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SOME REMARKS ON THE CHOICE OF DUCTILITY CLASS FOR EARTHQUAKE-RESISTANT STEEL STRUCTURES

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ABSTRACT

The implementation of the Eurocodes in current structural design practice has brought about a new emphasis on the design of earthquake-resistant structures. In some European countries, new earthquake zones have been defined; henceforth, the design requirements of many ongoing projects have changed as well. The choice of the ductility class of steel structures as one of the key design parameters, the consequences of this choice on design procedure, and some applications of the Eurocode 8 design criteria by comparing French and Slovak national practice are discussed, using a practical example of a structure.

KEY WORDS

- *Earthquake-resistant design,*
- *ductility,*
- *steel structure,*
- *Eurocode 8.*

1. INTRODUCTION

During recent decades we have seen the growth of urban areas in seismically active regions, which increases the risk that an earthquake will occur where there is a large concentration of a population. Extensive damage to human lives and property has led to more safety precautions in construction engineering and to tighter seismic specifications, even in countries where seismic activity is less important, but where there are numerous multi-storey structures, very attractive architectural projects, important energy plants, etc.

The challenge for a proper seismic structural design is to solve the balance between the seismic demand and structural capacity. The seismic demand corresponds to the effect of an earthquake on a structure and depends on modelling the ground motion. Structural capacity is the structural ability to resist these effects without failure.

Actually, the structural response can be predicted fairly confidently, but the prediction of ground motions is still far from being on a satisfactory level, due to the complexity of the seismic phenomena and the lack of communication between seismologists and structural engineers. Some shortcomings in the current practice have been identified by Gioncu and Mazzolani (2006).

In current engineering practice, an ordinary structural engineer counts on the available construction codes and their practical interpretation according to the demands of his building investor and/or control supervisor. In Europe, especially in countries where earthquake-resistant design does not have any reliable experience background, the new European codes (EN 1998-1) naturally raise many questions and doubts, despite the different design manuals available, e.g. the Manual for the Seismic Design of Steel and Concrete Buildings to Eurocode 8 (2010). The aim of this article is to discuss the choice of the ductility class for steel structures according to the Eurocode 8 (EN 1998-1), because it is one of the key parameters in seismic design, which may impact the design process on different levels from the choice of the material to the model behaviour of the structure and the quality of the testing methods after its completion.

Finally, some applications of the Eurocode 8 design criteria and the choice of the ductility class are discussed using a practical example of a structure in a real design situation.

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2. EARTHQUAKE-RESISTANT CONCEPTS ACCORDING TO EUROCODE 8

In terms of the actions applied to a structure, an earthquake means movement and, therefore, the displacement of the structure following the ground's acceleration. The structure is moving and changing its shape, so that only if it is well designed, will it be able to equilibrate this active energy by its resistance and its ductility.

Experience shows that steel structures subjected to earthquakes behave better than structures made from other materials. This may be explained by some of the specific features of steel structures: steel as a material is ductile, and a steel structure has many different structural members with rather small cross-sections and reliable geometric properties. These members have a relatively low sensitivity of the bending resistance of the structural elements to the presence of a coincident axial force; these elements and their connections can form numerous plastic hinges and so dissipate the seismic energy well. Thus the energy is not concentrated and conserved, but is dissipated by the structure.

This behaviour is different from that of concrete, because concrete structures are usually made of sufficiently large sections that are only subject to elastic stresses. But in this second case, the structure is heavy and behaves as a rigid body, while a steel structure is more flexible. Even if such a comparison is very superficial, one can understand the principle easily.

Eurocode 8 clearly defines such different behaviour types by introducing the notion of a dissipative structure, which is able to dissipate energy by means of ductile hysteretic behaviour and its predetermined dissipative zones, where the dissipative capabilities are mainly located. Therefore, in each project, an engineer has two design options which are said to lead to dissipative and/or non-dissipative (or low-dissipative) structures. The difference between these two behaviours is shown in Figure 1.

