A commentary on torsion in steel open sections

Adrián Cabello

adrian.cabello@tensys.com

Abstract

This article raises awareness on the fact that torsion in steel open sections is a complex phenomenon, especially when combined with other actions, and accurate predictions of the structural response are hard to make. The goal is to show, through a couple of practical examples, that aiming to utilisations close to 100% may not be always the most sensible decision if some aspects of the design are not fully defined. The first example shows how a shortcut frequently taken is not necessarily a safe design path, the second focuses on how a broadly-known case study from the literature is indeed behaving differently and is closer to failure than originally intended.

Keywords: Torsion; Open Sections; Design; FE modelling

Introduction

Steel open sections are not ideal to withstand torsion but sometimes the conditions are such that it can't be avoided. The first step to get a good representation of the structural behaviour is to understand the St.Venant/Warping contribution and calculating it is an involved task. The SCI publication P385 [1] or iStructE Technical Note [2] are good guidances where this is thoroughly covered. A complication is that this distribution of torsional effects depends highly on the type of loading, shape and length of the element, detailing at load application and at supports… For instance, it is many times ignored that full warping restraint at the supports implies a particularly robust end detail, not simply the flanges landing against a plate.

The first worked example shows that taking shortcuts to avoid the proper assessment of St.Venant/Warping proportions has to be done diligently because it is not always on the safe side. This is of particular interest when undertaking automated batch processing checks. The second example underlines the fact that it is paramount detailing goes hand-in-hand with the calculation assumptions. In both cases, the steel Eurocode [3] is the standard of reference –hereinafter referred to as EC3.

In both exercises cross-section axes are vertical-Z, horizontal-Y, axial-X.

Example 1

Some pieces of software and calculation spreadsheets do away with comprehensive torsion calculations by simply accounting for St. Venant contribution. If it is true this very conservative approach is most of the times on the safe side and is fit for the purpose of highlighting quickly when torsion needs attention, there are some circumstances, under combined axial, bending and torsion actions where this strategy is not prudent.

Considering St. Venant as the only mechanism to resist torsion shoots up very quickly the shear stresses in the flanges, firstly, and secondly in the web, according to $\tau = t\, T_{Ed}/I_t$ [4]. This will normally make the torsion utilisation (EC3 chapter 6.23) or the web shear utilisation via $V_{pl,T,Rd}$ (EC3 chapter 6.25) govern.

However, if the torsion moment capacity was to be assigned solely to the warping action, i.e. chiefly bending of the flanges –and a small shear–, or to a combination of St. Venant + warping, then the cross section would typically appear to be stronger. This proves that the simple and handy shear flow check is many times conservative. But, noteworthily, this is not always the case. On stocky beams (low L/a, being 'a' the torsional bending constant) with large warping stiffness, a combination of axial, bending and warping stresses, out of a correct St. Venant + warping interaction, can govern over a shear stress check where all the torsional moment is idealised as shear flow.

To illustrate that, it is chosen a beam HEB-160, 1m long and simply supported, being the ends free to warp. It is subjected to a UDL of 120 kN/m applied at a lateral eccentricity of 8.2 cm and a uniform axial force of 1725 kN –it could be tension or fully-braced compression, instabilities are not considered in this exercise. The steel grade is S355.

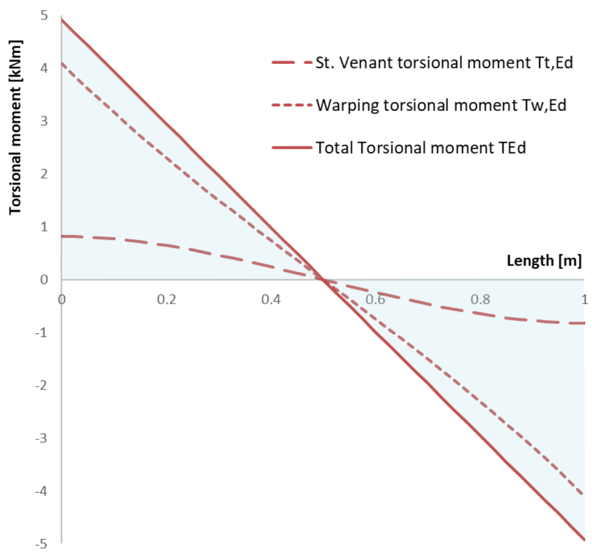

Figure 1. Contribution of St. Venant and Warping to the total torsional moment for Example 1

