Portal Frame Design Tips Seminar Proceedings

Author: *Clark Hyland*
Affiliation: *Steel Construction New Zealand Inc.*

Editor: *Kevin Cowie*
Affiliation: *Steel Construction New Zealand Inc.*
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Introduction

In October, 2009, Steel Construction New Zealand Inc., (SCNZ) ran technical seminars throughout New Zealand. One of the topics covered was 'Portal Frame Design Tips', presented by the Manager of SCNZ, Clark Hyland. These proceedings outline the main messages delivered on this topic at the seminar series and were edited by Kevin Cowie.

This paper summarises material predominantly from two Australian Steel Institute (Woolcock et al, 1999; Hogan et al, 1997), and one Steel Construction Institute (Salter, 2004) publications, contextualised for New Zealand practice in accordance with the New Zealand Steel Structures Standard NZS 3404 (SNZ, 2007). The use of these referenced documents in particular are gratefully acknowledged.

Portal Frame Types

Pitched Roof Portal (Fabricated from UBs)

A single-span symmetrical pitched roof portal frame (Figure 1) will typically have:

- A span between 15 m and 50 m
- An eaves height between 5 and 10 m
- A roof pitch between 3° and 5° is commonly adopted
- A frame spacing between 8 m and 12 m (the greater spacings being associated with the longer span portal frames)
- Haunches in the rafters at the eaves and apex.

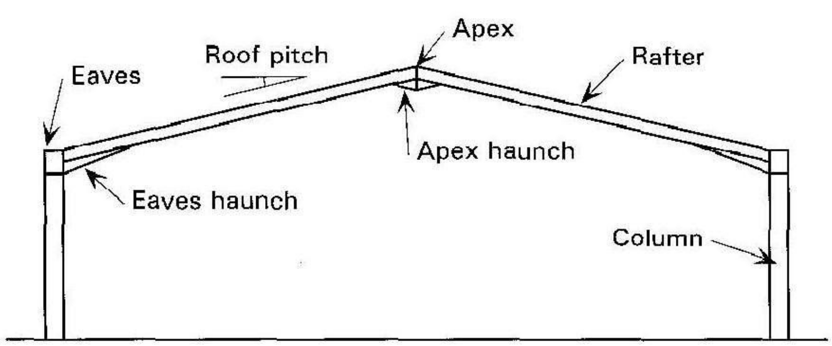


Figure 1: Single-span Symmetrical Portal Frame (Salter, 2004)

Propped Portal Frame

Where the span of a portal frame is large (greater than say 30 m), and there is no need to provide a clear span, a propped portal frame (Figure 2) can reduce the rafter size and also the horizontal thrust at the base, giving economies in both steelwork and foundation costs.

This type of frame is sometimes referred to as a "single span propped portal", but it is a two-span portal frame in terms of structural behaviour.

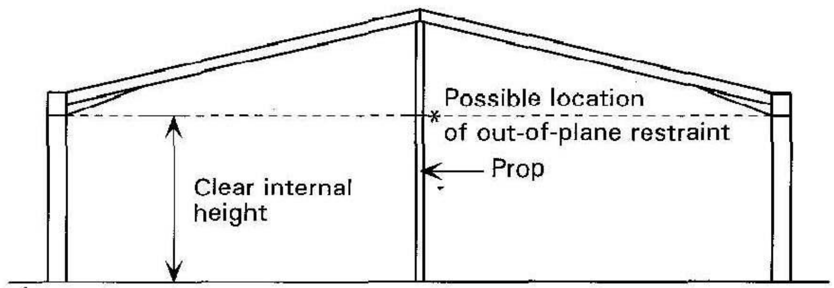


Figure 2: Propped Portal Frame (Salter, 2004)

Tapered Section or Cellular Beam Portal Frame

In recent years portal frames have been constructed using tapered welded sections and cellular beams. Cellular beam frames commonly have curved rafters (Figure 3), which are easily achieved using cellular beams or welded sections. Where splices are required in the rafter (for transport), they should be carefully detailed, to preserve the architectural features for this form of construction.

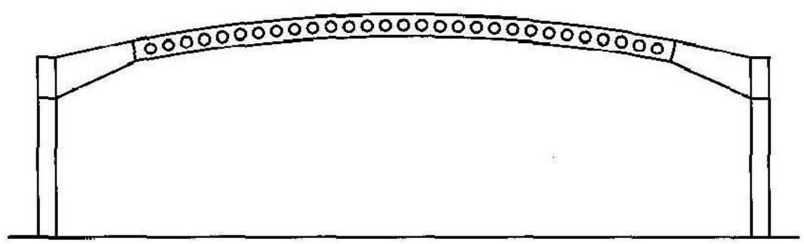


Figure 3: Tapered Section or Cellular Beam Portal Frame (Salter, 2004)

Many cellular beam portal frames in the span range of 40 m to 55 m have been constructed in the United Kingdom; greater spans are possible. Elastic design is used because the sections used cannot develop plastic hinges at a cross-section, which is an essential criterion for elastic design with moment redistribution.

Purlins

Purlin Deflections

The following deflection limits are recommended for purlins and girts.

- Under dead load alone: *Span/360*
- Under live load alone: *Span/180*
- Under serviceability wind load alone: *Span/150*

Purlin Bolts

The standard bolt is an M12-4.6/S which comes with loose washers. It should be remembered that washers under both the head and nut are essential. This is because the standard punched holes in purlins are too big for M12 bolt heads and nuts, even though the height of the hole through lapped purlins is less because of the lapping.

Frame Analysis

General

NZS 3404 permits a number of types of structural analysis, consisting of first and second order elastic analysis.

First order elastic analysis assumes the frame remains elastic and that its deflections are so small that secondary effects resulting from the deflections (second order effects) are negligible. First order analysis is generally carried out using plane frame analysis computer programs. Despite the basic assumption of first order analysis, second order effects are not negligible. Second order effects are essentially P-Δ effects which arise from the sway Δ of the frame, or P-δ effects which arise from the deflections δ of individual members from the straight lines joining the members' ends. NZS3404 requires that the bending moments calculated by first order analysis be modified for second order effects using moment amplification factors. First order elastic analysis of portal frames in accordance with NZS 3404 utilises a simple procedure that does not account for P-δ and P-Δ effects.

The use of moment amplification factors can be avoided by using second order elastic analysis. Second-order elastic analysis essentially involves a number of iterations of first order elastic analysis with the deflected shape of the previous iteration being used for the second and subsequent iterations until convergence is obtained. Second order elastic analysis programs are now widely available, and as the moments obtained do not require amplification and are generally less conservative than amplified first order elastic moments, second order elastic moments is recommended ahead of first order amplified elastic analysis.

It should be noted that second order analysis should only be performed for load combinations and not for individual load cases. Second order elastic analysis is performed on load combinations and *not* on individual load cases, since the second order analyses using the individual load cases cannot be superimposed. Therefore, it is necessary to have two separate sets of output for second order elastic analysis: the first for load cases and load case deflections (as obtained by first order elastic analysis) and the second for member forces and reactions for load combinations (as obtained by second order elastic analysis).

Elastic Analysis

Although the use of elastic analysis with moment redistribution of portal frames at the ultimate limit state is well established in New Zealand, it is not widely used internationally. Furthermore, there are situations where elastic analysis is more appropriate e.g. where:

- Tapered or cellular members are used.
- Instability of the frame is a controlling factor.
- Deflections are critical to the design of the structure

Elastic Analysis with Moment Redistribution

Plastic hinges may form in the members within the structure as their plastic moment capacity is reached as the structure redistributes moments. It assumes that the members behave elastically up to the full value of the plastic moment capacity, then plastically (without strain hardening) to allow redistribution of moments around the frame. Members required to redistribute ultimate limit state moments are required to have sufficient flexural torsional restraint to ensure development of the plastic section capacity of the section.

This method has several advantages including that it optimises the use of a single hot rolled section in a frame leading to fabrication simplification.

Frame Design with Haunches

For preliminary computer analysis, selection of the rafter and column sizes is from experience or by guesswork. The computer model should have at least two nodes near each knee joint to allow for modelling of the rafter haunches in the final design phase. Nodes at the mid-height of each column and at quarter points of the rafter can give useful bending moment diagrams in some cases, although this is generally unnecessary when using modern computer packages.

