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THE FUTURE OF SEISMIC DESIGN OF STEEL STRUCTURES PR EN 1998-1-1 AND PR EN 1998-1-2

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The first generation of Eurocode 8 has appeared some twenty years ago. At that time, it presented a brand new and modern seismic standard. But twenty years is a relatively long period and during it a lot has been happening in the field of seismic design of buildings. Latest scientific research together with the identified shortcomings in the current version of Eurocode 8 created a need for it to be updated. Bearing that in mind European commission began to work on the second generation of Eurocodes. They are expected to be finished and ready for adoption in a sequential manner at the start of 2026. In this paper the first draft of the second generation of Eurocode 8 (parts 1-1 and 1-2) is presented with specific attention being paid to seismic design of steel structures. The main part of Eurocode 8 which was EN 1998-1-1 is now divided into two parts: EN 1998-1-1 and EN 1998-1-2. These two documents combined give basic rules for the design of seismic resistant structures. They are massive, comprehensive and filled with novelties and latest scientific research in the area of seismic design. Bearing in mind that the two before mentioned drafts have over five hundred pages combined and that this paper can only be so long only the most important novelties are presented without going too much into details. The topics that are covered in this overview paper are general concept of seismic design and representation of seismic action, soil classification and site amplification effects, methods of seismic analysis and seismic design of steel structures. With all proposed changes and advances in mind it is safe to say that the second generation of Eurocode 8 will be a large step forward and will set a high bar for other seismic standards to reach. The advances that are made in field of steel structures will make them even more attractive in earthquake prone areas.

Keywords: second generation of Eurocode 8, spectral acceleration, site amplification effects, steel structures

1 INTRODUCTION

Eurocode 8 was amongst the first Eurocodes that was adopted in Montenegro. At the time it presented a brand-new approach for the design of seismic resistant structures embellished by the capacity-based approach. In early years there was some struggle in its implementation but as the time passed the civil engineers in practice become accustomed to it. Since then, years have passed and as it seems a second revolution is lurking around the corner. Namely, the first draft of the second generation of Eurocode 8 is finally came to light and to put it mildly it is a sight to see.

The main part of Eurocode 8 which was EN 1998-1-1 is now divided into two parts: EN 1998-1-1 [1] and EN 1998-1-2 [2]. These two documents combined give basic rules for the design of seismic resistant structures. They are massive, comprehensive and filled with novelties and latest scientific research in the area of seismic design. As it seems they will herald a new revolution when it comes to the design of seismic resistant structures.

The aim of this paper is to present an overview of the most significant novelties in before mentioned drafts with specific attention being paid to seismic design of steel structures. Bearing in mind that the two before mentioned drafts have over five hundred pages combined and that this paper can only be so long only the most important novelties are presented without going too much into details. For detailed information interested readers are referred to the references given in the literature section of the paper.

1.1 The inevitable future

Second generation of Eurocode 8 is expected to be adopted in 2026. Until then there are still two years but because of the sheer volume of novelties in parts EN 1998-1-1 [1] and EN 1998-1-2 [2], that are at this point simply inevitable, it is never too early to start preparing. This remaining time if wisely used should be spent on activities that will prepare the civil engineers in practice for this inevitable future and to shorten the necessary transition period. This paper is envisioned as a first small step towards that direction.

In the following sections of this paper the significant novelties that bring drafts EN 1998-1-1 [1] and EN 1998-1-2 [2] are presented. The topics that will be covered in this overview paper are:

- General concept of seismic design and representation of seismic action,
- Soil classification and site amplification effects,
- Methods of seismic analysis,
- Seismic design of steel structures.

1.1.1 General concept of seismic design and representation of seismic action

In the new version of Eurocode 8 the general concept of seismic design of structures has not changed. At its core capacity design is still present but pretty much everything else has been replaced.

Peak ground acceleration a_g as the main parameter representing the expected seismic action is replaced with spectral acceleration. For many authors [3] this change is justified and welcomed but it has resulted in a complete overhaul of everything else.

This means that the first step towards implementation of second generation of Eurocode 8 for each country, depended on the local seismic hazard, will be the creation of spectral acceleration maps.

In this maps seismic hazard should be described in terms of two parameters $S_{\alpha,ref}$ and $S_{\beta,ref}$.