The beam develops St. Venant and warping torsion (fig. 1) but, in the first instance, it is analysed under the assumption that all the torsion is withstood by St. Venant effect. This brings the ends to utilisation = 100% (with $\tau_{t,Ed,fl}$ = 205 MPa), and mid-span to approximately the same (fig. 2, left) under moment check ($M_{yy,N,V,Rd}$ for EC3 chapter 6.2.5) and biaxial check (also M + N + V) in eq. 6.41, chapter 6.2.9.1. Hence, it can be observed the particular geometric and loading parameters have been chosen to maximise utilisation. At mid-span, shear doesn't reduce the bending capacity, so that $M_{yy,V,Rd} = M_{yy,Rd}$ = 125.7 kNm but axial force reduces it significantly, to just 15 kNm because $N_{Ed} / N_{pl,Rd}$ = 89%.

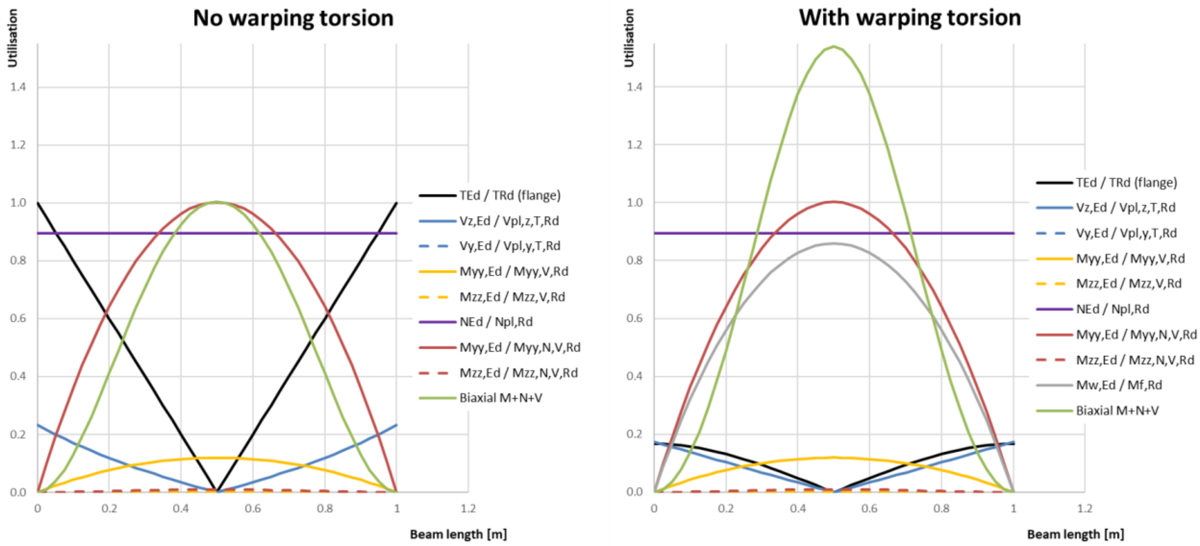

Figure 2. Verification of the beam for Example 1 ignoring warping torsion (left) and acknowledging it (right)

It should be noted that it has been assumed the beam is modelled as a number of finite elements, so that rotation is captured and, consequently, an additional moment $M_{zz,Ed} = \phi\, M_{yy,Ed}$ is included in the calcs, notably for the biaxial check. Whether the gyration ϕ is obtained as a function of St. Venant only or derived from the real interaction St. Venant-Warping is irrelevant for this example because it is in any case a small figure, given the short length of the beam.

