“**flop** *v.i.* **(-pp-).** *n.*, *& adv*, **1.** *v.i.* sway about heavily and loosely;”

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“No bracing. That’s what we want. And, no solid, fixed partitions.”

Sound familiar?

“Yes, that’s possible (but at a price)”

Introduction

With the advent of BS 5950-1: 2000 the treatment of second-order effects, particularly in relation to the susceptibility of buildings to ‘sway’, has come more to the fore. Many buildings, even conventionally braced structures, have been shown to be ‘sway sensitive’ when the clarified rules of BS 5950-1: 2000 are applied.

In addition, the layout of multi-storey structures has changed over recent years:-

- longer spans
- shallower construction (affecting connection depth)
- architectural preference for little or no bracing resulting in the greater use of moment resisting frames that are inherently not as stiff
- minimization of heavy masonry walls that can be assumed permanent
- lightweight/non-structural cladding

All of these have led to more reliance on the steel framing to resist lateral loads, with much less assistance from non-primary elements that have traditionally not been taken into account, and yet nevertheless significantly contributed to providing overall stability.

We have the technology!

With more reliance on the framing system alone to resist lateral loads, there is a tendency for buildings to be more ‘floppy’ (technical term) than in the past. In BS 5950-1: 2000 parlance, many structural frames are classified as “sway sensitive”. The technical distinction is when the elastic critical buckling load factor, λ_{cr}, is less than 10. In the vast majority of buildings second-order effects are dealt with simply and indeed may not affect the overall design at all in terms of resulting sections sizes[Ref. 1]. However, with the move to ever more slender structures as outlined in the Introduction, designers are increasingly facing designs in which λ_{cr} is quite low. In BS 5950-1: 2000 if λ_{cr} falls below 4, a rigorous second-order analysis is required.

Whilst software has helped designers understand the distinction between ' sway' and 'non-sway' frames and how to allow for second-order effects in the simplified ways given in BS 5950-1: 2000, many designers are unfamiliar with rigorous second-order analysis. Yet, there are many software packages that are capable of carrying out second-order analysis of 3D structures. To the newcomer to such sophistication it may come as a surprise that not all of the analysis approaches are the same – that could be the subject of an altogether different paper from this one. Simply be aware and wary. Given that the software available to you has such a facility and that it is sufficiently rigorous, then for 'floppy' structures an easy solution might simply be to press the appropriate button. We have the technology! However, such sophistication should be used only with a certain level of understanding and not as a replacement for such understanding.

On that basis and since the use of rigorous second-order analysis is not a panacea, the remainder of this article points the designer to some items that need consideration. Some of these are equally applicable when using the simplified methods of allowing for second-order effects given in BS 5950-1: 2000.

Where do I start?

Perhaps the first question to ask is do I need second-order analysis or perhaps more pertinently, why do I need second-order analysis? Some questions to pose to yourself:-

- am I content that the rest of the modelling of my structure is as rigorous as the analysis approach? We are after all modelling an idealised structure and not the real world
- are there better structural configurations that would be less 'floppy'?

- if it is ‘floppy’ in its final state, will there be difficulties during erection?
- can I (easily) solve all the other issues raised in this article?

In essence, take care that the ‘tail is not wagging the dog’!

That said, read on.

How ‘floppy’ is my building?

The measure of ‘floppiness’ is normally taken as the value of the elastic critical buckling load factor, λ_{cr}. BS 5950-1: 2000 provides a simple method for the determination of λ_{cr} (see Clause 2.4.2.6) based on the application of Notional Horizontal Forces (NHF) without any other loads being applied. For most structures this is surprisingly accurate even at quite low values of λ_{cr}[Ref. 2]. The code does provide some exceptions to the use of this method:-

- for single storey frames with rigid moment resisting joints i.e. portal frames
- for frames in which sloping members have moment resisting connections.

It should be noted that the simplified method of allowing for second-order effects in BS 5950-1: 2000, the Amplified Forces Method, can still be used for such structures providing an accurate value of λ_{cr} can be determined. Also, for a multi-storey building comprising several floors and a sloping roof with moment connections, it is suggested that the NHF method of determining λ_{cr} would still be appropriate, since the lowest value of λ_{cr} will tend to be governed by the ‘soft’ first storey. Second-order effects in the main structures can be based on this value of λ_{cr} but those in the portalised roof members would need to be estimated separately.

- if the value of λ_{cr} is ≥ 10, second-order effects are small enough to ignore
- if the value of λ_{cr} is < 10 but ≥ 4 either the simplified methods in BS 5950-1: 2000 or rigorous second-order analysis can be used
- below a value of λ_{cr} of 4 only rigorous second-order analysis is allowed.

A question often raised is whether there is a lower limit for λ_{cr} below which we should not stray? Definitively, the lowest limit is 1 since this would mean that the structure will not have attained its theoretical buckling load and will fail before reaching Ultimate Limit State. When λ_{cr} is less than 2, a rigorous second-order analysis will give a solution but this may be inaccurate. A full non-linear analysis would be more accurate. However, it must be remembered that λ_{cr} is a theoretical load factor and assumes the material and members have infinite strength, and have no imperfections e.g. members are perfectly straight. Obviously this is not true in reality and so its expected failure load factor (in buckling) is likely to be much less than 2. For a building with such a low value of λ_{cr}, it would be preferable to improve the stability of the structure or change the structural form. Hence, it is suggested that a value of 2 is the recommended lower limit for λ_{cr} (below which the designer should not stray).

What are the effects on design?

Given that you want to use rigorous second-order analysis, what are the effects on design?

Connection and base forces -

The first point to note is that for such an analysis the principle of superposition does not hold i.e. it is not correct to analyse the individual loadcases and then sum their individual effects (modified by appropriate load factors) in order to obtain the design forces at Ultimate Limit State. One of the first jobs, when designing a building in steel, is to obtain the connection and foundation forces.

