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AD 090: Deflection limits for pitched roof portal frames (Amended)

The deflection limits in Table 5 of BS 5950: Part 1 specifically exclude portal frames. This is because the deflections of portal frames have no direct significance for the serviceability of the portal frame itself, whereas the implications for the serviceability of the cladding depend on the type of cladding and on other constructional details outside the scope of the code. Guidance may, however, assist designers in providing suitably serviceable steel portal frames to satisfy the basic criteria given in paragraph one of Clause 2.5.1 in BS 5950: Part 1. It should be noted that portal frames which give large deflections may also encounter problems with frame stability at the ultimate limit state, but this is covered separately in the code.

Types of cladding

Side cladding

A distinction must be drawn, first of all, between buildings with their sides clad with sheeting and those with walls comprising brick, block or stone masonry, or pre-cast concrete panels. For sheeted buildings it is also necessary to distinguish between steel (or other metal) sheeting, fibre-reinforced cladding panels, curtain walling, and other forms of glazing. For buildings with masonry cladding one must distinguish between masonry, which is supported against wind loads by the steelwork, free-standing masonry, and pre-cast concrete units. Again, for supported masonry one must distinguish between walls with or without damp-proof courses made of compressible material.

Roof cladding

The type of roof cladding is also significant and a distinction needs to be made between corrugated or profiled sheeting, felted metal decking or other felted constructions, tiled roofs, and concrete roof slabs.

Deflections of portal frames

Types of deflection

Under gravity loads, the principal deflections of a pitched roof portal frame are outward horizontal spread of the eaves, and downward vertical movement of the apex. Under side loads due to wind, the frame will sway so that both eaves deflect horizontally in the same direction. Positive and negative wind pressure on the roof will also modify the vertical deflections due to gravity loads.

Loads to be considered

Depending on the circumstances, it may be necessary to consider:

(a) dead load

(b) imposed load

(c) all gravity loads (i.e. dead and imposed)

(d) wind load

(e) wind load plus dead load

(f) 80% of (wind load plus imposed load)

(g) 80% of (wind plus imposed) plus 100% of dead load.

Only the imposed load and the wind load are included in the serviceability loads. The dead load need normally only be considered where its effects are not already compensated for by the initial pre-camber of the frame.

Effects of cladding

The cladding itself often has the effect of reducing deflection of the frame. It may do this in three different ways: composite action with the frame, “stressed-skin” diaphragm action, and independent structural action. As a result, deflection limits and deflection calculations are normally related to nominal deflections based on the behaviour of the bare steel frame, unless otherwise stated. The actual deflections are generally less than the nominal values.

Behaviour of sheeted buildings

Composite action

Although composite action of the sheeting undoubtedly reduces deflections in many cases, the effect is very variable, due to differences between types and profiles of sheeting, behaviour of laps, behaviour of fixings, flexibility of purlin cleats, etc. Data are not widely available, and in some cases the behaviour of the more recent systems with over-purlin lining, double skin sheeting, etc. is probably different.

It is normal to ignore this effect in the calculations, but the recommended limits are based on experience and make some allowance for the difference between normal and actual deflections.

Stressed-skin action

Designs taking account of stressed-skin diaphragm action in the strength and stability of the structure at the ultimate limit state should also take advantage of this behaviour in the calculation of deflections at the serviceability limit state. Where stressed action is not explicitly taken into account in the design, it will nevertheless be present in the behaviour of the structure. Neglecting it is apparently on the safe side, but there is an important exception to this, as follows. Where significant stressed skin diaphragm action develops due to the geometry of the building, but the fixings of the sheeting are not designed to cope with the resulting forces, the fixings will be overstrained, including localized hole elongation and tearing of the sheeting. To keep this within acceptable limits at the serviceability limit state, differential deflections between adjacent frames have to be limited, otherwise in service the sheets may leak at their fixings.

Gable ends

Sheeted gable ends are generally so stiff in their own plane that their in-plane deflections can be neglected. The result of this is that it is generally the difference in deflections between the gable end and the next frame which is critical – at least for uniform spacing of frames. However, this may be affected by the presence of bracing (see Fig. 1). This applies both to the horizontal deflection at the eaves and to the vertical deflection at the ridge. It should be noted that where sheeted internal division walls are constructed like gable ends and not separated from the building envelope, the same relative deflection criteria apply.