Secondly, the same beam is checked taking into account the real St. Venant-Warping interaction (fig. 2, right), as per [1]. Torsion-induced shear stresses are not critical anymore. The beam rotates a maximum of $\phi = 0.6°$ at mid-span, being the max. rotation rate $\phi' = 1.9°/m$ at supports, and max. $\phi'' = -5.5°/m^2$ at mid-span. This produces a maximum warping moment of 6.63 kNm/flange, what shoots the biaxial utilisation up. If eq. 6.41 from EC3 is adopted to merge global ZZ bending and flanges bending, it gives:

$$\left[\frac{M_{yy,Ed}}{M_{yy,N,V,Rd}}\right]^2 + \left[\frac{M_{zz,Ed}}{M_{zz,N,V,Rd}} + \frac{M_{warp}}{M_{fl,Rd}}\right]^{\max(1,5n)} = 154\%$$

being $M_{fl,Rd} \approx M_{zz,N,V,Rd} / 2$. This equation shows that the beam fails with a utilisation equal to 154%. If, on the contrary, it is used the formula suggested by the P385 guideline then

$$\left[\frac{M_{yy,Ed}}{M_{yy,N,V,Rd}}\right]^2 + \left[\frac{M_{zz,Ed}}{M_{zz,N,V,Rd}} + \frac{M_{warp}}{M_{fl,Rd}}\right] = 187\%$$

Furthermore, a similar discussion could probably be held in relation to the suitability of EC3, clause 6.2.7 (7) for every possible scenario, when it says ‘as a simplification, in the case of a member with open cross section, such as I or H, it may be assumed that the effects of St. Venant torsion can be neglected’.

Example 2

The second example highlights the importance of the coherence between the analysis assumptions and the required detailing to meet them. This will be done by reviewing the first exercise from the P385 guideline.

This worked example defines a simply supported beam UKC 254 x 73 (S275), 4m long, and subjected to a point load of 100 kN (ULS) applied at mid-span but laterally eccentric by 75mm. The hand calcs proposed to verify the beam assume ideal conditions which don't correspond to the real behaviour of the beam. As it is detailed with the load hanging from a single bolt (fig. 3 a) the lower flange is likely to fail significantly earlier than the predicted collapse takes place, as a combination of global axial, bending and shear, but also local bending and shear. Hence, the utilisation achieved is much higher than the one announced by the hand calcs.

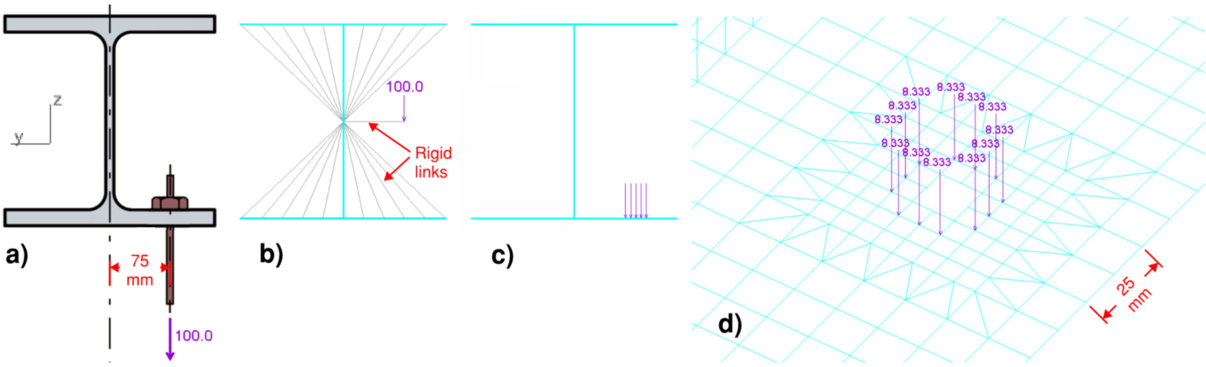

Figure 3. a) Problem definition; b) Ideal loading implementation for the problem; c) and d) Realistic loading implementation.

