THE NEW CHAPTER 9 OF EUROCODE 8 PART 3 FOR SEISMIC ASSESSMENT OF EXISTING STEEL STRUCTURES

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1. SUMMARY

The seismic assessment of existing steel structures necessitates the development of practice-oriented engineering models for use in nonlinear static and dynamic analysis procedures. These models should encompass the full range of the anticipated nonlinear behaviour of structural steel elements. Within such a context, this paper provides an overview of the newly developed Chapter 9 of Eurocode 8 Part 3. This chapter includes a comprehensive list of component models that idealize the nonlinear behaviour of the primary and secondary structural steel elements, including columns, steel and composite steel beams, beam-to-column joints, and steel bracings among others. The resistance and deformation models for seismic assessment rely on experimental data, which were systematically collected and curated in structural performance databases over the past two decades within the research group of the author. The paper summarizes some of these models for the nonlinear modeling of both code compliant and noncompliant structural steel elements. A publicly available web-based module has been developed that facilitates the use of the proposed models.

2. INTRODUCTION

Seismic assessment of new and existing structures relies on the development of practice-oriented component models that idealize the nonlinear behaviour of structural elements. These models are then used either in nonlinear static (i.e., pushover) or dynamic analysis procedures to facilitate the computation of engineering demand parameters (EDPs) and compare those with formally established limits, which are often called acceptance criteria (or verification of limit states). In North America, the current state-of-practice in seismic assessment evaluation of existing structures follows the guidelines summarized in ASCE 41 [1]. This document is an updated version of FEMA 356 [2]. More recently, the American Institute of Steel Construction (AISC) formulated separate guidelines [3] for the seismic assessment of existing steel structures. While available component models could be of high fidelity [4]–[7], the primary focus herein is on practice-oriented models that describe the general backbone curve of structural steel elements. Such a generalized force-deformation (Q-δ) curve is illustrated in Figure 1. In this case, the force, Q could be the axial, shear or flexural resistance of the structural steel element of interest. The deformation, δ could represent the axial displacement, shear deformation or chord rotation of the same element.

The basic input model parameters of the backbone curve can be inferred by a combination of first principles of mechanics and empirical formulations that are derived from pertinent experimental data.

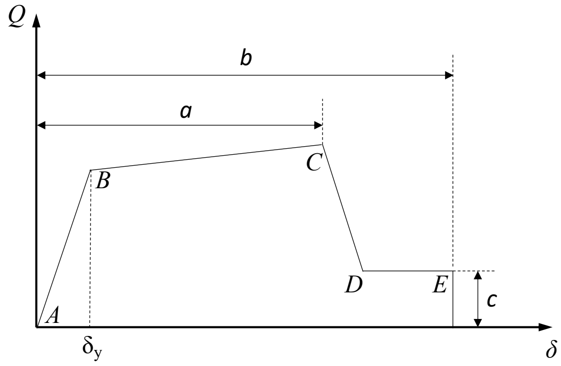

Fig. 1 Backbone curve for component modelling of structural steel elements.

In Europe, existing guidelines have been incorporated in Eurocode 8 Part 3 [8] with an emphasis at the deformation capacities at yield and ultimate of structural elements. While existing modelling guidelines in Europe are fairly well developed for reinforced concrete elements [9], those for steel and composite-steel structures are quite limited and fairly simplistic compared to the current state-of-the-art [10]–[13]. This was attributed to the lack of available experimental data that fully describe critical limit states in structural steel elements. In the last two decades the author has put considerable effort to establish structural performance databases that systematically document available experiments on structural steel materials [14] and elements [12], [13], [15]–[18]. This data along with corroborating numerical studies [19] has been taken into consideration for the development of comprehensive guidelines that have been formalized within Clause 9 of the new Eurocode 8 Part 3 [20]. These guidelines serve for the seismic assessment of existing steel structures as well as those for primary and secondary seismic and composite-steel concrete elements that conform with Ductility Class (DC) 2 and DC3 structures according to prEN1998-1-2:2022 [21].