Haunches don't need to be included in the initial computer run as they do not have much effect on the frame bending moments. However, significant reductions in deflection can be achieved later in the analysis.

Once the first computer analysis is run, the limit state bending moments in the column and in the rafters should be checked against the section capacities to check the assumed sizes.

For preliminary design, reducing the column bending moment to the underside of the haunch or reducing the section capacity to allow for coincident axial forces can be disregarded. The calculated moment at the knee should be checked against the column section capacity $\phi M_{sx.}$ Implicit in this check is that sufficient fly braces can be provided to ensure that the full section capacity is achieved.

The calculated bending moments in the rafter should be similarly checked against the capacity $\phi M_{sx.}$ except in the vicinity of the knee joints where haunches will probably be provided to cater for the peak rafter moments in these areas. Some small margin in flexural capacity should be retained in order to cater for axial forces. The member sizes assumed should then be adjusted accordingly and the frame analysis re-run.

Haunch Properties

Once the member sizes have been established with more confidence, it is appropriate to model the haunches. The standard AISC haunch (AISC, 1985) is formed from the same section as the rafter. It is common to model the haunch with two or three uniform segments of equal length although reference (Hogan et al, 1997) indicates that there is no benefit in using more than two segments.

The depth of the haunch is calculated at the mid-point of each segment and the section properties can he calculated accordingly. Some frame analysis programmes can calculate haunch properties automatically. Alternatively, the properties of standard UB's which are contained in standard software libraries can be used to model the haunch segments approximately.

A comprehensive AISC publication (Hogan et al, 1997) in 1997 investigated the design of tapered portal frame haunches fabricated from universal section members. The publication deals with detailing the cost of fabrication, the calculation of elastic and plastic section properties, computer modelling (including the effect of varying the number of segments), and section and member design to AS 4100, which is the source document for much of NZS 3404. It also reviews the testing of haunches in other literature.

Modelling Base Fixity

General

Column bases are usually considered as being nominally pinned at the ultimate limit state. This simplifies the design.

However a degree of base stiffness may be considered. Stiffness at the base can reduce the deflections and increase the stability of the frame considerably. However, foundations that are designed to resist moments are considerably larger than those designed for axial load and shear forces only and consequently, are much more costly (Salter et al, 2004).

Rigid Base

Where a column is rigidly connected to a suitable foundation, the following may be assumed:

- In elastic global analysis, the stiffness of the base should be taken as equal to the stiffness of the column for all ULS calculations. However, in determining deflections under serviceability loads, the base may be treated as rigid.
- In elastic analysis with moment redistribution, the assumed base stiffness should be consistent with the assumed base moment capacity, but should not exceed the stiffness of the column.

Nominally Pinned Base

Where a column is nominally pin-connected to a foundation that is designed assuming that the base moment is zero, the base should be assumed to be pinned when using elastic global analysis to calculate the other moments and forces in the frame under ULS loading.

The stiffness of the base may be assumed to be equal to the following proportion of the column stiffness:

- 10% when checking frame stability or determining in-plane effective lengths
- 20% when calculating deflections under serviceability loads

Nominally Semi-rigid Base

A nominal base stiffness of up to 20% of the stiffness of the column may be assumed in elastic global analysis, provided that the foundation is designed for the moments and forces obtained from this analysis.

Base Spring Stiffness Modelling

In practice, allowance for base fixity is usually by the use of spring stiffness or dummy members at the column base.

Ultimate Limit State

At the ultimate limit state:

- A nominally rigid base can be modelled with a spring stiffness equal to $4EI_{column}/L_{column}$
- A nominally pinned base can be modelled with a spring stiffness equal to $0.4EI_{column}/L_{column}$ for frame stability checks.

Serviceability Limit State:

- A nominally rigid base can be modelled with full fixity.
- A nominally pinned base can be modelled with a spring stiffness equal to $0.8EI_{column}/L_{column}$.

Rafters

Nominal Bending Capacity M_{bx} in Rafters

Simplified Procedure

NZS 3404 uses a semi-empirical equation to relate the nominal bending capacity M_{bx} to the elastic buckling moment M_o and the section strength M_{sx}, which for Universal and Welded Beams and Columns can be taken as $Z_{ex}f_y$. This philosophy uses a set of semi-empirical equations to relate the member strength to the plastic moment and the elastic flexural torsional buckling moment.

Equation 5.6.1.1(1) of NZS3404 expresses the nominal member bending capacity M_{bx} as

$$M_{bx} = \alpha_m \alpha_s M_{sx} \leq M_{sx}$$

where α_m is a moment modification factor to account for the non-uniform distribution of major axis bending moment, and α_s is a slenderness reduction factor which depends on M_{sx} and the elastic buckling moment of a simply supported beam under uniform moment M_o. The standard gives comprehensive values of α_m which would be met in practice. The conservative option of taking α_m equal to unity is also permitted.

For category 2 and 3 members in seismic resisting frames, $\alpha_m \alpha_s \geq 1.0$. For category 1 members $\alpha_m \alpha_s \geq \phi_{oms}$, reflecting the need to maintain stability under over-strength actions.

The slenderness reduction factor is expressed in Clause 5.6.1.1(3) of the standard as

$$\alpha_s = 0.6\left\{\sqrt{\left[\left(\frac{M_{sx}}{M_{oa}}\right)^2 + 3\right]} - \left(\frac{M_{sx}}{M_{oa}}\right)\right\}$$

Where M_{oa} may be taken as either (i) M_o which is the elastic buckling moment for a beam with a uniform bending distribution and with ends fully restrained against lateral translation and twist rotation but unrestrained against minor axis rotation; or (ii) a value determined from an accurate elastic buckling analysis.

The elastic buckling moment M_o may be determined from the accurate expression given in equation 5.6.1.1(4) as

$$M_o = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{L_e^{\,2}}\right)\left[GJ + \left(\frac{\pi^2 EI_w}{L_e^{\,2}}\right)\right]\right\}}$$

Where L_e is the effective length, and EI_y, GJ and EI_w are the flexural bending rigidity, the torsional rigidity and the warping rigidity respectively. Values of the section properties I_y, J and I_w are given in the ASI Design Capacity Tables for Structural Steel (AISC, 1997). The use of Equation 5.6.1.1(4) requires the effective length L_e, and determination of this is made using clause 5.6.3.

Alternative Procedure

Clause 5.6.4 of NZS3404 allows the designer to use the results of an elastic buckling analysis, although in most cases this is not practical for design offices and is really a research tool. If an elastic buckling analysis is to be used, then the elastic buckling moment M_{ob}, which allows for the moment gradient, restraint conditions and height of loading, is determined either from a computer program or from solutions given in the literature (Bradford, 1988).

Having obtained M_{ob}, the value of M_{oa} to be used in clause 5.6.1 or 5.6.2 is calculated from

$$M_{oa} = \frac{M_{ob}}{\alpha_m}$$

Where values of α_m are obtained either from the standard or from an elastic buckling analysis such that

$$\alpha_m = \frac{M_{os}}{M_{oo}}$$

The moment M_{os} is the elastic buckling moment corresponding to M_{ob} for the same beam segment with the same bending moment distribution, but with

- shear centre loading,
- ends fully restrained against lateral translation and twist rotation, and
- ends unrestrained against minor axis rotation.

The moment M_{oo} is the reference elastic buckling uniform bending moment M_o given by Equation 5.6.1.1(4) with L_e taken as the laterally unsupported length L.

In the event that the whole rafter is designed as a tapered member, an accurate elastic buckling analysis must be used. This also applies to the haunched segment of a conventional rafter. The values of M_{ob} and M_{os} for tapered rafters may be found in Reference (Bradford, 1988).