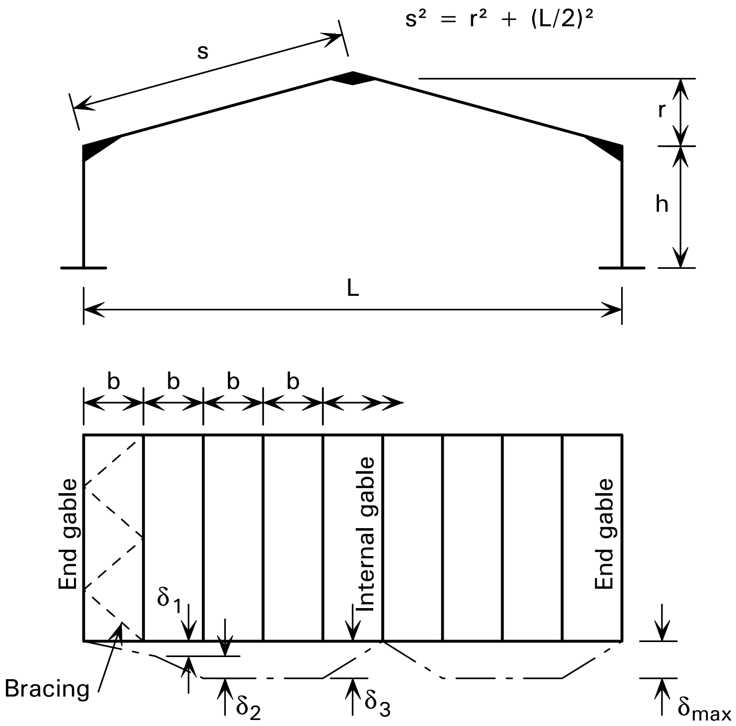

Fig. 1. *Portal frame definitions.* d_{max} = *maximum deflection.* d_1, d_2 *and* d_3 *are relative deflections.*

Behaviour of buildings with external walls

Free-standing side walls

When the side walls are designed free-standing, to resist the wind loads acting upon them independently of the frame, the only requirement is to ensure that, allowing also for construction tolerances, the horizontal deflections of the eaves are not such as to close the gap between the frame and the wall. The wall should either not contain a horizontal damp-proof course, or else have one composed of engineering bricks or other material which is capable of developing the necessary flexural resistance (see BS 5628: Part 3: Clause 18. 4.1).

Side walls supported by steel frames

When the side walls are designed on the assumption that they will be supported horizontally by the steel frame when resisting wind loads, then they should be detailed such that they can deflect with the frame, generally by using a compressible damp-proof course at the base of the walls as a hinge. The base hinge should also be taken into account when verifying the stability of the wall panels (see BS 5268: Part 3: Clauses 18.4.2 and 20.2.3).

Walls and frames sharing load

If a base hinge is not provided, but the side walls are nevertheless attached to the steel frame, their horizontal deflections will be equal and both the horizontal and the vertical loading will be shared between the frame and the walls according to their flexural stiffness. In such cases the walls should be designed in accordance with BS 5628 at both the ultimate and the serviceability limit states, for all the loading to which they are subject.

This procedure is only likely to be viable where either the steel frame is so rigid that it attracts virtually all the load, or the construction of the brick walls is of a cellular or diaphragm layout, capable of resisting relatively large horizontal forces. In both cases the design is outside the scope of these recommendations.

Analysis at the serviceability limit state

Serviceability loads

Although BS 5950 only defines a single level of serviceability loading, this is a simplification. In the case of the deflection of a floor beam leading to cracking of a plaster ceiling or other brittle finish, it is appropriate to consider the maximum value of the imposed load, or wind load, that is anticipated to occur within the design life of the building, even though its occurrence is rare.

For many other serviceability conditions it would be more logical to consider values of imposed and wind-loads that occur more frequently. However, for simplicity, only the maximum values are considered in BS 5950, with the limiting values adjusted accordingly.

Base fixity

Base fixity is covered in Clause 5.1.2.4 of BS 5950: Part 1, which requires use of the same value of base stiffness “for all calculations”. This clause is intended to apply to the ultimate limit state, and the requirement relates to consistency between the assumptions made for elastic frame analysis and those applied when checking frame or member stability and designing connections. When accurate values are not available, it permits the assumption of a base stiffness of 10% of the column stiffness for a nominal base, but not more than the column stiffness for a nominally rigid base.

It is a principle of limit state design that the verifications of the ultimate and serviceability limit states can be completely independent. At lower load levels, the base stiffness will generally be more than at ultimate, particularly for cases where it is as low as 10% at ultimate. Further, since BS 5950 was drafted, the requirements of the Health and Safety Executive in relation to erection have changed the normal detailing of nominal base connections from two to four holding-down bolts. Accordingly, it is recommended that a base stiffness of 20% of the column stiffness be adopted for nominally connected bases in analysis at the serviceability limit state. Similarly, for nominally rigid bases it is recommended that full fixity be adopted in analysis at the serviceability limit state, even though Clause 5.1.2.4 requires the adoption of partial fixity at the ultimate limit state.

Plastic analysis

Plastic analysis is commonly used in the design of portal frames for the verification of the ultimate limit state. Serviceability loading is less, typically 65-70% of ultimate, and the frame is assumed to remain elastic. Depending on the geometry, this is not necessarily the case under the rarely occurring maximum serviceability loads, but for many serviceability criterion the frequently occurring values are more relevant and the assumption is adequate. However, for such criteria as a portal frame hitting a free-standing masonry wall, or any other criteria related to damage to brittle components or finishes, any deformations due to the formation of plastic hinges under serviceability loading should also be allowed for. Such allowance should also be made where the elastic moments under serviceability loading exceed $1.5 \times M_p$.

