Limitations of current design procedures for steel members in space frames

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Abstract

The use of more advanced methods of analysis to design steel frames may lead to substantial material savings, in addition to simplicity in the design procedures. However, these benefits do not yet appear to be a powerful incentive for many structural engineers to abandon the familiar linear elastic analysis (LEA) based design procedures, even when dealing with steel structures that are not regular rectangular frames. This paper uses a heuristic example to demonstrate the serious limitations of the LEA based design procedures, whether alignment charts or system buckling analysis is used to determine the effective lengths of the compression members. It is shown that LEA based design procedures may lead to unsafe structures due to their inability to account for bending moment amplification in the rigidly connected tension members of a space frame. Furthermore, there is no allowance for the amplification of axial forces due to changes in the structure geometry, which is significant for the space frame example. Confidence in the system buckling analysis method for determining the effective lengths of compression members, based on linear buckling analysis, is shown to be potentially dangerous for certain types of frames. For the space frame example, the elastic buckling load is overestimated by over 200%. The conservatism inherent in the member capacity check equations specified in steel design standards is also illustrated.

Keywords

space, members, steel, procedures, design, frames, current, limitations

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LIMITATIONS OF CURRENT DESIGN PROCEDURES FOR STEEL MEMBERS IN SPACE FRAMES

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ABSTRACT

The use of more advanced methods of analysis to design steel frames may lead to substantial material savings, in addition to simplicity in the design procedures. However, these benefits do not yet appear to be a powerful incentive for many structural engineers to abandon the familiar linear elastic analysis (LEA) based design procedures, even when dealing with steel structures that are not regular rectangular frames. This paper uses a heuristic example to demonstrate the serious limitations of the LEA based design procedures, whether alignment charts or system buckling analysis is used to determine the effective lengths of the compression members. It is shown that LEA based design procedures may lead to unsafe structures due to their inability to account for bending moment amplification in the rigidly connected tension members of a space frame. Furthermore, there is no allowance for the amplification of axial forces due to changes in the structure geometry, which is significant for the space frame example. Confidence in the system buckling analysis method for determining the effective lengths of compression members, based on linear buckling analysis, is shown to be potentially dangerous for certain types of frames. For the space frame example, the elastic buckling load is overestimated by over 200%. The conservatism inherent in the member capacity check equations specified in steel design standards is also illustrated.

KEYWORDS

Effective Length, Frame Design, Member Capacity, Moment Amplification, Second-Order Analysis, Space Frame, Steel Structure, System Buckling Analysis, Tension Member

INTRODUCTION

Steel framed structures around the world have been traditionally designed on the basis of linear elastic analysis (LEA), whether using the classical methods such as the moment distribution method and the slope deflection method, or, after the advent of modern computers, the matrix analysis method. In order to account for the interdependency between the strength of a member and the strength of the frame, and to account for the bending moment amplification due to geometric nonlinearity (second-order effects), the effective length concept (ELC) was introduced in the 1960's (AISC 1963). As the ELC only approximates the second-order effects in a member, the moment amplification factor (MAF) for a member of *a given effective length and under a given axial force* tends to be over-conservative within its range of applicability. Nevertheless, and despite the perennial controversies regarding the ELC and the methods for determining the effective lengths of compression members under various conditions, the LEA based procedures continue to be widely used in the design of steel frames. Unless required by the specification, it takes more than material savings and simplicity in the design procedures (or, for that matter, speed in execution) for the more advanced methods of analysis and design to replace the traditional LEA based design procedures as the primary tool of design engineers, even for non-rectangular steel frames.

Ironically, the availability of high-speed personal computers may reinforce rather than diminish the natural inertia of the structural engineering profession to move to the second-order analysis based design procedures. In the past and until now, methods that make use of the alignment charts and the classification of the frame into either a braced or a sway frame have been employed to determine the effective length of a compression member. Such methods are known to be inaccurate in many cases, and are impractical to apply in certain cases. Many authors believe, with some merits, that the inaccuracy and awkwardness of the alignment chart based methods are circumvented via the use of the system buckling analysis (SBA) method. Davies (1996) argues that the SBA method, in which the effective length of a compression member is calculated from the linear buckling analysis result of the whole structure, is the most accurate one. Unlike in the past, the speed and the memory capacity of today's personal computers enable the linear buckling analysis of a large steel frame composed of thousands of members to be completed in minutes. The SBA method for determining the effective lengths of compression members have thus been gaining popularity in recent years, although the implicit assumption that all members of a frame buckle simultaneously dictates that the method overestimates the effective lengths of all but the most critical compression members. In addition, the importance of the linear buckling load factor of a frame is accentuated in recent design codes such as Eurocode 3 (Davies 1996).

