# Overall stability of multi-span portal sheds at right-angles to the portal spans

*SCI's Senior Manager for Standards, Charles M King, explains the approach for design of long-span portal sheds.*

## 1. Introduction

As portal sheds become wider and longer, the overall stability of the buildings at right-angles to the span of the main portals becomes increasingly sensitive. This article considers the issue and describes approaches for design for stability in this direction. Figure 1 shows the mode of deformation considered.

One issue that made the 2000 revision of BS 59501 necessary was the need for rules to ensure the in-plane stability of portal frames. The methods available to ensure the stability of the buildings at right-angles to the main portal frames were not so clearly defined.

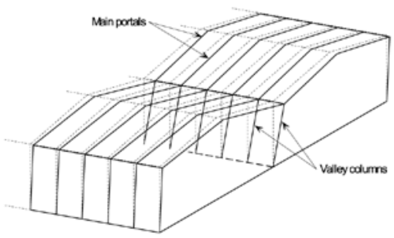



*Figure 1: Sway at right-angles to the span of the main portals.*

## 2. Stability systems

Every building must have a structural system that provides stability. In BS 59501:2000 clause 5.5 *Portal frames*, clause 5.5.1 states explicitly that frames "should be stabilised against sway out-of-plane" and refers to clause 2.4.2.5. The common structural arrangements are:

- Vertical bracing in the walls and in all of the planes of the valleys. This system gives the simplest method of resisting sway.
- Vertical bracing in the walls + plan bracing in the roof (wind bracing)
- Vertical bracing in the walls + portal frames in the plane of the valleys

There will be plan bracing in the roof to resist wind on the gable ends and to stabilise the portal rafters but it is used only in item 2 above to provide stability in the plane of the valley.

### 2.1 Vertical bracing in walls and the planes of the valleys

Figure 2 shows a shed with vertical bracing to provide stability in the walls and in the plane of the valley.

### 2.2 Vertical bracing in walls plus plan bracing in the roof

Figure 3 shows a shed with vertical bracing to provide stability in the walls but no vertical bracing in the plane of the valley. Stability in the plane of the valley is provided by the plan bracing in the roof connected to the vertical bracing in the walls. The plan bracing is commonly formed as a truss in which the chords are the rafters of one portal and the member along the top of the gable wall. It is often found that where the truss "depth" is only one bay, as shown in Figure 3, the truss is very flexible and may be insufficient to stabilise the valley columns. The "depth" of the plan bracing may need to be increased to two (or more) bays of the frame as shown in Figure 4. This plan bracing might develop major additional axial forces in the members forming the "chords". It is possible that the member along the top of the gable wall will need rather more demanding detailing than is commonly provided to avoid having many connections that might allow a significant accumulation of slip.

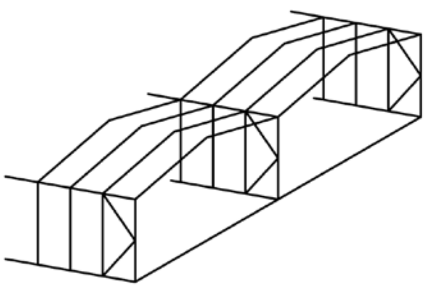


*Figure 2: Vertical bracing in the walls and in the planes of the valleys (plan bracing omitted for clarity).*

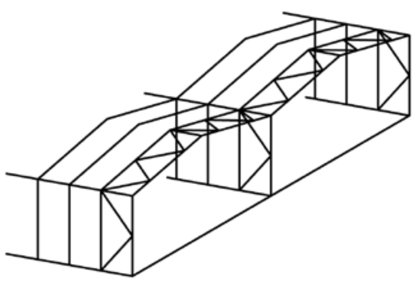


*Figure 3: Vertical bracing in walls plus plan bracing in the roof.*

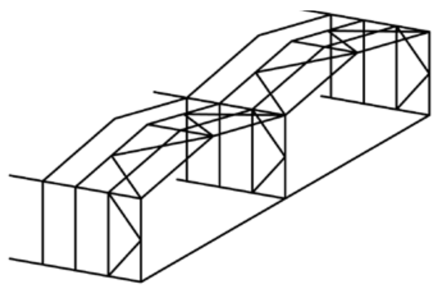


*Figure 4: Plan bracing in the roof with 2 bay "depth".*

### 2.3 Vertical bracing in walls + portals in the plane of the valleys

Portals, often called wind-portals, are used in the plane of the valley if diagonal bracing interferes with the use of the shed. This is shown in Figure 5.

