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Are Designers Sufficiently Instructed to Make the Most Rewarding Use of the Latest Steel Codes?

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Abstract
Eurocode 3 is going to be very soon substituted for most national codes within Europe. It introduces new concepts and, especially, a quite innovative approach for the design of structural joints. It shall result in more economical projects provided all these aspects are fully mastered by the practitioners. Presently the latter requisite is still not fully met. Therefore multiple trainings to Eurocode 3 were and are still held. They aim at giving the structural engineers the capability of using the relevant design rules in the most efficient manner. On the base of a personal experience in this field, it is less worth focusing on the latest design formulae than on the overtures offered by the European code. In present paper some main topics are identified and briefly commented on. The lessons drawn in the European context are likely to be more widely profitable.

1. Introduction

Defining waggishly "structural engineering" as "the art of molding a material that we do not know, into a form that we cannot really analyze, to resist forces that we cannot really assess, in such a way that public does not suspect" clearly points out the role and responsibility of the profession. The structural engineer is supposed to be deeply schooled to theoretical disciplines such as structural analysis, strength of materials, elastic stability and steel design. Once stepped in the professional life, he is expected to suitably merge these subjects so as to properly conduct the design in compliance with relevant codes in force. On the base of his technical knowledge and engineering judgment, he should get the most efficient and economical structural system.

Eurocode 3 is going to be substituted for former national codes and soon become the single design steel code throughout Europe. Actually that code consists in a set of separate documents. It is indeed subdivided in six parts, EN 1993-1 to EN 1993-6. Parts EN 1993-2 to EN 1993-6 address specific types of steel structures; they refer to generic rules given in EN 1993-1 and supplement them. For its own, EN 1993-1 is composed of twelve sub-parts, EN 1993-1-1 to EN 1993-1-12, which are devoted to specific topics. Especially its sub-part EN 1993-1-1 (CEN 2005a), entitled "General rules and rules for buildings", is the "master document" for all the other parts and sub-parts of Eurocode 3; in conjunction with mainly sub-part EN 1993-1-8 (CEN

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2005b), dedicated to joint design, it is especially concerned with present paper. Also Eurocode 3 is intended to be used in conjunction with EN 1090 (CEN 2009) in which all the technical requirements for fabrication and execution, including erection, of steel structures are specified. It is worth pointing out that, except otherwise contractually stated, a steel structure is designed for a working life which is specified in EN 1990 (CEN 2002a); accordingly it shall be designed against corrosion, designed for wearing and for accidental actions, detailed for sufficient fatigue life and duly inspected and maintained. For instance, the working life is 50 years for normal buildings and 100 years for monumental buildings; of course it may be alternatively determined in agreement with the client. Also the Eurocodes are used in conjunction with the relevant Euronorms, which are product standards; especially EN 10025 (CEN 2004) is concerned with hot-rolled shapes.

EN 1993-1-1 is significantly different from former national codes in many respects; that goes from details to much more important aspects such as not exhaustively:
- The way structural safety is accounted for;
- The concept of limit state design (LSD), which substitutes for the allowable stress design ASD) and significantly enhances the role of ductility;
- The acknowledgement that any structural system and its composing members are imperfect;
- The innovative approach for the design of structural joints;
- The structural modeling where joints and members are henceforth given a similar status;
- The benefit drawn from material yielding properties;
- The several ways to account, when necessary, for second-order effects;
- Refined design formulae for member and cross-section resistance when coincident effects;
- A same format of all the basic “stability checks” for a more comprehensive understanding.

Eurocode 3 is widely recognized as a potential source of savings in fabrication and erection costs of the structural frames. However the most rewarding use of the European code requires a largely new expertise and the practice of specific design tools, the availability of which is often disregarded if not ignored. Several interdependent ways are offered to go through it. Clearly a keen mastery of several inter-relations is requested but is still faulty. Also it is a matter of fact that technical education of “average” European designers is not yet fully consistent with the knowledge which is expected to draw the quintessence of Eurocode 3. Strongly regulated drafting and lack of commentaries, despite (Silva et al 2010), are as many additional handicaps for the designers. Last, some European countries did not take sufficiently care of the Eurocode work and have especially missed the advisability, as soon as 1992, to use the ENV version ENV 1993-1-1 of the code (CEN 1992) – in conjunction with the National Application Document – as an alternative to the national standard. For instance, the last national French code CM66 (AFNOR 1966) is fifty years old but was still largely used at the national level up to the present!

Such a context makes that many designers must cross the threshold but happen to come up against a blank wall. Then one question comes up to mind: are designers sufficiently instructed to make the most rewarding use of the latest steel codes? Asking it makes the answer hardly doubtful. Of course putting such a query should not be understood as evidence of disrespect to the structural engineering profession. It is simply prompted by the diagnosis which was made by the author and impacted the spirit of more than twenty trainings he contributed in the last decade and are still going on, not so much in his own country but in a neighbor one.

Instead of providing explicit answers to above question, present paper only ambitions to comment on the major general aspects which have to be first understood, then fully mastered by those wishing to get all benefit from the European steel code. Fore sake of conciseness, it is referred to plane frames composed of members with a uniform cross-section and statically loaded at room temperature. As far as possible the selected topics are reviewed according to the sequences of an actual design.

Though the following considerations are developed in a European context, it is felt that they might be profitable to a much larger circle of practitioners and teaching profession.

2. Concept of structural safety

In accordance with LSD, any check of cross-section resistance or member resistance requires that the design (index d) effect E_d normalized with regard to the relevant design resistance function R_d – or an appropriate combination of such ratios when coincident effects – does not exceed unity. Effects E are internal forces issued from a method of global analysis, which is usually conducted by means of adequate software. They result from combinations of actions F specified in "loading codes" (CEN 2002a and CEN 2002b). Resistance functions R are within the scope of design codes and standards, such as Eurocode 3 in Europe and LRFD (AISC 1999) in the United States.

Latest codes substituted limit state design (LSD) for former allowable stress design (ASD). In ASD, the design effects E_d are the characteristic (index k) effects E_k while the characteristic resistance functions R_k are divided by structural safety factors s to get the design resistance functions R_d. In LSD, characteristic effects E_k are magnified by partial safety factors for actions γ_F to get design effects E_d and design resistances R_d are obtained by dividing characteristic resistance functions R_k by global partial factors for resistance γ_M. In LSD, structural safety thus results from influences of both γ_F and γ_M. In case of 1st order elastic behavior, the $\gamma_F\ \gamma_M$ product is similar to s; that is no more the case in other situations, especially when the structural behavior is non linear. Moving from ASD to LSD thus changes the way structural safety is accounted for; with the new harmonized symbols applied for the purpose of the code, they are the very first impediments to which the still ASD dependent designers are faced.