This paper aims to demonstrate that the LEA based design procedures may lead to an unsafe design, especially when the frame is not a regular rectangular frame. It is hoped that this paper will help promote the use of more advanced methods of analysis in practical design of significant steel frames. References are made to AS 4100 (SA 1998), LRFD Specification (AISC 2000a) and Eurocode 3 (CEN 1993). However, the focus is on AS 4100 as it is more prescriptive than the other two in certain respects, and its commentary appears to be more comprehensive.

This paper also illustrates the conservatism inherent in the interaction equations specified in steel design standards for checking the capacity of a member under combined compression and bending. It is explained that this conservatism cannot be avoided by using a second-order elastic analysis.

LINEAR ELASTIC ANALYSIS BASED DESIGN PROCEDURES

Figure 1 depicts a simply supported space frame subjected to a concentrated gravity load at the apex. All the members are assumed to be rigidly connected to each other. This example has been chosen to illustrate the limitations of the moment amplification factor (MAF) approach as well as the system buckling analysis (SBA) method. The load value shown in Fig. 1 is the factored design load. In the linear buckling analysis, each member is modeled with two cubic beam elements (Teh 2001).

Figure 2 shows the linear elastic bending moment distribution in the frame. It can be seen that the bending moment in each bottom chord is uniform and is 2053 kNmm. The axial force in each bottom chord is 192.4 kN in tension. According to AS 4100 (SA 1998) and LRFD Specification (AISC 2000a), the linear elastic bending moment of a braced member needs not be amplified to allow for second-order effects unless the member is in compression. Therefore, if the bottom chords are considered to be braced members as there are no relative transverse displacements between their two ends, the design bending moment is equal to the linear bending moment (2053 kNmm).