$S_{\alpha,ref}$ is the reference maximum spectral acceleration corresponding to the constant acceleration range of the horizontal 5% damped elastic response spectrum. The parameter $S_{\beta,ref}$ represents the reference spectral acceleration at the vibration period of T_β = 1 s of the horizontal 5% damped elastic response spectrum.

Introducing spectral acceleration as a measure for seismic action has resulted in a newly defined response spectra.

For horizontal components of the seismic action elastic response spectrum $S_e(T)$ is defined by the following expressions:

$$0 \le T \le T_A:\ S_e(T) = \frac{S_\alpha}{F_A} \qquad T_A \le T \le T_B:\ S_e(T) = \frac{S_\alpha}{T_B - T_A}\left[\eta(T - T_A\right] + \frac{T_B - T}{F_A}$$

$$T_B \le T \le T_C:\ S_e(T) = \eta S_\alpha \quad T_c \le T \le T_D:\ S_e(T) = \eta\frac{S_\beta T_\beta}{T} \tag{1}$$

$$T \ge T_D: S_e(T) = \eta T_D \frac{S_\beta T_\beta}{T^2}$$

Where:

- $S_e(T)$ is the elastic response spectrum,
- T is the vibration period of a linear single – degree of freedom system,

S_α is the maximum response spectral acceleration (for 5% damping) corresponding to the constant acceleration range of the elastic response spectra,

- S_β is the 5% damped response spectral acceleration at the vibration period T_β,
- T_β is 1 s,
- T_A is the short period cu of associated to the zero period spectral acceleration,
- F_A is the ration of S_α with respect to the zero period spectral acceleration,
- $T_C = \frac{S_\beta T_\beta}{S_\alpha}$ is the upper corner period of the constant spectral acceleration range,

T_B is the lower corner period of the constant spectral acceleration range, with:

$$T_B = \frac{T_c}{\chi},\text{ if } 0{,}05\text{ s} \le \frac{T_c}{\chi} \le 0{,}10\text{ s or } T_B = 0{,}05\text{ s},\ \frac{T_c}{\chi} < 0{,}05\text{ s or } T_B = 0{,}10\text{ s},\ \frac{T_c}{\chi} > 0{,}10\text{ s}$$

T_D is the corner period at the beginning of the constant displacement response range of the spectrum,

η is the damping correction factor, with reference value η = 1 for 5% damping ratio.

Parameters T_A, χ, F_A and T_D can be defined in the National Annex. The recommended values are given in Table (1).

Table 1. Parameters T_A, χ, F_A and T_D [1]

T_A(s)	χ	F_A	T_D(s)
0,02	4	2,5	2 if $S_{\beta,RP} \le 1$ m/s^2 1+ $S_{\beta,RP}$ if $S_{\beta,RP} > 1$ m/s^2

If the value for damping is different than 5% the damping correction factor η can be determined using expression (2), where ξ is the damping ratio of the structure considered expressed as a percentage of critical.

$$\eta = \sqrt{\left(10 + \frac{T_C(\xi - 5)}{T_C + 30(T - T_A)}\right)/(5 + \xi)} \tag{2}$$

Newly defined elastic response spectra are shown in Figure (1).

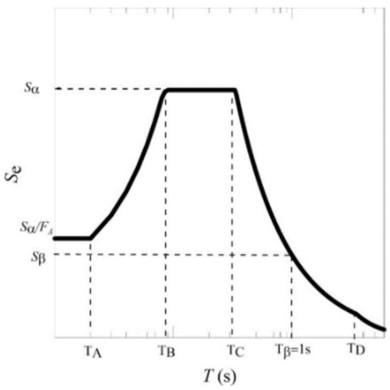


Fig. 1. Elastic response spectrum in pr EN 1998-1-1 [1]

Main input parameters S_α and S_β for the elastic response spectrum should take into account local site effects and the consequences of the collapse of the considered structure.

To take these factors into account S_α and S_β in pr EN 1998-1-1 [1] are defined in the following manner:

$$S_\alpha = F_T F_\alpha S_{\alpha,RP} \quad \text{and} \quad S_\beta = F_T F_\beta S_{\beta,RP} \tag{3}$$

With the parameters F_T, F_α and F_β local site effects i.e. local site amplification of seismic action is taken into account. Here we can observe a significant change in regard to the current version of Eurocode 8 [4] and local site effects.