### 3. Requirements of the stability system

The frames may have a valley column at every portal, or may be "hit and miss" or "hit-miss-miss" frames. Figure 6 shows a section through a typical shed showing the axial load $F_v$ in the valley column which includes the vertical reaction from any "miss" frames in the structure. Figure 7 shows the sway mode that the stability system is designed to resist.

Important features of the stability system are:

1 The stability system has to stabilise all of the columns
2 Stability systems often have relatively low sway stiffness, especially in sheds that are large and high.

The consequences of these two features are discussed below.

### 3.1 The stability system has to stabilise all of the columns

The stability system has to stabilise the total vertical load in the building, which is the sum of all the axial compression in all of the columns. Therefore, the design loads on the system in Load Combination 1 include the notional horizontal forces from all the columns that are stabilised by the system, including any office areas or other structures stabilised by the main shed. In other combinations, usually there are wind loads. There may also be horizontal forces from cranes and/or horizontal impact forces.

To calculate the internal forces and moments in the stability system, the analysis must be made with the columns supporting the total factored design load including the axial load from the analysis of the main portal frames as shown in Figures 6 and 7. Otherwise, the second-order effects in the plane of the system caused by these vertical loads will not be calculated.

### 3.2 Some stability systems have relatively low sway stiffness

The forces resisted by the stability system are generally small, so the members are small, resulting in a relatively low stiffness. Because of the low sway stiffness, the designer should expect to account for second-order effects in the plane of the valley frame. Often the design of the main portal frames also has to allow for second-order effects. It is important that where second-order effects arise, they are fully accounted for. This is required by BS 59501:2000 clause 2.4.2.5, Sway stiffness, to which designers are referred by clause 5.5.1. Therefore, if there are second-order effects in two directions, the effects in both directions must be considered.

Large buildings will often have large tonnages of steel in the stability systems. For these frames, it is probable that the most economical structures will be obtained by using second-order analysis software. Indeed, the stability systems will often be so flexible that they will be below the limit of $\lambda_{cr}$ for which BS 5950-1 allows simplified methods to be used. BS 59501 does not define a minimum

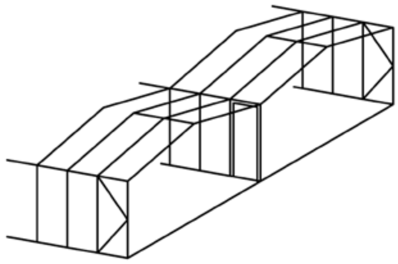


*Figure 5: Vertical bracing in walls + portals in the plane of the valleys (plan bracing omitted for clarity).*