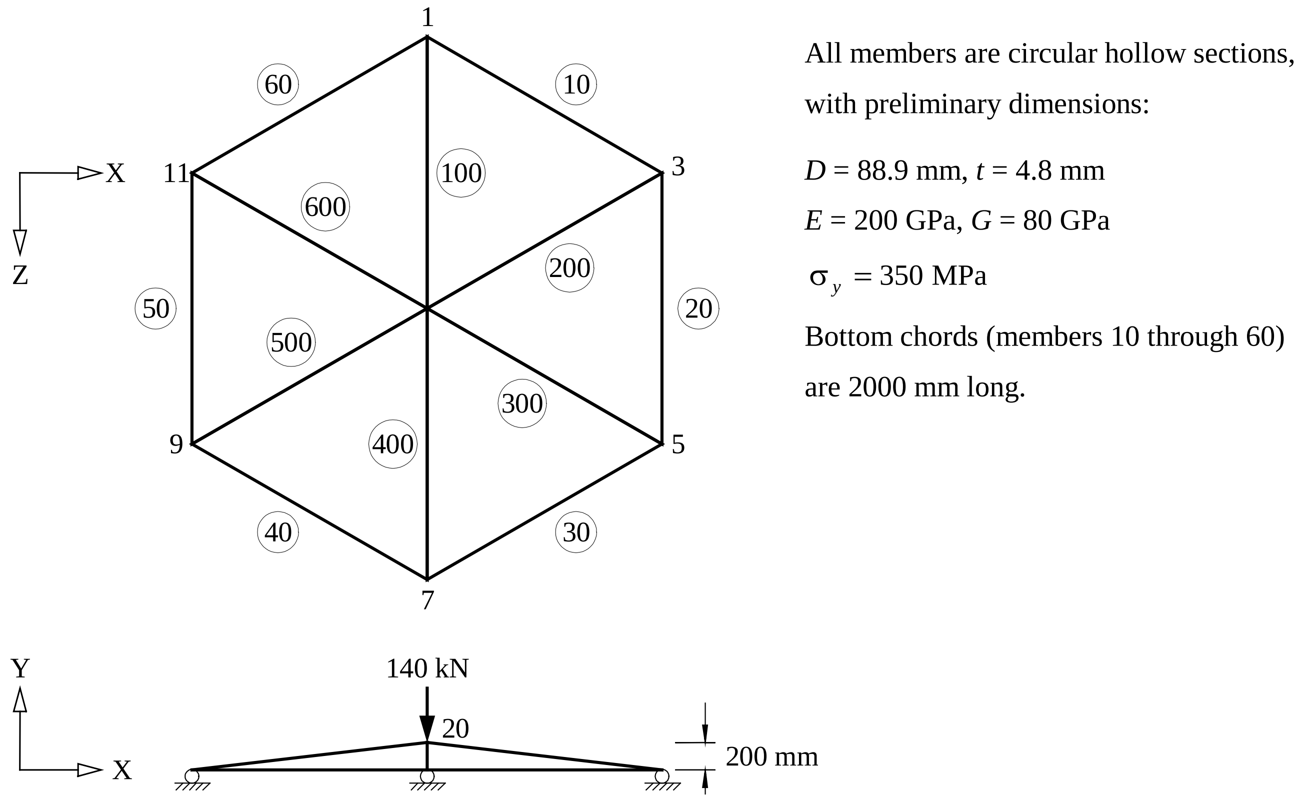


Figure 1: Space frame example

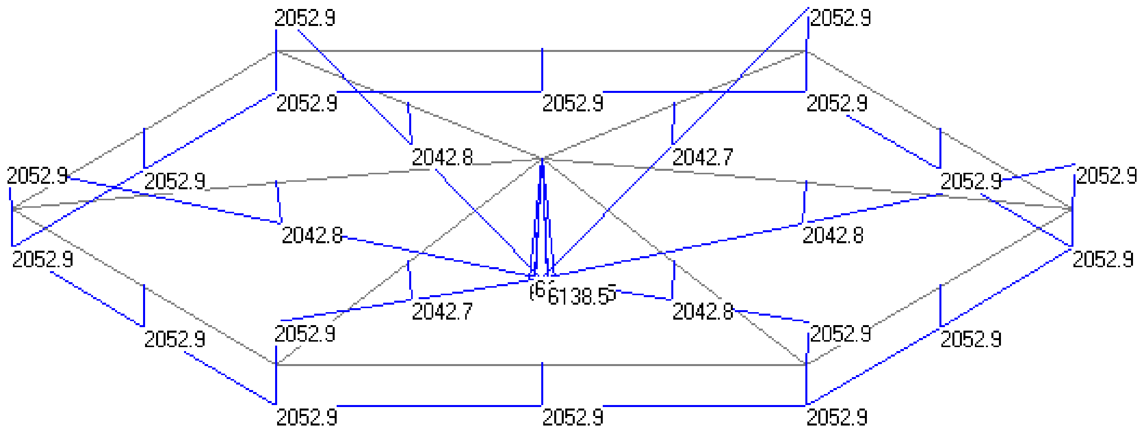


Figure 2: Linear elastic bending moment distribution under the factored design load

The commentary to AS 4100 (SA 1999) recognizes that the assumption of an amplification factor of unity for a braced member in tension is only an approximation, as the second-order effects in an adjacent compression member increase the former's bending moment. The commentary suggests that the largest amplification factor determined for all members of the frame be applied to every member. This suggestion appears to be particularly logical for the space frame having the topology depicted in Fig. 1. The moment amplification factor B of the sloping members may be computed in the manner specified in Clause 4.4.2.3(b) of AS 4100 (SA 1998) for a "sway" member in a non-rectangular frame

$$B = \frac{1}{1 - 1/\lambda_L} \tag{1}$$

in which λ_L is the linear buckling load factor of the frame. This is essentially the system buckling analysis (SBA) method - in this case it is used to determine a uniform moment amplification factor for all members. Such a method is believed by many to be conservative, which is not true for this example. As the linear buckling load factor of the frame is 4.48, the *B* factor is 1.29. Since this value is less than 1.4 (1.33 as per Eurocode 3), a second-order elastic analysis is not required. Applying this *B* factor to the bottom chords, the design bending moment becomes 2648 kNmm.