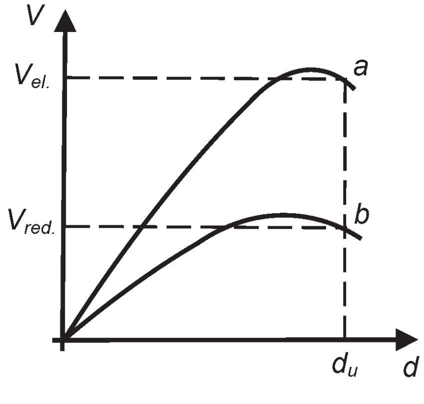

Fig. 1 *Examples of dissipative and non-dissipative global behaviours of structures (according to ArcelorMittal): a – low-dissipative structure, b – dissipative structure, $V_{el.}$ – structure designed to remain elastic under a design earthquake, V_{red} – structure designed to yield under a design earthquake.*

3. DUCTILE BEHAVIOUR OF A STEEL STRUCTURE

When designed according to Eurocode 8 (EN 1998-1), the ductile behaviour of a steel structure has an impact upon the different levels of its design. At the beginning of a project, the engineer has to choose to which ductility class the structure belongs:

- ductility class L (low),
- ductility class M (medium),
- or ductility class H (high).

It is important to know that the choice of ductility class only follows technical and economic reasons, except for contingent geographical limitations on the use of the ductility classes M and H, which may be found in the relevant National Annex. The final safety of the designed structure would necessarily be the same for each ductility class.

In terms of the Eurocode 8 design procedure, each ductility class is characterized by a more or less exhaustive application of the prescribed verification criteria, capacity design rules and constructional requirements.

In France, the use of the ductility class L is reserved only for cases where the ground acceleration $a_g = a_{gR}\,\gamma_I$ is between 0.78 m/s^2 and 1.1 m/s^2 (NF EN 1998-1/NA). For $a_g < 0.78$ m/s^2 or $a_g S < 1$ m/s^2, the seismic activity of the construction site is too low; the Eurocode 8 is not to be used, and the structure is to be designed by applying only the rules of Eurocode 3 (EN 1993-1-1). Some experts have introduced the notion of an intermediate ductility class DCL+ (DCL+ using $q = 2$, sometimes called DCM+), where a structural design is subjected to some specific conditions, i.e., $a_g S \leq 0.25\,g$, as mentioned by Aribert (2011).

Structures designed in accordance with the concept of dissipative behaviour should usually belong to structural ductility classes DCM or DCH. These two classes correspond to an increased ability of the structure to dissipate energy in plastic mechanisms. In most cases, seismic resistant structures are of the ductility class M, which matches best with the older French design code NF P 06-013 (also called PS 92). The use of the ductility class H enables the solution of some particular cases where the use of the M-ductility class would lead to an implausible economic solution.

In Slovakia the limit for very low seismicity defined in the National Annex matches $a_g S = 0.05\,g = 0.49$ m/s^2. For lower values of $a_g S$ the provisions of EN 1998 do not need to be observed at all. The ductility class DCL can be used for cases where the product $a_g S$ lies between $0.05\,g$ (0.49 m/s^2) and $0.1\,g$ (0.98 m/s^2).

Along the lines of the design procedure, the choice of the ductility class of a dissipative steel structure exerts an influence on:

- the quality of the material for structural members, including their connection elements,
- the structural type and value of the behaviour factor,
- the cross-sectional class of the structural members,
- the types and behaviour of the connections, etc.

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3.1 Material properties

The structural steel used for earthquake-resistant structures should conform to the standards referred to in EN 1993-1-1 with the following requirements:

- ratio $f_u / f_y \geq 1.10$,
- ductility $\varepsilon_u / \varepsilon_y \geq 15$,
- deformation at the failure $\geq 15\%$.