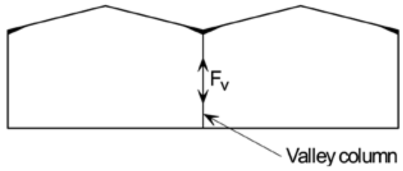



*Figure 6: Section showing vertical load, $F_v$, in valley column.*

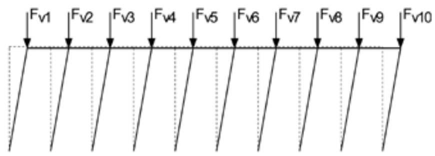
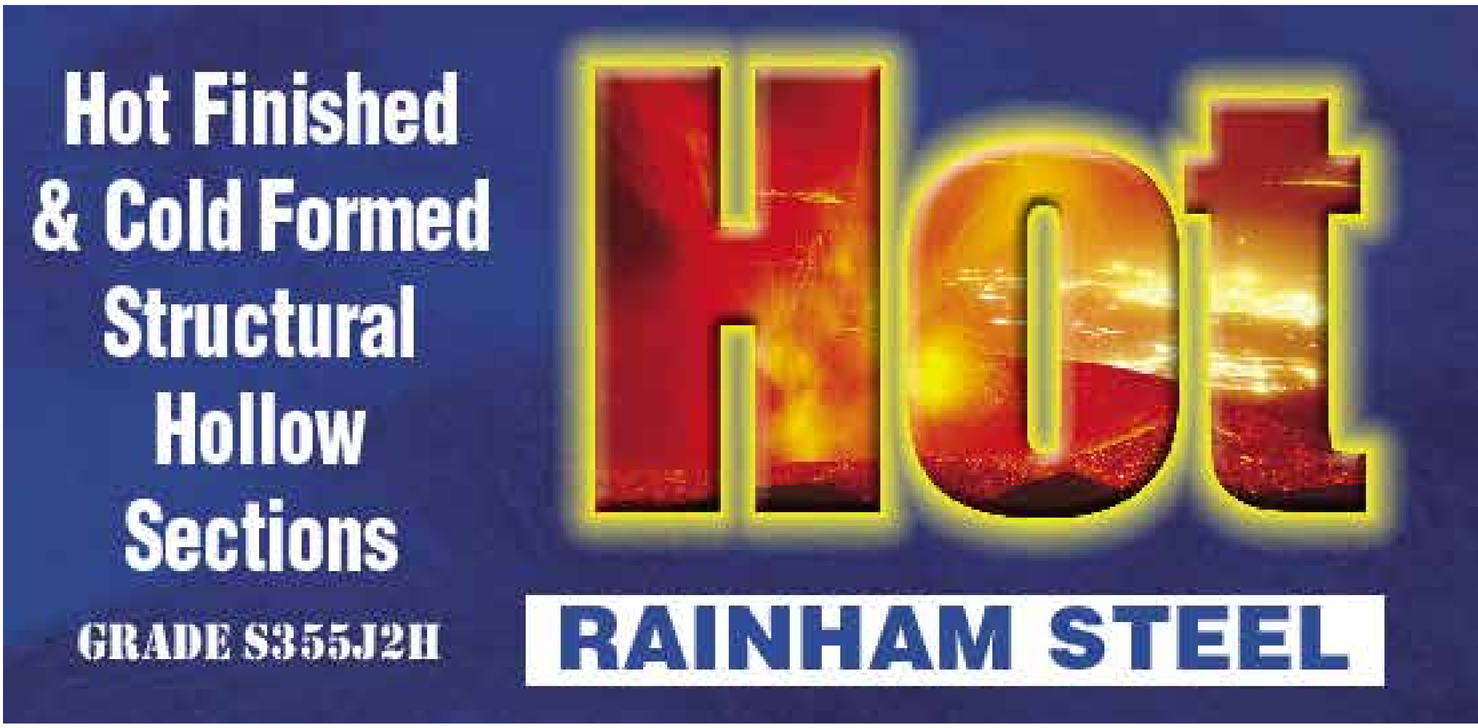



*Figure 7: Valley frame sway mode showing column loads $F_v$.*

value of $\lambda_{cr}$ for second-order analysis, but it is recommended that designers should be cautious where $\lambda_{cr} < 4$ and that frames should not have $\lambda_{cr} < 2$. This is because any connection slip or flexibility reduces $\lambda_{cr}$ below the value shown by frame design software, as also does any plasticity (especially in moment resisting frames), and the frame will collapse at $\lambda_{cr} = 1$ unless there is something else to hold it up.

If second-order software is not available, the designer needs to choose another way to allow for any second-order effects. Guidance on the use of the simplified methods in BS 59501 is given below.

## 4. Modelling and design

To understand the stability of a building, it needs to be considered initially as a 3D structure, even if it is modelled as several 2D frames. If there is more than one stability system, the horizontal loads should be shared between the systems in proportion to the sway stiffness of each system. The most common example of this is where there is vertical bracing at both ends of a line of columns. If the bracing is the same at each end, only half of the columns are stabilised by each bracing. It might be simplest to model the entire line of columns and all the bracing.

### 4.1 Vertical bracing in the walls and in each plane of valley-columns

The frame can be modelled as separate 2D frames in which the total vertical load in the plane of the frame must be stabilised by the bracing in that plane.