3. Limit state design and material properties

ASD refers to the stress concept; its use is thus restricted to both elastic global analysis and elastic design checks of cross-sections and members. In contrast, limit state design (LSD), which prevails in the latest codes, allows for inelastic material behavior. Steel yielding is usable at two stages: cross-section resistance and frame resistance.

3.1 Possible plastic redistribution: requirements and consequences

The substitution of inelastic material properties for elastic ones is still challenging for many designers. Therefore elementary but fundamental comments or reminders shall be addressed, especially to point out that material compatibility is locally violated under the reservation of appropriately ductile material and structural behavior. The concepts of plastic stress redistribution and moment redistribution, within respectively the cross-section and the structure, as well as the effects of such redistributions on both the frame stiffness and structural stability must be clearly delivered. Indeed clear understanding of the detailed mechanical behavior, up to the ultimate, of a frame subjected to monotonously increased loading is fundamental. Deliberately the terminology “plastic design” is avoided herein; though it obviously presupposes material yielding, it does not clearly indicate to which extent advantage is taken of it.

Plastic cross-section bending resistance is developed when the yield strength is attained everywhere within the cross-section. It is reachable under the reservation that i) the compression plate components of the structural shape do not buckle prematurely, and ii) the material has sufficient ductility, i.e. capacity of fibers to strain before their failure occurs. The concept of plastic hinge is associated to such full stress redistribution. A plastic hinge has no more elastic flexural stiffness. Once such a hinge is formed, the frame stiffness decreases, and repeatedly, under increased loading, till the exhaustion of the whole structural stiffness. That ends with the formation of either an incomplete or a complete plastic mechanism according as how much the progressive loss of frame elastic stability interferes with the development of the successive plastic hinges. After its onset any plastic hinge must continue to rotate – though its moment resistance cannot further increase – till the frame carrying capacity is reached. The demand in rotation capacity of cross-sections where plastic hinges occur is therefore significant. A similar demand applies when, as an alternative, linear elastic global analysis is used but is followed by a lump redistribution of peak moments in continuous beams.

It goes without saying that the capability of cross-sections to develop their plastic resistance and further sufficiently rotate is a requisite for plastic global analysis. Widely, but wrongly, some designers still believe that checking cross-sections for plastic resistance is subordinated to prior plastic global analysis. For sure, elastic global analysis – the most usually practiced – followed by plastic resistance checks is an approach which is consistent with Eurocode 3.

At many places of Eurocode 3 the concepts of ductility, rotation capacity and local plate buckling are not explicitly referred to but are simply underlying. Designers must be fully aware of them otherwise they are kept from fully realizing the aim or the meaning of several design specifications.

3.2 Structural role of ductility

LSD consists in checking ultimate limit states (ULS) and service limit states (SLS); the latter are disregarded herein. ULS is suitably identified with the exhaustion of the ultimate resistance of cross-sections and/or members and/or frames; by extension, it is often associated with failure, what is sometimes misleading. For instance, the ultimate resistance of a bolted tensile member is given as the lower of two cross-section resistance functions corresponding respectively to yield strength in the gross cross-section and ultimate strength in the net cross-section. Gross cross-section resistance disregards material strain-hardening; yielding develops along most of the member length and results in elongation of the member which is deemed to be inadmissible. In contrast, net section resistance is reached when almost the tensile strain capacity of steel material is exhausted. When ductile behavior is requested, the design of the tensile member shall be governed by the gross cross-section resistance. What is a requirement in seismic design (to allow for energy dissipation) is only a recommendation in static loading (to have premonitory signs to potential collapse).

Ductility is a determinative key-point in the design of joints. That is illustrated by means of two types of joints. The first example is the lap bolted joint. In the elastic range, all the bolts of the joint do not experience a same proportion of the load. Such connections are however designed on the base of an even share of the load, i.e. the joint components are ductile enough to enable the load being evenly redistributed amid all the bolt rows. That happens indeed in regular joints. In contrast, the strain capacity of the outer bolts in so-called "long" lap joints is exhausted before complete plastic redistribution takes place; the plastic resistance of the joint has then to be affected by a penalty factor. Let us now examine the more complex beam-to-column bolted joint. For a long time, its resistance was based on an elastic distribution of the moment in the connection and governed by the attainment of the resistance of the most tensile bolt. Also the area surrounding the so-called "rigid" beam-to-column connection was stiffened for fear, i.e. without due justification or actual prove that these stiffeners are required (see the German appellation "angststeifung"). Nowadays, according to EN 1993-1-8 (CEN 2005b), the resistance of such joints accounts for some plastic redistribution, to the amount permitted by the ductility of respective joint components; also the need for and the role of joint stiffeners can be clearly investigated in order to get the best joint detailing.

For steel structures, a minimum ductility is required from the material; it is governed by minimum values of the ultimate-to-yield strength ratio, elongation at failure and ultimate strain; nonetheless structural steel grades covered by Euronorms are deemed to fulfill all these requirements. Furthermore not only the material but also the structural components made of that material must exhibit a sufficiently ductile behavior.

In LSD ductility (and/or rotation capacity) is given a similar status as resistance and stiffness properties. In the design process, it is more qualitatively than quantitatively referred to. Though very helpful, it remains misrecognized and unsuitably mastered by many designers. Let us for instance refer to a braced frame with simple beam-to-column joints. Such joints are deemed to experience no bending moment whilst their detailing clearly invalidates this assumption. Is it anyway conservative to conduct global analysis by assuming pinned joints? When the loading increases, the bending moment M_{Ej} experienced by the actual joint ends in reaching the (small) joint moment resistance M_{Rj}. Then the connected members exhibit a relative rotation ϕ_j deduced from the limit M_j-ϕ_j joint resistance curve. Further increment of moment results in spreading material yielding: the joint stiffness decreases and some redistribution of the internal forces occurs within the frame so that the design bending moment $M_{Ed,j}$ in the joint decreases while the design shear force $V_{Ed,j}$ increases. Provided the joint is capable of sufficient rotation capacity, the joint behavior then evolves along the limit joint resistance curve. The more the joint area yields, the more $M_{Ed,j}$ decreases and the relative rotation ϕ_j increases so that the hinge-type behavior is progressively approached. At the ultimate, the assumption of simple joints holds.