Instead of one factor (factor S) that is present in the current version of Eurocode 8 [4] now we have three. This is a significant step forward because local amplification of seismic action is a rather complex problem to take into consideration. There are many influential parameters that must be analysed (local geological profile, topography, seismic motion, cyclic soil strength and stiffness, etc.) and simply taking them into account altogether by one factor proved to be inadequate. This issue was recognized already by some countries (Germany). In German National Annex for EN 1998-1 a two-factor formula for determining factor S is given.

Also, the recommended values for factor S in some specific cases were proven not to be on the safe side by a large amount. So, all things considered this new formulation is a significant step forward. The values for parameters F_T, F_α and F_β are going to be discussed in the next subsection of the paper.

$S_{\alpha,RP}$ and $S_{\beta,RP}$ are the representative values for spectral acceleration S_α and S_β. These representative values take into account the fact that all structures don't necessarily have the same importance during an earthquake. Some must remain fully operational during strong earthquakes while in others damages are permitted. So representative values of S_α and S_β must reflect the consequence class of the building and the associated limit state. Bearing that in mind $S_{\alpha,RP}$ and $S_{\beta,RP}$ are defined as follows:

$$S_{\alpha,RP} = \gamma_{LS,cc} S_{\alpha,ref} \quad \text{and} \quad S_{\beta,RP} = \gamma_{LS,cc} S_{\beta,ref} \tag{4}$$

Performance factors $\gamma_{LS,cc}$ depend on the considered limit state and the consequence class of the building. For different structures their values are given in the relevant parts of EN 1998. Limit states that must be considered in the scope of the new version of Eurocode 8 are completely redefined in regard to the current version.

The new adopted definition is that that the seismic performance of the building is measured by its state of damage under a given seismic action. With that in mind four limit states are identified:

– Limit state of near collapse (NC),
– Limit state of significant damage (SD),
– Limit state of damage limitation (DL),
– Fully operational limit state (OP)

NC limit state is defined as one in which the structure is heavily damaged but it retains its vertical load bearing capacity. Most ancillary components are collapsed. In SD limit state the structure is significantly damaged with moderate permanent drifts. Ancillary components are damaged but not collapsed. The structure is expected to be repairable but in some cases it may be uneconomic to do so. DL limit state is defined as one in which the structure is only slightly damaged and economic to repair. In fully operational limit state the structure is only slightly damaged allowing continuous operation of systems hosted by the structure.

Significant damage and near collapse limit states are considered as ultimate limit states while damage limitation and fully operational limit states are considered as serviceability limit states. This new definition of limit states ensures better seismic performance of the structures and is in line with other modern seismic standards (like in USA or Japan).

It is important to point out that pr EN 1998 is conceived in such a way that for a large majority of new structures the SD non – exceedance requirement implies avoiding NC exceedance under a seismic action meaningfully more severe than that of design as well avoiding DL exceedance under a seismic action less severe than that of design.

The return period of 475 years for SD limit state and consequence class CC2 is considered as a reference period. For other limit states and consequence classes the recommended return period and performance factors are given in Table (2) and (3).

Table 2. Return period of seismic action and the associated limit state [2]

Limit state	Consequence class (CC)			
	CC1	CC2	CC3-a	CC3-b
NC	800	1600	2500	5000
SD	250	475	800	1600
DL	50	60	60	100

Table 3. Performance factors [2]

Limit state	Consequence class (CC)			
	CC1	CC2	CC3-a	CC3-b
NC	1,2	1,5	1,8	2,2
SD	0 ,8	cs	1,2	1,5

1.1.2 Soil classification and site amplification effects

Local site effects and seismic action are strongly dependent on the soil characteristics on which the building is founded. Bearing that in mind in current version of Eurocode 8 [4] five categories of soils are identified with regard to the shear wave velocity. The current soil classification procedure proved in some cases unreliable and impractical. This is because the average shear wave velocity is calculated for layers of soils existing in the top 30 m.

This formulation created some problems in cases where there is a relatively thin soft deposit over a thick significantly stiffer one. In this case by using the average value for shear wave velocity in order to classify the soil the influence of the thin soft deposit is practically ignored.