### 4.2 Vertical bracing and plan bracing

Where valley columns depend on plan bracing for stability, the flexibility of the complete system of vertical bracing plus plan bracing must be included in the calculation. If the vertical and horizontal bracing are analysed as separate models, the lateral deflection of the vertical bracing must be added to the lateral deflection of the plan bracing.

### 4.3 Portals in each plane of the valley columns

As in the case of vertical bracing in each valley plane, the structure may be modelled in 2D provided that the advice above about calculating second-order effects in the plane of the valley frame is followed (ie including all the loads in all the columns when calculating the internal forces and moments to allow for the destabilising effects).

It is recommended that the main portal frames are analysed without the valley portals because in the normal orientation, shown in Figure 8, the valley portal leg has insignificant effect on the stiffness of the main portal valley column in the plane of the main portal.

Where the main portal valley column and the valley portal leg are welded together, they will act as a compound member. In the plane of the valley portal, this has a significant effect on the column stiffness which may be worth calculating to obtain the maximum column stiffness.

Because many large sheds are very high, base stiffness is often very helpful in providing stability to the frame. Guidance on base stiffness is given in BS 59501:2000 clause 5.1.3.

Plastic design is not recommended for these stability frames because

1 in frames supporting major loads on the valley beam, such as "miss" frames, there is commonly significant sway after formation of the first plastic hinge
2 extensive plasticity reduces the sway stiffness
3 special care is needed to avoid forming hinges at the beam-column connections, which do not have adequate ductility.

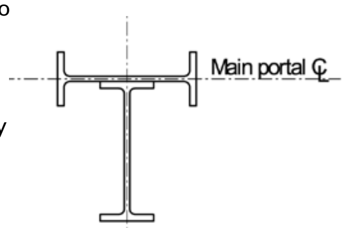



*Figure 8: Valley column and valley portal leg.*

## 5 Simplified methods to allow for second-order effects

### 5.1 Vertical bracing in the walls and in each plane of valley-columns

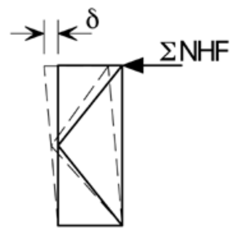



*Figure 9: Elevation of sway mode showing vertical bracing.*

The stability can be checked using BS 59501:2000 clause 2.4.2 as if each 2D frame is an ordinary braced frame. Figure 9 shows an elevation of the bracing system. In the figure, ΣNHF denotes the sum of the Notional Horizontal Forces from the columns in that plane that are stabilised by the bracing system shown. The figure also shows the deflection, δ, at the top of the columns arising from the Notional Horizontal Forces.

The procedure is as follows:

1 Calculate the total notional horizontal force from the total vertical load in the plane of the bracing (0.5% of the sum of the column loads).
2 Apply the total notional horizontal force to the bracing in the plane
3 Calculate $\lambda_{cr}$ as BS 59501:2000 clause 2.4.2.6. If $\lambda_{cr}$ is less than 4.0, the method should not be used.
4 If $\lambda_{cr}$ < 10, calculate $k_{amp}$ as BS 59501:2000 clause 2.4.2.7 and amplify the horizontal forces applied to the bracing.

The calculation may be done independently for each load combination for greatest economy.

### 5.2 Vertical bracing and plan bracing

Where valley columns depend on plan bracing for stability, the flexibility of the complete system of vertical bracing plus plan bracing must be included in the calculation. Figure 10 shows a perspective view of the bracing system in the sway mode. Figure 11 shows a plan view and Figure 12 shows an elevation. In these figures, ΣNHF denotes the sum of the Notional Horizontal Forces from the columns in that plane that are stabilised by the bracing system shown. The figures also show the deflections arising from the Notional Horizontal Forces