The choice of structural steel for any dissipative structure has to obey several code requirements, which guarantee that the dissipative zones will yield before the other zones working in the elastic range during the earthquake. In practice this means that in the design phase of a project, an overstrength factor is introduced to increase the nominal yield strength specified for the steel grade in a dissipative zone. Then, in the execution phase, the real value of the actual maximum yield strength $f_{y,max}$ of the structural steel in dissipative zones should not exceed $1.1.\gamma_{ov}.f_y$ (§6.2 (3) of EN 1998-1), where γ_{ov} is the material overstrength factor defined in EN 1998-1. Its nominal value depends on the steel grade (NF EN 1998-1/NA), or its value is taken as 1.25, but the value $1.1.\gamma_{ov}.f_y$ must not exceed f_u (STN EN 1998-1/NA). In such a case, the γ_{ov} value has to be adjusted.

Eurocode 8 also requires that the required toughness of the steel and welds at the lowest service temperature adopted in combination with the seismic action should be defined in the project specifications (§6.2 (7) of EN 1998-1).

It can be observed that the French National Annex to Eurocode 8 (NF EN 1998-1/NA) specifies additional requirements for the steel properties of the dissipative members of structures, the steel grade depending on the chosen ductility class, and the climate. The recommended steel grades are given in the following table in which the original part selected from the French National Annex is marked in gray. Some additional requirements for weldability are presented according to the Guide of Earthquake-Resistant Constructional Requirements for Steel, Concrete, Timber and Brick Structures by AFPS (2005).

In practice, such a definition of the minimally necessary steel grade as a function of the external temperature is easier to work with than to specify which service temperature should be adopted as a steel grade choice criterion. The final data, pinpointed from the above-mentioned table, is then specified on each project drawing.

Tab. 1 *The minimal construction steel grade and required toughness of welds as a function of the temperature (AFPS, 2005).*

Structures and Altitude		Ductility class	Base steel grade	Plate thickness (mm)	Required weld toughness	Exigence in Charpy notch test
Exterior structures or interior structures in unheated spaces	H < 500 m	DCL	(JR)	($t \leq 50$)	-	-
		DCM	J0	$t \leq 30$	27 J at 0°C	R*
			J2	$30 \leq t \leq 50$	27 J at -20°C	
		DCH	J2	$t \leq 30$	27 J at -20°C	R
			K2 (M, N)	$30 \leq t \leq 50$	40 J at -20°C	
	500 ≤ H < 1000 m	DCL	J0	$t \leq 50$	27 J at 0°C	-
		DCM	J2	$t \leq 30$	27 J at -20°C	R*
			K2 (M, N)	$30 \leq t \leq 50$	40 J at -20°C	
		DCH	K2 (M, N)	$t \leq 30$	40 J at -20°C	R
			L2 (ML, NL)	$30 \leq t \leq 50$	60 J at -20°C	
Interior structures in heated spaces		DCL	-	-	-	-
		DCM	JR	$t \leq 30$	27 J at +20°C	R*
			J0	$30 \leq t \leq 50$	27 J at 0°C	
		DCH	J0	$t \leq 30$	27 J at 0°C	R
			J2	$30 \leq t \leq 50$	27 J at -20°C	

* *If the toughness of the weld material used is higher than the required one and the absorbed energy exceeds the necessary minimal one, we can do without a test.*

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3.2 Behaviour factor

A behaviour factor reflects the capacity of a structure to deform plastically. The energy dissipated in plastic mechanisms can contribute significantly to the energy absorption in a structure submitted to an earthquake. The behaviour factor *q* is thus an approximation of the ratio of the seismic forces that a structure with a completely elastic response would experience to the seismic forces that may be used in the design to still ensure a satisfactory response of the structure.

Estimating the exact values of behaviour factors is a complex problem, so the seismic code recommends the values of *q* to be associated with a typology of structure reflecting its potential to form numerous dissipative zones. These values depend on the ductility class chosen for a given design and are influenced by the plastic redistribution parameter α_u/α_1, which characterizes the structural typology (see Tab. 2). They are to be considered as the maximum values allowed by the code, and a designer is free to choose values of *q* lower than those indicated.