This problem is corrected in the new version of Eurocode 8 by the use of the equivalent shear wave velocity of the superficial soil deposit for the soil classification instated of the average one. Equivalent value of the shear wave velocity of the superficial soil deposit is defined as follows:

$$v_{s,H} = \frac{H}{\sum_{i=1}^{N} \frac{h_i}{v_i}} \quad (5)$$

Where

- h_i – is the thickness of the i^{th} soli layer,
- v_i – is the shear wave velocity of the i^{th} soil layer,
- N – is the total number of soil layers from the ground surface to the depth H.
- The reference depth H is defined in the following manner:
- H = 30 m if $H_{800} \geq 30$ or H = H_{800} if $H_{800} < 30$

The parameter H_{800} represents the depth of the seismic bedrock formation identified by v_s which is at least equal to 800 m/s. By introducing the H_{800} the before mentioned problem of relatively soft deposits over a thick significantly stiffer ones is solved.

Soil classes and their categorization according to pr EN 1998-1-1 [1] are shown in the following table.

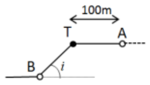
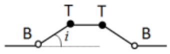
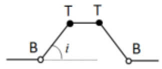
Table 4. Soil classes according to pr EN 1998-1-1 [1]

	Ground class	Stiff	Medium stiff	Soft
Depth class	$v_{s,H}$ range/ H_{800} range	400 m/s ≤ $v_{s,H}$ < 800 m/s	250 m/s ≤ $v_{s,H}$ < 400 m/s	150 m/s ≤ $v_{s,H}$ < 250 m/s
Very shallow	$H_{800} \leq 5$ m	A	A	E
Shallow	5 m < $H_{800} \leq 30$ m	B	E	E
Intermediate	30 m < $H_{800} \leq 100$ m	B	C	D
Deep	H_{800} > 100 m	B	F	F

In regard to the defined soil class site amplification factors are defined. The values for these factors can be found in revelant tables in pr EN 1998-1-1 [1].

In some cases, local topography can play an important role when it comes to local site amplification. This problem is not covered in the current version of Eurocode 8. Some slight improvement is present in the draft of the new version of EN 1998-1-1. In the case of topographic irregularities of height greater than 30 m with average slope angle larger than 15° a period independent amplification factor FT is defined. The values for FT are shown in the Table (5). These values refer only to the ground types A and B.

Table 5. Topography factor in pr EN 1998-1-1 [1]

Topography description	F_T	Simplified sketch
Flat ground surface, slopes and isolated ridges with average slope angle i < 15° or height < 30 m	1,0	
Slopes with average slope angle i > 15°	1,2	
Ridges with width at the top much smaller than at the base and average slope angle 15° < i < 30°	1,2	
Ridges with width at the top much smaller than at the base and average slope angle i > 30°	1,4	

1.1.3 Methods of seismic analysis

In order to verify the resistance of structural members first the design seismic action must be defined. According to pr EN 1998-1-1 this can be done using one of the following two approaches:

- The forced based approach,
- The displacement-based approach.

In forced based approach a linear analysis is carried out and through behavior factor q over strength and nonlinear response are taken into account. This can be done using either lateral force method or response spectrum method.

These two methods are in principle the same as in the current version of Eurocode 8 [4] with some slight modifications and additions.

Force based approach may be used for the verification of SD, DL and OP limit state.

In displacement-based approach the structural nonlinear response is explicitly accounted. This approach should be used for the verification of NC limit state.

As was stated before in linear methods with behavior factor q over strength and nonlinear response is taken into account. The behavior factor value reflect the structure capacity for the dissipation energy. This capacity is influenced by many parameters like inherent over strength, system redundancy, structural concept and etc.

In current version of Eurocode 8 [4] it is unclear how much each of the before mentioned factors influence the overall system response i.e. behavior factor value. This cloak and dagger situation is remedied in the draft of the new version of Eurocode 8. In pr EN 1998-1-1 [1] behavior factor q is defined in terms of three factors.

$$q = q_R q_S q_D \tag{6}$$

Redistribution of seismic action effects in redundant structures is taken into account with factor q_R. The reference value for q_R is 1 unless otherwise specified in the relevant parts of EN 1998. With factor q_S inherent over strength of the structure and members is taken into account. The reference value for q_S is 1,5. Factor q_D reflects the deformation and energy dissipation capacity of the analysed structure. Values for q_D are defined in order to keep a significant margin with respect to the ultimate deformation capacity of the structure and are depended on the selected ductility class.