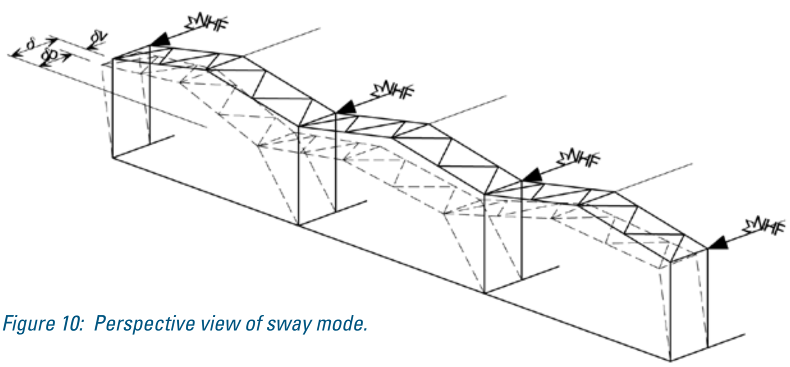



*Figure 10: Perspective view of sway mode.*

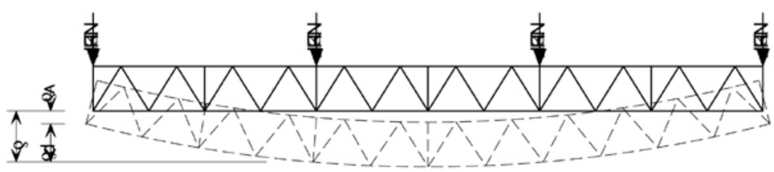


*Figure 11: Plan view of sway mode showing plan bracing.*

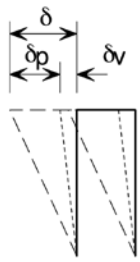



*Figure 12: Elevation of sway mode at valley columns.*

$\delta_V$ is the deflection at the top of the vertical bracing

$\delta_P$ is the maximum plan bracing deflection at the top of any column

$\delta$ (= $\delta_V + \delta_P$) is the maximum total deflection at the top of any column. If a 3D model is used, δ is found directly.

The procedure is as follows:

1 Calculate the total notional horizontal forces in each plane of columns from the total vertical loads in each plane of columns (0.5% of the sum of the column loads).
2 Apply the total notional horizontal force in each plane to the bracing system at each plane.
3 Calculate $\lambda_{cr}$ as BS 59501:2000 clause 2.4.2.6 using $\delta$ (= $\delta_V + \delta_P$). If $\lambda_{cr}$ is less than 4.0, the method should not be used.
4 If $\lambda_{cr}$ < 10, calculate kamp as BS 59501:2000 clause 2.4.2.7 and amplify the horizontal forces applied to the bracing.

The calculation may be done independently for each load combination for greatest economy.

### 5.3 Portals in each plane of the valley columns

Figure 13 shows a section through the shed showing the elevation of the valley frame and a potential sway failure in the plane of the valley columns

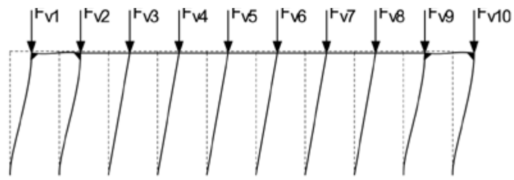



*Figure 13: Valley frame sway mode showing column loads $F_v$.*

Valley portals are single storey frames with moment resisting joints, for which BS 59501 clause 2.4.2.6 requires that reference is made to clause 5.5. This gives methods of calculating the resistance of frames. In addition to second-order analysis in clause 5.5.4.5, there are two simplified methods in which second-order effects are allowed for by the additional load factor $\lambda_r$ being greater than 1.0 for frames in which these effects are significant. These are:

1 the Sway-check method
2 the Amplified moments method

#### *5.3.1 The Sway-check method*

The Sway-check method is in clause 5.5.4.2. When applying this method, only the $h_i$/1000 approach should be used and the $L_b$/D formula approach should not be used because it cannot allow for the loads on all the valley columns. The notional horizontal forces applied should be calculated from the total load on all the valley columns stabilised by the portal. This ensures that the destabilising effects of all the column loads have been included in the calculation.