This paper summarizes part of this work with an emphasis on the structural steel material properties and on the resistance and deformation models for assessment as discussed in [20]. Clause 9 of EC8 Part 3 considers other information regarding the identification of geometry, construction details and materials for existing steel structures (i.e., buildings and bridges). However, these are not presented herein due to brevity.

3. STRUCTURAL STEEL MATERIALS

Table 1 summarizes structural steel materials used in buildings and bridges that are classified based on the respective year of production. The same table provides indicative values of the nominal yield strength, f_y and an ultimate tensile strength, f_u. Noteworthy stating that cast iron made before 1920 to resist tensile stresses should not be considered for use in seismic retrofitting. Moreover, the yield and ultimate tensile strength of structural steel materials between 1955 and 1993 may be assumed to be the same as those prior to 1955 or after 1993 depending on the identified steel material grade. The modulus of elasticity, E for cast and wrought iron should be taken as 140 GPa and 200 GPa, respectively. Other values may be considered when material testing is conducted from in-situ sampling, or pertinent documentation.

Table 1. Nominal yield and ultimate tensile strength of structural steel materials

Date of Production	Material Grade	Nominal yield strength, f_y [MPa]	Nominal tensile strength, f_u [MPa]
Before 1901	Pre-standardized structural steel	70	120
1850-1900	Wrought iron and homogeneous iron	220	320
Before 1920	Cast iron	Not applicable	Not applicable
1900-1940	Homogeneous iron	235	335
1925-1955	Mild steel	235	360
1993 - current	S235	According to [22](see Table 5.1)	According to [22](see Table 5.1)
	S275		
	S355		
	S420		
	S460		
1993 - current	S260	According to [22](see Table 5.2)	According to [22](see Table 5.2)
	S315		
	S355		
	S420		

Rivetted steel construction is quite common in Europe. As such, default yield and ultimate strength values for rivets are provided in Table 2 depending on the year of construction. Similar values are made available for fasteners and weld metals where the filler metal is either listed or missed.

Table 2. Nominal yield and ultimate tensile strength for rivets

Date of Production	Material Grade	Nominal yield strength, f_y [MPa]	Nominal ultimate strength, f_u [MPa]
1850-1900	Wrought iron	220	320
1890-1940	Homogeneous iron	220	320
From 1925	Mild steel	335	350

4. OVERVIEW OF METHODOLOGY FOR THE DEVELOPMENT OF RESISTANCE AND DEFORMATION MODELS

The methodology for the development of generalized force-deformation relationships for the seismic assessment of steel structures relies on experimental data that has been systematically digitized, documented and curated in structural performance databases within the research group of the author throughout the years. A fully searchable web-based version of these databases with interesting data visualization features has been made publicly available (https://resslab-hub.epfl.ch/). These feature data on structural steel materials, beam-to-column connections, steel columns and steel bracings among others. Figures 2a and b illustrate examples of typical cyclic load – deformation relationships for a steel beam (see Figure 2a) and a steel bracing (see Figure 2b). The first cycle envelope is first deduced for each one of the assembled test data as shown in the same figures. This envelope is then approximated with a linear idealization as shown in the same figure based on rules that have been developed in prior work [10].