3.3 Structural role of local plate buckling

National building codes pay little attention to buckling aspects. Hot-rolled shapes – the most usual sections – are indeed almost insensitive to elastic plate buckling because technical rolling requirements prevent from producing shapes with thin plate elements. The scope of the latest codes is extended to welded sections, which are characterized by larger slenderness of their plate elements and often cannot develop the elastic resistance of their gross cross-section. There is thus a need for discriminating cross-sections in terms of elastic or plastic resistance and possibly in rotation capacity; that is met through the classification of cross-sections.

For plastic cross-section resistance being reached, plate buckling – also termed local buckling – has to occur in the inelastic range and once a sufficient strain is reached. That required strain – to which corresponds a demand in rotation capacity – is significantly larger for plastic global analysis being permitted. Also the larger the yield strength, the earlier the onset of plate buckling compared to the attainment of the yield strain.

The web of welded sections is usually very slender. Then plate buckling occurs in the elastic range or in the early elasto-plastic range. Within the compression area it penalizes the elastic extensional rigidity of the buckled fibers and has a detrimental influence on the rotation capacity.

The class of a cross-section is given as the worst class of the individual plate components which are subjected to compression across their whole width or a part of it only. Each class of plate element is determined by a maximum width-to-thickness ratio, the magnitude of which involves the yield strength and a numerical factor that reflects altogether the relevant ductility demand and the effects of stress distribution and boundary conditions. The class of a given cross-section is not an intrinsic property; it is likely to be less favorable if a higher steel grade is used.

The choices of, respectively, global analysis – elastic or plastic – and reference cross-section resistance – elastic or plastic – are clearly governed by the cross-section classification.

Elastic critical buckling (normal or shear) stresses are no longer design resistance criteria. Nowadays they are just parameters involved in the ultimate resistance models on which the design specifications of sub-part EN 1993-1-5 (CEN 2006) relative the buckling resistance of plate elements are based. Many designers have a poor, if not very poor, knowledge in the plate buckling aspects and might therefore consider some latest design specifications as magic formulae. The latter get sense if it is commented on the plate behavior, the conditions for a biaxial behavior taking place in the post-buckled range, the ultimate resistance models and the effects reflected through them. Some recent documentation does exist in this respect (Johansson et al 2007, Beg et al 2010).

3.4 Material strengths

Nominal values of yield strength and ultimate tensile strength of structural steel grades are material properties, which are involved in the resistance functions. Of course both vary with the steel grade. Moreover they do not remain constant within a given steel grade but decrease when the (flange) thickness increases; that is worth being physically justified. Structural steel grades shall comply with Euronorms. For hot-rolled sections, values of above properties are given in product standard EN 10025 for 5 to 6 classes of thickness ranges. That metallurgist's approach is all but appropriate for design purposes so much the values of the design yield strength should be adapted within a project. Therefore Eurocode 3 allows for a much pragmatic approach by distinguishing two thickness ranges only. Besides the physical justification of the thickness-dependency, the inconstancy between product standard and design standard has to be stressed out as well as the fact that EN 1993-1-1 clearly permits that reference be made to either standard. Designers will most often seize the more pragmatic alternative.

EN 1993-1-1 is dedicated to steel structures made of steel grades up to S460 grade. Increasing the steel strength beyond the latter grade – what is unusual in the field of buildings – makes the ductility decrease and there is a price to pay for that: some design rules are still valid but subordinated to some limitations or restrictions while some other ones are merely modified. High strength is the scope of sub-part EN 1993-1-12 (CEN 2007).

4. Member and frame imperfections

In accordance with the European code, the carrying capacity of a frame can be more closely approached because the material is better exploited. *De facto* the structural model shall be in closer consistency with the real, by nature imperfect, frame. Member and frame imperfections detrimentally influence the global frame stability and the carrying capacity of prone to buckling members. Therefore EN 1993-1-1 pays due attention to such imperfections.

4.1 Frame imperfection

Due the erection process a vertical file of columns is supposed to exhibit an initial sway. That "frame imperfection" is taken as an out-of-plumb in the frame plane; its reference magnitude is 1/200. The latter is moderated by two factors, which depend respectively on the number of columns present in the storey and on the total height of the frame. The probability of having all the columns fitted with the maximum out-of-plumb is indeed the lesser as the number of columns is large; also erection strives of course to balance any misalignment observed on site at the time the column segments are spliced. Frame imperfection generates so-called P-Δ effects and therefore enhances the frame proneness to global buckling according to a sway mode. In view of structural global analysis, it is modeled by:

- Either fitting the column files with the appropriate out-of-plumb,
- Or keeping the column files vertical but replacing the frame imperfection by equivalent horizontal forces applied at each floor and roof level and their counterbalanced resultants at the column bases to make foundations reaction free in the absence of directly applied loads.

Allowance is made for disregarding frame imperfection when directly applied loads produce, at the base of each storey, a horizontal reaction which is larger than 10% of the vertical one.

4.2 Equivalent member imperfection

Though all members are presumably imperfect, member imperfections may be considered for compression members only. There are two types of member imperfections:

- A "bow imperfection" results from the fabrication process. It is assumed to be sinusoidal and present once, and only once, in either of the buckling planes. Eurocode 3 does not specify the magnitude of the reference bow imperfection though it is worth knowing it. People aware of how the European buckling curves were developed shall remember that the amplitude of the bow imperfection was drawn from measurements and taken conservatively as a lower characteristic measured value rounded to 1/1000 of the member length.
- Structural imperfections consist in residual direct stresses due to a non uniform cooling down at the end of the rolling or welding process. In each cross-section, these residual stresses have a self-equilibrated distribution, the shape and magnitude of which resulted from measurements. It is accepted that the effects of residual stresses include those due to the scatter of the yield strength. Several characteristic distributions are selected as representative of sets of structural shapes which differ mainly by the fabrication process, the massiveness of the cross-section and the thickness of the plate elements.

Both types of member imperfections forestall the very first material yielding. Further yielding reduces the elastic stiffness of the frame and increases the sway displacements, and therefore the so-called 2nd order effects. Subsequently the resistance of members prone to buckling is reduced by an amount which is the more significant as the magnitude of imperfections is large.

