INTERIM REPORT

DESIGN OF PORTAL FRAMES TO EUROCODE 3: AN OVERVIEW FOR UK DESIGNERS

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T: +44 (0)1344 636525
F: +44 (0)1344 636570
E: reception@steel-sci.com

www.steel-sci.com

To report any errors, contact:
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FOREWORD

In the UK, designers of portal frame buildings have commonly used bespoke software, specifically written for the analysis and member design in accordance with BS 5950. At the time of writing (March 2013), the view has arisen that design to Eurocode 3 is not practical. This short guide was written to refute that impression.

The guidance aims to show how existing analysis software (for use in design to BS 5950) may be utilised, with appropriate adjustments, and how (at least in the interim, until Eurocode-specific software is widely used) portal frame member design can be carried out manually at critical locations without undue complexity.

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SUMMARY

Currently, the analysis and design of portal frames is almost exclusively completed using bespoke software, due to the complexities of elastic-plastic analysis, the inclusion of second-order effects and the sheer volume of checks to be completed for a number of combinations of actions (load cases). The available software has been developed for design in accordance with BS 5950.

In advance of widespread use of analysis and design software for design to Eurocode 3, this guide explains how existing analysis software (both bespoke and general) may be used, in conjunction with limited manual member verification, to design portal frames in accordance with the Eurocodes.

This guide complements the guidance given in SCI Publication P397, which covers the elastic design of symmetrical, single span frames, and anticipates more comprehensive general guidance in SCI publication P399, due to be published in late 2013.

It is expected that, in due course, comprehensive bespoke software will be used for portal frame design to Eurocode 3.

INTRODUCTION

For most portal frame buildings, the frames will be analysed and designed using bespoke software, written specifically for portal frame design, rather than using a general analysis and design program. Such software alleviates the burden of the lengthy evaluations needed for such frames.

Conceptually, it is helpful to separate analysis and member design. Analysis is generally independent of the design standard. Thus with appropriate modelling, and using appropriate loading, existing analysis software will give results that will be appropriate for use in Eurocode designs.

Detailed member design is strongly dependent on the relevant design standard. In the Eurocodes, the design of members, referred to as "member verification" should generally be in accordance with BS EN 1993-1-1[1]. Until bespoke portal frame software is used, designers can make use of the following:

- General element design software to BS EN 1993-1-1 (already available).
- Manual design, using the Blue Book[2], for ease.

Connections should generally be designed in accordance with BS EN 1993-1-8[3]; detailed guidance on the design of moment-resisting connections is available in SCI P398[4].

ANALYSIS OF PORTAL FRAMES

Elastic or plastic analysis can be used for the analysis of portal frames. Bespoke software for portal frame design is almost certain to complete a plastic analysis, as this generally produces a more economical (lighter weight) design.

2.1 Frame analysis for BS EN 1993-1-1

For frames designed according to BS EN 1993-1-1, the frame analysis must account for:

- Second-order effects
- Frame imperfections (e.g. lack of verticality)
- Member imperfections (e.g. straightness and residual stress)

These issues are discussed in more detail below.

2.1.1 Second-order effects

All structural frames experience some degree of second-order effects. Analysis software may always make allowance for second-order effects (even when such effects are small) or may make allowance only when they are significant.

BS EN 1993-1-1 states that, as an alternative to explicitly allowing for second-order effects in the analysis, allowance may be made for second-order effects by modifying a first-order analysis – generally by applying an amplification factor to the lateral loads.

Sensitivity limits for portal frames

In BS EN 1993-1-1, the parameter used to assess sensitivity to second-order effects is designated as α_{cr}, which is the ratio of the elastic critical buckling load for the frame to the design value of actions. BS EN 1993-1-1 prescribes values of α_{cr} for elastic and plastic analysis above which second-order effects are small enough to be ignored. The UK National Annex[5] modifies the limits for plastically designed portal frames that pass certain geometric limits (see clause NA.2.9 of the UK NA), when the stiffening effect of cladding has been ignored (in the calculation of α_{cr}). The limits are presented in Table 2.1.

Table 2.1 Liming values of α_{cr} in BS EN 1993-1-1 and its UK NA

TYPE OF ANALYSIS	COMBINATION OF ACTIONS	SECOND-ORDER EFFECTS SMALL ENOUGH TO BE IGNORED	SOURCE
Elastic	All	$\alpha_{cr} > 10$	BS EN 1993-1-1
Plastic	Gravity loads and EHF	$\alpha_{cr} > 5$	UK NA
Plastic	All other combinations	$\alpha_{cr} > 10$	UK NA

Note that the UK NA allows a less onerous limit for plastically designed frames in the "gravity" combination of actions. Logically, this lower limit should also be appropriate for elastically designed frames in the "gravity" combination of actions.

SCI recommends that the lower limit of $\alpha_{cr} > 5$ is also appropriate for frames designed elastically, in the "gravity" combination, if the frame passes the geometric constraints listed in the UK NA. Using this lower limit for an elastically analysed frame would (currently) mean that a frame design could not be said to comply with the Eurocode.