To prove this, it has been modelled in *Strand7* a FE model of the beam with a load application that replicates the ideal conditions from the hand calc assumption – 'Ideal' model (fig. 3 b); and another where the load application matches the detail – 'Real' model (fig. 3 c, d). The 'Real' model assumes the load is distributed by a washer of diameter 25mm –not specified on P385's wording. Boundary conditions for both cases are shown in Fig. 5 a, these have not been varied in order to focus the study on the load application area. The analyses are run as GNLA. The mechanical properties are those suggested by EC3, clause 3.2.6. The mesh size is typically ~25x25 mm^2 and it becomes finer until it reaches ~6.3x6.3 mm^2 in the vicinity of the bolt. The elements are *Quad4* shells formulated as Discrete Kirchhoff Quadrilateral [5].

The 'Ideal' model's output is first compared with the P385 results to see if this is a valid contrast model, i.e. mesh density, boundary conditions and load application match the hand calc assumptions.

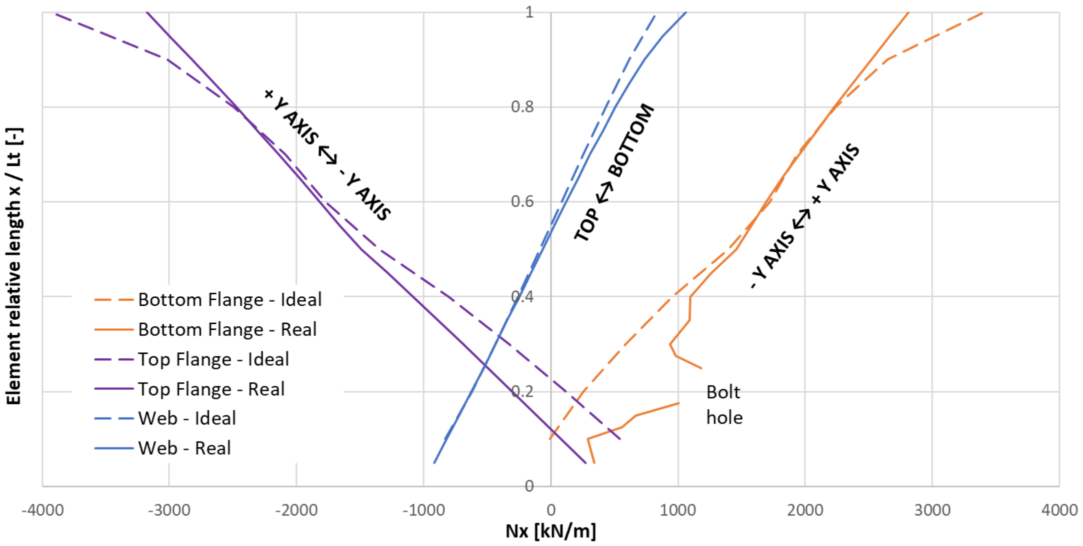

Figure 4. Axial forces in the longitudinal direction of the beam

The variable N_x [kN/m] (fig. 4) has been extracted and integrated over the flanges and web and the equivalent moments M_{yy}, M_{zz} and M_{warp} have been reverse-engineered from them, leading to:

	P385	FE model
M_{yy} [kN/m]	102.0	102.3
M_{zz} [kN/m]	5.4	6.0
M_{warp} [kN/m]	21.1	22.1

It should be noted that the rotation according to the hand calc (1st order) is 3.1° and the iterative GNLA shows 3.4°. This explains the difference in M_{zz} and probably part, or all, of the difference in M_{warp}.

Once it is accepted the good agreement between the hand calcs and the 'Ideal' FE model, the 'Real' model can be interrogated. The latter shows a very distinct local deflection around the load application (fig. 5 b, c) and less warping action (fig. 4).