Regarding their ductility, structures assigned to the DCL are to be calculated with a lower value of *q* than those in DCM or DCH. The use of DCL means taking into consideration the highest design forces, but only when performing the usual static design checks (EN 1993-1-1). For class DCH, the highest possible behaviour factor *q* is considered, and this approach results in the smallest possible design earthquake actions. The design forces M_{Ed}, V_{Ed}, N_{Ed} are reduced, often significantly, in comparison to those considered in the design of a non-dissipative structure; however, this is not a case for displacements which are the same for any value of *q*, because they are only related to elastic design spectra.

A design engineer should not forget that the *q*-values given in Table 2 are not only the maximum ones, but also reference the nominal values, so other corrections may be necessary with regard to the general features of the structure. For instance, if the structure is irregular in elevation, the code mandates a reduction in *q* of 20% (§6.3.2 (2) of EN 1998-1).

The use of different values of *q* in two horizontal directions of the structure is allowed; however, dissipative zones in both directions must be of the same ductility class (§4.3.3.5.1 (4) of EN 1998-1).

3.3 Structural members and their connections

The choice of ductility class is also important with regard to the design of individual structural members and their connections. Depending on the ductility class and the behaviour factor *q* used in the design, the Eurocode 8 prescribes the cross-sectional classes for all the dissipative steel elements (see Table 3).

Tab. 3 *Requirements for the cross-sectional class of dissipative elements, depending on the ductility class and behaviour factor.*

Ductility class	Reference value of behaviour factor q	Required cross-sectional class
DCM (sometimes called DCM+ or DCL+)	$1.5 < q \le 2$	1, 2 or 3
DCM	$2 < q \le 4$	1 or 2
DCH	$q > 4$	1

The aptitude of steel members for consuming energy is essential in dissipative zones. Therefore, with higher ductility, the use of cross-sectional class 1 is very important. This means that the given cross-section can form a plastic hinge with the rotation capacity required by the plastic analysis without any reduction of its resistance.

Tab. 2 *The maximum values of behaviour factor q according to EN 1998-1.*

Structural type	Ductility class		
	DCL	DCM	DCH
Moment resisting frames (MRF)	1.5 (2*)	4	$5\ \alpha_u/\alpha_1$
MRF with concentric bracing	1.5 (2*)	4	$4\ \alpha_u/\alpha_1$
MRF with unconnected concrete or masonry infill in contact with the frame	1.5 (2*)	2	2
MRF with masonry infill disconnected from the frame	1.5 (2*)	4	$5\ \alpha_u/\alpha_1$
Concentric diagonal bracing (Saint-Andrea's X crosses)	1.5 (2*)	4	4
Concentric V-bracings	1.5 (2*)	2	2,5
Eccentric bracings	1.5 (2*)	4	$5\ \alpha_u/\alpha_1$
Inverted pendulum	1.5 (2*)	2	$2\ \alpha_u/\alpha_1$

** The French National Annex can allow q=2 in class DCL only when prescribed conditions are met. In Slovakia the value 1.5 is prescribed.*

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A strict application of the capacity design required by Eurocode 8 is essential to ensure the reliability of the dissipative zone and to guarantee the good earthquake behaviour of the structure in seismic areas. The capacity design applied to different members (for example, §6.6.3 (1)P of EN 1998-1 for columns of moment-resisting frames, or §6.7.4 (1) for beams and columns of structures with concentric bracings, etc.) is to be applied to the related connections as well. The design rule for rigid full strength connections is common to all types of structures. It says that the resistance R_d of non-dissipative connections should satisfy:

$$R_d \geq 1.1\, \gamma_{ov}\, R_{fy}$$

where R_{fy} is the plastic resistance of the connected dissipative member, based on the design yield strength, and not on the design force calculated in this member. The factor γ_{ov} is the material overstrength factor. The rule applies to non-dissipative connections using fillet welds or bolts. When full penetration butt welds are used, they automatically satisfy the capacity design criterion.