For design purposes explicit allowance for residual stresses means a tedious task and, in addition, is usually not empowered by commercial software programs. Therefore Eurocode 3 introduces the concept of “equivalent bow imperfection” as a single geometric imperfection having presumably the same detrimental effect as above two types of coincident member imperfections. Magnitudes of the equivalent bow imperfection are given in EN 1993-1-1; they range from L/350 up to L/150 for elastic analysis or L/300 up to L/100 for plastic analysis. The difference between the above sets of amplitude is physically understandable; indeed the less restrictive the buckling limit state criterion, the larger the equivalent bow imperfection. Deviation of the equivalent member imperfection from the reference value L/1000 of the sole geometric imperfection thus reflects the influence of the residual stresses.

In principle, equivalent bow imperfections should be modeled in view of global analysis. However no recommendation is given in EN 1993-1-1 regarding the direction and possibly the distribution of equivalent bow imperfections of members within the frame. The designer is only bound to model and discretize any compression member which experiences a design axial load N_{Ed} larger than 25% of its elastic critical load N_{cr}; that criterion corresponds to unusually slender columns. In practice, most members do not need to be modeled as imperfect ones and are thus kept straight; of course global analysis is then unable to detect any member instability so that buckling resistance of individual columns shall be checked independently.

Trainees make rather easily their mind to these imperfections provided that their origin is clearly and physically explained. Those who are still ASD dependent may however feel disturbed by the fact that residual stresses – about which they just heard – do not influence the ultimate cross-section resistance but well the buckling resistance of members.

5. Joint classification and modeling

Joints are a matter to which much attention has been paid during the last thirty years with the consequence of significant progress regarding the structural joint design. A part of the new knowledge is included in Part 1-8 of Eurocode 3 (CEN 2005b); another part that was meant not worth being codified has to be found in several published documents (Maquoi 1998, ECCS 2009). Recollecting all this information and making one’s mind without further assistance is not easy. In contrast, well conceived trainings help much to demystify the new concepts and show their workability in practice. The main innovations are briefly examined below.

5.1 Joint classification

Where a beam is attached to a column, not only the connection but a whole surrounding area, designated “joint”, is involved in the transfer of internal forces from one member to the other one. For global analysis, members are modeled by their longitudinal axis and joints are localized directly adjacent to the intersection of connected members. In a statically indeterminate frame, the elastic distribution of internal forces due to a given combination of actions is governed by the distribution of elastic stiffness throughout the structure, i.e. EI/L for members and S_j for joints. In the “traditional design approach”, the joints are assumed either rigid ($S_j = \infty$) or simple ($S_j = 0$) and joints are detailed at a later stage so as to comply with joint resistance requirement and assumed joint stiffness. In some countries, members and joints are designed by different staffs or even different companies.

In accordance with EN 1993-1-8, a joint is seen as a mechanical model involving as many springs as there are individual “joint components”. Each component is characterized in terms of strength, elastic stiffness and deformation capacity. Then component properties are “assembled” in full consistency with the mechanical model so as to get similar properties for the joint. Joint characterization by a hand calculation is a very tedious task. Fortunately appropriate software (CoP 2012) is available for general use; more especially it enables to have a deep insight on the properties of all the joint components and on the respective contributions of the latter to the joint structural response. Thus the efficiency of joint stiffeners and the consequence of removing some of them can be easily investigated, making the “angststeifung” practice (see §3.2) irrelevant. Also optimization of joints becomes child’s play. Above referred software was developed by the drafters of EN 1993-1-8 – therefore the trust in it – and intensively used to produce useful catalogues of standard joints (Weynand et al 2012, SCI 2010, Senin 2007).

In view of elastic global analysis, joints can thus be given a similar status as members. Each of them is modeled by a single resultant spring characterized by its elastic joint stiffness (initial stiffness). That “consistent approach” includes the “traditional approach”: a joint is considered rigid (resp. simple) under the reservation that its elastic stiffness lays within a certain stiffness range. The boundaries of that range are $S_j = \infty$ (resp. $S_j = 0$) and another limit stiffness, the magnitude of which depends on the ratio between joint stiffness S_j and beam stiffness EI_b/L_b. Accordingly implicit allowance is made for some inaccuracy on the magnitude of internal forces compared to those obtained with the assumption of rigid (resp. simple) joints. Of course resistance and assumed stiffness have still to be proven after global analysis. The “consistent design approach” does no longer focus on rigid or pinned joints only. When initial joint stiffness is not within either range, the joint is said “semi-continuous” and it is modeled as such; because elastic global analysis is common practice, only stiffness property is concerned so that the terminology “semi-rigid” is used instead.

In contrast with a current belief, the classification of structural joints as rigid, semi-rigid or simple is not an intrinsic joint property but relatively to the connected member and to the type of frame. Indeed the joint is the stiffer as the stiffness EI_b/L_b of the connected member decreases, i.e. the length L_b of the member (with a given inertia I_b of the connected member) increases.

5.2 Joint modeling

The M-ϕ moment-rotation curve of a structural shape bent about its major principal axis is assumed to remain elastic till the moment resistance of the cross-section is reached; this simplification is acceptable because the range of non linearity is quite small. In contrast, a joint is a complex assembly of individual components having unlike behavior so that its actual M_j-ϕ_j structural response is highly non linear and exhibits significant strain hardening. So-called plastic joint moment resistance is a very conservative assessment of the actual ultimate joint resistance. Also elastic joint moment resistance is significantly lower than plastic joint moment resistance; according to the EN 1993-1-8, it amounts roughly 2/3 of the latter.

Of course the initial joint stiffness is only representative of the joint behavior within the elastic range. That influences the further design procedure. The designer is indeed offered some choice:

- Either he performs the elastic global analysis with initial joint stiffness S_j and the moment experienced by the joint shall then be compared to the elastic joint moment resistance;
- Or he is willing to benefit from the plastic joint moment resistance but has then to account for the significant effect of non linear joint behavior on the distribution of internal forces at ULS; according to EN 1993-1-8, global analysis has to be performed with a pseudo-secant joint stiffness S_j/η. Should SLS not be fulfilled at the level of service loads, global analysis may be started again with initial joint stiffness S_j; then, besides the regular SLS checks, it shall be verified that the elastic joint moment resistance is not exhausted in service conditions.

