knee brace (X=10% of span and Y=3% of span)	
	The vertical reaction at each base ; V_{Ed} = 104 kN	
	The horizontal reaction at each base ; H_{Ed} = 51 kN	
	The maximum axial force in the rafter ; $N_{R,Ed}$ = 64 kN	
EN 1993-1-1	The axial compression is significant if $N_{ed} > 0.09 N_{cr}$	
Cl.5.2.1	$$N_{cr,R} = \frac{\pi^2 EI_y}{L_{cr}^2}$$	
	Lcr is the developed length of the rafter pair from column to column	
EN 1993-1-1	Lcr = 20/cos 10 = 20.31 m	
Cl.3.2.6	E = 210000 N/mm^2	
	Iy = 10200 cm^4 (Rafter = 356x127x39 kg/m UB)	
	$N_{cr} = \dfrac{\pi^2 \times 210000 \times 10200 \times 10^4 \times 10^{-3}}{(20.31 \times 10^3)^2}$ = 512 kN	
	0.09 Ncr = 46.08 kN	
	$N_{R,Ed}$ = 64 kN > 46.08 kN	
	Therefore, the axial compression in the rafter is significant and α_{cr} from EN 1993-1-1 is not applicable.	
Appendix – B	Calculation of $\alpha_{cr,est}$	
	The notional horizontal force is ;	
	$H_{NHF} = \dfrac{1}{200} V_{Ed} = \dfrac{1 \times 104}{200}$ = 0.52 kN	
	modeling is done with dummy member, and got following results	
		
	δ_{NHF} = 1.82 mm	
	Then, $\alpha_{cr,s,est}$ calculated as follows;	
	$$\alpha_{cr,s,est} = 0.8 \left\{1 - \left[\frac{N_{R,Ed}}{N_{R,cr}}\right]_{max}\right\} \left\{\frac{1}{200} \frac{h}{\delta_{NHF}}\right\}$$	
	$\alpha_{cr,s,est}$ = 0.8 {1 -[64/512]} {(1/200)/(6000/1.82)} = 11.54	> 10
EN 1993-1-1	Since α_{cr} is greater than 10, first order elastic analysis can be used and	
Cl.5.2	second order effects can be neglected	