The French Technical Industrial Institute for Steel Structures (CTICM) has developed several papers recommending the principles of the concept, the calculation procedures and the testing methods for steel connections used in seismic zones. Aribert (2011) presents some of them. Some typical examples of non-dissipative connections are given in Figure 2.

Similarly to the cross-sectional requirements, sufficient rotation capacity is prescribed for the steel member connections as well. For frame structures, for example, Table 4 gives the required minimum values of the plastic rotation capacity of a beam-to-column connection. The symbol θ_p corresponds to the rotation angle of the half-length of the beam related to the column axis.

Tab. 4 *Required minimal plastic rotation of the beam-to-column connection of a moment-resisting frame structure according to EN 1998-1.*

Ductility class	DCM	DCH
Rotation angle θ_p	25 mrd	35 mrd

Such a requirement means that upon completing a structural design, a design engineer should calculate the rotation capacity of the connection and verify if the possible rotation angle is higher than the required minimal value.

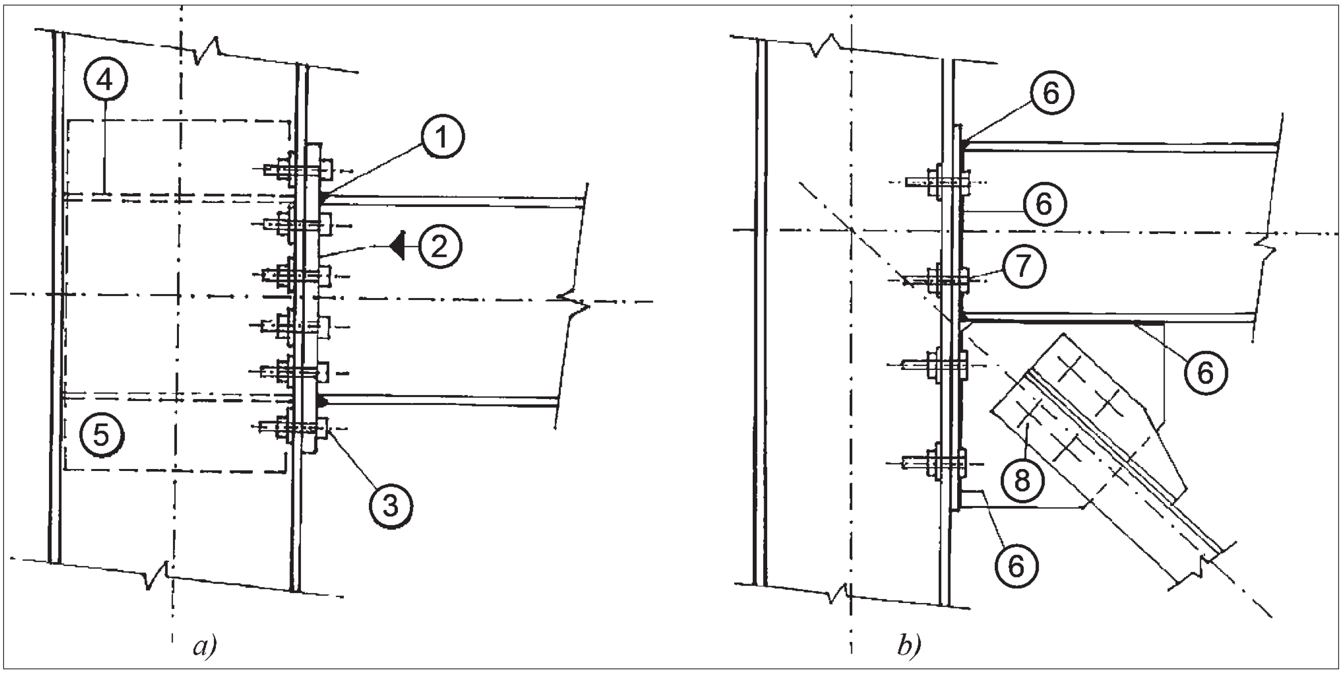

Fig. 2 *Examples of non-dissipative connections conceived to satisfy the capacity design: a) moment-resistant beam connection; b) diagonal bracing connection; 1 – full penetration welds in DCM or DCH, fillet or partial penetration welds in DCL+; 2 – full penetration welds in DCH or fillet welds in DCM; 3 – high resistance bolts; 4 and 5 – transversal stiffeners or supplementary web plates if necessary; 6 – fillet or partial penetration welds; 7 – preloaded bolts; 8 – category C bolts in DCH and category B bolts in DCM or DCL+.*