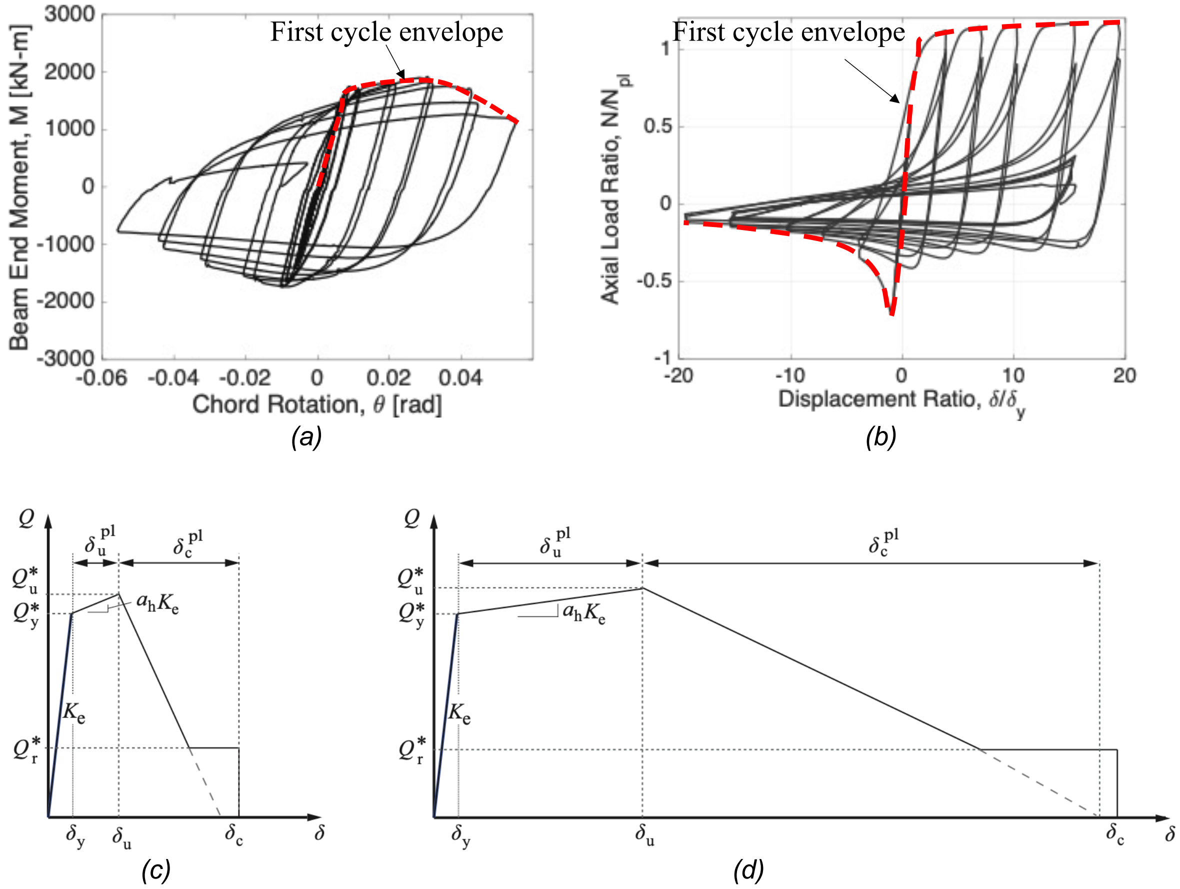

Fig. 2 General definition of piecewise linear load-deformation relationship for steel and composite-steel concrete elements; (a) limited ductile behaviour; (b) ductile behaviour.

The resistance and deformation parameters of interest are then extracted. These include the elastic stiffness, K_e of the structural element and its resistance and deformations at yield (δ_y, Q_y^*) and ultimate $\delta_y, Q_u^*)$. The asterisk (*) denotes that the resistances refer to the first cycle envelope to distinguish from the monotonic backbone curve of the same element. The first cycle envelope considers the effects of cyclic hardening on the respective resistances. Unlike reinforced concrete and masonry elements, the post-ultimate deformation capacity, δ_c of code-conforming structural steel elements is considerable. In certain cases (e.g., steel beams and bracings), a residual resistance, Q_r^* may be attained due to stabilization of local geometric instabilities such as inelastic local buckling. Referring to Figure 2, the linear softening branch is defined by a plastic deformation at post-ultimate, δ_c^{pl}. Depending on the damage mechanism(s) in steel and composite-steel concrete elements, two distinct cases are defined. The first one is termed limited ductile behaviour (see Figure 2c) whereas the second one is called ductile (see Figure 2d). The primary difference between the two is the amplitude of δ_c^{pl} as shown in Figures 2c and d. Details regarding some of the damage mechanisms are summarized in the next section.

The resistance models are typically determined based on first principles of mechanics whereas the deformations above yield are determined based on empirical relationships, which are derived from multivariate regression analyses [23].