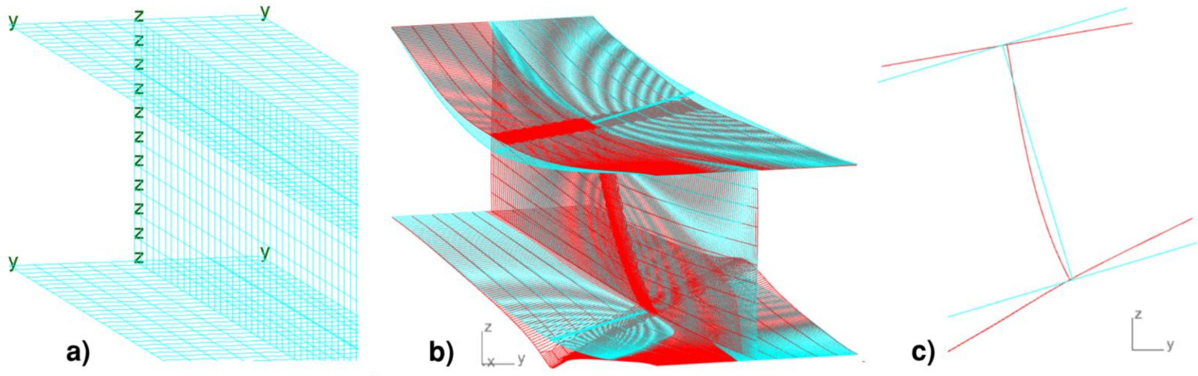

Figure 5. a) Boundary conditions; b) Deformed shape, magnified x5, for 'Ideal' (blue) and 'Real' (red); c) Cross-section cut at mid-span, magnified x5, for 'Ideal' (blue) and 'Real' (red)

Lastly, the 'Real' beam FE model has been analysed under GMNLA, incrementally up to the load that theoretically would provide a 100% utilisation according to the hand calcs (P_{Rd}). The constitutive model used is bilinear elasto-plastic with just a minimal non-zero strain hardening to ensure convergence. Ignoring buckling, the ULS load of 100 kN provided a stress utilisation of 51%. Replicating the torsion calculation a load P_{Rd} = 161 kN should lead to a 100% utilisation. On the contrary, the results of the non-linear analysis in figure 6 show that at approximately between 70% to 85% of P_{Rd}, depending on the washer diameter, a Von Mises stress = f_{yd} would be reached at every fibre through the bottom flange thickness across a surface area wide enough to produce a local failure of the plate. Therefore, the real utilisation of that beam, if built as detailed, would probably be much higher than expected.