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4. EXAMPLE OF THE SEISMIC DESIGN OF A STEEL STRUCTURE

4.1 Input data

The project is situated in France. The example given presents a simple industrial structure with a rectangular basis of 8.60 m x 9.00 m. The structure is composed of two parts: the first, rather low with the highest roof point at +4.80 m, and the second, a tower-like multi-storey part reaching +17.30 m (including two roof shelters). Its four intermediate floors, as well as its roof, all have two square openings, which enable the passage of tube equipment with chimneys. The construction elements are all made from standard steel profiles (I or H for beams and columns and circular tubes for bracings). All the exterior walls are covered in simple sheet metal cladding.

The standard design situation for the project considered the permanent loads of the structure itself, the wall and roof cladding, the floor grating and other small structural accessories. The service load is the same for all four floors (250 kg/m²). Other design actions considered were: the temperature (±27°C), snow (the characteristic value in the region was taken as equal to 65 kg/m²) and wind (a characteristic wind pressure of 103 kg/m² at the ground level and 135 kg/m² at +17.30 m).

By agreement with the client, an accidental design situation to take into account the possibility of extreme climatic loads was stipulated (exceptional snow load of 135 kg/m², and extreme wind pressure of 235 kg/m² considered for the highest roof point).

The third design situation was the seismic one. In this situation, the permanent load remains the same, but the service load on the floors is reduced to 50 %.

The project's contractual documentation (calculation hypothesis and design methodology) established according to preliminary guide drawings assumed that all the steel structures were to be designed using a dissipative concept and were all to be of the medium ductility