Effective Length and Moment Modification Factors for Bending Capacity

General

If the simplified design procedure in Clause 5.6.1 of NZS 3404 is used, then the effective length L_e of the rafter must be determined in accordance with Clause 5.6.3. The effective length depends on the spacing and stiffness of the purlins and fly braces, and the degree of twist and lateral rotational restraint for a chosen segment as follows:

- Whether the connection between the purlins and rafter is rigid, semi-rigid or pinned.
- The flexural rigidity of the purlins, in that regard NZS 3404 classifies purlins qualitatively as flexible or stiff. No numerical yardstick is given.
- The load height in that regard NZS 3404 allows, for example, for the destabilising effect of loads applied at or above the shear centre in a beam subjected to downward loads.
- Whether the top or bottom flange is the critical flange. For a portal frame, the compression flange is the critical flange.
- The degree of lateral rotational restraint provided at the ends of a segment by adjoining segments.

Top Flange in Compression

Under gravity loads, the top flange is mostly in compression, except near the knees. Purlins provide lateral restraint to the top flange, but full twist restraint to the rafter from the purlins cannot always be relied upon because standard oversized slotted holes are often used in purlins. However even where the holes in the purlins are slotted, the bolts are tightened and so the purlin to rafter connection using a standard purlin cleat and two bolts can be regarded as a partial twist restraint connection in terms of Figure 5.4.2.2 in NZS 3404. Fortunately, the standard permits partial twist restraint at the critical flange (in association with lateral restraint) to be classified as full restraint of the cross-section as shown in Figure 5.4.2.1(b). Therefore, for each segment between purlins when the top flange is in compression, both ends are fully restrained (FF) and the *twist restraint factor* k_t is 1.0.

Although gravity loads are applied through the purlins at the top flange, the *load height factor* k_t of 1.4 in Table 5.6.3(2) in NZS 3404 does not apply because the load is not free to move sideways as the member buckles. In other words, the load is applied at a point of lateral restraint and k_t *should be taken as 1.0.*

The degree of lateral rotational restraint provided at each end of the segment by adjoining segments depends on whether the adjoining segments are fully restrained laterally or not, as described in Clause 5.4.3.4 of NZS 3404. (A fully restrained segment in accordance with Clause 5.3.2 is essentially one with M_b not less than M_s which means its $\alpha_m\alpha_s$ value is greater than unity.) The standard permits full lateral rotational end restraint or none. No intermediate option is provided. While segments between purlins under downward loading are short and are likely to be fully restrained laterally, full restraint in accordance with Clause 5.3.2 cannot be guaranteed. It follows that lateral rotational restraint should strictly speaking be disregarded. There is, however, a high degree of lateral rotational restraining which would allow k_r to be taken safely as 0.85.

In summary, the effective length L_e is given by $k_t k_l k_r L$ as $L_e = 1.0 \times 1.0 \times 0.85 S_p = 0.85 S_p$

The spacing between purlins is short in comparison with the length of the rafter (Figure 4), so the moment modification factor α_m should usually be taken as 1.0.

Bottom Flange in Compression

With Fly Bracing Under Uplift

Under uplift, most of the bottom flange of a portal frame rafter is in compression. In such cases, the rafter is attached to the purlins at the tension flange level, and the compression flange of the rafter is unrestrained. In order to achieve increased member capacity, it is customary to restrain the bottom flange of the rafter laterally by providing fly bracing using small angle section members joining the bottom flange to the purlins.

With the bottom flange in compression, NZS 3404 classifies a fly brace restraint as a full or partial cross-sectional restraint depending on whether the purlins are flexible or stiff. No numerical criterion is given for assessing the flexibility or stiffness of purlins. Therefore if partial cross-sectional restraint is assumed conservatively at each end of the segment (PP), the twist restraint factor k_t will be greater than 1.0 in accordance with Table 5.6.3(1) of NZS 3404. However, unless fly braces are closely spaced or the rafter has an unusually high flange to web thickness ratio, k_t will normally be close to 1.0. Considering that the partial restraint assumption is probably conservative, a k_t value of 1.0 is recommended for simplicity.

It may appear that there should be a useful reduction in effective length because the wind loads act at the more favourable tension flange level. However, the benefit of this is not significant as most of the bending moment within a segment is due to end moments, and the segment should not be likened to a simply supported beam under uniformly distributed load applied at the tension flange level. Moreover, the reduction in effective lengths of a simply supported beam under such loads is limited in some cases and NZS 3404 offers no concession for bottom flange loading. For this reason, k_l should be taken as 1.0.

For a segment between fly braces and with the bottom flange in compression, the lateral rotational restraint provided at the ends of the segment by adjoining segments should strictly speaking be disregarded because it is unlikely that the adjoining segments are fully restrained laterally in accordance with Clause 5.4.3.4 of NZS 3404. There is, however, a degree of lateral rotational restraint which would allow k_r to be taken as 0.85.

In summary, the effective length L_e for segments between fly braces for uplift conditions is given by $k_t k_e k_r L$ as $L_e = 1.0 \times 1.0 \times 0.85 S_f = 0.85 S_f$.

The moment modification factor α_m for segments between fly braces will usually be greater than 1.0. For segments which have a reversal of moment, part of the segment will have its compression flange restrained by purlins but this benefit should be ignored.

Without Fly Bracing under Uplift

Although some fly bracing is recommended, it is interesting to consider the rafter behaviour under uplift where there is no fly bracing at all. In this case, the full portal span should be taken as the effective length, and α_m should be based on the bending moment distribution across the rafter span. Even though the validity of this approach for a kinked member is doubtful, the large effective length should equate to such a low capacity that some fly bracing will be necessary.

Designers often feel that the lateral restraint offered by purlins to the tension flange under uplift conditions should also increase the lateral buckling capacity. However, theoretical and experimental studies (Dux et al, 1986; Wong-Chung, 1987) of the bracing of beams have confirmed that translational restraint alone acting at the level of the tension flange, such as that provided by purlins, is virtually ineffective. These studies show that if the lateral restraint is combined with some twist restraint, the buckling capacity is increased. It is possible to design the purlin-rafter connection for some rotational capacity by providing two or four friction bolts to the cleat, or by using wider cleat plates with more bolts. There may be architectural advantages in avoiding fly bracing, such as when a ceiling is required above the bottom flange level.

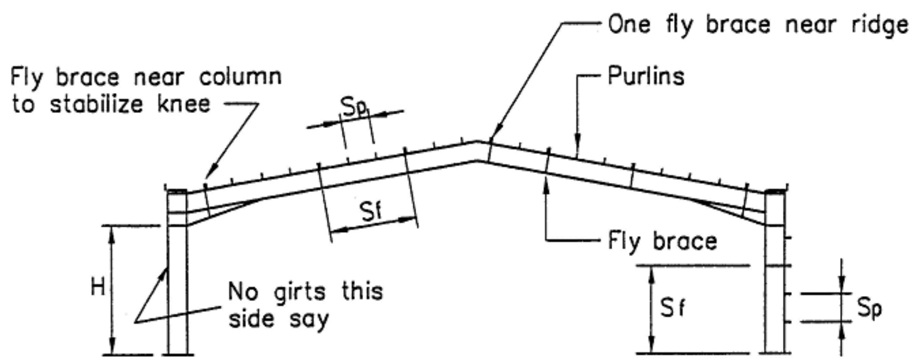
Investigations have been carried out (Wong-Chung, 1987) into the effectiveness of standard purlin connections in providing rotational restraint to the rafters. The results revealed in part that the requirement for rotational stiffness is a function of the initial geometric imperfections in the rafter. That is, for very crooked rafters, greater stiffness in the brace is required. The theoretical and experimental studies have so far indicated that ordinary or standard purlin connections are effective to some degree, provided that the bolts are properly tightened.

With Fly Bracing under Downward Load

The effect of the bottom flange near the columns being in compression due to gravity loads or other loading should be considered even though most of the bottom flange of the rafter is in tension. A fly brace is recommended near each knee and near the ridge to restrain the inside corners of the frame at kinks. A stiffener between column flanges as indicated in Figure 4 effectively extends the bottom flange of the haunch to the outside column flange which is restrained by girts. This effectively provides some restraint to the inside of the knee. However, a fly brace near the knee is still recommended. With fly braces at least at the knees and the ridge, the effective length will be 0.85 times the spacing between fly braces.