5. RESISTANCE AND DEFORMATION MODELS FOR ASSESSMENT

Resistance and deformation models for assessment depend on the associated damage mechanism(s) in steel and composite-steel concrete elements. Ductile mechanisms are those associated with flexural and shear yielding, local and member buckling. Conversely, limited ductile mechanisms are associated with weld and bolt fracture(s) in beam-to-column joints. Weld fractures in splices or bracing-end connections are classified as brittle. And their plastic deformation capacity is assumed to be zero. Consequently, the seismic assessment is only based on a resistance/force criterion in this case. This section provides a summary of resistance and deformation models of primary and secondary structural steel elements for assessment of steel structures. Emphasis is placed on steel beams in rigid full-strength beam-to-column joints, steel columns, the beam-to-column web panel zone as well as steel bracings. An extensive list of generalized force-deformation relationships can be found in [20] and [24] for existing and new steel structures, respectively.

5.1 Steel beams in rigid full-strength beam-to-column joints

Steel beams in rigid full-strength beam-to-column joints are categorized to those with/without compliant seismic weld details. As the paper covers existing steel structures, the primary focus herein is on steel beams with noncompliant seismic weld detailing. Referring to Figure 2c, the effective flexural resistance and chord rotation at yield should be calculated according to the following equations:

$$M_{\mathrm{y}}^{*} = 1{,}1 W_{\mathrm{el}} f_{\mathrm{y}} \tag{1}$$

$$\theta_{\mathrm{y}} = M_{\mathrm{y}}^{*}/K_{\mathrm{e}} \tag{2}$$

where, W_{el} is the elastic cross-sectional section modulus of the beam; K_{e} is the flexural stiffness of the steel beam, which depends on its boundary conditions. For rigid-end beams in contraflexure, $K_{\mathrm{e}} = 6EI_{\mathrm{b}}/L_{\mathrm{b}}$; where I_{b} and L_{b} are the second moment of area and clear span of the steel beam, respectively. The effective flexural resistance, M_{u}^{*} and the associated plastic rotation, $\theta_{\mathrm{u}}^{\mathrm{pl}}$ at ultimate are computed according to Equations (3) and (4), respectively,

$$M_{\mathrm{u}}^{*} = M_{\mathrm{y}}^{*} + a_{h} K_{\mathrm{e}} \theta_{\mathrm{u}}^{\mathrm{pl}} \tag{3}$$

$$\theta_{\mathrm{u}}^{\mathrm{pl}} = 0{,}048 - 0{,}000433h \tag{4}$$

Referring to Figure 2c, the chord rotation at collapse, θ_{c} may be computed as follows,

$$\theta_{\mathrm{c}} = 0{,}056 - 0{,}000433h \tag{5}$$

Where h is the full depth of the steel beam in millimetres; a_{h} is the steel material hardening ratio and may be considered equal to 0,03 for all structural steels. Other acceptable values in the literature (e.g., [11]) may be considered based on experimental evidence. Equations (4) and (5) suggest that the plastic rotation at ultimate and collapse, respectively, of steel beams with noncompliant welds is only dependent on their beam depth. This corroborates with observations from pertinent experiments [25]. Conversely, the plastic rotation capacity of steel beams in rigid beam-to-column joints with compliant seismic weld detailing according to Annex E of [21] is dominated by inelastic local buckling within the anticipated dissipative zone. Consequently, the plastic rotation capacity in this case is dependent on the local slenderness ratios of the steel profile as well as the unbraced length among other geometric and material parameters [11].

5.2 Steel columns

Steel columns exhibit inelastic deformations that depend on their cross-sectional and member geometric characteristics as well as the compressive axial load ratio. The new provisions of Eurocode 8 Part 3 provide explicit modelling recommendations for both I- or H-shaped and hollow structural steel elements. Referring to Figure 2d, the effective flexural resistance and chord rotation at yield for planar loading with respect to the strong axis of the column should be computed by Equations (6) and (7), respectively.