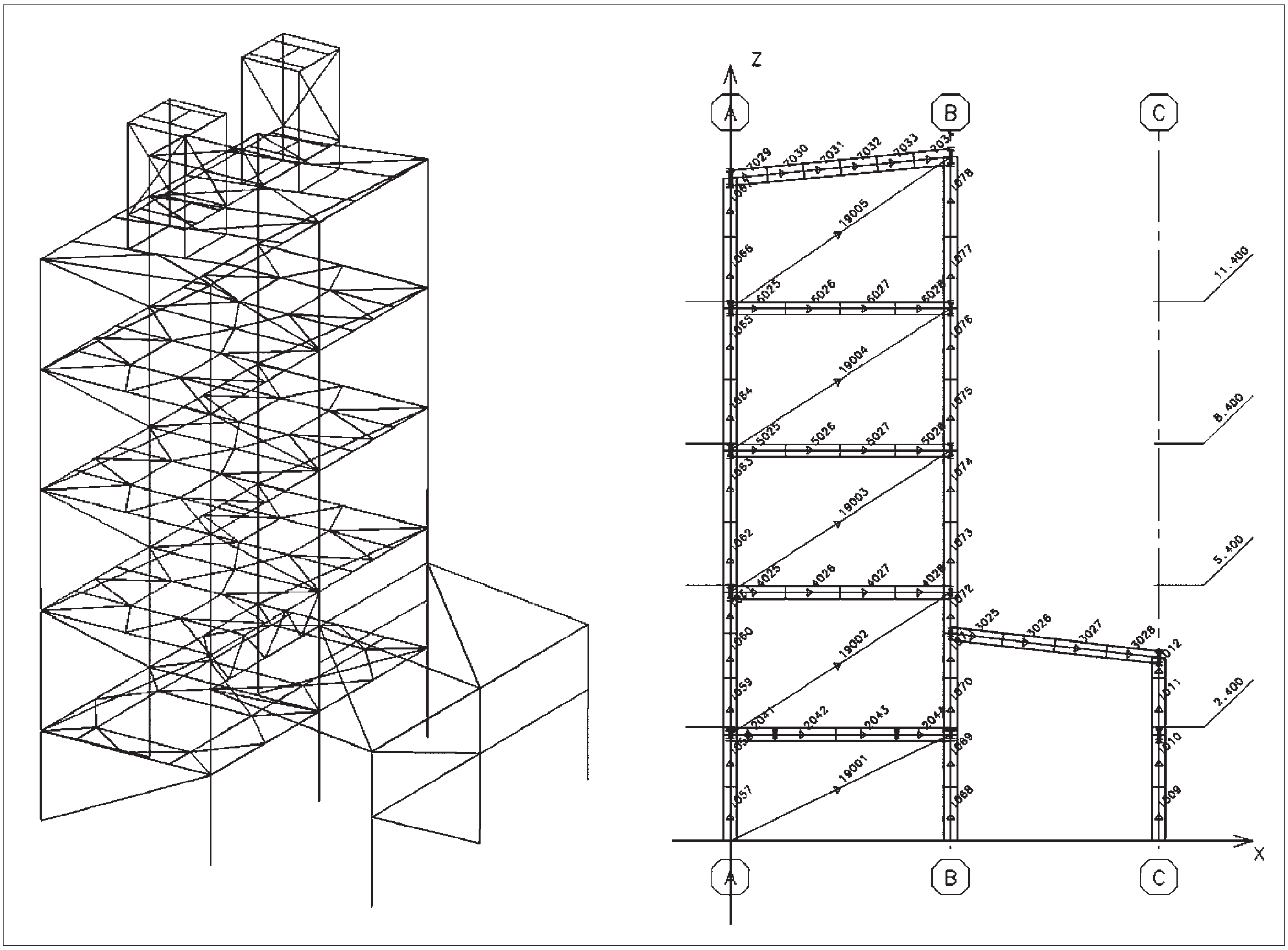

Fig. 3 *Geometry of the calculation model: perspective view (on the left), profile view of one of the walls.*

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class (DCM). Such a hypothesis was common for all the structures to be built on this industrial site.

According to EN 1998-1, this simple structure represents a multi-storey system of frames with concentric bracings, where the dissipative zones are mainly located in active tension diagonals. The compression diagonals were ignored.

The same approach was adopted for the finite element calculation model drawn up in HERCULE (Socotec). All the primary structural elements were defined as beams (horizontal beams or columns) with appropriate hinge connections. For each concentric bracing, only one diagonal was modelled (Figure 3) and defined as a truss member supporting only axial loading. The column foundations were represented by simple hinges. The final computational model was composed of 628 finite elements and 393 nodes.

With provisions for the modal response analysis and spectral analysis to be performed, only the linear behaviour of the model was considered. For this reason, the sign of the design axial forces in the diagonals was always considered as positive (tension), even if the calculation gave the opposite sign. The nominal value of the corresponding behaviour factor q is equal to 4.

The following figure shows the horizontal design spectrum in which the design value of the behaviour factor $q = 3.2$ is already included (a reduction of 20% due to the irregularity of the shape).

Regarding a seismic event, only the horizontal earthquake components EX and EY were considered, because the vertical acceleration was less than 0.25 g (§ 4.3.3:5.2 (1) of EN 1998-1). This simplified the set of the twenty-four Newmark seismic combinations to only eight combinations.

4.2 Modal analysis of the structure

As mentioned in 4.1, a reduction of 50% was applied to the floor service load in the seismic design situation. The total volume of the active seismic mass represented 81 t (permanent mass 60 t, and service mass during earthquake 21 t).

In order to obtain precise calculations, the modal analysis was performed up-to 15 vibration modes. However, only the first two modes were sufficient to mobilize over 80% of the active mass. Their characteristics are given in Tab. 5.

As a supplement, a residual mode was also considered. A complete quadratic method was used to combine the different modal results (load cases EX and EY), which were signed according to the two simple load cases, considering the uniform acceleration fields in the X and Y directions.