5.3 Operating the "consistent approach"

In the "consistent approach", the designer pays due consideration, as soon as the preliminary design stage, to the properties of both members and joints so as to conduct global analysis accordingly. That approach is not incompatible with the possible, and often customary, separation of the design tasks, relative respectively to members and joints. Two situations can be encountered.

<u>Share of the whole design task</u>

Two parties are responsible for member design, on the one hand, and joint design, on the other hand; for sake of simplicity, they are respectively designated "engineer" and "fabricator". The "initial guess" is a valuable non-codified tool for the preliminary design of structural joints; it is available in published documents (Maquoi et al 1998). An approximate magnitude $S_{j,app}$ of the initial stiffness of each joint is determined by the "engineer" on the sole base of a – to his mind – credible type of joint with regard to preliminarily designed members and to the single-sided or double-sided joint configuration. A stiffness range is attached to $S_{j,app}$ so as to make due allowance for a permissible inaccuracy in the internal forces. The boundaries of that range result from simple expressions which involve the approximate joint stiffness $S_{j,app}$ and the beam stiffness EI_b/L_b. Once global analysis and final member design are satisfactorily completed, the "engineer" provides the "fabricator" with the internal forces to be transferred by each joint as well as the relevant joint stiffness boundaries. Then the "fabricator" first opts for a type of joint, which is not necessarily the one imagined by the "engineer" and results from technical or economical considerations (available equipment, preference for fabrication, easiness for erection, requirements imposed by finishing…); then he prepares the joint detailing so as to fulfill the joint resistance criteria and get an initial joint stiffness which is within the targeted stiffness range provided by the "engineer". That range is large and a large variety of suitably detailed joints surely lay within it. As said above, the detailed joint characterization is hopefully conducted with appropriate software or design aids.

<u>No share of the whole design task</u>

A single party – the "engineer" or alternatively the "fabricator" – is responsible for both global analysis and member and joint design. First he makes a preliminary sizing of members; on that base and possibly with regards to requirements of any kind, he opts for joint type(s) and proceeds to their complete detailing and further characterization. Global analysis is activated. At its end, the "engineer" – or alternatively the "fabricator" – runs all the final design checks for members and joints. Of course advantage may still be taken of the admissible stiffness range as above when possible further adjustment of joint detailing is required.

5.4 Benefit to be drawn from innovative joint design

In building projects, applying new concepts for the structural joint design is widely recognized as a potential source of significant economy. Fabrication of joints is expensive because labor consuming and requiring manipulations in the workshop; it significantly contributes the total cost of the bearing structural system. Thus important cost savings can result from a good mastery of the whole joint context. This approach is relatively easy to integrate into everyday practice by mainly using available design aids tools and aids as well as adequate documentation. However many designers are still reluctant to change their mind and habits regarding joint design and thus do not give due consideration to the joint properties at the very first steps of the design process. In other words, they stop using the latest code to what they did in the past. This "all or nothing" attitude prevents them from drawing benefits offered by the consistent approach in many circumstances:

- Rigid joints no longer need to be strongly stiffened.
- Compared to rigid joints, semi-rigid joints are hardly stiffened or not stiffened at all. When necessary – for instance, when SLS become determinative – the larger sway displacements can be compensated by using slightly stiffer columns while beam sizes do not change.
- Compared to pinned joints, semi-rigid joints reduce the sagging bending moments in the beams so that slightly lighter beam size is expectable. Of course columns are then subjected to additional bending, the adverse effect of which on column stability is more than counterbalanced by the stabilizing effect due to end restraints provided by the joints. As a consequence, the assumption of axial compression only may still be used for the preliminary sizing of columns. For similar reason the eccentricity – with regard to the column axis – in simple beam-to-column joints be disregarded as long as the joint is not a mechanical hinge (pin connection).

In a more general way, it is worth adjusting the joint stiffness so as to strike the best balance between the cost of joints and the cost of beams and columns. In braced frames, semi-rigid joints are more costly than pinned joints but are likely to reduce beam sizes. In unbraced frames, substituting rigid joints by less costly semi-rigid joints implies a slight increase of column sizes. Economical analysis conducted some years ago in Western Europe and North America has concluded to savings peaking at least 10% and 5% of the cost of the bearing structure (including painting, transportation and erection) for respectively unbraced and braced frames. Of course these gains are achievable only if:

- The cost of the structure is not established on an outright price/unit weight irrespectively of unlike workmanship contributions;
- Column buckling length is not too roughly assessed (for instance taken as the system length in rigid frames) but well with due account taken of end restraints.

Also rigid joints can often be achieved without requiring costly stiffeners.

6. Frame global analysis

Global analysis is an important step of the design process. However, Eurocode 3 pays very few – if not no – attention to it. That theoretical matter is supposed known and to be fully mastered by the designers; the reality is however quite else, especially when plasticity and 2nd order effects are concerned. Also global analysis is operated once the loads and load combinations are known, and the structural elements are preliminary sized and characterized on the base of professional experience and/or relevant routine methods. For each load combination, global analysis provides the magnitude of internal forces at reference cross-sections and their distribution within the whole frame. It is conducted according to either first-order (1) or second-order (2) theory, on the one hand, and with account taken of either elastic (E) or plastic (P) behavior, on the other hand.

As a result there are four methods of global analysis: 1E, 1P, 2E, 2P. These methods are not equivalent because they do not internally cope with same resistance problems. For instance, when elastic global analysis is performed, the resistance of cross-sections has still to be checked independently; moreover member buckling resistance cannot be cared for when elastic global analysis is first-order. Also methods which account for geometric non linearity (2) or/and material non linearity (P) are iterative and thus more time consuming. Despite the availability of software's with extended capabilities, many "average" designers are still attracted by using the most simple and efficient method of global analysis with regards to their own needs, with due consideration to the consequence of their choice on further design checks.

Method 1E (1^{st} order elastic) was and is still the most practiced. However, in accordance with Eurocode 3, that simplest approach might be insufficient. During the trainings, designers must be given guidance so that they are able to balance their design work, to their liking, between global analysis and further design checks. Also the relation between the method of global analysis and the extent of further design checks, including the way to conduct the latter, has to be stressed out.

6.1 First-order or second-order theory?

In what follows, it is implicitly assumed there is no need for modeling member equivalent imperfections. As a consequence, the terminology "first-order" or "second-order" used herein, as well as "second-order effects", are relative to the global frame behavior. In 1^{st} order theory, equilibrium is expressed with regard to the initial configuration of the non loaded frame. In contrast, 2^{nd} order theory refers to the deflected configuration; more especially sway displacements result in additional destabilizing internal forces – designated as 2^{nd} order P-Δ effects – in presence of gravity loads.

