Unbraced Frames - Sway Stability

David Brown

Having looked at braced frames in the November/December 2002 issue of New Steel Construction, this article considers unbraced frames, often known as continuous frames. In fact, the process is exactly the same as for unbraced frames, in that \( \lambda_{cr} \), the measure of sway sensitivity, is calculated by applying the Notional Horizontal Forces (NHF) to the frame. The more interesting issues are the methods of dealing with sway-sensitive frames and the identification of the sway effects.

Sway stability - the reprise:

- calculate the NHF. Note that these are 0.5% of the factored vertical loads, and thus are loadcase dependant;
- apply the NHF to the otherwise unloaded frame, and note the lateral deflections;
- calculate \( \lambda_{cr} \) for each storey, from

\[
\lambda_{cr} = \frac{h}{200d}
\]

- the sway sensitivity of the whole frame is taken as the minimum value from any storey.

Designers are positively encouraged to include base stiffness in their frame models. Even the modest stiffness of a nominally pinned base will reduce the lateral deflections of a frame, thus improving the sway stability, but may even be enough to make the frame non-sway. Note that because sway stability is an Ultimate Limit State (ULS) consideration, the ULS rules for base stiffness apply (the column stiffness for a "rigid" base; 10% column stiffness for a base; zero stiffness for a "rigid" base). 10% column stiffness for a "rigid" base; 10% column stiffness for a "rigid" base; 10% column stiffness for a "rigid" base; 10% column stiffness for a "rigid" base.

Are real connections truly rigid?

We might observe that the model of the frame assumes truly rigid connections. The calculation of \( \lambda_{cr} \) and subsequent treatment of the frame is thus affected by this assumption. However, living in the real world, we know the connections will not be truly rigid - but is the difference significant? The proper course is to follow the guidance in Clause 2.1.2.1, which may be paraphrased as "whatever assumptions about the joints you made in design, the actual details must deliver those assumptions". In other words, if we have assumed rigid joints in the model, the actual joints should be rigid. The Green Book on moment connections (ref. 3) offers advice on making joints rigid, prescribing "Mode 3" behaviour in the joint. In this mode, the end plate on the beam and the flange of the column are both relatively thick (or stiffened), which minimises joint flexibility. In addition, there is a limitation on the shear in the column web panel - again minimising flexibility in the joint.

Frame types

All frames, including unbraced frames, may be categorised in BS 5950-1: 2000 as either "non-sway" or "sway-sensitive". If the frame has cladding or some other stiffening, and yet this beneficial stiffening has been ignored in the calculation of \( \lambda_{cr} \), then the frame is categorised as "non-sway" if \( \lambda_{cr} \geq 10 \). If \( \lambda_{cr} \) is less than 10 for this type of frame, and for all other frames, the frame is categorised as "sway-sensitive" and second order effects must be addressed. If \( \lambda_{cr} < 4 \) for either frame type the simple approaches in the Standard are not appropriate for such a floppy frame - a second order analysis must be carried out.

Non-sway unbraced frames

Design can proceed, using effective lengths factors taken from Annex E, which will be 1.0 or less. This is a manual, laborious process - it may be more cost effective to adopt effective length factors at 1.0.

Sway-sensitive unbraced frames

In Section 5 (Clause 5.6.4), there are two options for sway-sensitive unbraced frames:

- effective length method, or
- amplified sway method

Each method allows for second order effects - the first by down-rating the resistance of the columns, the second by increasing the load effects. Interestingly, note that in addition to amplifying the moments on the columns, the amplified sway method also amplifies the moments in the beams and connections. The effective length method only affects the column design, with a restriction that the beams must remain elastic.

Which method?

Annex E is simple, but would be horribly laborious for a frame with many columns. It is also a manual method. The amplified sway method is also simple, and rather more appropriate if one is using computer analysis and design packages. My personal recommendation is for the amplified sway method.

Amplified Sway Method

Simply, the sway effects must be amplified: the amplifier, \( k_{amp} \), depends on the type of frame, as did for braced frames.

<table>
<thead>
<tr>
<th>Frame Type</th>
<th>( k_{amp} )</th>
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<tbody>
<tr>
<td>Add additional stiffness present, but ignored in model.</td>
<td>( \frac{\lambda_{cr}}{1.15 \lambda_{cr} - 1.5} )</td>
</tr>
<tr>
<td>Add additional stiffness accounted for (or no additional stiffness, e.g. no cladding).</td>
<td>( \frac{\lambda_{cr}}{\lambda_{cr} - 1} )</td>
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The members and connections are designed for the increased forces and moments, making it ideal for computer applications. The single problem is to identify the sway effects.

Fig. 1. Frame with vertical load alone - but considerable sway.
Sway effects are, of course, caused by any lateral loads. However, frames will also sway under the action of vertical load, if either the frame or the loading, or both, is asymmetric. Check the frame in Figure 1, and observe the sway under vertical load only!

Clause 2.4.2.8 provides two methods to identify the sway effects, so that these can be amplified. Whilst entirely satisfactory, both approaches in the Standard involve manual intervention in subtracting, amplifying and re-combining loadcases, which may be inconvenient if you prefer a software-based approach. One straightforward procedure, which can be completed almost without leaving the keyboard, is as follows:

1. Provide horizontal supports at each floor and roof level to prevent sway. (These supports should not provide vertical support.)
2. Apply the loadcase to the frame (the ULS combination being considered);
3. Analyse the loaded frame, and note the horizontal reactions at these imaginary supports;
4. Remove the imaginary supports. Amplify the horizontal reactions by \((k_{\text{amp}} - 1)\) and apply them back to the loaded frame in the reverse direction.

This procedure will produce the (non-sway) plus (amplified sway) moments, and we can then proceed to the design of the members.

The frame in Fig. 2 may be used as a test example. If you choose to check your own results:

- the bases were nominally pinned, and were modelled for the calculation of \(\lambda_{cr}\);
- the bases were modelled as truly pinned for the ULS bending moments (else the bases and foundations would have to be designed for the resulting moment);
- the connections were modelled as fully rigid;
- all loads are ultimate;
- cladding is present, but ignored in the model.

\[\lambda_{cr} = 6.52, \quad k_{\text{amp}} = 1.087.\]

The (non-sway plus amplified sway) bending moment diagram (columns only, for clarity) is shown in Fig. 3. Note that the modelling of the nominally pinned bases is effective if the base stiffness is ignored; \(k_{\text{amp}}\) increases to 1.16.

**Important points to note:**
- This process should not be followed for the wind-moment method (ref. 4). The procedures in the wind-moment method allow for second-order effects
- Note the last two paragraphs of Clause 4.8.3.3.4. The equivalent uniform moment factor, \(m\), can only be applied to the non-sway moments. This will prove irritatingly difficult at the design stage, so the conservative approach of setting \(m = 1.0\) is recommended. In the test frame shown, the non-sway moments are such a small part of the total moments that the loss of economy is not significant.
- For folk with time to spare, Clause 5.6.4 (b) allows the effective lengths for the columns to be calculated from Annex E when following the amplified sway method. This will give effective length factors less than 1.0, but will involve considerable manual intervention. My personal recommendation is to leave the effective length factors as 1.0 (to ease the design process). This conservatism may offer some reassurance about the ‘less than rigid’ connections.
- Clauses 5.6.3 and 5.6.4 offer advice on load combinations and pattern loads.

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References