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An Overall Interaction Concept for an alternative approach to steel members design

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ABSTRACT

The present paper focuses on a new, alternative design philosophy: the Overall Interaction Concept (O.I.C.). This concept, based on the well-established resistance-instability interaction and the definition of a generalised relative slenderness, was thought and built to i) improve actual design practice, ii) increase accuracy, iii) advance simplicity and consistency, and iv) provide a sound framework for computer-assisted resistance predictions. This paper first details the bases and features of the O.I.C. approach, and provides mechanical interpretations of its application steps. Comprehensive sets of results at the cross-sectional level are then presented, for both H-shaped and hollow sections. Ayrton-Perry-based χ-λ design relationships for hollow structural shapes are proposed and shown to lead to more accurate, consistent and safe resistances when compared to Eurocode 3 rules, in addition to being significantly simpler in application. As for the behaviour and response of members, numerous F.E. results are reported, demonstrating the potential of a χ-λ approach to successfully apply to members under combined load cases. Also, the challenging case of coupled instabilities is investigated, and the O.I.C. approach is showed to be very efficient and appropriate. Further developments towards the derivation of full O.I.C. procedures to steel members are currently under way.

Abbreviations

O.I.C.	Overall Interaction Concept
E.W.M.	Effective Width Method
D.S.M.	Direct Strength Method
C.S.M.	Continuous Strength Method
ρ	Plate reduction factor (according to the E.W.M.)
λ_p	Relative plate slenderness
ψ	Ratio of longitudinal stresses at plate edges or end moment ratio
b	Width of profile
h	Height of profile
t	Thickness of plate
k_σ	Plate buckling coefficient
M_{pl}	Plastic bending moment
M_{el}	Elastic bending moment
λ_L	Generalised cross-section relative slenderness (includes influence of *local* buckling behaviour)
λ_{L+G}	Generalised member relative slenderness (includes influences of *local* and *global* buckling behaviour)
χ_L	Generalised cross-section *local* buckling factor
χ_{L+G}	Generalised member *local* and *global* buckling factor
χ_{FE}	Generalised buckling factor calculated numerically by finite elements
χ_{EC3}	Generalised buckling factor calculated according to Eurocode 3 equations
$\chi_{proposal}$	Generalised buckling factor calculated according to proposed approach
R_{pl}	Load ratio to reach to "resistance" limit (*plastic* capacity)
$R_{cr,L}$	Load ratio to reach to cross-sectional (*local*) "stability" limit
$R_{cr,G}$	Load ratio to reach to member (*global*) "stability" limit
$\sigma_{cr,p}$	Plate critical stress
ε	Strain
ε_y	Strain at first yield (elastic)
α_L	Generalised imperfection factor (cross-section level)
β	Factor accounting for strain hardening effects
δ	Factor accounting for post-buckling resistance reserves
λ_0	Non-dimensional length of plateau for resistance curve

1. Motivation – context

The present paper focuses on a new, alternative design philosophy: the Overall Interaction Concept (O.I.C.). This concept, based on the

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well-established resistance-instability interaction and the definition of a generalised relative slenderness, was thought and built to i) improve actual design practice, ii) increase accuracy, iii) advance simplicity and consistency, and iv) provide a sound framework for computer-assisted resistance predictions.

The idea of developing this new approach rose as a response to current specific problems – both conceptual and practical – and as an anticipation to the emergence of new materials (e.g. high or ultra-high strength steels) and design tools (i.e. software).

As one of the current main issues in steel design, the cross-section classification concept bears many inconsistencies and practical difficulties. Cross-section classification consists in a preliminary step to section and member resistance checks, that intends at i) informing the designer on the possibility to resort to a plastic analysis (class 1 – plastic – sections possessing sufficient ductility and rotation capacity to allow developing a complete frame plastic collapse mechanism), and ii) orientating the designer to either plastic (classes 1 and 2), elastic (class 3) or effective (class 4) resistance checks. Classification of a section is achieved by means of b/t limit ratios which provide, knowing the actual stress distribution on each element and its "support conditions" (i.e. flange or web element), the class of the cross-section, determined as the class of its worst element.

First of all, various papers have evidenced the inaccuracy of the proposed b/t limits, and both unsafe and over-conservative resistance predictions are reported, especially for hollow sections ([1,2]). Also, several discrepancies can be reported, e.g. as the class 3–4 border, where a section may happen to be classified as class 4 according to Eurocode 3 Part 1-1, thus requiring the calculation of effective properties following Eurocode 3 Part 1-5 design guidance, but eventually found to be class 3 and fully effective from the latter part of the code ([3]).

This can be primarily attributed to the inaccurate definitions of the b/t limits as in Eurocode 3 (see Table 2). These limits, for the sake of simplicity, were basically derived for compression, mono-axial bending, and mono-axial bending with compression ([2]). Their background lies in i) the somewhat arbitrary adoption of plate slenderness limits $\lambda_p = 0.5$ and $\lambda_p = 0.6$ to define class 1–2 and class 2–3 limits ([4]), respectively, and in ii) the use of the so-called "Winter formulae" ([5]) to set class 3–4 limits. Additional background information may be found in [4].