First-order analysis is admissible only when sway displacements are small enough for 2^{nd} order effects being negligible; in accordance with EN 1993-1-1, it is therefore required that elastic critical buckling load V_{cr} of the frame according to sway buckling mode is not less than ten times the resultant V_{Ed} of design gravity loads, i.e. $\alpha_{cr} \equiv V_{cr}/V_{Ed} \geq 10$. Frames fulfilling with above condition are said "non sway" or "rigid"; if not, they are said "sway" or "flexible" and then 2^{nd} order effects must be taken into account for further member and cross-section resistance checks.

Second-order analysis may be used without restriction. It provides internal forces which include 2^{nd} order effects. As an alternative, these forces are assessed by magnifying appropriately some results of a 1^{st} order global analysis. Several ways are available for that purpose; they differ by the expected degree of accuracy. Possible non-conservativeness of the internal forces obtained accordingly (loading term) are counterbalanced by safe – and sometimes unduly safe – assumptions regarding the buckling length of compression members (resistance term). It may thus happen that a further member resistance check has to be run with different data depending on the way the 2^{nd} order effects are reflected. The designers' attention shall be drawn to this latter fact as well as on some additional aspects:

– "Rigid" or "flexible" is not an intrinsic frame property; indeed that depends on the magnitude

of the resultant gravity load and therefore on the combination of actions under consideration.
- Accordingly it may happen that a given frame is flexible under such a combination but rigid under another one.
- The critical load involved in the frame classification is usually obtained by bifurcation analysis, which gives eigenvalues and buckling eigenmodes. The elastic critical amplifier α_{cr} to be selected for classification purposes must be relative to the sway buckling mode; thus it does not necessarily correspond to the lowest of above eigenvalues.
- For portal frames with shallow roof slopes and multistorey plane frames in buildings, the elastic critical amplifier α_{cr} is approached with a good accuracy as the smallest of the elastic critical buckling amplifiers calculated in each storey on the base of the sway displacements obtained from 1st order elastic analysis.
- A steel frame is said braced when a bracing system does exist and contributes at least an 80% reduction of the sway displacement of the frame without its bracing system. Accordingly a braced frame is not *ipso facto* rigid. Similarly an unbraced frame is often but not always flexible.

6.2 Elastic or plastic global analysis?

Elastic global analysis is allowed without restriction. Steel material as well as member and joint behavior are assumed indefinitely elastic and no peculiarly rotation capacity is required. Clearly such a global analysis is not able to care for cross-section resistance. The latter shall be checked independently.

Plastic global analysis is subjected to specific requirements regarding mainly material properties and cross-section classification, i.e. the demand in rotation capacity is much larger than above. In the daily practice, it is conducted with a simplified elastic-perfectly plastic constitutive law. It copes internally with cross-section resistance.

6.3 Inter-relation between global analysis and subsequent design checks

Under the reservation of appropriate modeling, 2nd order plastic global analysis is of course the most powerful method. Should indeed both frame and member imperfections be modeled and discretized accordingly, the internal forces are known and duly include 2nd order geometric effects and material non linearity (yielding). The design consists in verifying that the deflected shape of the frame under the design load combination is associated to a converging state of equilibrium. Otherwise speaking, global analysis copes internally with almost all the cross-section and in-plane member limit states so that further design checks reduce to nearly nothing. The checks relative to out-of-plane behavior shall be performed separately when, as it is usual, the structure is modeled as a 2D one.

The use of any other method of global analysis results in a more or less large number of design checks, which shall be conducted once the internal forces are known. These design checks are the main scope of the European code. They shall only address the aspects that global analysis did not yet cope with, either explicitly or implicitly. In view of giving a physical meaning to these checks and to the way they are performed, the designers should always keep in mind the following elementary principles:
- Provided member imperfections are duly modeled and discretized and account is taken of elastic 2nd order effects, the check of the member buckling resistance – often designated

stability check – reduces to cross-section resistance along the member under the action of internal forces including these 2nd order effects.

– The more sophisticated the global analysis, the less and simpler the ULS design checks still required at the end of global analysis. The total effort involved in the design process is thus shared in uneven proportions between, respectively, global analysis and design checks depending on the designer's choice regarding global analysis.

6.4. Current practice

There are several reasons why plastic global analysis is not expected to be or become the current practice; they are not worth being commented herein. However it may not be hidden that plastic global analysis can be very helpful in some circumstances: to clear up a structural collapse, or to assign an existing elastically designed structure for supporting higher loads, or to design one-storey industrial buildings for which SLS criteria are usually more liberal. Because plastic global analysis is only rewarding in a restricted range of situations, it is disregarded herein so that only (1st order or 2nd order) elastic global analysis is mainly referred to in what follows.

The range of validity of 1st order elastic global analysis is limited by the onset of either the first plastic hinge or the very first yielding; cross-section resistance may be checked either plastically or elastically depending on the class of the cross-section. In addition it shall be proved that the frame as a whole and its composing members remain stable under the action of the design loads; therefore the need for checking in-plane and out-of-plane stability of both the frame and its members under the action of internal forces exclusive of 2nd order effects. First-order elastic analysis provides a safe basis for design as long as the predicted response of the frame deviates only slightly from the actual response over a large range of loading. It is the case of rigid frames at ULS because they have a small sensitivity to sway displacements. Also 1st order elastic analysis provides generally a realistic response (in terms of displacements and/or deflections) of frames and their elements under service loads.

Internal forces obtained with 2nd order elastic global analysis directly include all 2nd order effects if both frame and member imperfections are modeled and the frame is discretized accordingly; then only cross-section resistance has still to be checked. Should member imperfections not be modeled, then member buckling resistance shall be checked in addition.

With 2nd order elastic analysis, the efficient and widely used "principle of superposition" (internal forces, displacements) is no more workable; thus global analysis has to be conducted as many times as there are load combinations to be examined. That might be a severe drawback for some designers. In order to alleviate it, alternative approaches may be contemplated; in a first step, a 1st order global analysis is performed and, in a second step, the "sway moments" are magnified by means of an amplification factor so as to get an assessment of the internal forces including indirectly 2nd order effects. Sway moments are those moments resulting from the horizontal translation of the top of a storey with respect to its bottom; they are due to the horizontal loads but can also result from vertical loads when either the frame or the loading is asymmetrical. The amplification factor is computed as $\alpha_{cr}/(\alpha_{cr} - 1)$. When α_{cr} is lower than 4, the sway moment amplification method is not allowed because of insufficient accuracy; then, 2nd order global analysis is required.