$$M_{\mathrm{y}}^{*} = 1{,}15\omega_{\mathrm{rm}}\left(1-\frac{N_{\mathrm{Ed,G}}}{\chi_{\mathrm{z}}N_{\mathrm{Rk}}/\gamma_{\mathrm{M1}}}\right)\chi_{\mathrm{LT}}M_{\mathrm{y,Rk}}/\gamma_{\mathrm{M1}} \tag{6}$$

$$\theta_{\mathrm{y}} = M_{\mathrm{y}}^{*}/K_{\mathrm{e}} \tag{7}$$

where, $1{,}15\omega_{\mathrm{rm}}$ accounts for the effects of cyclic hardening and the steel material randomness; $N_{\mathrm{Ed,G}}$ is the compressive axial load due to the non-seismic actions in the seismic design situation; N_{Rk} is the characteristic value of the cross-sectional resistance to compression axial force; χ_z and χ_{LT} are the flexural buckling reduction factor and the lateral torsional reduction factor, respectively, according to [22]; γ_{M1} may be considered equal to 1,0; K_{e} is the elastic flexural stiffness of the column depending on its boundary conditions. The ultimate-to-yield effective flexural resistance ratio, $M_{\mathrm{u}}^{*}/M_{\mathrm{y}}^{*}$ is an indicator of the effective hardening ratio. Lignos and Krawinkler [11] has shown that this is a more stable parameter than the post-yield stiffness to describe the post-yield hardening of structural steel elements. The $M_{\mathrm{u}}^{*}/M_{\mathrm{y}}^{*}$ ratio may be computed as follows,

$$M_{\mathrm{u}}^{*}/M_{\mathrm{y}}^{*} = 7{,}6\left(\frac{c}{t_w}\right)^{-0{,}4}\left(\frac{L_b}{i_z}\right)^{-0{,}16}\left(1-\frac{N_{\mathrm{Ed,G}}}{N_{\mathrm{pl,e}}}\right)^{0{,}2} \leq 1{,}2 \text{ and } M_{\mathrm{u}}^{*}/M_{\mathrm{y}}^{*} \geq 1{,}0 \tag{8}$$

The plastic chord rotation capacity of I- and H-shaped steel columns at ultimate, $\theta_{\mathrm{u}}^{\mathrm{pl}}$ may be computed as follows,

$$\theta_{\mathrm{u}}^{\mathrm{pl}} = 7{,}37\left(\frac{c}{t_w}\right)^{-0{,}95}\left(\frac{L_b}{i_z}\right)^{-0{,}5}\left(1-\frac{N_{\mathrm{Ed,G}}}{N_{\mathrm{pl,e}}}\right)^{2{,}4} \leq 0{,}15 \text{ rad} \tag{9}$$

Referring to Figure 2d, the chord rotation at collapse, θ_{c} may be computed as follows,

$$\theta_{\mathrm{c}} = 20\left(\frac{c}{t_w}\right)^{-0,9}\left(\frac{L_b}{i_z}\right)^{-0,5}\left(1-\frac{N_{\mathrm{Ed,G}}}{N_{\mathrm{pl,e}}}\right)^{3,4} \leq 0{,}07\ \mathrm{rad} \tag{10}$$

Similar expressions have been proposed for HSS steel columns. From the above two equations it is evident that the plastic rotation capacity of I- or H-shaped steel columns is mostly dependent on their web slenderness ratio, c/t_{w}, which is colinear to the flange slenderness ratio. This parameter is strongly dependent on local buckling within the anticipated dissipative zone, which often acts synergistically with lateral torsional buckling. This global instability mode depends on the member slenderness ratio, $L_{\mathrm{b}}/i_{\mathrm{z}}$. Steel columns are subjected to coupled lateral drift demands with compressive axial load. This is found to have a dominant effect on $\theta_{\mathrm{u}}^{\mathrm{pl}}$ and θ_{c}. However, studies on the seismic stability of steel columns [26] has shown that the axial load contribution from the non-seismic action in the seismic design situation has the most contribution on the plastic deformation capacity of a column.