An alternative approach is to consider the rafter segment between the column and point of contraflexure if accurately known, or nearest purlin beyond the inflection point. The inflection point is considered to be unrestrained in determining the effective length. This approach is described in an example by Clifton, Goodfellow and Carson (1989)

The value of the moment modification factor α_m for the segment should be determined using one of the three methods in NZS 3404, but using a specifically calculated α_m in Clause 5.6.1.I(a)(iii) is likely to be most appropriate if there is no intermediate fly brace between the knee and ridge. It is recommended that any haunch should be ignored in determining the design bending capacity ϕM_{bx} of the segment, but the applied bending moments should be reduced by factoring the moment at any haunch section by the ratio of the elastic section modulus of the unhaunched section to the corresponding elastic modulus of the haunched section. Alternatively if each end of the haunch happens to be fly braced as in the design example, the haunch may be treated as a tapered segment in accordance with clause 5.6.1.1.1 of NZS 3404.



LOCATION	EFFECTIVE LENGTH
Outside flange in compression	0.85 Sp
Inside flange in compression	0.85 Sf
Column without girts or fly bracing	0.85 H

Figure 4: Effective Length Factors for Bending in Rafters and Columns (Woolcock et al, 1999)

Major Axis Compression Capacity N_c

In NZS3404, the nominal member capacity N_c for buckling in plane about the major axis is required in the combined actions rules for determining the in-plane member capacity in Clause 8.4.2.2. It is obtained from Clause 6.3.3 as

$N_c = \alpha_c k_f A_n f_y$

where A_n is the net rafter cross-sectional area, which is generally the gross area for portal frame members (see Clause 6.2.2 of NZS3404). The member slenderness reduction factor α_c is given in tabular form in the standard for values of the modified slenderness ratio $\lambda_{nx} = (L_e / r_x)\sqrt{k_f}\sqrt{f_y/250}$ where L_e is the effective length equal to $k_e L$ based on the actual rafter length L from the centre of the column to the apex.

Two effective lengths need to be used under Clause 8.4.2.2 of NZS 3404. For combined actions, the effective length factor k_e should be taken as 1.0. The rafter also needs to be checked under axial load alone using effective lengths determined from the frame elastic buckling load factor. λ_c . This factor can be obtained either by using the approximate method in clause 4.7.2.1 or 4.7.2.2 of NZS 3404, or by using commercially available computer packages. The check under axial load alone is unlikely to be critical for portal frames without cranes because they are principally flexural frames with low axial loads in all members.

The form factor k_f which accounts for local plate buckling are given in the steel producers' section handbooks.

Minor Axis Compression Capacity N_{cy}
The nominal member capacity N_{cy} for buckling about the y axis is required in the combined action rules of NZS 3404 for determining the out-of-plane capacity in Clause 8.4.4.1. It is obtained by taking the effective length L_e as the distance between purlins, since the purlins are restrained longitudinally by roof sheeting acting as a rigid diaphragm spanning between the roof bracing nodes. The theoretical effective length of an axially loaded member (rafter or column) with discrete lateral but not twist-rotational restraints attached to one of the flanges may be greater than the distance between the restraints. Unfortunately, there is no simple method of determining the effective length of such a member. In the case of a rafter restrained by purlins, some degree of twist-rotational restraint would also exist. The combined full lateral and partial twist-rotational restraint provided by the purlins to the outside flange should be effective in enforcing the rafter to buckle in flexure between the purlins. The capacity N_{cy} is obtained using the minor axis modified slenderness ratio in clause 6.3.3 of NZS 3404.

$$\lambda_{ny} = (L_e/r_y)\sqrt{k_f}\sqrt{f_y/250}$$

Combined Actions for Rafters
The effect of axial tensile or compressive forces in rafters combined with bending should be included in the design as described in clause 8.4.4.1 and 8.4.4.2 of NZS 3404.

Flexural Torsional Buckling Restraints

Fly Braces
As discussed previously, fly braces are diagonal members bracing the bottom flange of rafters back to purlins, or the inside flange of columns back to girts to stabilise the inside flange when in compression. Fly braces can take many forms, with the most common being a single angle each side of the bottom flange, as shown in Figure 5.

The design bracing force is determined from Clause 5.4.3 of NZS 3404, which gives criteria for the strength of braces to prevent lateral displacement of the braced compression flange. For each intermediate brace, the design force is 2.5% of the maximum compression force in the braced flange of the segments on each side of the brace. In this case, a segment is the length of the member between fly braces. Sharing between multiple intermediate braces is not permitted but each bracing force is related to the local maximum flange compression force rather than to the maximum flange compression force in the whole rafter or column. It should be noted that NZS 3404 permits restraints to be grouped when they are more closely spaced than is required for full lateral support, the actual arrangement of restraints being equivalent to a set of restraints which will ensure full lateral support.

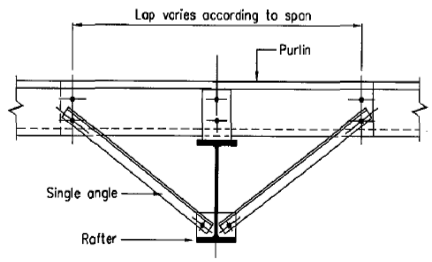


Figure 5: Double Fly-brace (Woolcock et al, 1999)

Under these conditions, the capacity of single bolted fly brace angles will be close to their concentric capacity based on minor axis (y-y) buckling. For this case, even the smallest angle, a 25x25x3, has the capacity in compression to sustain the force calculated. However, it is not really practical to use a bolt smaller than an M12, and a 25x25 angle is too small for an M12 bolt whose washer diameter is 24 mm. The smallest angle which can accommodate an M12 bolt is a 40x40x3 angle. It seems unnecessary to use fly braces on both sides of the rafter when a small angle on one side is quite adequate.

In some cases, there may be practical or aesthetic objections to fly braces because of the presence of a ceiling above the bottom flange of the rafter. This could occur in a supermarket for example. In this case, a wider purlin cleat and four high strength bolts, and a web stiffener on one or both sides to prevent cross-sectional distortion, as shown in Figure 6 could be used to brace the bottom flange. The bolt shear forces in the friction type joint can be calculated for the combined case of purlin uplift and moment due to the lateral bracing force at the bottom flange level. The disadvantage of this approach lies in the non-standard purlin cleats and non-standard holing of purlins.

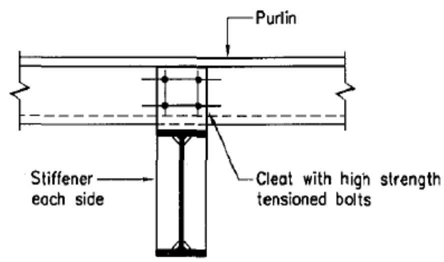


Figure 6: Alternative Torsional Restraint (Woolcock et al, 1999)

Purlins as Braces
Where the top flange is in compression, it was assumed previously in the rafter design section that the purlins provided adequate restraint to the top flange. NZS 3404 permits restraints to be grouped when they are more closely spaced than is required for full lateral support, the actual arrangement of restraints being equivalent to a set of restraints which will ensure full lateral support.

In summary, where the top flange is in compression, it is recommended that the restraint spacing necessary to provide the required member capacity be determined. If the required restraint spacing is much greater than the purlin spacing, then some of the purlins can be ignored as restraints, and two or three purlins near the notional brace point could be considered as sharing the required bracing force at that point.

Central Props

General
In large span industrial buildings, a central prop is often used to reduce the rafter span and to limit rafter and external column sizes. An efficient central prop is a square hollow section (SHS) as central props are typically long and can buckle about both axes. Other sections such as UB's, UC's, WB's or WC's can also be used effectively, particularly if the lateral stiffness requirements of the portal frame are a problem. The columns can be detailed with flexible or rigid connections to the rafter. In both cases, there is a need to determine the effective lengths both in-plane and out-of-place in order to calculate the compression capacity under axial load alone. In the case of a rigid top connection, there will be in-plane bending moments generated in the column, and these moments will need to be amplified if a first order elastic analysis has been carried out. If a flexible

connection between the column and rafter is detailed, it would be prudent to check the central column for both pinned and rigid top connections as there will be some in-plane moments generated with most practical flexible connections.

