Practical application of the "General Method" of EN 1993-1-1

In the second article of this series, Dr József Szalai of ConSteel Solutions demonstrates practical examples where the "General Method" of EN 1993-1-1 shows advantages compared to the conventional approaches.

The key component: general elastic stability analysis

When verifying the stability of a structural component the critical issue is always the correct assessment of the possible forms of buckling. In conventional design procedures the buckling forms are taken into account by calculating the elastic critical resistances (N_{cr} and M_{cr}) for the associated buckling shapes. Thus the determination of the appropriate elastic critical force is the most significant step of the design method (Step 3 in the previous article) but at the same time has the potential to be the most complicated step. It is very important to recognise that the calculation of an elastic critical force always implies an assessment of the buckling shape – the resistance and the buckling shape are linked.

Designers are used to the concept of using an effective length (to determine a buckling length) though this can be a significant simplification of the problem, particularly if the system of applied forces and the restraint system is complex, because the buckling mode in these circumstances is not easily identified or assessed. In the general method the elastic critical load amplifiers are calculated considering the complex system of forces based on more realistic compound buckling shapes. Thus the general method provides solutions for a number of design irregularities – irregular members such as tapered or haunched members, irregular buckling shapes due to special (for example eccentric) supports or restraints on the member – by the calculation of the compound buckling shape of a realistic structural model.

The general method requires that designers consider the buckling behaviour, which in turn has the advantage of placing the designer more directly in control of the frame behaviour, which may well lead to more appropriate solutions.

Although elastic critical forces calculated using the buckling length are suited for simple hand calculation, the determination of the elastic critical load amplifiers required for the general method are usually much more complicated and the use of some appropriate numerical method or software is necessary. The efficient use of the general method requires the use of software in which the all the necessary compound buckling modes – lateral, torsional, lateral-torsional buckling, buckling about an eccentric restraint axis etc. – can be calculated properly. Currently, such software products are quite rare since the general analysis of buckling which includes torsional modes requires particular analysis tools. One solution is to use shell finite elements when creating the global structural model. This can make modelling the building very complicated and the analysis results can be difficult to handle, meaning this option is practically never used by engineers.

Another solution is the use of special beam finite elements which are able to accommodate torsional buckling modes. There are a number of published finite elements fulfilling these requirements, including a 7 degree-of-freedom finite element which includes the effect of torsion and warping of the cross section. This element is implemented into the structural design software ConSteel which has been used in this article to calculate the design examples.

Design examples

Two simple examples are presented to examine the consequences of the two main simplifications of the conventional stability design method: the isolation of the structural member from the surrounding structure and the separation of the flexural and lateral-torsional buckling modes.

Example 1: Influence of buckling mode separation

The first example, shown in Figure 1, is a simple column fixed at the bottom and pinned at the top subjected to compression and bending. The column has two intermediate (eccentric) supports to one of the flanges.

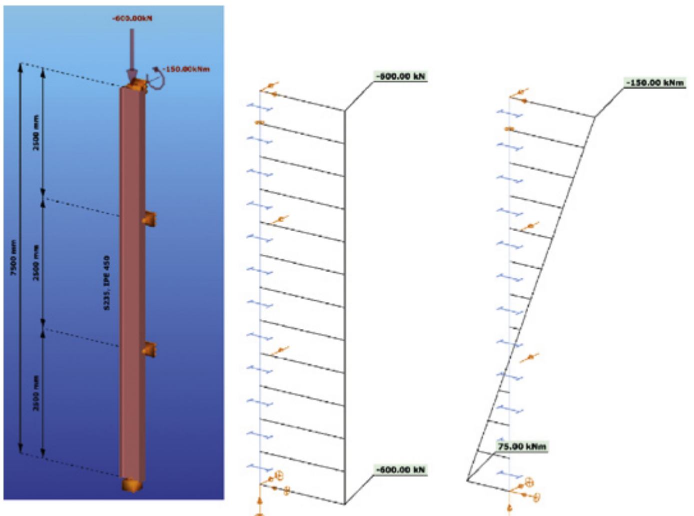


Figure 1. Geometry, loading and internal forces of Example 1

The primary problem with this simple column is that the eccentric intermediate supports generate an irregular buckling situation where neither of the well-known pure buckling forms (pure lateral or pure torsional buckling for compression or pure lateral-torsional buckling for bending) can be separated. EN 1993-1-1 provides some simplified design formulas for beams supported on the compression flange (Section 6.3.2.4) but only for those subjected to pure bending. For pure compression there are no applicable rules for columns with intermediate restraints to only one flange. Moreover these formulas cannot be applied in any procedure that includes interaction of the pure buckling forms. If the conventional method is to be applied considering the pure buckling forms, the problem should be somehow modified in order to be able to determine the pure elastic critical forces. Two usual simplifications are considered: (1) to assume concentric intermediate lateral supports and (2) to assume concentric intermediate lateral and torsional supports. In Table 1 (over page) the actual configuration is illustrated by the elastic critical forces and associated buckling shapes for →