5.3 Beam-to-column web panel joint

The beam-to-column web panel joint (also known as simply the panel zone) is subjected to high shear demands, thereby exhibiting shear yielding as shown in Figure 3. Research [27]–[29] has shown that shear yielding is generally a stable damage mechanism and may be approximated with a trilinear load-deformation relationship as shown in Figure 3. The shear resistance at characteristic shear distortions, γ of a beam-to-column web panel joint should be computed as discussed in this section.

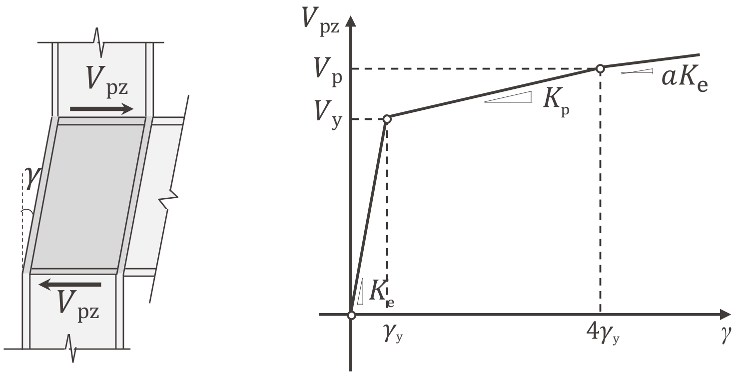

Fig. 3 Panel zone force-displacement relationship

The shear resistance at yield, V_{y} of the web panel may be computed by assuming uniform shear yielding and shall be determined according to prEN 1998-1-2:2022 [22]. The full shear plastic resistance of the panel zone, V_{p} should be computed according to Equation (11) based on recent work by Skiadopoulos et al. [29],

$$V_{\mathrm{p}} = \frac{f_{\mathrm{y}}}{\sqrt{3}}\left[1{,}1(h_{\mathrm{c}}-t_{\mathrm{cf}})t_{\mathrm{p}} + \left(0{,}93\frac{K_{\mathrm{f}}}{K_{\mathrm{e,c}}}+0{,}015\right)(b_{\mathrm{cf}}-t_{\mathrm{cw}})2t_{\mathrm{cf}}\right]\sqrt{1-v_{\mathrm{Ed}}^{2}} \tag{11}$$

where h_{c} is steel column cross-sectional depth; t_{cf} is the flange thickness of the column cross section; t_{p} is the web panel thickness including the total thickness of the column web doubler plates, if any; b_{cf} is the flange width of the column cross section; t_{cw} is the web thickness of the column cross section; v_{Ed} is the axial load ratio of the steel column due to the seismic actions in the seismic design situation and $v_{\mathrm{Ed}} = N_{\mathrm{Ed}}/N_{\mathrm{pl,e}}$. Referring to Formula (11), $K_{\mathrm{f}}/K_{\mathrm{e,c}}$ is the column flange-to-panel zone stiffness ratio and shall be computed as follows:

$$K_{\mathrm{f}} = \frac{2Eb_{\mathrm{cf}}t_{\mathrm{cf}}^{3}}{h_{\mathrm{b}}^{2}+2(1+v)t_{\mathrm{cf}}^{2}} \tag{12}$$

$$K_{\mathrm{e,c}} = \frac{12Et_{p}(h_{\mathrm{c}}-t_{\mathrm{cf}})I_{\mathrm{c}}}{t_{\mathrm{p}}(h_{\mathrm{c}}-t_{\mathrm{cf}})h_{\mathrm{b}}^{2}+24(1+v)I_{\mathrm{c}}} \tag{13}$$

5.4 Steel bracings

Steel bracings exhibit highly asymmetric behaviour under tensile and compressive axial load demand as shown in Figure 4. The effective axial resistance of steel bracings at yield in tension, $N_{\mathrm{pl,e}}$ and at buckling, $N_{\mathrm{b,e}}$ should be computed according to Equations (14) and (15) respectively,