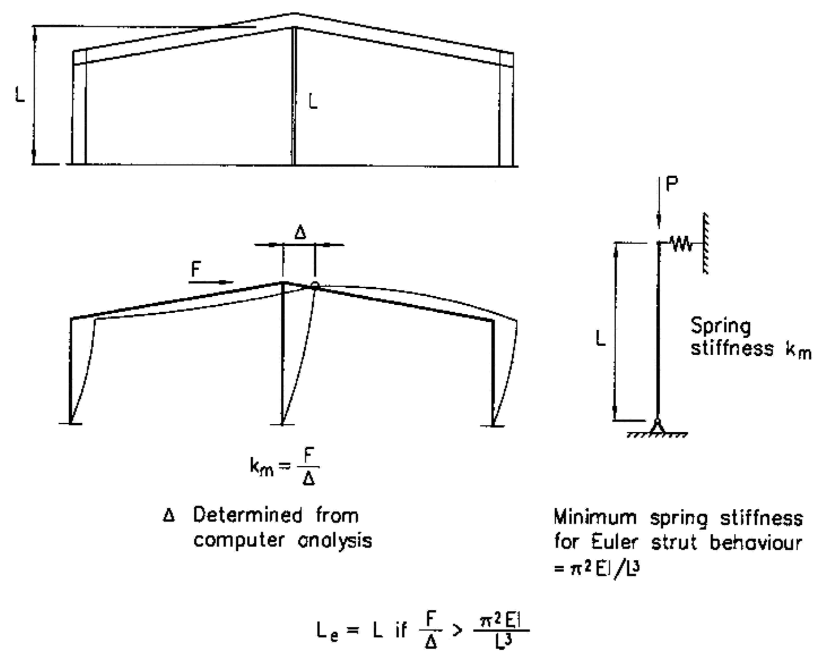


Figure 7: Effective Length of Central Prop (Woolcock et al, 1999)

There can be some uncertainty about how to calculate the effective length for determining the nominal capacity N_{cx} in the plane of the portal frame (see Figure 7). The uncertainty arises partly because the top of the rafter is attached to the apex of a portal frame which can sway sideways. This is dealt with in the following sections.

Effective Lengths of Props for Axial Compression

Top Connection Pinned

If the top of the central column is connected to the portal frame by a flexible connection such as a cleat perpendicular to the plane of the frame, it would be reasonable to regard this connection as pinned. In this case, the central column does not interact in flexure with the frame, but the frame must have a certain minimum stiffness to effectively brace the top of the columns as shown in Figure 7. For a pinned base column the minimum spring stiffness to ensure that its effective length L_e is equal to and not greater than the length L of the column is $\pi^2 EI_c / L^3$.

In practical frames, the side-sway stiffness of the rigid frame with its relatively stiff side columns and rafter is usually quite sufficient to brace the top of a slender central column. Designers can readily determine the sideways stiffness by analysing a special load case with a single horizontal load at the apex of the frame.

Top Connection Rigid

If the top connection is rigid, then there should logically be some reduction in effective length of the central column. However, in accordance with NZS 3404, it is not possible to determine directly the effective length of *individual* members in non-rectangular frames. The standard in Clause 4.9 requires a rational buckling analysis of the whole frame to determine the frame elastic bulking load factor λ_c. The only practical way of determining λ_c is by means of a frame analysis. These programs also convert the λ_c value for each load combination into effective lengths for each member by use of Equation 4.5.

Combined Actions with First Order Elastic Analysis

If the top connection is rigid, the frame elastic buckling load factor λ_c for each load combination is used in Clause 4.4.3.3.2(b) to determine the amplification factor δ_c which is applied to any bending moments from a first order elastic analysis. The capacity of the central column is then checked under Clause 8.4.2.2 of NZS 3404 using an effective length factor k_e of 1.0 for combined actions, and also an effective length factor calculated from λ_c for axial load alone.

If the top and bottom connections are assumed to be pinned, there will be no moments from the frame analysis but a nominal eccentricity in each direction is recommended. The effective length factor k_e will then be 1.0 for both combined actions and for axial load alone if the minimum spring stiffness in Section 4.6.2.1 is provided.

Combined Actions with Second Order Elastic Analysis
Ironically, if a designer has access to programs such to determine λ_c for amplifying first order moments, then it is likely that the designer also has access to the second order elastic analysis option of these programs. In this case, a designer would ideally use the second order elastic analysis as this obviates the need to amplify the moments. The capacity of the central column is then checked as described in the previous section.

Frame Deflections

General

Portal frames are generally designed on the basis of strength first, and are checked for the serviceability (deflection) limit state according to some arbitrary criteria. Deflection limits can govern the design of portal frames, and it is therefore important that any deflection limits be realistic.

The selection of deflection criteria for industrial steel frames is a subjective matter. In general, standards are not prepared to give specific recommendations, probably because deflection limits have not been adequately researched. The Australian steel code AS4100 states that the responsibility for selecting deflection limits rests with the designer, but still gives some recommendations. For a metal clad building without gantry cranes and without internal partitions against external walls, the standard suggests a limit on the horizontal deflection of the eave as column height/150 under serviceability wind loads. This limit reduces to column height/240 when the building has masonry walls. The limits suggested in Appendix B of AS 4100 are based on the work in (Woolcock et al, 1986).

Problems of Excessive Deflection

The potential problems of excessive deflections in industrial buildings include:

- Damage to cladding and fixings thereby affecting the hold down capacity of fixings and water tightness.
- Ponding of water on low pitched roofs and possible leakage because of ponding or insufficient pitch.
- Visually objectionable sag in rafters or suspended ceilings whose ceiling hangers are difficult to adjust for sag, e.g. heavy acoustic ceilings.
- Visually objectionable sag in the ridgeline because of the deflection of the apexes of internal rafters relative to the end wall apexes. The end wall rafters do not sag because they are supported by end wall columns.

The results of the survey were reported in (Woolcock et al, 1986). It is interesting to note that in many answers, there was no clear consensus of opinion among engineers. What is regarded as acceptable to one engineer is not necessarily acceptable to another. The results of the survey were rationalised, and deflection limits were proposed. These are summarised in Figure 8, Figure 9 and Figure 10. It is emphasised that these limits should be used for guidance rather than as mandatory limits. Further research is required to establish deflection limits with more confidence.

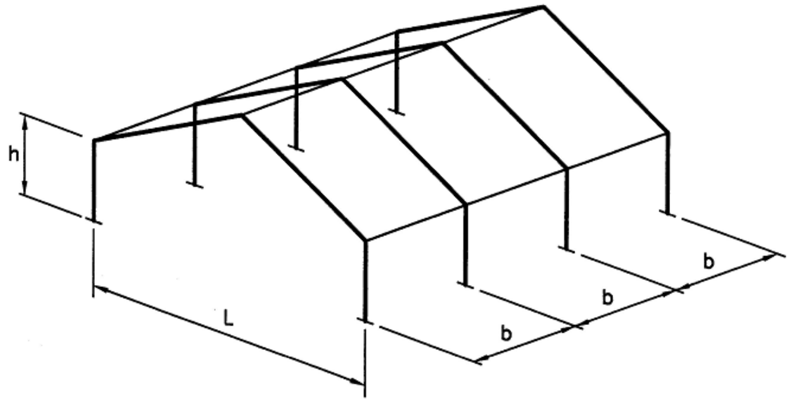


Figure 8 Notation for Deflection Limits (Woolcock et al, 1999)

Type of Building and Load	Deflection Limit	Comments
INDUSTRIAL BUILDINGS		
(a) Dead Load	$L/360$	For roof pitches > 3° (see footnotes)
	$L/500$	For roof pitches < 3° but check for ponding or insufficient roof sheeting slope (see footnotes)
(b) Live Load	$L/240$	Check spread of columns if gantry crane present
(c) Wind Load	$L/150$	If no ceilings
FARM SHEDS		
(a) Dead load	$L/240$	Check for ponding if roof pitch < 3°
(b) Live load	$L/180$	
(c) Wind load	$L/100$	

Figure 9: Recommended Rafter Deflection Limits (Woolcock et al, 1999)

Type of Building	Limits	Comments
INDUSTRIAL BUILDINGS (a) Steel sheeted walls, no ceilings, no internal partitions against external walls or columns, no gantry cranes	$h/150$ $b/200$	Relative deflection between adjacent frames
(b) As in (a) but with gantry cranes	$h/250$ $b/250$	(i) h may be taken at crane rail level (ii) $h/300$ should be used for heavy cranes
(c) As in 1(a) but with external masonry walls supported by steelwork	$h/250$ $b/200$	
FARM SHEDS	$h/100$ $b/100$	

Figure 10: Recommended Lateral Deflection Limits (Woolcock et al, 1999)

Notes:
The wind load deflection limits apply to serviceability wind loads.