7. Member buckling resistance

The choice of a method of global analysis is subordinated to some requirements specified in Eurocode 3. Also personal preference, software availability and desired balance of efforts dedicated respectively to global analysis and further ULS design checks can be determinative.

The major part of LSD design codes is devoted to "ultimate limit states", a heading which mostly addresses two main subjects: resistance of cross-sections and buckling resistance of members. Design criteria which must be fulfilled for the design being acceptable are given.

Amongst the designers there is still a strong common belief that both subjects must be visited and the design criteria evenly checked, irrespectively of the type of global analysis. In this respect, it shall be tireless reminded that checking member buckling resistance of a member is nothing else than checking, along the member, cross-sectional resistance for actual internal forces, i.e. including 2^{nd} order effects (it is reminded that the analytical expression of the European column buckling curves was developed from that principle). Otherwise speaking, should these internal forces be known and correspond to a state of equilibrium, no further stability check would be required. In contrast, the sources of 2^{nd} order effects which are not taken into consideration at the stage of global analysis are the cause of further design checks.

Yet it was said above that equivalent geometric imperfection of compression members is usually not modeled in the structural model used for global analysis. The corollary is that buckling member resistance has to be checked separately. Two main approaches are available for that purpose; they are commented below for the case of a pin-ended column subjected to uniform axial compression.

7.1 Check of column buckling by member global analysis

A 2^{nd} order analysis of the column fitted with the equivalent imperfection is conducted step by step up to reaching the design axial force. The equivalent imperfection makes that each cross-section is subjected to axial force and bending moment including 2^{nd} order effects. That means that the effects of equivalent imperfection are explicitly included in the loading term. When plastic 2^{nd} order plastic analysis is performed, the column is stable as long as the amplitude of the deflected configuration has a finite magnitude. Nowhere the plastic cross-sectional resistance under combined bending and axial force is reached and no further check is required. In contrast, an elastic 2^{nd} order analysis does not handle material yielding; therefore the need for separate cross-sectional resistance checks. Of course column buckling resistance is the magnitude of the axial force which corresponds to the exhaustion of the cross-sectional resistance.

Though that iterative approach is workable, it is expectedly not common practice. The engineering approach described below does largely prevail.

7.2 Direct engineering check of column buckling

In the engineering approach of column buckling, reference is made to the perfectly straight column. Column buckling resistance is given as the cross-sectional resistance reduced by a reduction factor χ, which is a function of: a) the reduced slenderness $\bar{\lambda}$, and b) the imperfection parameter that reflects the detrimental influence of member imperfection. Effects of both material yielding and equivalent imperfection are implicitly accounted for through the selection of the appropriate column buckling curve; *in fine* they are accounted for in the resistance term. The column is stable and its design satisfactory when its (design) buckling resistance obtained accordingly is at least equal to the (design) value of the axial force in the column.

Fulfilling the member buckling resistance for an axially loaded compression member is clearly more severe than checking the cross-section for axial load only. That makes the latter check useless. In contrast, such a conclusion does not hold in any generality, especially when the member is subjected to coincident axial compression N and bending moment distribution $M(x)$ (beam-column). The member design check involves the concept of equivalent bending moment, i.e. a uniform 1st order moment distribution along the member which is deemed to produce same maximum 2nd order effects as the actual 1st order moment distribution. Member design check handles the resistance of the "critical" cross-section within the member length whereas the coincident loading ($N + M$) at either end section can be more critical. Therefore, in contrast with the axially loaded compression member, the stability checks of a beam-column shall necessarily be supplemented by cross-section resistance checks at member ends.

Moreover the buckling length to be used in the stability checks depends on what is included in the internal forces. Common practice is to use buckling lengths relative to frames the stories of which are either allowed or not allowed to sway. Nowadays the questions are more precisely: a) are the 2nd order effects sufficiently large (sway frames) or not (rigid frames) for them having to be accounted for? and, if they are, b) how are these effects included in the internal forces? The answers are as follows:

- Non sway (or rigid) frames: the buckling lengths are those for the non sway buckling mode because 2nd order effects are negligible and disregarded;
- Sway (or flexible) frames with 2nd order elastic global analysis: the buckling lengths are those for the non sway buckling mode because sway effects are directly included in the internal forces obtained from global analysis;
- Sway (or flexible) frames with 1st order analysis and sway moments further amplified by $\alpha_{cr}/(\alpha_{cr} - 1)$ provided $\alpha_{cr} > 4$: the buckling lengths are those corresponding to the non sway buckling mode because sway effects are accounted for indirectly and with sufficient accuracy, by amplifying the sway moments.

Design charts, as well as their analytical expressions, do exist to evaluate the buckling length factor as a function of the end restraint factors at both member ends. Usually one of these charts is said relative to buckling lengths in "rigid" frames and the other one to buckling lengths in "flexible" frames. These charts may be kept but with henceforth reference to respectively "non sway buckling mode" and "sway buckling mode". This slight difference of appellation is of paramount importance. End restraint factors are comprised between 0 and 1. They are expressed in terms of beam and column stiffness only. When semi-rigid joints, the restraint factors still hold but beam stiffness shall be understood as the stiffness of an "equivalent beam" composed of the beam properly and the elastic spring representative of the joint.

Inter-relation between the determination of internal forces and buckling lengths was clearly commented on in the preliminary draft ENV of Eurocode 3. Unexpectedly this is no longer the case in the final EN draft on the grounds that structural stability is part of the technical knowledge of the "average" designer and therefore not worth being codified. That hiatus is the more regrettable as a more correct evaluation of buckling lengths is understandably a source of significant economy in column design.

The assessment of buckling resistance according to the so-called direct engineering approach is closely linked with the selection of the appropriate buckling curve. Whatever the type of instability, the relevant resistance curve has two bounds: a) cross-section resistance in the range of small slenderness where effects of instability are about zero and b) elastic critical (postcritical for plates with adequate aspect ratio) resistance in the range of the large slenderness where yielding does not significantly spread out. Yielding and instability interact more in the range of medium slenderness. For member buckling (column buckling, lateral torsional buckling) there are several buckling curves, which thus account for uneven detrimental effects of imperfections depending on several parameters (type of fabrication, type of shape, cross-sectional massiveness, flange thickness, direction of buckling, yield strength).