$$N_{\mathrm{pl,e}} = \omega_{\mathrm{rm}}f_{\mathrm{y}}A \tag{14}$$

$$N_{\mathrm{b,e}} = \omega_{\mathrm{rm}}N_{\mathrm{b,Rd}} \tag{15}$$

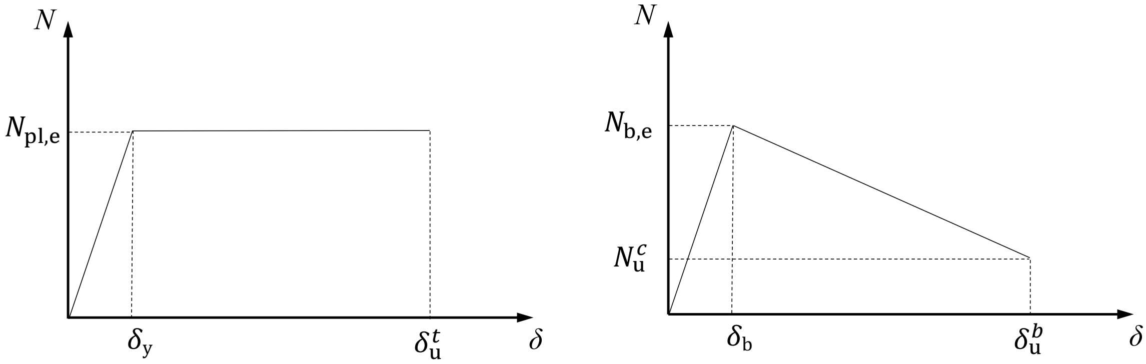

Fig. 4 General axial force-displacement relationship of steel bracings for axial tension and compression.

Where A is the cross-sectional area of the steel brace; $N_{\mathrm{b,e}}$ is the buckling resistance of the steel brace according to [22]. The effective axial resistance at ultimate in tension, $N_{\mathrm{u}}^{\mathrm{t}}$ and compression, $N_{\mathrm{u}}^{\mathrm{c}}$ (see Figure 4), should be computed based on Equations (16) and (17), respectively. Note $N_{\mathrm{u}}^{\mathrm{c}}$ is empirically calibrated based on available test data from the literature [18], [30],

$$N_{\mathrm{u}}^{\mathrm{t}} = N_{\mathrm{pl,e}} \tag{16}$$

$$N_{\mathrm{u}}^{\mathrm{c}} = 0{,}20\ \omega_{\mathrm{rm}} N_{\mathrm{b,Rd}} \tag{17}$$

Indicatively, the axial displacement at ultimate for steel bracings with rectangular or square hollow sections under compression and tension should be computed according to Equations (18) and (19), respectively,

$$\delta_{\mathrm{u}}^{\mathrm{c}} = 3{,}75 \left(\frac{\lambda_{\mathrm{f}}}{\lambda_{\mathrm{Cl.1}}}\right)^{-1{,}0} \left(\frac{L_{\mathrm{cr}}}{i}\sqrt{\frac{\omega_{\mathrm{rm}} f_{\mathrm{y}}}{E}}\right)^{0{,}4} \delta_{\mathrm{b}} \tag{18}$$

$$\delta_{\mathrm{u}}^{\mathrm{t}} = 5{,}8 \left(\frac{\lambda_{\mathrm{f}}}{\lambda_{\mathrm{Cl.1}}}\right)^{-1{,}0} \left(\frac{L_{\mathrm{cr}}}{i}\sqrt{\frac{\omega_{\mathrm{rm}} f_{\mathrm{y}}}{E}}\right)^{0{,}24} \delta_{\mathrm{y}} \tag{19}$$

where, δ_{y} and δ_{b} are the axial displacements at yield and buckling and should be computed based on first principles of mechanics; λ_{f} is the width-to-thickness ratio of rectangular or square hollow sections; and $\lambda_{\mathrm{Cl.1}}$ is the limit of width-to-thickness ratio for Class 1 for members in compression according to prEN 1993-1-1:2022, 7.6 [22]; L_{cr} is the buckling length in the buckling plane considered according to prEN 1993-1-1:2022, 8.3 [22]; and i is the is the radius of gyration of the bracing cross section in the buckling plane. An implementation of all resistance and deformation models for design can be found in http://resslab-hub.epfl.ch.