- L is the rafter span measured between column centrelines.
- Precamber or pre-set may be used to ensure that the deflected position of the rafter under dead load corresponds to the undeflected design profile, or is within the above limits of the undeflected design proflle. Even so, pre-set may be advisable for internal rafters to avoid visual sag in the ridge line.
- For low roof pitches, the check for ponding is really a check to ensure that the slope of the roof sheeting is nowhere less than the minimum slope reconnnended by the manufacturer. The slope of the rafter in its deflected state can be determined from the joint rotations output from a plane frame analysis program. The slope of the roofing should also be checked mid-way between rafters near the eaves where purlins are more closely spaced and where the fascia purlin may be significantly stiffer than the other purlins.
- Where ceilings are present, more stringent limits will probably be necessary.

Differential Deflections

Generally, where a rafter and post frame has been used, it will be braced and will therefore be much stiffer than the adjacent portal frames. In practice this is also true with a portal frame gable wall because it will be stiffened by the cladding. Differential deflection between the gable frame and penultimate frame can therefore be relatively large, and may be of particular concern if there are cranes, masonry construction, or sensitive cladding attached to the frame.

Ways of reducing differential deflections include:

- Bracing in the roof between the gable frame and the adjacent frame will reduce the deflection of the adjacent portal frame to some extent, but this is normally not quantifiable without a 3-D analysis of the whole structure.
- A penultimate frame can be provided of greater stiffness than the other frames to reduce the differential deflection due to eaves spread and wind loading. This is not usually a sensible option in terms of fabrication efficiency.
- The portal frames should be pre-set carefully to ensure that all dead load deflections result in frames that line up with the gable frame under dead load only, thus reducing to some extent the differential deflection due to eaves spread.

Column Bases

Base Plates

Mild steel Grade 4.6 bolts are preferred because they can be adjusted by bending on site particularly if there is a sleeve or pocket around the holding down bolt for this purpose. Mild steel bolts can also be tack welded into a cage, whereas Grade 8.8 bolts should not be tack welded because welding can have an adverse effect on steel properties in the vicinity of the weld. Regardless of the steel grade, it is recommended that holding down bolts be hot dip galvanised.

Holding Down Bolt Design Criteria

There are many considerations in the design of holding down bolts (Trahair et al, 1998), the most important being as follows:

- The bolts themselves should have sufficient capacity in combined tension and shear.
- The grouting or bedding under the base plate should have sufficient capacity in compression to cater for applied compression and bending moment at the base of the column.
- The concrete or the grout filling the space around the bolts and sleeves should have sufficient strength in bearing to transmit the shear force in the bolt.
- If the bolts do not have a suitable head or other anchor at the head to prevent pullout or bearing failure under the head, the bolts must be sufficiently long or must be suitably cogged or hooked to satisfy the anchorage requirements for plain deformed bars (as appropriate) in the concrete standard NZS 3101 (SNZ, 2006).
- If the bolts have a suitable head or anchor, the embedment must be sufficient to prevent the bolts pulling out a cone of concrete (cone failure).
- If there is insufficient edge distance, the bolts must be lapped or anchored with reinforcing bars in accordance with the concrete standard.
- Account should be taken of fabrication and erection tolerances when detailing and installing holding down bolts.
- The likelihood of corrosion must be considered carefully. Hot dip galvanizing is recommended.
- A minimum of four bolts rather than two bolts is favoured by riggers to assist in supporting columns during erection.

Base Moments for Foundation Design

It should be noted that, as far as the base moment (and associated forces) for foundation design is concerned, the following applies:

- Where partial base fixity is used to reduce the moments for which frame members have to be designed (compared to those obtained assuming pinned bases) the base moments should be taken into account in designing the foundations. This applies for both elastic analysis and elastic analysis with redistribution of moments.
- Where a nominal 10% base stiffness is used only in assessing effective lengths (or elastic critical load factors) or in determining whether an unbraced frame is 'sway-sensitive' or 'non-sway' it is not necessary to take account of the base fixity moment in foundation design.

Roof & Wall Bracing

General

Roof and wall bracing often consist of panels of double diagonals which are so slender as to have negligible capacity in compression. Such members include pre-tensioned rods, slender tubes and angles. In the design of double diagonal tension bracing, one of each pair of diagonals is assumed to act in tension as shown in Figure 11, depending on the direction of wind loading, and the other diagonal is usually ignored. In addition to tension forces, roof bracing diagonals have to carry their own weight whether by cable action in the case of rods, or by beam action in the case of tubes and angles.

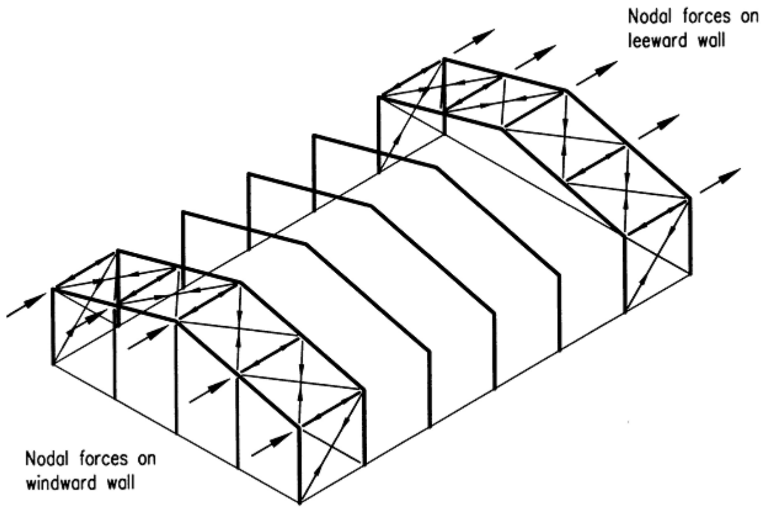


Figure 11: Roof and Wall Bracing (Woolcock et al, 1999)

As common as tension bracing is, there is not a widely accepted method of design which accounts for tension and self weight. This problem was investigated in References (Kitipornchai et al, 1985; Woolcock et al, 1985).

Temporary Bracing
Portal frames can collapse during construction if adequate care is not taken to use permanent or temporary bracing to withstand wind gusts. The procedure to be used varies from building to building depending on the type and location of the permanent roof and wall bracing bays and whether the end wall frame is a braced frame or a portal frame.

Roof Plane Bracing
Roof plane bracing is placed in the plane of the roof. The primary functions of the roof plane bracing are:
- To transmit horizontal wind forces from the gable posts to the vertical bracing in the walls.
- To provide stability during erection.
- To provide a stiff anchorage for the purlins that are used to restrain the rafters.

Rafter Bracing Forces
In addition to the longitudinal wind forces, the bracing system could also be considered as resisting accumulated, coincidental purlin or fly brace forces. When the top flange is in compression, the purlins act as braces whereas fly braces restrain the bottom flange when it is in compression. However, it is unclear whether the bracing forces should be accumulated. Purlins and fly braces together could be considered as providing a rotational restraint system in accordance with Clause 5.4.3.2 of NZS 3404. In this case, it would not be necessary to treat the compression flanges of rafters as parallel restrained members in accordance with Clause 5.4.3.3, and therefore it would not be necessary to accumulate the forces. On the other hand, purlins and fly braces could be considered as providing restraint against lateral deflection of the compression flange (Clause 5.4.3.1) and in this case the bracing forces would be accumulated.

It is interesting to compare roof trusses as far as accumulation of bracing forces is concerned. The bottom compression chord of a series of large span roof trusses under net uplift is usually braced back to the end bracing bays by a system of struts or ties. In this case, the bracing forces should be accumulated and then combined with forces due to longitudinal wind. When the top chord is in compression, it is usually regarded as being braced by purlins back to the end bracing bays. Logically, the top chord bracing forces should also be accumulated, but as the compression in the top chord is generally due to gravity loads, there are no other longitudinal forces in combination and so the loads on the end bracing bays are not likely to be critical.