Surprisingly, for given conditions except the yield strength, the influence of the yield strength is not continuous. That is worth being commented on. When the European buckling curves were developed and calibrated, the magnitude of the residual stress distribution in a specific structural shape was expressed as a constant proportion of the material yield strength. At that time S235 was by far the most popular steel grade – compared to S275 and almost S355 – and experiments were mostly – though not exclusively – performed on sections made of so-called mild steel S235. Therefore an appropriate buckling curve was – and is still – attached to a given structural shape irrespectively of the yield strength in the range up to S355. Nowadays it is well known that the magnitude of the residual stress is closer to an absolute value. Accordingly the larger the yield strength, the smaller the relative detrimental influence of the residual stresses on the column buckling resistance and, consequently, the more favorable the buckling curve. That real physical behavior is not recognized by the European buckling curves except for latest steel grade S460. What looked a conservative attitude in the past is today exceptionally distorted for reasons which have little to do with science. That anomaly may not be passed over in silence when it is a matter of explaining the physical background of the European column buckling curves.

8. Briefly on some other specific aspects

8.1 About reference axes

In some national standards, the principal axes of structural shapes were *x* for the major axis and *y* for the minor axis. The latest code reserves the *x* axis for the longitudinal direction while *y* and *z* are used for the cross-section major and minor axis respectively. Of course this change is a question of detail but it is source of disturbance when moving to Eurocode 3.

8.2 About robustness

In clause 2.1(4) of EN 1990 (CEN 2002a), it is specified that “a structure shall be designed and executed in such a way that it will not be damaged by events – such as explosion, impact and the consequences of human errors – to an extent disproportionate to the original cause”. The events to be taken into account are those agreed for an individual project with the client and the relevant authority.

Any structure designed in accordance with EN 1993 has to comply with that robustness requirement. However many designers are still the more unaware of that stringent principle as very few applications rules to fulfill it are provided. Thus the qualitative aspects of robustness are not accompanied with quantitative tools; that is a severe deficiency of the code.

For lack of scenarios to be agreed with the client, the intensive European research work in the field of robustness is more on the track of referring to the alternative load path method, taking due account of membrane behavior of the beams and providing members,, and more especially joints with large ductility by means of adequate joint design and detailing.

8.3 About the static theorem of the plasticity theory

Under the heading “Resistance of cross-sections – General” of EN 1993-1-1 (CEN 2005a), one finds the clause 6.2.1(6): “The plastic resistance of cross sections should be verified by finding a stress distribution which is in equilibrium with the internal forces and moments without exceeding the yield strength. This stress distribution should be compatible with the associated plastic deformations”.

That statement simply renders the static theorem of the plasticity theory. Expressed in that way, it is understandable by initiates only so that few designers will probably take advantage of it. That theorem pertinently illustrates the paramount role of material ductility.

Trainings and education should more thoroughly explain above specification and point out, by means of worked examples, how much that clause, used as a design tool, is helpful and powerful.

8.4 About effects of plasticity when combined axial force and bending

In strictly elastic design, the resistance of cross sections is governed by the sum of elastic direct stresses due respectively to axial force and uniaxial/biaxial bending moment. Provided the class of section is adequate, the plastic resistance to such coincident forces is significantly larger, especially in the range where both axial force and bending moment, normalized with respect to the relevant resistances, interact the most.

A similar conclusion holds for members too despite the involvement, in the so-called beam-column interaction formulae, of penalty factors reflecting the effects of member buckling. As much as it is permitted, the designers concerned with the project economy should draw advantage of plasticity. In this respect, the innovative formulae of EN 1993-1-1, which are dedicated to the resistance checks of members subjected to coincident axial force and bending, deserves a special mention. The format of the design formulae is similar to what it was in previous ones but the involved coefficients – which aim at reflecting several types of interactions – are much better assessed. More than enabling an excellent agreement between experimental and numerical results, they are almost source of significant economy. How to extract the member from the structure and check correctly its resistance is delicate. In this respect, complete documentation and worked examples may be found in a dedicated publication (ECCS 2006) and appropriate software is freely downloadable on the website of the Graz Technical University (Semi-Comp 2012).

In the latter bibliographical reference, a physically validated proposal is made for a continuous transition resistance model for semi-compact (Class 3) cross-sections so as to remove the sudden jump (from plastic to elastic level) of resistance at the Class 2 to Class 3 border.

9. Conclusive remarks

Eurocode 3 is going to become the single code in force for steel structures amid the European countries. It includes the recent knowledge and research results. Compared to some national standards, it represents significant changes. Material yielding capacities are valorized and ductility plays a more important role. The design procedure is not one-way but well multi-ways. Especially the design of joints is quite innovative. Only the design checks are concerned with the code which remains silent on the effects of joint behavior on the global analysis.

The common attitude should be *"As you must do it, so better make the best of it"*. Therefore education has to be revisited so as to be suitably adapted to the new philosophy of design while senior designers have to change their mind and past habits and get new expertise through adequate training and information.

Without being appropriately educated and/or trained to the latest code, future and present designers fail to understand clearly and completely the inter-relation between: i) the way the internal forces are determined, ii) the effects yet accounted for at this stage, iii) the extent of the still required design checks, and iv) the way these checks are conducted accordingly. Actually the latest steel codes are generally said more time-consuming than former ones and the code developers are burdened with a useless search of complexity. Such criticism however vanishes once a sufficient mastery is acquired, knowing that nothing is gained effortless.

Therefore three concluding remarks are as follows:

- The teaching profession has to identify itself with the design profession so as to deliver an improved knowledge of the inter-relation between global analysis and design;
- When conducting trainings to latest design codes, stressing above inter-relations and commenting on the impact, on the design checks, of an appropriate choice of a method of global analysis it is much more worth than just focusing on the differences between past and present formulae to check a specified phenomenon.
- Training to the latest codes shall address worked examples with appropriate critical comments on the relevant specifications and their physical interpretation, and inform on existing appropriate documentation which supplement very usefully the use of code specifications.

Any effort aiming at the most rewarding use of the latest steel codes should be inspired by:

"Teach me and I forget,
Tell me and I remember,
Involve me and I learn"

an idiom that is true for to many fields of human activity.

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