Figure 3 shows the second-order elastic bending moment distribution in the frame under the factored design load. It can be seen that the design bending moment in each bottom chord is 3522 kNmm, which is 71.5% and 33% higher than the values determined previously using the linear elastic analysis based procedures. These results may be surprising to many as the braced members are in tension, and the moment amplification factor computed "conservatively" from Equation (1) is 1.29.

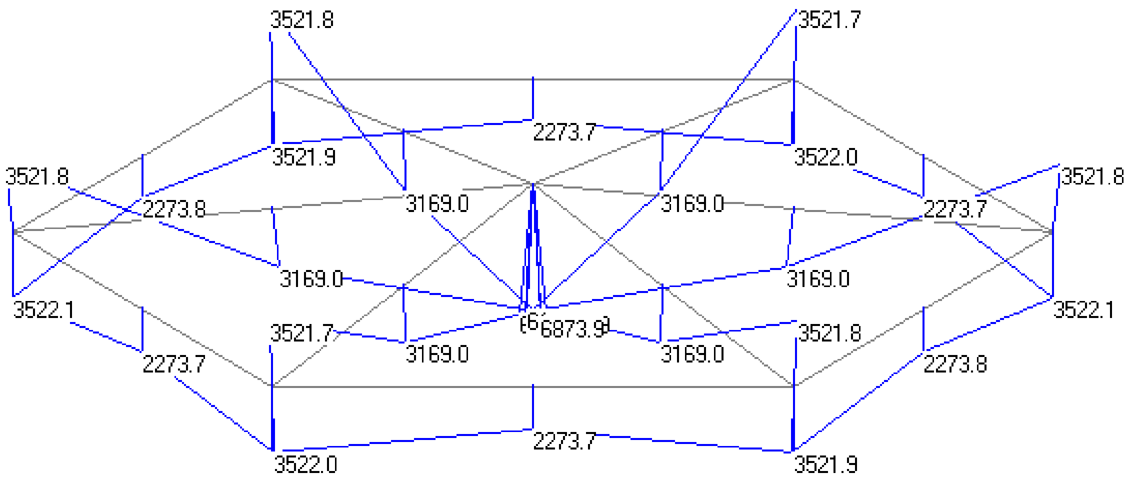
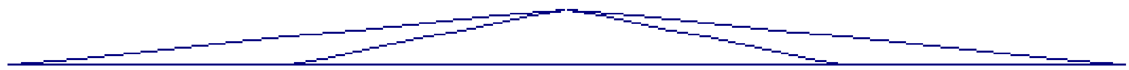
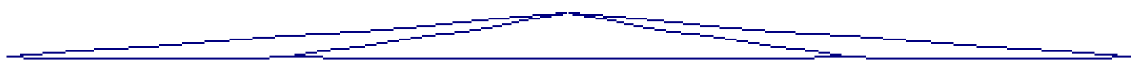


Figure 3: Second-order elastic bending moment distribution under the factored design load

Figure 4 illustrates the second-order elastic deformations of the space frame under the factored design load. Note that the projected plane is *Y-Z*. The apex deflects vertically by 40.6 mm.



Before deformation



After deformation

Figure 4: Second-order elastic deformations under the factored design load

The primary source of the unsafe error in determining the amplification factor *B* using Equation (1) is the neglect of changes in the structure geometry under loading, illustrated in Fig. 4, in the linear buckling analysis. Figure 5 shows the load-deflection graph of the apex as obtained from a second-order elastic analysis. The limit point takes place at a load factor of 1.48, which is just one third of the linear buckling load factor of 4.48. Such a huge overestimation of the elastic buckling load by the linear buckling analysis may be uncommon in practice, but is nevertheless real even though the second-order elastic deformations under the factored design load are "modest".