2.1.2 Frame imperfections

According to BS EN 1993-1-1, the effect of frame imperfections should be accounted for in every combination of actions, unless the external horizontal actions are relatively large, compared to the vertical actions (see clause 5.3.2(4)).

SCI advice is that it is simplest (and in fact makes only a modest difference to the eventual results) to include the allowance for frame imperfections in every combination of actions. In BS EN 1993-1-1, the effects may be allowed for by a system of Equivalent Horizontal Forces (EHF).

It will probably be simplest to create a load case comprising of just the EHF, and include this in subsequent combinations of actions. As the EHF are a proportion of the factored vertical actions, the EHF will vary with combination. In most cases, however, the EHF and their effects are small, so it is not unduly conservative to calculate the EHF for the most onerous combination of actions (the "gravity" combination) and use this value of EHF in all subsequent combinations.

For more finesse, and to be expected in software written for the Eurocode, the EHF will vary in each combination.

The EHF are applied at the top of each column, as horizontal point loads, in the same direction on each column.

Calculation of the EHF

The EHF should be calculated as the factored vertical reaction at the base, multiplied by $\frac{1}{200}\alpha_h\alpha_m$, in which:

α_h is a reduction factor due to the height of the columns, taken as $\alpha_h = \frac{2}{\sqrt{h}}$, but $\frac{2}{3} \le \alpha_h \le 1.0$

h is the height of the column, generally taken to be the height to the centreline node of the column and rafter.

α_m is a reduction factor for the number of columns, taken as $\alpha_m = \sqrt{0.5\left(1+\frac{1}{m}\right)}$

m is the number of columns in the frame. For a two-span frame with a central column (a "hit" frame), $m = 3$.

Both α_h and α_m may be set to 1.0; this is conservative.

2.1.3 Member imperfections

According to clause 5.2.2(3) of BS EN 1993-1-1, member imperfections (an initial out of straightness) may be accounted for either within the global analysis, or within the member verifications.

SCI recommends that in-plane member imperfections are accounted for in the global analysis. This recommendation is elegant because it has been found that, for orthodox frames, the impact of initial member imperfections in the global analysis is small enough to be ignored. This conclusion has been reached after comparing the analysis results for a range of orthodox frames and assessing the impact of in-plane member imperfections. The results of this study are published elsewhere[6].

The attractive outcome from this approach is that no in-plane member verifications are required for members in orthodox portal frames. More advice is given in Section 5.1.

It is convenient to allow for out-of-plane member imperfections in the member verification, as described in Section 5.2. The member resistances calculated in accordance with Section 6.3 of BS EN 1993-1-1 incorporate the effect of member imperfections, as clause 5.3.4(1) confirms.

2.1.4 Base fixity

BS EN 1993-1-1 is silent on allowance for the base fixity of columns (such allowance is familiar to BS 5950 designers). However, Non-contradictory complementary information (NCCI) is available[7] which confirms that the base fixity rules used with BS 5950-1 remain appropriate for use with BS EN 1993 1-1.

Thus, for a nominally pinned base, 10% of the column stiffness may be used to model the stiffness of the base when assessing frame stability. Bespoke software for portal frame design may already have this stiffness as an optional choice.

2.2 Use of bespoke BS 5950 software for portal frame analysis

2.2.1 Sensitivity to second-order effects

Bespoke software will calculate and report the parameter λ_{cr}. This parameter is the exact equivalent of α_{cr} required by BS EN 1993-1-1.

Software used for designs to BS 5950 will include (or allow for) second order effects, as required.

If designing to the Eurocodes and second-order effects are significant, the two approaches available are:

1. to allow for the second-order effects in the analysis, or
2. amplify all lateral loads (this includes the externally applied axial loads and the EHF – see Section 2.2.2)

Either of these approaches should be chosen in the software. Options described as "sway-check method" or a reference to clause 5.5.4.2.1 (the BS 5950-1 clause) should not be selected.

Allowing for second-order effects in the analysis may be achieved by a number of methods within the software. Any references to "P292" or similar refer to a method of allowing for second-order effects described in SCI publication P292[8]. This approach is perfectly satisfactory.

2.2.2 Frame imperfections

In BS 5950-1, frame imperfections are allowed for by Notional Horizontal Forces (NHF) but these only appear in the so-called "gravity" load combination.

2.2.3 Member imperfections

No allowance for member imperfections is generally made in bespoke software prepared for portal frame design to BS 5950-1. Following the guidance in Section 2.1.3, no allowance need be made for orthodox frames designed in accordance with BS EN 1993-1-1.

2.2.4 Base fixity

As noted above, bespoke software for design to BS 5950-1 generally has options for base fixity which may be selected by the user. These options remain appropriate for design of portal frames to BS EN 1993-1-1.