In pr EN 1998-1-1 [1], like in the current version of Eurocode 8 [4] three ductility classes are defined. However, in the second generation of Eurocode 8 they are completely redefined and now called DC1, DC2 and DC3.

In structures with ductility class 1 (DC1) only the inherent over strength capacity is taken into account. In structures with ductility class 2 (DC2) local over strength, deformation and energy dissipation capacity is taken into account. The formation of the global plastic mechanism is controlled. The structure belonging to ductility class 3 has the ability to form a global plastic mechanism at SD limit state and local over strength, deformation and energy dissipation capacity are taken into account.

For relevant ductility class and structural system maximum values of factor q_D are given in the various parts of EN 1998.

1.1.4 Steel structures

Steel as a structural material due to its very high strength to weight ratio and pronounced ductile behavior in tension and bending can be regarded as a good seismic resistant material. Nevertheless bearing in mind the experience from past earthquakes special attention is needed when designing seismic resistant steel structures.

The new version of Eurocode 8 significantly widens, slightly changes and complements the current rules for seismic design of steel structures. Due to paper length restrictions and the sheer volume of changes and additions only the most important changes/additions will be discussed without going further into details.

In pr EN 1998-1-2 [2] besides the current seven structural types that are covered two more are added. Frames with buckling restrained bracings and light weight steel systems using flat strap bracing or sheathed with steel sheets or wood sheeting or gypsum sheeting are now in the scope of the standard.

For each of these now nine categories of structures detailed design rules are given. The design rules follow the philosophy of capacity design with regard to the different expected behavior and dissipation of energy for DC2 and DC3. Behavior factors as a rule are higher than in the current version of Eurocode 8 [4].

In order to achieve the expected structural behavior clear hierarchy of ductile and non – ductile elements must be ensured. In moment resisting frames, the most commonly used structural system, beams are regarded as dissipative members while columns are considered as non-ductile.

Dissipative zones (usually beam ends) should be designed so that yielding or local buckling don't affect the overall stability of the structure. According to pr EN 1998-1-2 [2] beams should be verified using the following expressions:

$$M_{Ed} \leq M_{Rd,b} \quad \text{and} \quad N_{Ed} \leq 0{,}15\, N_{Rd,b} \quad \text{and} \quad V_{Ed} \leq \begin{cases} 0{,}5 V_{Rd,b} \text{ for } q > 2 \\ V_{Rd,b} \text{ for } 1{,}5 \leq q \leq 2 \end{cases} \tag{7}$$

Where

- M_{Ed}, N_{Ed} and V_{Ed} are the bending moment, the axial and the shear force, respectively in the seismic design situation,
- $M_{Rd,b}$, $N_{Rd,b}$ and $V_{Rd,b}$ are the corresponding design resistances of the cross sections.

For beams in DC3 frames design shear force V_{Ed} should be calculated as follows:

$$V_{Ed} = V_{Ed,G} + V_{Ed,M} \tag{8}$$

Where

- $V_{Ed,G}$ is the design shear force in non-seismic combination,
- $V_{Ed,M}$ is the design shear force due to the formation of plastic hinges at the ends of the beam.

For non dissipative members (columns) different rules are given for DC2 and DC3 structures. In DC2 structures where the global plastic mechanism is controlled the resistance and stability of the member should be verified using the following action effects:

$$N_{Ed} = N_{Ed,G} + \Omega N_{Ed,E} \quad \text{and} \quad M_{Ed} = M_{Ed,G} + M_{Ed,E} \quad \text{and} \quad V_{Ed} = V_{Ed,G} + V_{Ed,E} \tag{9}$$

Where

- $N_{Ed,G}$, $M_{Ed,G}$ and $V_{Ed,G}$ are the effects of the non-seismic actions in the seismis design situation
- $N_{Ed,E}$, $M_{Ed,E}$ and $V_{Ed,E}$ are the effects of the design seismic action.