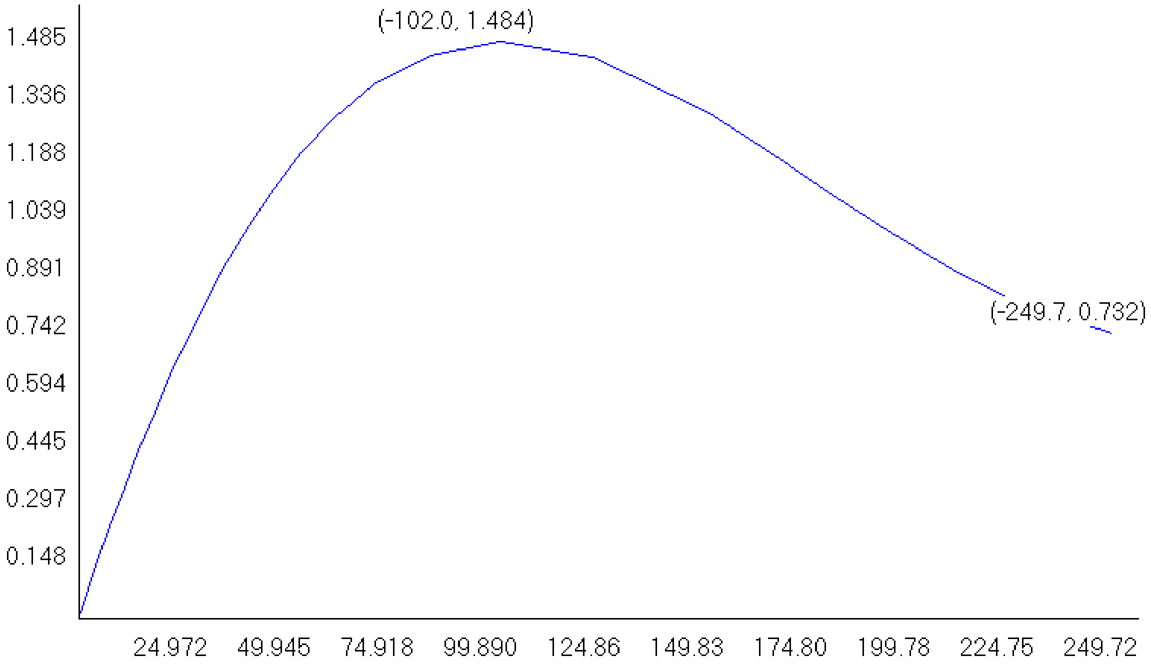


Figure 5: Second-order elastic load-deflection graph

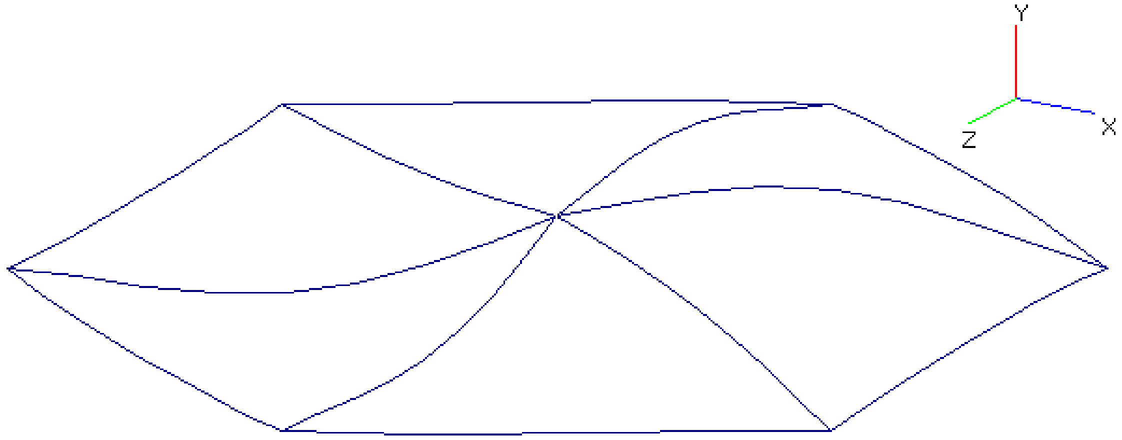
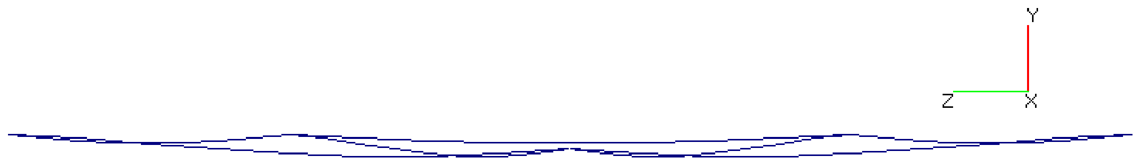


Figure 6: Linear buckling mode

Deformation at the load factor of 1.48



Deformation at the termination of analysis (see Fig. 5)

Figure 7: Buckling mode predicted by second-order elastic analysis

The linear buckling analysis not only overestimates the elastic buckling load by over 200%, but also fails to capture the snap-through buckling mode. Figure 6 depicts the asymmetric buckling mode predicted by the linear buckling analysis, while Fig. 7 depicts the snap-through buckling mode. It should be noted that the branch-switching technique (Teh & Clarke 1999) is irrelevant here.