2.3 Use of general analysis software for portal frame analysis

2.3.1 Sensitivity to second-order effects

General analysis software may not calculate α_{cr}. Some software completes a buckling analysis of frames, and it may be possible to isolate the first in-plane buckling mode, with its associated eigenvalue, by carefully inspecting the graphical output. It may be that a reasonably large number of buckling shapes need to be analysed and reviewed before the first in-plane mode is observed. The number of buckling analyses required is likely to be set via an interface, so this number may need increasing.

It is generally prudent to introduce out-of-plane restraints around the frame before completing the buckling analysis, to avoid being swamped by out-of-plane buckling modes. The eigenvalue for the first in-plane buckling mode is α_{cr}.

If the software does not calculate α_{cr} or λ_{cr} (the direct equivalent), or complete a buckling analysis, it is possible to calculate α_{cr} by assessing deflections under Notional Horizontal Forces (NHF). Full details are given in P397[9].

2.3.2 Allowing for second-order effects

A general analysis package may have the facility of a second-order analysis. If not, the alternative is to calculate an appropriate amplifier and multiply the horizontal loads in a new combination of actions. This is described in detail in P397.

2.3.3 Frame imperfections

The same guidance as given in Section 2.2.2 is appropriate.

2.3.4 Member imperfections

No allowance for member imperfections is normally made in general analysis software. Following the guidance in Section 2.1.3, no allowance need be made for orthodox frames designed in accordance with BS EN 1993-1-1.

2.3.5 Base fixity

The same guidance as given in Section 2.1.4 applies. In general analysis software, it may be appropriate to model the effects of base stiffness with a dummy member – details are given in P397.

ACTIONS

The frame analysis is required to determine the effects (forces, moments, etc.) due to the combinations of 'actions' acting on the structure in a range of design situations. The actions on portal frames are principally the loads due to self-weight, imposed and snow loads on roofs, and forces due to wind pressure. The actions also include the EHF that represent the frame imperfections (discussed in Section 2.1.2).

3.1 Characteristic actions

Characteristic values of actions are defined in the various Parts of BS EN 1991 and their National Annexes.

3.1.1 Imposed loads on roofs

Imposed loads on roofs are defined in BS EN 1991-1-1[10] and its UK National Annex[11]. Table NA.7 of the UK NA states that the imposed load for roofs not accessible except for normal maintenance and repair should be taken as 0.6 kN/m^2 for roof slopes up to 30°.

3.1.2 Snow load

Snow loads are defined in BS EN 1993-1-3[12] and its UK National Annex[13]. Drifted snow is considered to be an accidental action, as discussed in Section 3.2.3.

3.1.3 Wind actions

Wind actions are defined in BS EN 1993-1-4[14] and its UK National Annex[15].

Detailed advice on the calculation of wind actions is given in P394[16]. For portal frames, SCI recommends that the internal pressure coefficient c_{pi} is calculated for the wind direction being considered. Further details are given in P397.

3.2 Combinations of actions at ULS

All analysis software will allow the creation of combinations of actions (loadcases), in which characteristic values of actions are multiplied by appropriate factors to give design values for the combinations. The Eurocode values must be used when creating combinations of actions, as they are significantly different from those in BS 5950-1. Note that, in some cases, the combination factors may be set by default to the BS 5950-1 values in current software.

As noted in Section 2.1.2, SCI recommends that, for simplicity, the EHF are included in every combination.

3.2.1 "Gravity" combinations of actions

BS EN 1991-1-1, 3.3.2(1) states that, on roofs, imposed loads and snow loads or wind actions should not be applied together simultaneously.

The "gravity" combinations of actions for portal frames are therefore:

1. Permanent + the more onerous of imposed or snow + EHF
2. Permanent + snow + wind + EHF
3. Permanent + wind + snow + EHF

In these combinations, the wind loadcase that results in the most onerous downward actions should be selected. This loadcase will have positive external pressure coefficients on the roof.

For the gravity combinations of actions, it is advantageous to use expression 6.10b from BS EN 1990[17]. The resulting expressions, combining the partial factors, reduction factors and combination factors specified in the UK NA[18], are given in Table 3.1.

Table 3.1 "Gravity" combinations of actions

COMBINATION	DESIGN VALUE OF ACTIONS
1	1.25 Permanent + 1.5 (more onerous of imposed and snow) + EHF
2	1.25 Permanent + 1.5 Snow + 0.75 Wind + EHF
3	1.25 Permanent + 1.5 Wind + 0.75 Snow + EHF

In the expressions in Table 3.1, the combination factors for snow assume that the site altitude is less than 1000 m above sea level.

Note that the EHF are implicitly factored, as they are a proportion of the design values of vertical reactions at the base. A conservative approach is to use the EHF as calculated for combination 1 in all three combinations.

3.2.2 Uplift combinations of actions

The uplift combination of actions is:

1. Permanent + wind + EHF

In this combination, the wind loadcase that results in the most onerous uplift should be selected. It is likely that two design situations will need to be considered: one with wind on the side of the building, with negative external pressure coefficients, and one with wind on the end of the building, blowing parallel to the line of the apex.

For uplift combinations of actions, it is advantageous to use expression 6.10 from BS EN 1990. The resulting expression, combining the partial factors and combination factors given in the UK NA, is given in Table 3.2.