For colums in DC3 structures where global plastic mechanism is expected design action effects should be calculated as follows:

$$N_{Ed} = N_{Ed,G} + \omega_{rm}\omega_{sh}\, \Omega_d N_{Ed,E} \quad \text{and} \quad M_{Ed} = M_{Ed,G} + \omega_{rm}\omega_{sh}\, \Omega_d M_{Ed,E} \tag{10}$$

$$V_{Ed} = V_{Ed,G} + \omega_{rm}\omega_{sh}\, \Omega_d V_{Ed,E} \tag{11}$$

Where

- ω_{rm} is the material ovestrength factor for steel in disipative zone,
- ω_{sh} is the factor accounting for hardening of the dissipatibe zone.

Factors ω_{rm}, ω_{sh} and Ω are defined in the appropiate tables in pr EN 1998-1-2 [2] dependent on the structural system and adopted plastic mechanism.

It is interesting to point out that the values for material overstength factor ω_{rm} are significantly higher than in the current version of Eurocode 8 (factor γ_{ov}) [4]. This corection is a welcomed one because as shown in [5] the current value of 1,25 for steel grade S235 proved to be inadequate.

According to pr EN 1998-1-2 [2] dissipative zones can be located either in the members or joints. Although this is also the case in the current version of Eurocode 8 simply there aren't specific design rules on how to design dissipative joints. This is about to change. As a result of the Equal joints project [6] a number of prequalified joints are defined with specific and comprehensive set of design rules. For these joints both the required resistance and stiffness is defined. These new design rules will ensure better performance of joints in seismic design situation and they are a much needed and welcomed addition to the steel part of Eurocode 8.

Another important addition in pr EN 1998-1-1 [1] is the fully described plastic hinge behavior. Plastic hinge behavior is described for flexure and compression so pushover analysis can be carried out without searching for plastic hinge behavior in the literature as now is the case.

In the steel part of the pr EN 1998-1-2 [2] there are further changes and additions (like second order coefficient) and a lot more to unpack. These changes will be addressed in several other papers that are under preparation. This paper was meant to provide only a glimpse in a future that is to come.

2 CONCLUSION

In this paper the first draft of the second generation of Eurocode 8 (parts 1-1 and 1-2) is presented. It is safe to say that these drafts are bringing a large number of changes and novelties when compared to the current version. The importance of these changes' ranges from cosmetic to conceptual.

As the most important conceptual change the shift from peak ground acceleration to spectral acceleration as the main measure for expected ground motion can be identified. Spectral acceleration proved to be a better measure of the potential damage to structures and thus will lead to better seismic performance of the newly designed buildings. This conceptual change has a number of implications.

First of all, it created the need for spectral acceleration maps. Now the first step in adoption of the second generation of Eurocode 8 for each country would be their creation. Secondly it has resulted in a brand new response spectrum.

Significant advances are also made in the field of the local site amplification of seismic action. Factor S that is present in the current version of Eurocode 8 is disregarded in the favor of three factor formulation. Using factors F_{α}, F_{β} and F_T the influence of local site effects as well as topography is better described and thus leads to a safer design.

Various practical situation for soil categorization that are troublesome in the current version of the Eurocode 8 will be a thing of the past with the introduction of the equivalent shear wave velocity of the superficial soil deposit.

In order to ensure even better and closely monitored seismic performance of the newly designed buildings limit states and ductility classes are completely redefined. Now Eurocode 8 will be in trend with other modern seismic standards like in the USA or Japan where performance-based design is an integral part.

When it comes to the design of seismic resistant steel buildings there are no great conceptual changes only advances. Detailed design rules are given for two more structural categories as frames with buckling restrained bracings and lightweight steel systems using flat strap bracing or sheathed with steel sheets or wood sheeting or gypsum sheeting. Plastic hinge behavior in flexure and compression is described thus making pushover analysis more easily performed.

The most important advance is made in the field dissipative joints. A set of prequalified joints is defined with clear rules for their design. Dissipative behavior can now be safely designed in joints or simultaneously in joints and members thus leading to a more economic design.

With all these changes and advances in mind it is safe to say that the second generation of Eurocode 8 is a large step forward and will set a high bar for other seismic standards to reach. The advances that are made in field of steel structures will make them even more attractive in earthquake prone areas.

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