The results of this example indicate that a linear buckling analysis of a pitched-roof portal frame may overestimate the elastic buckling load, especially as the rafters are often more critical than the columns. Also, the linear buckling analysis may predict an asymmetric buckling of the rafters similar to that of the sloping members shown in Fig. 6, while the second-order elastic buckling mode is one of snap-through. This possibility supports the requirement of Clause 5.2.6.1(4) of Eurocode 3 (CEN 1993) that a non-triangulated pitched roof be checked for snap-through buckling, especially if the columns tend to move apart under the applied loads. A second-order analysis (elastic or plastic) is suggested by the present results. Using a linear buckling analysis with the Sturm sequence property check, the snap-through buckling mode of the space frame is detected as the fourth mode at a load factor of 6.52, more than four times the correct elastic buckling load factor.

In addition to underestimating the design bending moment of the bottom chords, LEA based design procedures also underestimate the design axial forces by 18%. To the author's knowledge, no allowance is made in any steel design standard for the amplification of axial forces due to changes in the structure geometry when linear elastic analysis is used to compute the member forces.

For the sloping members of the space frame, which are in compression, the SBA method in conjunction with Equation (1) overestimates the moment amplification factor B by 15%. However, this overestimation does not necessarily lead to a conservative design as the design axial force and the effective length factor are significantly underestimated. Besides, the significant underestimation of the design forces of the bottom chords means that the space frame as a whole is likely to be inadequately designed to carry the factored design load, irrespective of the sloping members.

SECOND-ORDER ELASTIC ANALYSIS BASED DESIGN PROCEDURES

As mentioned in the preceding section, the LEA based design procedures not only underestimate the design bending moment of the bottom chords and the design axial forces (tension and compression), but also underestimate the effective length factor to be used in checking the member capacity. The procedures thus underestimate the required capacity and tend to overestimate the available capacity. In the context of current design standards such as AS 4100 (SA 1998), LRFD Specification (AISC 2000a) and Eurocode 3 (CEN 1993), the use of second-order elastic analysis *largely* solves the problem of determining the member design forces. However, this is not the case with the effective length factor to be used in checking the member capacity.

Some researchers argue that a second-order elastic analysis accounts for the member stability effects in the analysis itself, and therefore only requires a section capacity check for the formation of the first plastic hinge (no member capacity check is required). It is also argued that such an approach is conservative due to the neglect of the inelastic reserve strength of a redundant frame. These arguments are probably valid for most rectangular frames, especially where yielding is not extensively distributed in the steel members. However, the compression members of a reticulated space frame are more likely to fail by inelastic instability than by forming a plastic hinge. The first plastic hinge approach clearly does not account for this possibility. For the present example, in which residual stresses are assumed to be non-existent, it was found through a second-order plastic-zone analysis that the frame ultimate capacity is reached at the load factor of 1.06. The maximum percentage of cross-sectional yielding at the ultimate load is about 55% only, distributed over a significant length of the sloping members. Therefore, it is not always feasible to neglect the member capacity check even if the proper geometric imperfections are modelled explicitly in the second-order elastic analysis.

The notional horizontal load approach used in conjunction with the second-order elastic analysis provides a satisfactory solution to the stability design problems of sway rectangular frames (Clarke & Bridge 1995). In this approach, the nominal member capacity under compression is computed based on the actual unsupported length, and there is thus no issue concerning the effective length factor. Unfortunately, the notional horizontal load approach is not applicable to many space frames such as the present example. Likewise, the recommendations of White & Hajjar (1997) regarding steel frames that can be designed without calculating the effective length factors may not be applicable to space frames such as the present example.