Table 3.2 Uplift combination of actions

COMBINATION	DESIGN VALUE OF ACTIONS
1	1.35 Permanent + 1.5 Wind + EHF

3.2.3 Accidental actions

Two accidental design situations are generally considered in portal frame design:

1. With drifted snow (in valleys, and against parapets, *etc.*)
2. When a dominant opening that was assumed to be closed (for the gravity and uplift combinations) is open.

Expression 6.11b from BS EN 1990 should be used when considering accidental actions. The resulting expressions when the accidental action is drifted snow, combining the partial factors and combination factors given in the UK NA, are given in Table 3.3.

Table 3.3 Accidental combination of actions

COMBINATION	DESIGN VALUE OF ACTIONS
1	Permanent + Accidental action + EHF
2	Permanent + Accidental action + 0.2 Wind + EHF

Although expression 6.11b includes the effects of a leading variable action and other accompanying actions, imposed roof load or uniform snow do not appear in the above expression because:

- The UK NA states that the combination factor to be used for the leading variable action is ψ_1, which is zero for imposed loads on roofs
- Snow cannot be both the accidental action (drifted) and a variable action (uniform) at the same time

More advice on dominant openings and how they should be treated is given in P397.

ANALYSIS OUTCOMES

In general, the recommended strategy to determine design effects (whether completed manually or automatically within the software) is, for each design situation:

- To carry out a frame analysis and determine α_{cr}
- Depending on the value of α_{cr} and the combination being considered:
  - To carry out a second-order analysis, or
  - To carry out a first-order analysis in which the lateral loads are amplified to allow for second-order effects

If the frame is sufficiently stiff, α_{cr} will exceed the limiting values and no second-order effects need to be allowed for. However, as noted in Section 2.1.1, software may make allowance for second-order effects in all cases.

The outcome of this process will depend on the type of analysis undertaken.

4.1.1 Plastic analysis using bespoke software

The outcome will be a set of forces and bending moments around the frame, including (if necessary) an allowance for second order effects. The software will report α_{cr} (though this may be reported as λ_{cr}) and λ_{p}, the ratio of the plastic collapse loads to the ULS loads. For a satisfactory outcome, the value of λ_{p} should always be greater than 1.0.

4.1.2 Elastic analysis using general structural analysis software

The outcome will be a set of forces and bending moments in the members of the frame, including (if necessary) an allowance for second order effects.

MEMBER VERIFICATION

For design to BS 5950-1, existing bespoke software will invariably complete exhaustive member checks for every length between restraints, for every load combination. The designer will probably have to specify the location and type of restraint, and may be able to select the type of check to be carried out over a particular length. Several different types of check are available in BS 5950-1.

The range of member verifications required in BS EN 1993-1-1 is similar to the range of checks in BS 5950-1. Until software to BS EN 1993-1-1 is used for the verification of members in portal frames, designers will need to complete manual verifications of members. It is recommended that, in manual verification, designers limit themselves to the critical verifications for the relevant load combinations and observe by inspection that less onerous cases are satisfactory.

Member verification involves:

- Classification of cross-section
- Calculation of cross-sectional resistance
- Calculation of buckling resistance

In general, members subject to axial compression and bending are verified using expressions 6.61 and 6.62 in BS EN 1993-1-1, which cover in-plane and out-of-plane verification respectively. If the interaction coefficients needed for expressions 6.61 and 6.62 are selected from Annex B of BS EN 1993-1-1, the expressions may be considered to address in-plane and out-of-plane buckling. This cannot be said if the interaction coefficients are selected from Annex A.

SCI recommends that the interaction coefficients are selected from Annex B, for simplicity, and, to be consistent with that choice, only out-of-plane checks are required for members in portal frames.

Detailed guidance on member verification in elastically designed frames is given in P397. The following Sections provide additional guidance for the verification of members in plastically designed frames, and modify the guidance in P397 relating to in-plane member verification.

5.1 In-plane member verification

There is no need to verify the members using expression 6.61, as second-order effects and all imperfections have been allowed for in the global analysis. Cross-section verification is all that is required.

The cross-section verification is simply to demonstrate that the design bending resistance exceeds the design moment. In general, the bending resistance of a member may be reduced because of the effect of compression or shear, but this is unlikely to be the case for most portal frames because the compression and shear forces will be relatively small.

5.2 Out-of-plane member verification

Out-of-plane verification is carried out for lengths of members between restraints (segments of members). The requirements differ for segments adjacent to plastic hinges, and for segments in which all the effects are within the elastic range.

Comments on flange restraint and the identification of critical segments are given in Sections 5.3 and 5.4.

5.2.1 Segments adjacent to plastic hinges

Positions where plastic hinges will develop always need lateral and torsional restraint; this defines one end of the segment. An additional restraint must be provided within a distance L_{m} from the plastic hinge and this defines the other end of the segment. The value of L_{m} is given by expression BB.5, in clause BB.3.1.1 of BS EN 1993-1-1.