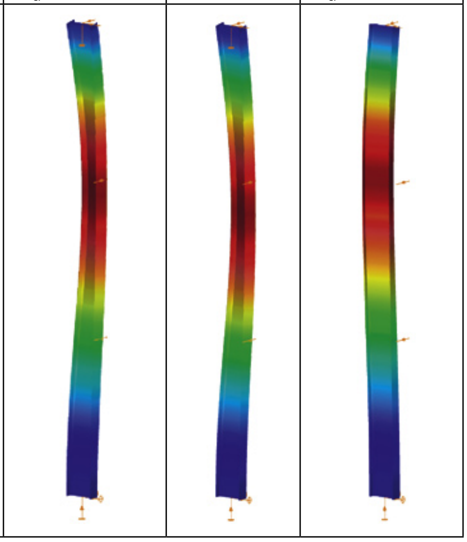
supports	actual eccentric support conditions		
loading	$N + M$	pure N	pure M
elastic critical forces	$\alpha_{cr} = 2.05$		
	$N_{cr} = 1230$ kN	$N_{cr} = 2052$ kN	
	$M_{cr} = 307.5$ kNm		$M_{cr} = 712.5$ kNm
buckling shapes			

Table 1

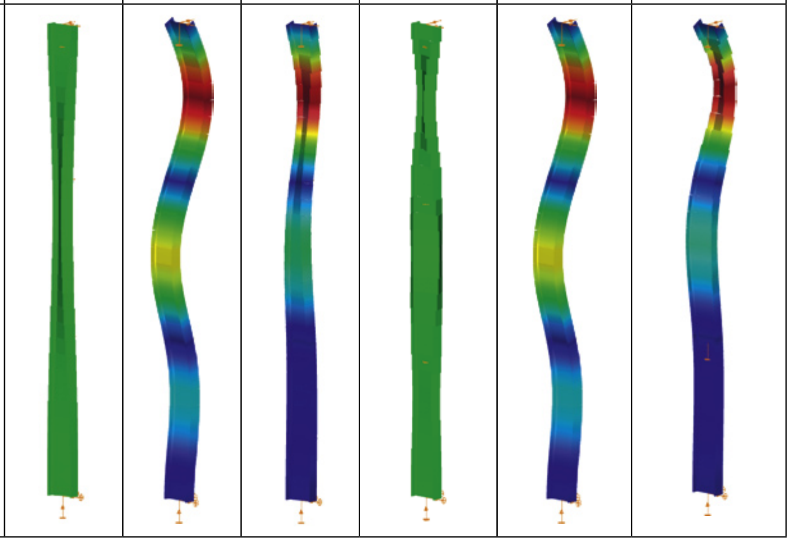
supports	concentric intermediate lateral restraints			concentric intermediate lateral and torsional restraints		
loading	pure N		pure M	pure N		pure M
elastic critical forces	$N_{cr,x} =$ 3192 kN	$N_{cr,z} =$ 6306 kN	$M_{cr} =$ 1647 kNm	$N_{cr,x} =$ 9732 kN	$N_{cr,z} =$ 6306 kN	$M_{cr} =$ 2246 kNm
buckling shapes						