It could be similarly argued that the top or bottom flange bracing forces of UB or WB rafters, whichever flange is in compression, should also be accumulated. However, even if the lateral restraint argument (as opposed to the rotational restraint argument) is accepted, the accumulated bracing forces are usually a small part of the total longitudinal force for portal frame buildings. It is therefore considered reasonable for UB or WB rafters, to ignore accumulated bracing actions in the design of the roof and wall bracing bays.

A summary of the advantages and disadvantages of various options for bracing layouts is as follows as shown in Figure 12.

Option I: Two End Bays Braced

This is the simplest and most direct option. Intermediate eaves and ridge struts are sometimes used as shown dashed. However, purlins are usually sufficient to brace internal rafters so that no intermediate struts are required.

Longitudinal wind loads, as a combination of pressure on the windward wall, suction on the leeward wall and friction, could be shared between braced bays if purlins have the capacity to transfer some compression load from one end to the other. However, it is recommended that the bracing at each end be designed to resist loads from external pressure and internal suction on the adjacent end wall (plus half of the frictional drag forces if applicable). This keeps the purlin design simple as purlins can then be designed without considering combined actions. Diagonals are crossed which means that CHS sections, which are efficient as long ties under self weight, cannot easily be used. This option also excludes the use of the top flange as a bracing plane with angle diagonals crossed back to back unless higher purlin cleats are used. End bay bracing can have detailing difficulties at the end wall rafter.

Option II: Double Diagonal Bracing Over Two Bays at Each End

- Diagonals intersect at rafters and therefore tubes can be used as diagonals without difficulty if they are not crossed.
- The number of diagonals is the same as for Option I but more struts are required.

Option III: Second Bay from Each End Braced

- This option can overcome any detailing difficulties associated with end bay bracing but extra struts are required to transfer the end wall wind loads to the braced bays unless the purlins can act as struts.

Option IV: One Bay Braced

- Struts in the unbraced bays are required to transfer end wall wind loads to the braced bay which is expensive unless the purlins can act as struts.

Option V: Single Diagonal Tension Bracing at Each End

- Unstable during erection.
- The windward braced bay takes all of the longitudinal wind loads.
- Purlins are usually sufficient to brace internal rafters as in Option I. Leeward end wall forces are transmitted to the active braced bay at the windward end by purlins in tension.
- Tubes can be used for diagonals without difficulty as they are not crossed.
- Single diagonal rods with turnbuckles should not be used as there is nothing to tension against.
- Temporary diagonals may be necessary to create a double diagonal bracing system for erection purposes in which case there is little advantage in a single diagonal system.

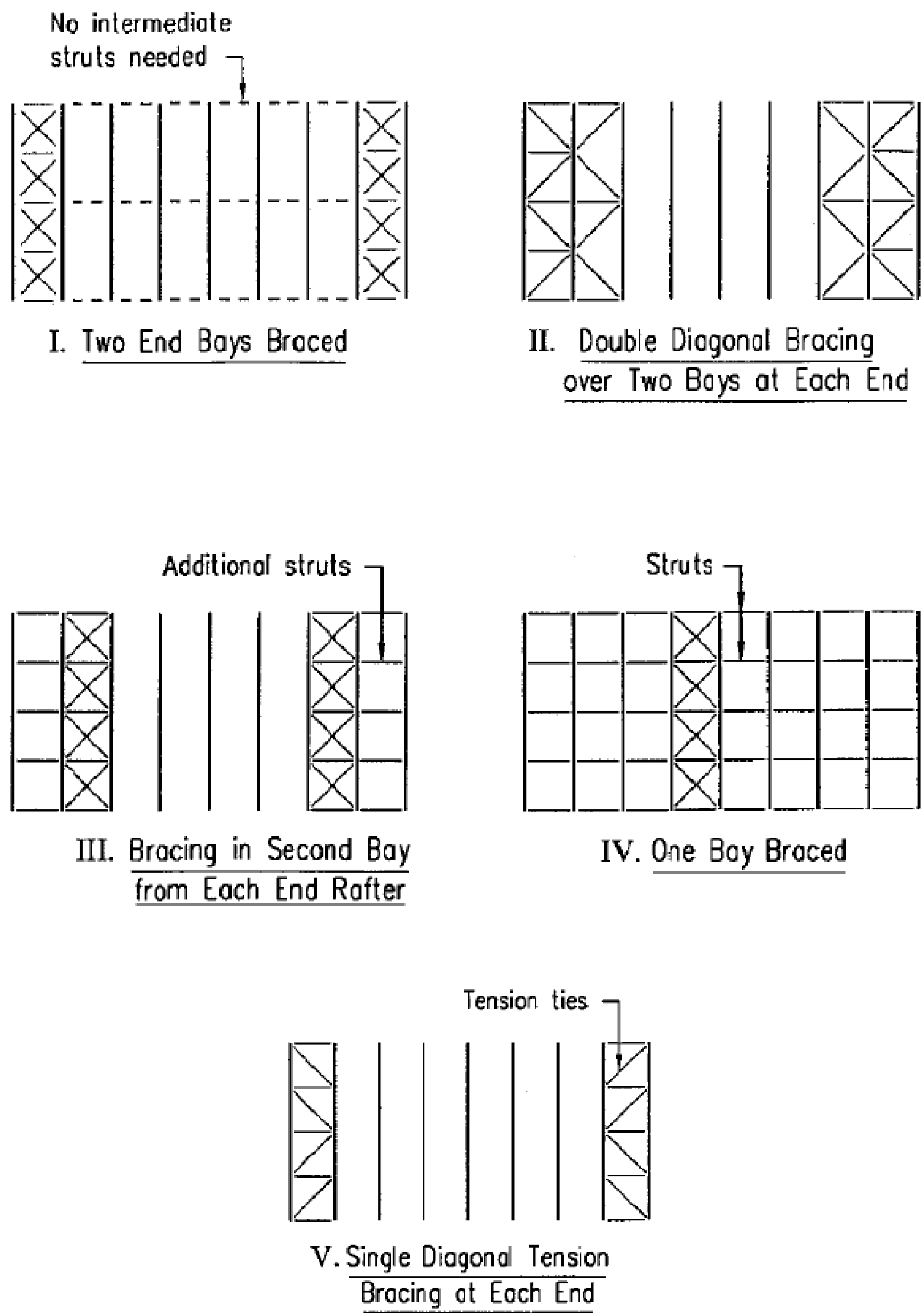


Figure 12: Bracing Layout Options (Woolcock et al, 1999)

Bracing using Circular Hollow Sections
In the United Kingdom circular hollow section bracing members are generally used in the roof and are designed to resist both tension and compression. Many arrangements are possible, depending on the spacing of the frames and the positions of the gable posts (Salter et al, 2004).

Tension Rod Bracing
Rods cater for the lower end of the range of tensile forces, and are very common in light industrial buildings. Rods differ from tubes and angles in that they must be pre-tensioned to reduce their self weight sag. However, there are certain aspects of rod pre-tensioning which are not widely understood. The aspects which need to be considered are as follows:

- The minimum level of pretension force needed to reduce the sag sufficiently to avoid undue axial slack in the rod.
- The level of pretension used in practice. The effect of pretension on the tensile capacity.
- The effect of pretension on the end connections, and on the adjacent struts in the roof bracing system, when wind loads are applied.

Pre-tension actions should be 10% to 15% of the yield capacity. While these levels of pretension may be adequate, it is not practical to measure or control the pre-stress level in practice. To answer the questions above properly, it is necessary to examine the behaviour of pre-tensioned rods in some detail.