The length L_{m} is similar to (but not identical to) L_{m} as defined in BS 5950 1, clause 5.3.3.

5.2.2 Elastic segments

Expression 6.62 should be used to verify segments between restraints. Detailed numerical examples of typical verifications are given in P397. With no minor axis bending (which is the usual case for segments in portal frames), expression 6.62 reduces to:

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

For preliminary verification, the value of k_{zy} may be taken as 1.0 (a typical range is from 0.96 to 1.0).

Out-of-plane member verification can readily be carried out using resistance values taken from the Blue Book[2]. The axial resistance to buckling in the minor axis, $N_{\mathrm{b,z,Rd}}$ should be based on the distance between restraints. The bending resistance $M_{\mathrm{b,Rd}}$ should be based on the same length, and should account for the shape of the bending moment diagram using the C_1 factor. It is conservative to use a smaller C_1 value.

5.3 Restraint to one flange

If only one flange is restrained (typically the tension flange), an increased resistance can be calculated, compared to an unrestrained member.

Before the benefit of tension flange restraint can be utilised, the restraints must be demonstrated to be sufficiently close to be effective. In P397, this demonstration is made by making sure that the intermediate restraints are no further apart than the distance L_{m}, the limiting length for a segment adjacent to a plastic hinge, as described in Section 5.2.1.

For elastic segments, the use of L_{m} is conservative. It is recommended that in an elastic segment of a member, expression 6.62 (see Section 5.2.2) is used to determine whether the intermediate restraints are spaced sufficiently closely to be effective.

5.4 Member segments to be verified

This Section describes the critical checks to be undertaken, presuming that the member verifications will be completed manually, until software is used.

The following guidance assumes that a minimum of three restraints to the inside flange are *always* provided at the following locations:

- To the column, at the underside of the haunch
- To the inside flange of the rafter, at the sharp end of the haunch
- To the inside flange of the rafter, adjacent to the apex

These "default" restraints are shown in Figure 5.1.

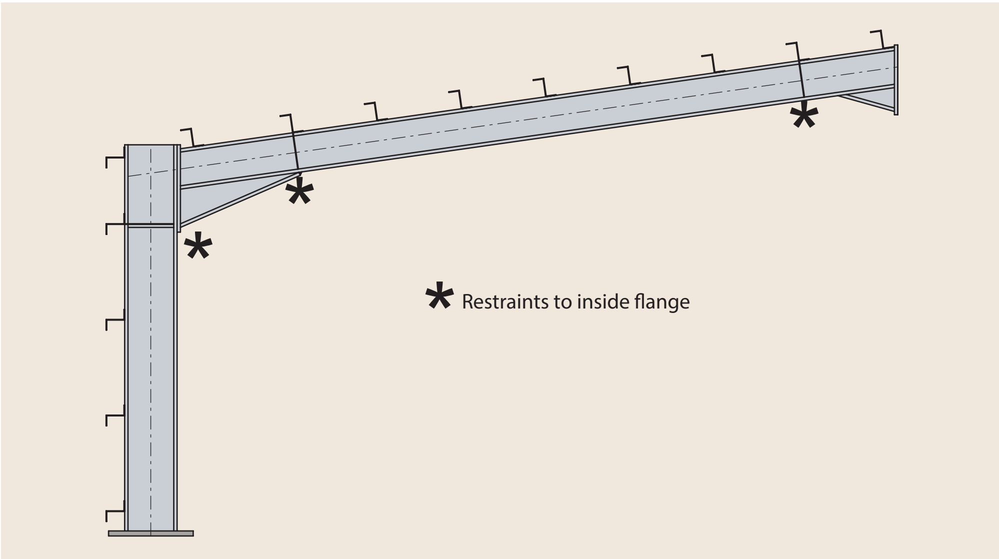

Figure 5.1
Minimum restraints to the inside flange

5.4.1 Columns – "gravity" combination

Each column must be verified over the length from the underside of the haunch to the base, unless intermediate restraints to the inside flange are provided. If intermediate

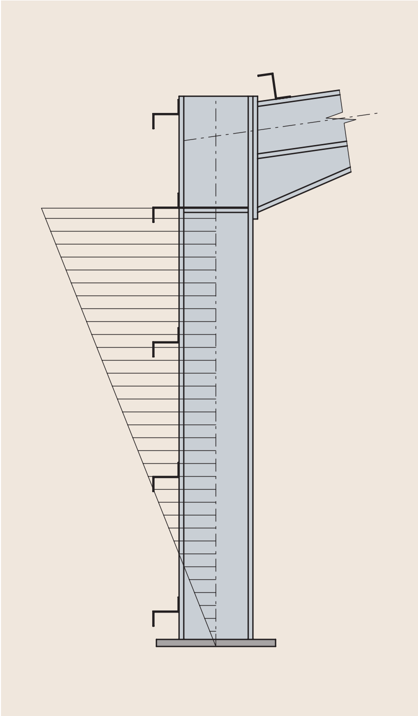

Figure 5.2
Bending moment in column leg – "gravity" combination

restraints are provided, the column must be verified between each restraint (alternatively, the intermediate restraints may be ignored). The bending moment diagram will vary linearly between restraint positions as shown in Figure 5.2; as noted in Section 5.2.2, it is important to allow for the effect of the shape of the bending moment diagram when determining the resistance to buckling.