Building with overhead crane gantries

Where a portal frame supports gantry girders for overhead travelling cranes, not only will deflections be produced in the frames by crane loads, but also deflections of the crane girders will be produced by wind and gravity loads on the building envelope. Although vertical deflections may also be produced, the most significant parameter is variation in the horizontal dimension across the crane track from one rail to the other. Standard overhead cranes can only tolerate a limited variation in this gauge dimension, whereas, with crane brackets added to an otherwise standard pitched roof portal frame, the relative horizontal deflections of the two crane girders will be relatively large.

It is therefore a question of deciding, on the merits of each individual case, whether it will be more cost-effective to have a special crane with greater gauge dimension tolerances, or whether to design a special stiffer form of frame. Horizontal ties at eaves level help reduce spread of the crane track. Base fixity is also beneficial, especially with stepped crane columns. The use of stepped columns, rather than cantilever brackets, to provide supports for the crane girders will also reduce deflections, provided that the upper part of the column is not too slender.

Crane manufacturers are often very reluctant to provide crane gantries with more than a very limited play in the gauge, and it is important to ascertain what is available at the earliest possible stage. In any case, it is advisable to use relatively rigid frames where cranes are carried, otherwise significant horizontal crane forces may be transferred to the cladding. Unless the cladding fixings have been designed accordingly, damage to cladding or fixings may result. It is also advisable to limit the differential lateral movements between the columns in adjacent frames, measured at crane rail level.

Visual appearance

Deflection limits based on visual appearance are highly subjective. As noted above, the values under frequently occurring loads are actually relevant, but equivalent values under maximum serviceability loads are used. The main criterion concerned is the verticality of columns, expressed as a limit on the lateral deflection at the eaves. However, for frames supporting false ceilings, limits on the vertical deflection at the ridge are also relevant.

Indicative values

Values for limiting deflections appropriate for pitched roof portal frames without cranes, or other significant loads supported from the frame, are given in Table 1 for a range of the more common side and roof cladding materials. In this table, side cladding comprising brickwork, hollow concrete blockwork or pre-cast concrete units is assumed to be seated on a damp-proof layer, and supported against wind by the steel frame. In using this table for horizontal deflections, the entries for both the side and roof cladding should be inspected, and the more onerous adopted. For the vertical deflection at the ridge, two criteria are given; both should be observed.

The values for differential deflection relative to adjacent frames apply particularly nearest each gable end of a building, and also to the frames adjacent to any internal gables or division walls attached to the external envelope. Note, however, that differential deflections may be reduced by roof bracing (see Fig. 1).

Amendment Note: This Advisory Note was amended to correct the labelling in Figure 1.

Note: See also AD092, AD097 and AD194

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Table 1. *Indicative deflection limits for pitched roof steel portal frames. The symbols are defined in Fig.1.*

(a) Horizontal deflection at eaves level, due to wind load or imposed roof load or 80% (wind and imposed)

Type of cladding		Absolute deflection	Differential deflection relative to adjacent frame
Side cladding	Profiled metal sheeting	$\leq \frac{h}{100}$	—
	Fibre reinforced sheeting	$\leq \frac{h}{150}$	—
	Brickwork	$\leq \frac{h}{300}$	$\leq \frac{(h^2 + b^2)^{1/2}}{660}$
	Hollow concrete blockwork	$\leq \frac{h}{200}$	$\leq \frac{(h^2 + b^2)^{1/2}}{500}$
	Pre-cast concrete units	$\leq \frac{h}{200}$	$\leq \frac{(h^2 + b^2)^{1/2}}{330}$
Roof cladding	Profiled metal sheeting	—	$\leq \frac{b}{200}$
	Fibre reinforced sheeting	—	$\leq \frac{b}{250}$
	Felted metal decking	—	$\leq \frac{b}{400}$

(b) Vertical deflection at ridge (for rafter slopes ▯ 3°), due to imposed roof load or wind load or 80% (imposed and wind)

Type of roof cladding		Differential deflection relative to adjacent frame
	Profiled metal sheeting	$\leq \frac{b}{100}$ and $\leq \frac{(b^2 + s^2)^{1/2}}{125}$
	Fibre reinforced sheeting	$\leq \frac{b}{100}$ and $\leq \frac{(b^2 + s^2)^{1/2}}{165}$
Felted metal decking	Supported on purlins	$\leq \frac{b}{100}$ and $\leq \frac{(b^2 + s^2)^{1/2}}{250}$
	Supported on rafter	$\leq \frac{b}{200}$ and $\leq \frac{(b^2 + s^2)^{1/2}}{250}$

Note: See also AD097