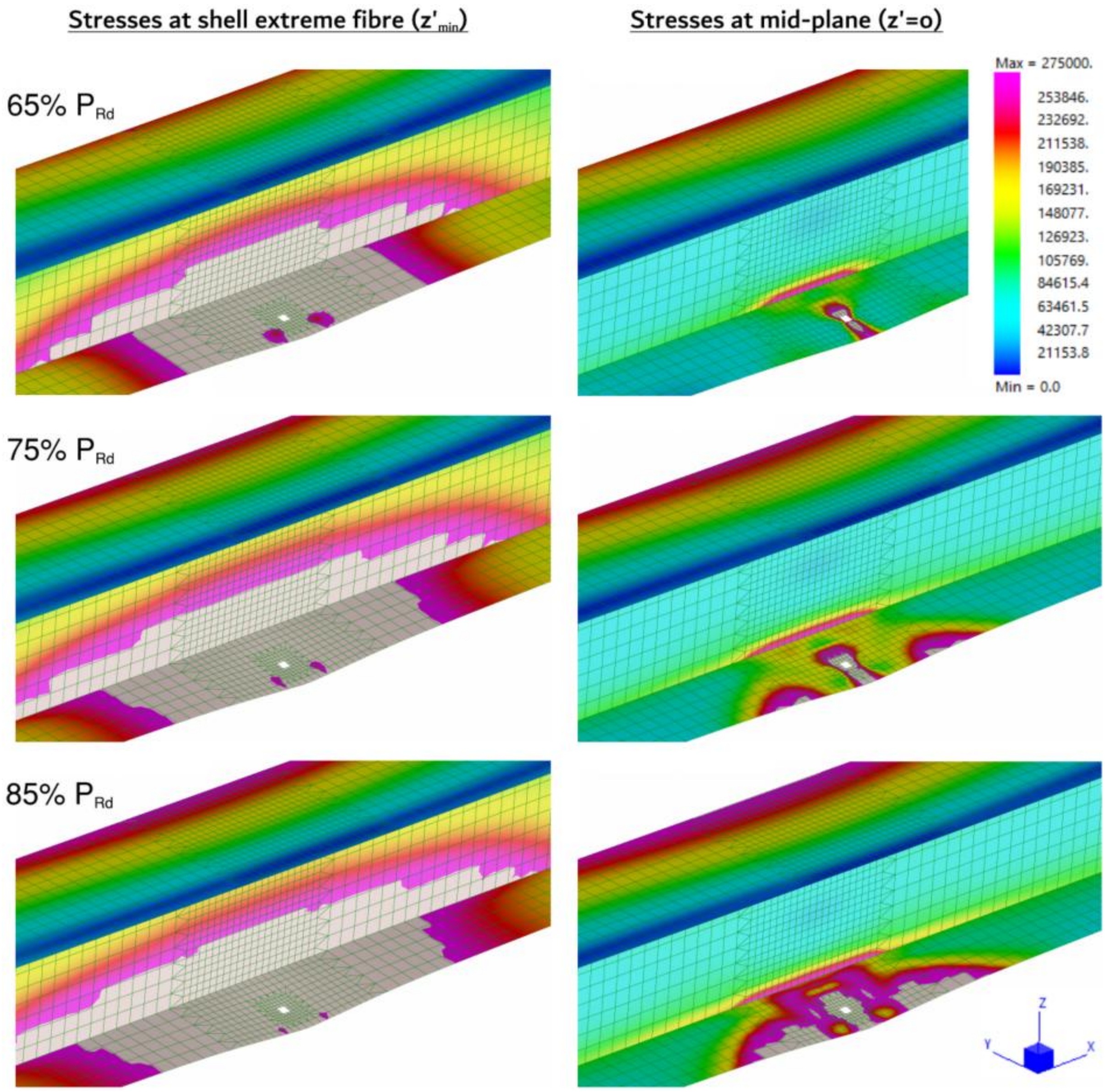

Figure 6. Von Mises stresses [kPa] at the extreme fibre z'_{min} (Left) and at mid-plane (Right). Stresses are cut-off when reaching f_{yd} = 275 MPa. Stresses at extreme fibre z'_{max} have been omitted because of their similarity to those at z'_{min}.

To achieve a structural response closer to the assumed by the hand calcs vertical stiffeners, for instance at both sides of the bolt, would help to mobilise better the whole section in torsion, consequently achieving lower utilisation than the one that can be found on the 'Real' model.

It is unfortunate this local failure could turn up because, purely torsion-wise, Farwell [6] and Pi [7] demonstrated through experimentation on simply supported beams subjected to a central torque, well distributed across the central section, that torsion had a stiffening effect and final failure of the beams was due to tensile fracture at the flange tips, at torques higher than those which predicted plastic collapse.

Conclusion

In brief, this text's intention has been to give a word of caution when dealing with torsion verifications and aiming to optimise the structural element to a high degree. If it is the case, it has to be done prudently, avoiding shortcuts and paying attention to the detail(s). The article has illustrated with two examples the importance of the above.

References

[1] A. F. Hughes et al. (2011) 'SCI Publication P385: Design of Steel Beams in Torsion'. The Steel Construction Institute.

[2] R. Henderson (2023) 'Designing for torsion in open-section steel members'. Technical Guidance Notes; Level 3, No. 1

[3] CEN (2014). 'BS EN 1993-1-1:2005 + A1:2014 Design of Steel Structures Part 1-1: General rules and rules for buildings'.

[4] S. Timoshenko (1930) 'Strength of Materials. Part II – Advanced Theory and Problems'. D. Van Nostrand Company.

[5] Strand 7. 'Reference Manual' (www.strand7.com/strand7r3help/)

[6] Farwell, C.R., and Galambos, T.V. (1969). 'Non-Uniform Torsion of Steel Beams in Inelastic Range', Journ. Struct. Dvn, ASCE, 95 (ST12) 2813-2829.

[7] Pi, Y.L., and Trahair, N.S. (1995). 'Inelastic Torsion of Steel I-Beams', Journal of Structural Engineering, ASCE, 121 (4) 609- 620.