A full numerical example for a portal column in an elastically analysed frame is provided in P397.

In a plastically designed frame, if there is a hinge at the underside of the haunch, the next restraint must be at a distance no greater than L_m (see Section 5.2.1). Verification below the segment adjacent to the hinge is identical to that for an elastically analysed frame.

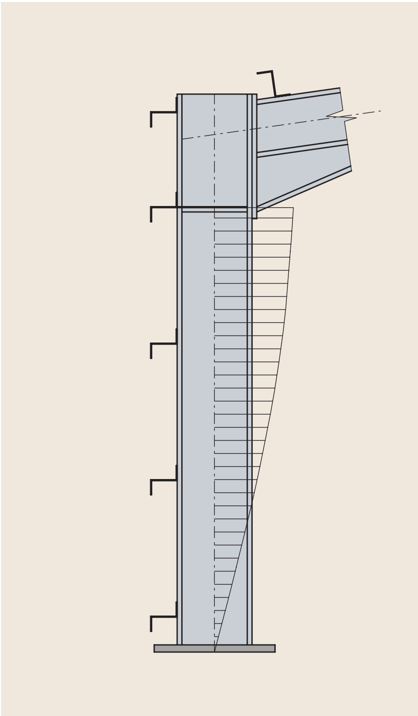

Figure 5.3
Bending moment in column leg – uplift combination

5.4.2 Columns – uplift combination

Verification should be carried out for each segment between restraints –generally the lengths between side rails on the outside flange.

The bending moment diagram is likely to be smaller in absolute magnitude than the "gravity" combination, and the bending moment diagram at the top of the column is probably reasonably uniform, as shown in Figure 5.3. Unless the side rail spacing varies, the critical verification is likely to be the upper segment of the column, with an approximately uniform bending moment diagram.

5.4.3 Rafters – "gravity" combination

In the gravity combination of actions, two zones are likely to be critical, as illustrated in Figure 5.4:

- The sagging zone, near the apex, where the moment is almost uniform, with an unrestrained length between the purlins (which provide restraint to the compression flange).
- The hogging zone, adjacent to the sharp end of the haunch, where the moment is varying, with an unrestrained length between the sharp end of the haunch (assuming a restraint to the inside flange is provided at that point) and at least as far as the point of contraflexure. It is conservative to assume the unrestrained length

extends to the first purlin past the point of contraflexure. If the rafter is not stable over this length, then additional restraints may be provided. The segment lengths are then the lengths between restraints.

In a plastically designed frame it is likely that there will be a plastic hinge at one or both of the points identified in Figure 5.4. As noted in Section 5.2.1, all plastic hinges should be restrained, and a further restraint is needed within a distance L_m of the hinge position.

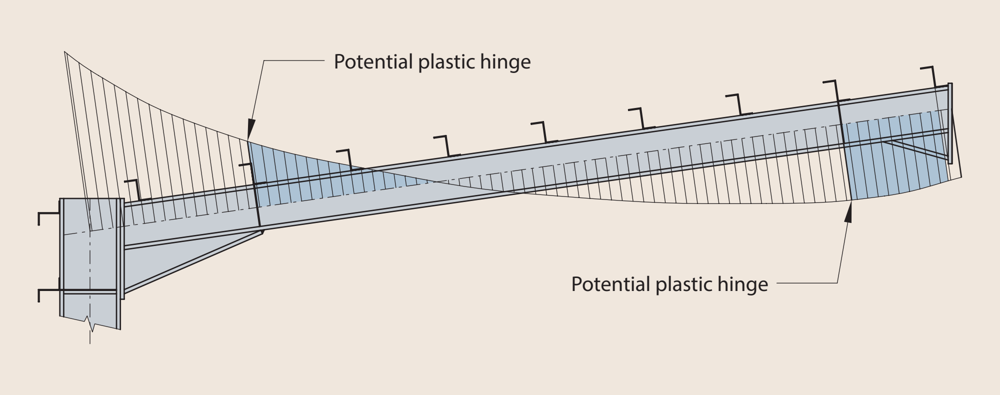

Figure 5.4 Rafter critical check zones – "gravity" combination

5.4.4 Rafters – uplift combination

A typical bending moment diagram for an uplift combination is shown in Figure 5.5. Under uplift combinations of actions, the absolute magnitude of the bending moment is generally smaller than the sagging moment in the "gravity" combinations, so a plastic hinge is unlikely.

In the uplift combination, the critical verification will be for the segment adjacent to the apex. It may be that the rafter is stable all the way to the point of contraflexure; if not, an additional restraint will need to be introduced at some point along the rafter.

Figure 5.5 also shows typical C_1 factors to be used when calculating the resistance to lateral torsional buckling.