Table 2

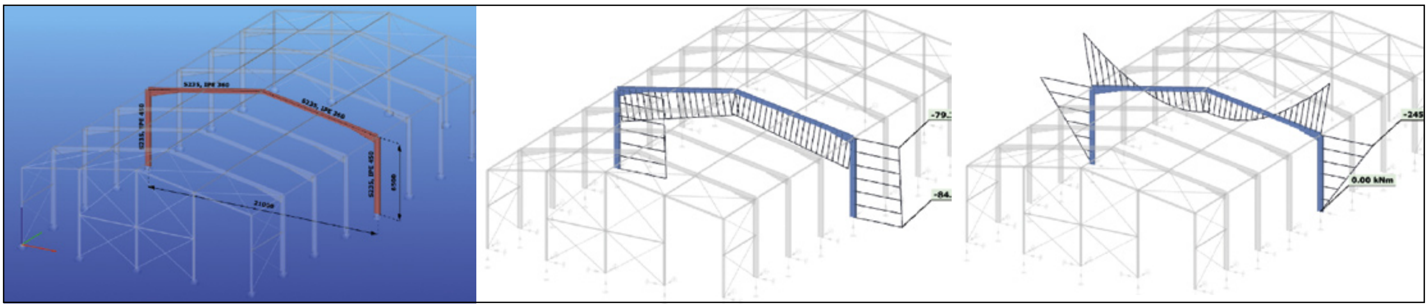


Figure 2: Geometry and internal forces of Example 2

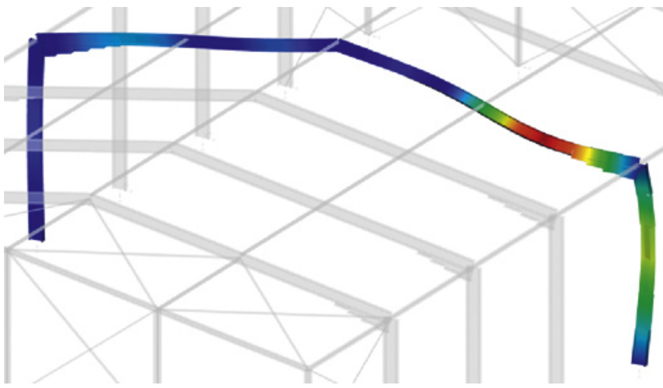
the combined loading – used in the general method – and for the pure cases (N; compression, M; bending). It can be seen that even the pure cases do not belong to pure lateral and lateral-torsional buckling shapes so the appropriate pure mode separation and accordingly the determination of the buckling lengths are impossible.

In Table 2 the two modified support situations are illustrated. If concentric lateral supports are assumed, a pure lateral buckling resistance can be determined, but this is clearly a different buckling behaviour than under the actual restraints. The intermediate lateral supports do not prevent the twist of the member, with different buckling lengths for flexural and torsional buckling, meaning conventional calculation of the elastic critical bending moment is impossible. Also, the relevant mode (torsional) is not taken into account in the interaction formula.

Example 2: Influence of structural member isolation

In this example the frame shown in Figure 2 is examined as a part of a complete structure. The frame is haunched and has pinned bases, braced at the corners and in the middle of the rafters and subjected to 6.75 kN/m distributed vertical load on the rafters. The stability design of the right column is presented. In Figure 3 the buckling of the frame is illustrated due to the compression and bending moments in the members and the results of the general method are shown calculated at the top section of the column.

It can be seen that according to the general method the column is slightly inadequate (105.6%). The column resistance may be recalculated by the conventional interaction method defined in EN 1993-1-1 6.3.3 using the Method 1 (Annex A) for the calculation of the interaction factors. The buckling lengths for both the lateral and lateral-torsional buckling are taken as the system length, assuming that this is a correct estimation. The final utilization →37



Global Stability resistance (dominant)			
Utilization	105.6%		
Applied part of standard	6.3.4 (2)-(3), (4)b - (6.63, 6.64, 6.66) formula		
$\alpha_{ult,k}$	1.522	β	0.750
$\alpha_{cr,op}$	1.450	χ_{LT}	0.624
λ_{op}	1.025	N_{Ed}	-79.5 kN
α	0.340	$M_{y,Ed}$	-249.3 kNm
Φ	1.165	$M_{z,Ed}$	0.0 kNm
χ	0.582	N_{Rk}	2 322.3 kN
α_{LT}	0.490	$M_{y,Rk}$	400.3 kNm
Φ_{LT}	1.047	$M_{z,Rk}$	61.9 kNm
λ_0	0.400	γ_{M1}	1.0

Figure 3: Buckling shape and the results of the general method for the column

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value was 82.1%, meaning that according to the conventional method the column is adequate. The reason for the big difference between the two utilization values is that the buckling shape of the frame involves both the column and the rafter, so these two members should form a consistent unit in stability design. The separation of the column and the determination of the buckling length independently of the rafter produce an overestimation for the column buckling resistance. This can be understood clearly if it is appreciated that as one unit, the column and the beam together reach the elastic critical state at a lower level of load than the column alone.

Conclusions

This article presented some examples of the application of the general method. It was demonstrated that if a more realistic modelling and structural analysis is possible – i.e. a general stability analysis – then a more realistic and natural way for the stability design is to use the general method. The examples also showed the importance of an accurate assessment of the buckling shapes and the associated elastic critical values which can lead to safer – and in other cases more economic – structural design.