For the former this presents no difficulty as the P-Delta analysis for combinations of loadcases will give the ULS forces for the design of the connections. Any moment connections will (correctly) include any additional moments from P-Delta effects. However, for foundation forces it is normal to provide the unfactored reactions – these are not available from the P-Delta analysis. The forces from an elastic analysis could be used especially for pinned bases where only vertical reactions and shears exist. The sum of these reactions will, of course, be exactly the same as those from the P-Delta analysis since in both cases the total reaction must equal the total applied load. The distribution of forces will vary due to the second-order effects and whether this is significant is a question of judgement. For bases that attract moment, the additional P-Delta moments will be missing from the results of an elastic analysis. This might be more significant but again is a question of judgement.

Connections and bases stiffness -

As well as the forces on connections and bases, their stiffness also has to be considered. Where the lateral load resistance of the structure (its sway resistance) is reliant upon moment resisting frames, the stiffness of the beam to column connections becomes very important. Two pieces of advice are relevant here:-

- BS 5950-1:2000 states in Clause 2.4.2.5, “Where moment resisting joints are used to provide sway stiffness, unless they provide full continuity of member stiffness, their flexibility should be taken into account in the analysis.”
- the SCI/BCSA Green Book on Moment Connections[Ref. 3] states in Section 2.5, “Connection rotational stiffness is inherent to the safety of this type of frame[1]. Flexibility in the connections adversely affects frame stability and serviceability”.

The above statement from the ‘Green Book’ refers to elastic analysis. The importance of connection stiffness increases significantly for ‘floppy’ structures where P-Delta effects are significant. There is some guidance in that publication on how to deal with this and the forthcoming Eurocode for steel construction provides a method for calculating connection stiffness. In practical terms, haunched connections and extended end plate connections might be expected to have sufficient stiffness in this context but flush end plates on shallower beams are unlikely to be adequately stiff to justify discounting their influence on the analysis.

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[1] “this type of frame” refers to moment resisting frames that provide lateral load resistance

Connection stiffness will not only affect the distribution of the forces around the frame but will also affect the deformations of the structure. With a λ_{cr} around 4 at ULS, the value of λ_{cr} will be around 6 at Serviceability Limit State. In this case the use of an elastic analysis is likely to be sufficiently accurate to determine the deformations of the structure. However, for lower values of λ_{cr} the second-order deformations might need to be taken into account. In this case a P-Delta analysis using SLS loads would be required.

A modicum of base stiffness is often used when assessing the sway stability of a structure (10% of the column stiffness) or when checking the deformations at Serviceability Limit State (20% of the column stiffness). BS 5950-1: 2000 allows these figures to be used without taking into account in the foundation design, the moments that such base fixity attracts. In the former case this helps to reduce the impact of second-order effects when using the simplified methods in the Code (reduces λ_{cr} and hence the amplification factor k_{amp}). Any base fixity included with a P-Delta analysis will attract moments to the bases. Since this is a ULS analysis these resulting moments must be catered for in the design of the base and foundations.

Imposed load reductions -

Since the principle of superposition does not hold for P-Delta analysis, the introduction of ‘imposed load reductions’ is very difficult and somewhat contrived to the extent that whilst not impossible it is impractical. For multi-storey buildings the column sizes tend to be axial force dominated and so the use of imposed load reductions is very important. For lower rise buildings that use moment frames as their primary resistance to lateral loads, the important column members tend to be moment dominated. Imposed load reductions are less important on two counts – the reductions are small for lower rise buildings and the moment component of the combined design forces must not be reduced. The inference is that applying P-Delta analysis to more conventional structures would be inefficient.

Uniform moment factors -

For members subject to combined buckling e.g. columns and unrestrained beams with biaxial moments and/or axial load, use is often made of the ‘uniform moment factors’ (‘m’ factors) to improve the efficiency of design for this condition. In Clause 4.8.3.3.4 of BS 5950-1: 2000, the ‘m’ factors (other than that for lateral torsional buckling) should be applied only to the ‘non-sway’ component of the moment. Whilst not impossible, the absence of superposition in P-Delta analysis makes identification of the non-sway moments impractical. The clearer and quicker solution is to always set these particular ‘m’ factors to 1.0. Unfortunately that can lead to some inefficiency in design.

Summary

The use of rigorous second-order analysis is not a panacea and whatever method of allowing for second-order effects is used, the impact of using inherently less stiff moment resisting frames to resist lateral loads needs to be considered:-

- the first requirement is understanding – the Steel Construction Institute runs an excellent course on “Frame Stability” that deals with second-order effects and on a number of occasions CSC have been called upon to provide half-day seminars on this topic.
- if λ_{cr} is > 4 be aware of the downsides of using rigorous second-order analysis as a substitute for the clearer and equally effective simple methods given in BS 5950-1: 2000
- if λ_{cr} is < 4 and hence BS 5950-1: 2000 forces you into using rigorous second-order analysis, consider the impact on constructability, cost effectiveness and behaviour at ultimate and working loads before reaching for that button.

‘Floppy’ frames can be designed efficiently and effectively to provide robust and safe buildings that do not “sway about heavily and loosely”.

References

Ref. 1 Rathbone, Alan J, Second-order effects – who needs them? The Structural Engineer, Vol. 80, No. 21, 5 November 2002, p19.

Ref. 2 Horne, M. R. An approximate method for calculating the elastic critical loads of multi-storey plane frames. The Structural Engineer, Vol. 53, No. 6, June 1975, p242.

Ref. 3 SCI/BCSA. Joints in Steel Construction. Moment Connections. SCI Publication P207, SCI 1995.