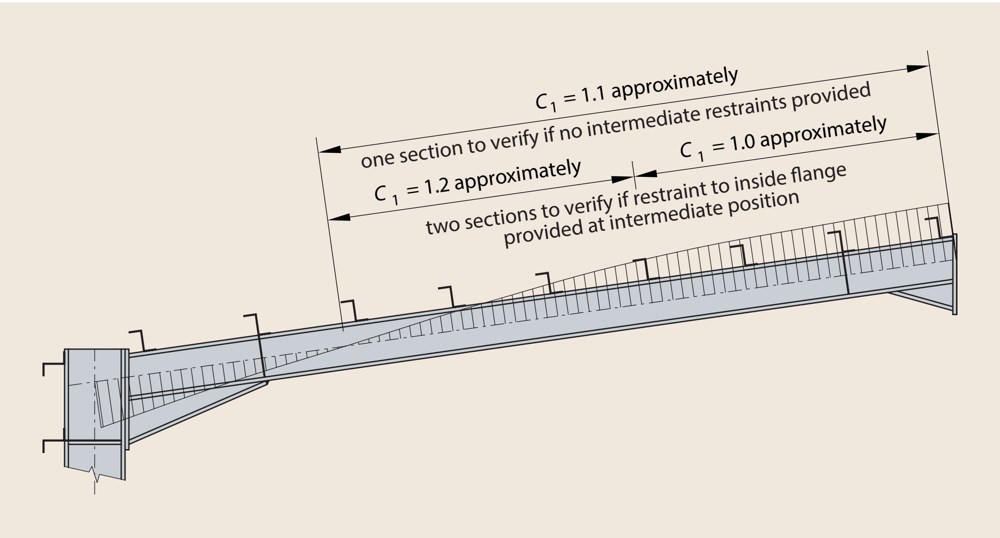

Figure 5.5 Rafter – uplift combination

5.4.5 Haunch verification

BS EN 1993-1-1 provides rules for stable lengths of tapered members (haunches), but only for lengths adjacent to plastic hinges. These rules are given in Section BB.3 of the Standard.

SCI intends to provide appropriate rules for elastically designed frames, and for when there are no plastic hinges adjacent to the haunch, in a future publication[6]. In the meantime, for elastic haunches, the plastic stability requirements may be used (they will be conservative). Alternatively, an equivalent tee approach may be used, considering the compression flange of the haunch in isolation, and verifying its resistance as a strut between restraints. The latter approach is demonstrated in P397.

SERVICEABILITY LIMIT STATE

The Eurocode does not provide any recommendations for limiting deflections of portal frames at SLS.

SCI recommends that previous practice should continue to be followed in the UK.

JOINTS

In both BS 5950-1 and BS EN 1993-1-8[3], connection design uses a component based approach. As components will have almost the same design strength according to both Standards, connections will have very nearly the same resistance.

The design approach described in P207[19], the SCI/BCSA guide to moment-resisting connections designed to BS 5950-1, followed the approach laid out in the early drafts of the Eurocode. This approach has hardly changed in the current version of BS EN 1993-1-8, so there are very few differences between the guidance in P207 and that in P398, for joints designed to BS EN 1993-1-8. Some minor differences are noted below.

The calculation of joint stiffness is an important requirement of BS EN 1993-1-8, to demonstrate that the assumptions made in the analysis are appropriate. In portal frames, the analysis will assume that the joints at eaves and apex are rigid. Instead of requiring the calculation of stiffness, BS EN 1993-1-8 allows a joint to be classified on the basis of previous satisfactory performance; the considerable experience in the UK is considered sufficient to demonstrate satisfactory performance of orthodox joint types.

The UK NA[20] is very explicit, stating that connections designed in accordance with the Green Book (P207) may be classified in accordance with the guidance in that publication. For portal frames, P207 states that well-proportioned connections designed for strength alone may be assumed to be rigid. It is expected that the UK NA will be updated to refer to P398 in the same manner.

Until software for the design of moment-resisting joints to BS EN 1993-1-8 is used, design using software to BS 5950-1 will generally be satisfactory.

At the time of writing (March 2013), practice in some areas appears to be to take the design forces and moments at joints from an analysis using Eurocode values of actions and to increase them by a factor before using BS 5950-1 software to design the connection. Such amplification is not necessary.

7.1 Component resistances

The following Sections describe the very few significant changes between the advice in P207[19] (the “BS 5950 Green Book”) and that in P398[4] (the “Eurocode Green Book”). The comments assume that the reader is familiar with P207.

7.1.1 Yield line patterns

When calculating the resistance of the equivalent T-stub, the yield line patterns are identified in BS EN 1993-1-8 as “circular” and “non-circular”.

For Mode 1, both circular and non-circular patterns are considered.
For Mode 2, only the non-circular patterns are considered.

This is described in Table 6.4 of BS EN 1993-1-8. In P207, the minimum pattern would have been taken for both modes. In practice this has little impact. The approach in P207 is conservative, if anything.