Long rods behave like cables whose self-weight is carried by tension alone; the tension being inversely proportional to the sag. For small sags in roof bracing situations, the tensile stress f_{at} versus sag y_c relationship has been shown [13] to be independent of the rod diameter and is given by

$$f_{at} = 9.62 x 10^{-6} \left(\frac{L^2}{y_c} \right) \text{MPa}$$

in which L is the length of the rod and both y, and L are in mm. This relationship is presented graphically in Figure 13. Using this equation, it can be demonstrated that as a rod is tensioned, very little force is required to reduce the sag until the sag gets to about span/100. The rod then begins to stiffen suddenly and behave as a straight tension member. This is shown graphically in Figure 14. Therefore, the maximum sag of a rod to avoid undue axial slack should be about span/100. Surprisingly, a stress of only 20 MPa is required to reduce the sag of a 20 metre cable to the L/100 deflection. However, typical stress levels in practice could be much higher.

In experiments at the University of Queensland (Woolcock et al, 1985), six different laboratory technicians were asked to tighten rods ranging in diameter from 12 mm to 24 mm with spans up to 13 metres long. They were told to tighten the nuts as if they were working on site. Once tightened at one end, the force in the rod was measured with a calibrated proving ring connected to the other end. The experiments revealed that the average level of pretension force was well in excess of the value of 10% to 15% suggested in (Gorenc et al, 1996). In fact, it was found that 16 mm diameter rods were tensioned close to their design capacity, while 20 mm rods were tensioned to between 40% and 55% of their design tensile capacity. Because of these unexpectedly high pretension forces, excessive sag is not a problem, even for a 20 metre span.

The presence of pre-tension does not affect the ultimate tensile capacity of the rod itself. However, there are a few other factors that need to be considered in the design of roof bracing rods. In some cases of over-tensioning, the active tension diagonal may yield under the serviceability wind load, although yielding will relieve the pretension in the system to some extent. Fortunately, the fracture capacity of the threaded section typically exceeds the yield capacity of the rod itself. This means that the main body of the rod will generally yield before failure of the turnbuckle section. Because of the pretension, the rod connections should be designed so that their ultimate or fracture capacity is equal to or greater than the ultimate or fracture capacity of the rods. This is particularly important because oversized rods are often used. For example, a 20 mm diameter rod may be used because of its robustness where only a 16 mm diameter rod is required. This philosophy for the end connection design of rods is covered in Clause 9.1.4(b)(iii) of NZS3404.

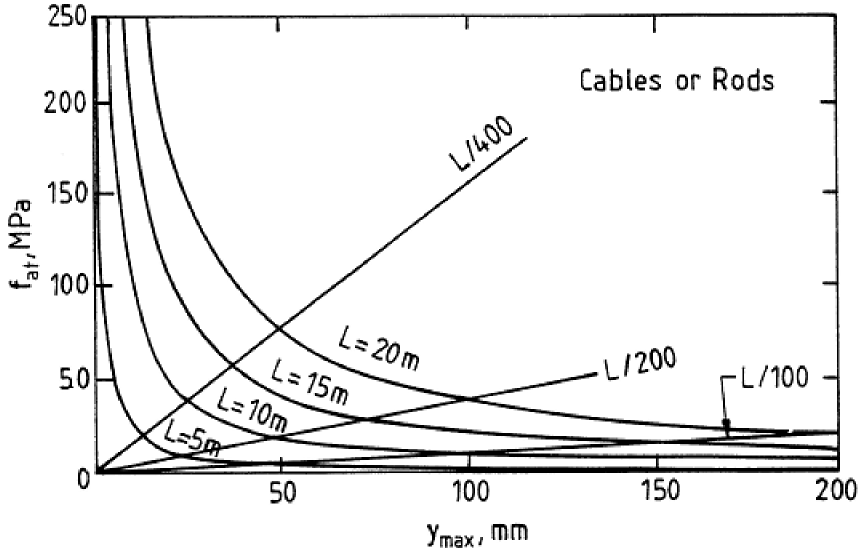


Figure 13: Effect of Axial Stress on Cable and Rod Deflections (Woolcock et al, 1999)

Pre-tensioning could also result in overloading of the struts in the roof bracing system, especially if rods larger than that required are used. A check should therefore be made in the design of the struts to cater for forces in the diagonals due to combined pretension and wind load as shown in the design example.

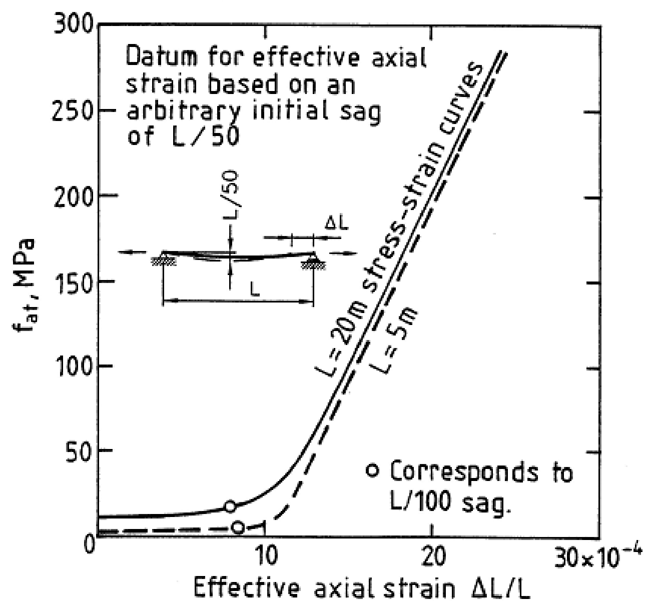


Figure 14: Effective Axial Stiffness of Cables and Rods (Woolcock et al, 1999)

Tubes and Angles in Tension

In contrast to rods, tubes and angles are not easily pre-tensioned and must be sized as beams to limit self weight sag. The uncertainties for designers, as far as tube and angle section members are concerned, are firstly the effect of self weight bending on tensile capacity, and secondly deflection limits. Some engineers combine self weight bending actions with axial tensile actions, while many engineers intuitively ignore the bending actions.

It can be shown theoretically (Woolcock, 1985) that self weight bending has a marginal effect on the ultimate fracture capacity of a tube or angle. This is because the sag and self-weight bending moments reduce as the tension increases. It can therefore be concluded that self-weight bending actions need not be considered in combination with axial tension.

As proposed for rods, a maximum sag of span/100 is suggested to avoid undue slack. However, it is advisable to limit deflections to span/150 to avoid lack of fit without propping during erection, and for aesthetic reasons. Note that even with a span/150 deflection, there is occasionally concern expressed during construction as the sag can be quite evident if one sights along the member. The sag is not generally obvious from floor level.

Of course, the designer has the option of suspending the diagonals from the purlins, but very flexible diagonals (other than rods) can be difficult to erect before the purlins are in place because of lack of fit. If the purlins are erected first, the stability of the portal frames without bracing may be inadequate and lifting the diagonals into place will be more difficult because of obstruction from the purlins. Furthermore, the extra labour necessary to drill and suspend may cost more than the material saved. The effect of purlin uplift loads on the capacity of diagonals should also be taken into account. With all these factors considered, suspending very flexible diagonals from purlins is not recommended.

Side-wall Bracing

General
The primary functions of vertical bracing in the side walls of buildings are:
- To transmit the horizontal loads, acting on the end of the building, to the ground.
- To provide a rigid framework to which side rails may be attached so that they can in turn provide stability to the columns.

To provide temporary stability during erection, the bracing system will usually take the form of:
- Circular hollow sections in a V pattern.
- Tension only cross-braced rods.
- Circular hollow sections in a K pattern.
- Crossed flats (within a cavity wall).
- Crossed hot rolled angles.

The bracing may be located at:
- One or both ends of the building, depending on the length of the structure.
- At the centre of the building (but this is rarely done due to the need to begin erection from one braced bay at, or close to, the end of the building).
- In each portion between expansion joints (where these occur).

Where the side wall bracing is not in the same bay as the plan bracing in the roof, an eaves strut is required to transmit the forces from the plan bracing into the wall bracing.

Side-wall Bracing Using Circular Hollow Sections
Circular hollow sections are very efficient in compression, which eliminates the need for cross bracing. Where the height to eaves is approximately equal to the spacing of the frames, a single bracing member at each location is economic. Where the eaves height is large in relation to the frame spacing, a K brace may be used.

An eaves strut may be required in the end bays, depending on the configuration of the plan bracing. In all cases, it is good practice to provide an eaves tie along the length of the building.

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