AS 4100 (SA 1998) assumes an effective length factor of unity in checking the member capacity under combined compression and bending, provided the "actual" effective length factor is used to check the member capacity under compression alone. The commentary to the standard (SA 1999) argues that the effects of end restraints on the member stability have already been taken into account in determining the design bending moment, either in amplifying the linear elastic bending moment, or in carrying out a second-order analysis. However, an extensive study on sway rectangular frames (without the use of the notional horizontal load approach) has shown that the use of an effective length factor of unity in checking the member capacity under combined compression and bending may lead to unsafe errors (AISC 2000b). Therefore, the "actual" effective length factor should ideally be used in checking the capacity of the sloping members, as required by LRFD Specification (AISC 2000a).

There are at least two possible approaches that can be used for determining the effective length factor of the sloping members of the present space frame: using a linear buckling analysis (the SBA method) and using a second-order elastic analysis. The second approach requires that the nonlinear analysis be performed until the load limit point, usually well beyond the factored design load. The effective length factor computed using the SBA method is 0.80, while that computed from the axial forces existing at the second-order elastic limit point is 1.03. It should be noted that the axial forces existing at the elastic limit point, which are about 513 kN for all members, cannot be computed by multiplying the linear elastic values with the elastic limit point load factor of 1.48.

The effective length factor of 0.80 obtained through the SBA method is apparently incorrect as it implies that the sloping members are braced members with some end-restraints. The effective length factor of 1.03 appears to be a more appropriate value, and happens to be close to unity. However, as will be seen next, this value leads to a conservative design due mainly to the conservatism of the linear interaction equation used to check the member capacity under combined compression and bending, and partly to the inability of the second-order elastic analysis to redistribute the bending moments as yielding progresses in the frame. The use of an inelastic effective length factor may improve the situation, but its calculation is far from trivial.

The design bending moment of the sloping members is overestimated by 7% (relative to the plastic-zone analysis result). Using the effective length factor of 1.03 and the design forces determined from the second-order elastic analysis, and assuming a capacity factor of unity, the member capacity check using Clause 8.4.2.2 of AS 4100 (SA 1998) suggests that the moment capacity of the sloping members falls short of the required capacity by 40%. Although the member capacity check accounts for an assumed geometric imperfection, the result is obviously incorrect as the ultimate load factor of the space frame is 1.06 (as found through the second-order plastic-zone analysis of the geometrically perfect model).

The conservatism of the member capacity check described above also applies to other steel design standards such as LRFD Specification (AISC 2000a) and Eurocode 3 (CEN 1993), although the degree of conservatism may vary from code to code. Clause 8.4.2.2 of AS 4100 (SA 1998) specifies a nonlinear interaction equation for square hollow sections, which is less conservative than the linear interaction equation specified for circular hollow sections. However, the application of the nonlinear interaction equation to the sloping members of the space frame still leads to conservatism.

CONCLUDING REMARKS

This paper presents a heuristic example involving a space frame to demonstrate the serious limitations of linear elastic analysis based design procedures. It is shown that, contrary to the specification-sanctioned assumption of a moment amplification factor of unity for a tensioned braced member, in certain types of frames the rigidly connected tension members may be subject to a very large bending moment amplification due to the second-order effects transmitted from the adjacent compression members. Even when the largest amplification factor determined for all compression members of the frame is applied to the tension members, the system buckling analysis method may underestimate the bending moment amplification due to the overestimation of the elastic buckling load using linear buckling analysis. Furthermore, the potentially significant amplification of the axial forces due to changes in the structure geometry is neglected in linear elastic analysis based design procedures. For non-rectangular frames subject to significant pre-buckling deformations, second-order analysis should be used to determine the member design forces.

The use of second-order elastic analysis in determining the member design forces and the effective lengths of compression members represents a significant improvement over the use of linear elastic analysis and linear buckling analysis. However, second-order elastic analysis based design procedures often lead to conservative designs due to the conservatism inherent in the interaction equations for checking the member capacity under combined compression and bending, and to the inability of the analysis to redistribute the bending moments of yielded members.

The only way to completely avoid the requirement for member capacity checks is to use the advanced analysis/design method (SA 1998), which is basically a second-order inelastic analysis method. However, it is not suggested that the use of second-order inelastic analysis in the design of steel frames does not encounter difficulties. The appropriate modelling of geometric imperfections in a nonlinear frame analysis/design is largely an unsettled area except for sway rectangular frames.

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