7.1.2 Web resistance

Reduction for coexisting shear

In BS EN 1993-1-8, a reduction factor ω is applied to both the column web tension resistance and the column web compression resistance. The value of the reduction factor depends on the shear force in the column web panel and is based on a transformation parameter β, which is given in Table 6.3 of BS EN 1993-1-8. In a one-sided connection (a typical eaves connection) $\beta = 1$, and the reduction factor ω is equal to ω_1 as defined in Table 6.3 of the Standard.

The value of ω_1 varies with the column section, and the effective width, and is given by the expression:

$$\omega_1 = \frac{1}{\sqrt{1+1.3\left(b_{\mathrm{eff}}t_{\mathrm{wc}}/A_{\mathrm{vc}}\right)^2}}$$

where

b_{eff} is the effective width of column web in tension or compression

t_{wc} is the column web thickness

A_{vc} is the shear area of the column, as defined in BS EN 1993-1-1.

Web tension resistance

In P207, the length of the web assumed to resist tension is limited by geometry. Clause 6.2.6.3(3) of BS EN 1993-1-8, specifies that the effective width of column web in tension should be taken as equal to the effective length of equivalent T-stub, which is invariably longer than the physical length assumed in P207. To some degree therefore, the effect of the reduction factor ω is offset by the increased effective length assumed.

Generally, web tension is not the limiting component resistance, by a significant margin, although it may be the limiting feature in some connections. Assuming orthodox connection geometry for column/rafter connections, indicative values of the reduction factor ω $(= \omega_1)$ have been calculated and are given in Table 7.1; these values will enable a manual check to be completed, based on software that provides values of web tension resistance according to BS 5950-1.

Table 7.1
Typical values of reduction factor ω, for web tension resistance

COLUMN SIZE	REDUCTION FACTOR ω	
	SINGLE BOLT ROW	TWO BOLT ROWS
610 × 305 × 149	0.94	0.88
533 × 210 × 122	0.92	0.84
533 × 210 × 82	0.92	0.84
457 × 191 × 67	0.89	0.80
356 × 171 × 45	0.85	0.71

Web compression resistance

In compression, the effective width is the same as the stiff bearing length and depends on the thickness of the rafter flange, the weld, the thickness of the end plate and the dispersion through the column flange. The effective width is given in expression 6.11 of BS EN 1993-1-8 for a bolted connection.

For typical combinations of rafter and column, the reduction factor ω is shown in Table 7.2. An end plate of 20 mm and an 8 mm fillet weld has been assumed.

Table 7.2
Typical values of reduction factor ω, for web compression resistance

COLUMN SIZE	RAFTER SIZE	REDUCTION FACTOR ω
610 × 229 × 101	457 × 191 × 74	0.93
533 × 210 × 82	457 × 152 × 52	0.93
457 × 191 × 67	406 × 140 × 39	0.92
406 × 178 × 54	356 × 127 × 33	0.91

In most cases, the reduction in the unstiffened web resistance in compression is irrelevant, as the portal column is likely to need a compression stiffener. In the unlikely event of the column not needing a stiffener, a manual check can be carried out on the unstiffened resistance, using the reduction factors in Table 7.2 and a value of web compression resistance according to BS 5950-1.

DESIGN OF SECONDARY MEMBERS

8.1.1 Hot rolled sections

Gable posts, gable rafters and bracing can all be designed to BS EN 1993-1-1. Software may be used, or the resistance values taken from the Blue Book[2].

8.1.2 Cold formed sections

Some manufacturers make available performace data based on resistances determined in accordance with BS EN 1993-1-3[21]. The resistances may be calculated, based on the Standard, or determined by test.

Design resistances to lateral torsional buckling calculated in accordance with BS EN 1993-1-3 tend to be lower than those determined in accordance with BS 5950-5[22].

8.1.3 Purlins and side rails

Compared to previous practice, based on BS 5950-1, side rails and purlins with greater resistance are likely to be needed, for the following reasons:

- Design values of moments in the gravity loadcase are likely to be greater, as any positive wind loads on the roof will be additive to the permanent and imposed loads. This will be offset to some degree by the reduction in partial factors on the permanent and imposed loads.
- In the uplift combination, the actions will include the wind actions, factored by 1.5, rather than the 1.4 according to BS 5950-1.
- The pressures and suctions on the side rails will be factored by 1.5, rather than the 1.4 according to BS 5950.
- As noted above, cold formed purlins and rails have a lower lateral torsional buckling resistance, which will reduce their resistance when their compression flange is not restrained (uplift for purlins; net suction on side rails).

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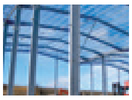	**Cover**	Sainsbury's Distribution Centre, Basingstoke
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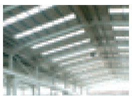
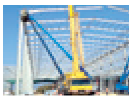
	**8**	Siemens Facility, Lincoln
	**21**	Tesco's Distribution Centre, Reading

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SCI
Silwood Park, Ascot, Berkshire. SL5 7QN UK
T: +44 (0)1344 636525
F: +44 (0)1344 636570
E: reception@steel-sci.com
www.steel-sci.com