6. VERIFICATION OF LIMIT STATES

The deformation capacity, δ_{NC} of primary or secondary structural elements corresponding to the limit state of Near Collapse (NC) should be computed by the deformation at ultimate, δ_{u}, or collapse, δ_{c}, whichever is applicable relating to each retrofit method, divided by the corresponding partial factor of resistance (deformation), γ_{Rd}. This factor accounts for uncertainty in the ultimate deformation of the respective element or connection. It is evaluated by considering the uncertainty of all parameters involved in the corresponding deformation model such as those of Equations (4), (5), (9), (10), (20) and (21) presented earlier. Values of γ_{Rd} are tabulated in [20] and have been derived with a reliability-based methodology as summarized in [31]. Similarly, the corresponding displacement capacity, δ_{NC} of primary or secondary structural elements corresponding to the limit state of Damage Limitation (DL) should be computed by the deformation at yield, divided by the corresponding γ_{Rd}, which may be taken as 1,1 for primary structural steel elements or 1,0 for secondary structural steel elements.

7. CONCLUSIONS

This paper provides a brief summary on the recent developments regarding the resistance and deformation models for the seismic assessment of existing steel structures according to the new Eurocode 8 Part 3 [24]. The new chapter provides quantitative information regarding the nominal properties of structural steel materials depending on the time of construction that dates back from early 1900s. The generalized force-deformation relationships that are proposed to describe the primary behavioral characteristics of primary and secondary structural steel elements are based on a combination of first principles of mechanics and empirical formulations. These have been derived from available experimental data that have been consistently put in comprehensive structural performance databases by the author. These databases along with a web-based application that allows for the exploitation of the resistance and deformation models are available from http://resslab-hub.epfl.ch. Finally, the verification of two primary limit states, i.e., near collapse and damage limitation, are presented.

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ΤΟ ΝΕΟ ΚΕΦΑΛΑΙΟ 9 ΤΟΥ ΕΥΡΩΚΩΔΙΚΑ 8 ΜΕΡΟΣ 3 ΓΙΑ ΤΗΝ ΑΠΟΤΙΜΗΣΗ ΤΗΣ ΣΕΙΣΜΙΚΗΣ ΣΥΜΠΕΡΙΦΟΡΑΣ ΥΦΙΣΤΑΜΕΝΩΝ ΜΕΤΑΛΛΙΚΩΝ ΚΑΤΑΣΚΕΥΩΝ

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ΠΕΡΙΛΗΨΗ

Η σεισμική αποτίμηση υφιστάμενων μεταλλικών κατασκευών προϋποθέτει την ανάπτυξη πρακτικών προσομοιωμάτων για χρήση της μη γραμμικής στατικής και δυναμική ανάλυσης των κατασκευών αυτών. Τα προσομοιώματα αυτά θα πρέπει να καλύπτουν όλο το φάσμα της μη γραμμικής συμπεριφοράς των εν λόγω μελών του εκάστοτε φορέα. Στο άρθρο παρουσιάζεται μια περίληψη του νέου κεφαλαίου 9 του Ευρωκώδικα 8 μέρος 3. Το κεφάλαιο αυτό καλύπτει μεγάλο αριθμό προσομοιωμάτων μεταλλικών και σύμμικτων υποστυλωμάτων, δοκών καθώς και τυπικών συνδέσμων δυσκαμψίας. Τα προτεινόμενα προσομοιώματα βασίζονται σε εκτενή πειραματικά δεδομένα τα οποία έχουν συλλεχθεί και επιμεληθεί σε βάσεις δεδομένων, οι οποίες έχουν αναπτυχθεί από το ερευνητικό γκρουπ του συγγραφέα τις τελευταίες δύο δεκαετίες με σκοπό τη σεισμική αποτίμηση νεόδμητων και υφιστάμενων μεταλλικών και σύμμικτων κατασκευών. Το άρθρο παρουσιάζει κάποια από αυτά τα προσομοιώματα καθώς και μια εφαρμογή σε ιστότοπο η οποία έχει αναπτυχθεί με στόχο την ευρεία χρήση τους από πολιτικούς μηχανικούς.