Winter formulae for "*internal* compression plate elements" (webs) used for setting class 3–4 limits are as follows:

$$\rho = \frac{\overline{\lambda}_p - 0.05(3+\psi)}{\overline{\lambda}_p^2} \leq 1 \text{ (Winter) and} \tag{1}$$

$$\rho = \frac{\overline{\lambda}_p - 0.055(3+\psi)}{\overline{\lambda}_p^2} \leq 1 \text{ (modified Winter)}$$

For "*outstand* compression plate elements" (flanges), the so-called "Winter formula" reads:

$$\rho = \frac{1}{\overline{\lambda}_p} - \frac{0.188}{\overline{\lambda}_p^{\,2}} \leq 1 \tag{2}$$

Depending on the support conditions of the plate or element (i.e. web or flange) and its associated stress distribution, these assertions lead to Table 1 b/t limits.[1] It shall be noted here that, for the sake of consistency with Eurocode 3 Part 1.5 rules for plate buckling ([6]), the "modified Winter formulae" have been used in Table 1. The recommendations of Eurocode 3, in turn, provide b/t limits as in Table 2.

As can be seen, multiple differences can be evidenced, for which quite limited scientific justification could be made. In the particular case of b/t limits for webs, several authors ([3,7,8]) have reported significant unconservative resistance predictions triggered by the seemingly too optimistic $b/t = 38\ \varepsilon$ value at the class 2–3 border. This is clearly seen in the comparison between Tables 1 and 2, and further evidenced by Fig. 1a, where the comparison between different standards exhibits large and questionable differences – note in particular that Eurocode 3 proposes a 4 ε-wide class 3 range, from 38 ε to 42 ε, while other standards propose an end of class 3 at 38 or 34 ε, i.e. an end of the class 3 range well below the beginning of that of Eurocode 3.

Also, it is now widely recognised that smooth and continuous resistance transitions along the b/t range shall be made available in modern design codes, in order to provide consistent design provisions from plastic to slender situations. In Eurocode 3 ([9]) in particular, the current rules shall be improved, as suggested in [3] in order to avoid the gap of resistance at the class 2–3 border (see Fig. 1b), which is mechanically meaningless and unacceptable. It may be noted here that several standards ([10,11,12]) have already included such continuous provisions.

As another point suffering criticism, the assumption of "ideal support conditions" for the element plates comprised within the whole section brings further inadequate and inaccurate resistance predictions. The interaction between elements is indeed usually disregarded ([13]), each element being presumed to behave discretely; flanges are assumed to behave as under pinned-free support conditions, while webs are assumed as pinned-pinned. It is however clear that elements are interacting, and that this may lead to both over-conservative results (e.g. a weak web associated with strong flanges so that the web is close to clamped-clamped support conditions), or unsafe results (e.g. a slender, locally-buckled web may be attributed a "negative" stiffening effect to flange stability [14]). Several attempts to account for it directly ([15]) or indirectly through interdependent b/t limits for flanges and webs may be found in the literature. Early results ([8,16]) have shown that the assumption of "ideal support conditions" may lead to significant differences with respect to more rigorous modelling. In this respect, the possibility to consider the cross-section as a whole in the design procedure represents a great improvement, which the O.I.C. allows for (see Section 2.1).

Eventually, one may report on major practical application difficulties of the cross-section classification concept, for sections under compression and biaxial bending or for the determination of the plastic neutral axis of hollow sections under $M_y + M_z$ for example; the latter situations lead to disproportionate efforts regarding the information it provides, when related to the consecutive design checks, in which designers are primarily interested.

Besides issues associated with the cross-section classification concept, the adoption of the Effective Width Method (E.W.M.) in major design standards is known to bring further practical difficulties. Indeed, while the assumption of neglecting parts of the elements that are most concerned with local buckling can make sense from a Structural Mechanics point of view, it triggers long and tedious calculations of the cross-section effective properties – sometimes even through an iterative process, cf. [17]. While the calculation of effective properties may be considered as affordable for large girders in sophisticated structures (e.g. bridges), this may be deemed as unacceptable in case of standard, simple building elements. This point is expected to become of greater importance in the near future through the increasing use of high strength steels, where the relative importance of instabilities is growing (especially local buckling) and situations where local-global coupled instabilities have to be accounted for are also met more often.

In addition, the material response of high strength steels being known to sometimes be more non-linear than mild steels (i.e. no yield plateau), the concept of plastic resistance makes no sense anymore[2]; other similar cases include stainless steel members, cold-formed

[1] Note that $\varepsilon = \sqrt{(235/f_y)}$ intends at unifying b/t limits for various steel grades, acknowledging for the well-known more important relative influence of local buckling for higher steel grades.

[2] Indeed, the "disappearance" of the plastic plateau prevents from using typical constant stress blocs diagrams, causing the usual determination of plastic resistance inappropriate.