The procedure is as follows:

1 Check that the geometry of the portal is within the limits of clause 5.5.4.2.1. This is true for most common valley portals.
2 Calculate the total notional horizontal force from the total vertical load in the plane of the portal (0.5% of the sum of the column loads).
3 Apply the total notional horizontal force to the portal
4 Calculate the deflections $\delta_i$ and check that $\delta_i \leq h_i/1000$. If this condition is not fulfilled the method should not be used.
5 Check the frame for the gravity load case (= gravity loads plus Notional Horizontal Forces)
6 Calculate $\lambda_{sc}$ and $\lambda_r$ as 5.5.4.2.3 and check the portal for the horizontal load case (= gravity loads + horizontal loads, eg wind)

### *5.3.2 The Amplified Moments method*

The Amplified Moments method is in clause 5.5.4.4. In applying this method, the calculation of $\lambda_{cr}$ must be made using a model that includes the vertical loads on all the valley columns stabilised by the portal. This method requires that the value of $\lambda_{cr}$ includes the effect of any axial load in the valley beam. This will be very small if there is a valley column in each portal frame, but it might be significant in "hit & miss" frames or "hit-miss-miss" frames. This is because the vertical loads applied by the "miss" frames produce a horizontal shear at the bases and thus an axial force in the valley beam. If software is not available to calculate $\lambda_{cr}$, then it may be calculated using the formula:

$$\lambda_{cr} \geq 0.8\left(1-\frac{F_{R,ULS}}{F_{R,cr}}\right)\frac{h}{200\delta}$$

where $\frac{h}{200\delta}$ is as defined in BS 59501:2000 clause 2.4.2.6

$F_{R,ULS}$ is the axial compression at ULS in the valley beam in the relevant load case

$F_{R,cr}$ is the valley beam Euler buckling load in the plane of the portal in which L is taken as the span of the valley portal and I is $I_x$ if the web of the beam is vertical.

The procedure is as follows:

1 Calculate the total notional horizontal force from the total vertical load in the plane of the portal (0.5% of the sum of the column loads).
2 Apply the total notional horizontal force to the portal in the plane
3 Calculate $\lambda_{cr} \geq 0.8\left(1-\frac{F_{R,ULS}}{F_{R,cr}}\right)\frac{h}{200\delta}$ .

   If $\lambda_{cr} < 4.6$, the method should not be used.
4 Calculate $\lambda_r$ as BS 59501:2000 clause 5.5.4.4.
5 If using elastic design of the portal, follow clause 5.5.2 which requires that the output forces from the analysis are multiplied by $\lambda_r$. (Note that the same result is achieved by multiplying all the applied forces by $\lambda_r$, which might be more convenient as a design procedure.) The calculation may be done independently for each load combination for greatest economy.

## 6 Effective length of valley columns

Valley columns potentially fall into two categories:

1 Columns stabilised by an independent structural system
2 Columns forming the stabilising system

### 6.1 Columns stabilised by an independent structural system

Columns that are stabilised by an independent bracing system may be designed as "non-sway", as clause 5.1.4. This means that non-sway effective lengths may be used for these columns even if $\lambda_{cr}$ for the stabilising structure is less than 10. It is recommended that an effective length of 1.0 is taken

### 6.2 Columns forming the stabilising system

Where the stabilising system is a truss system as shown in Figures 10, 11, 12 and 13, the effective lengths are those appropriate to normal truss design.

Where the stabilising system is a portal system which is checked using the methods in BS 5950-1:2000 clause 5.5, there is no requirement to consider the in-plane stability of the individual members forming the portal because these methods allow for the in-plane buckling effects through the factor $\lambda_r$. Only out-of-plane member stability need be checked.

## 7 Compound columns in valley portals

The compound section created by welding the valley portal leg to the valley column of the main portal has high gross inertia in the plane of the valley frame, but it is susceptible to torsional-flexural buckling which is not covered by BS 5950-1:2000. To avoid the complications of design for torsional-flexural buckling, it is simplest to observe the common practice of considering the main portal and the valley portal as independent frames for the strength calculations. If the designer chooses to calculate the strength of the compound section, guidance on torsional and torsional-flexural buckling is available in references 1 and 2 below.
It is important to remember that the load from the main portal is not concentric with the centroid of the compound section.

## References


1 Design of cruciform sections using BS 59501:2000,
New Steel Construction, Vol 14, No 4, April 2006
2 Design of mono-symmetric and asymmetric sections in compression using BS 59501:2000
New Steel Construction, Vol 14, No 6, June 2006