Tab. 5 *Calculated characteristics of first two vibration modes.*

Mode n°	Period (s)	Frequency (Hz)	Active mass (%)	
			X (west-east)	**Y** (south-north)
1	0.4558	2.19	0.0	**81.7**
2	0.4476	2.23	**79.1**	0.0

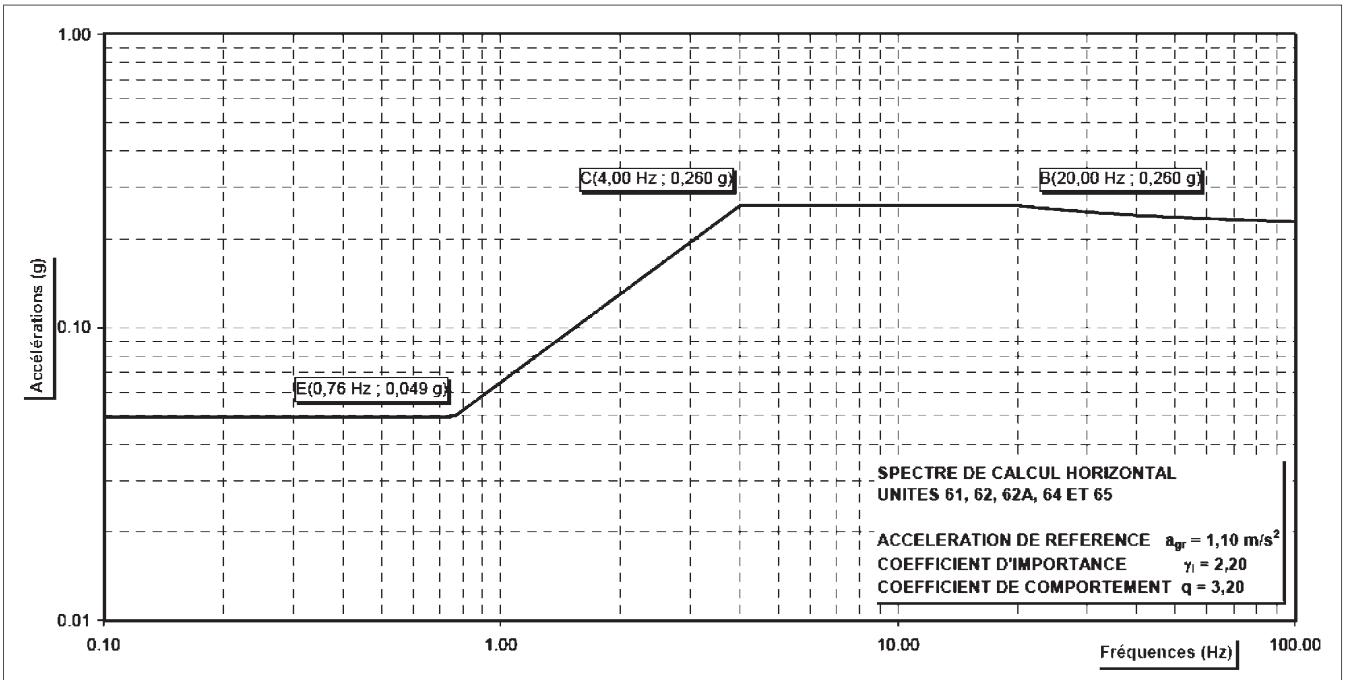

Fig. 4 *Design spectrum applied in both horizontal directions.*

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4.3 Principal earthquake design criteria according to EC 8

In a structural design according to EN 1998-1, the first order analysis was sufficient, because the maximum value of θ did not exceed 0.035 (§ 4.4.2.2).

The diagonal members were verified in tension by considering the most unfavourable design axial force issued from the standard, seismic and accidental design situations following these sequences:

- Inspection of the cross-sectional class (class 1 required contractually),
- Verification of the non-dimensional slenderness $\overline{\lambda}$ as defined in EN 1993-1-1. According to EN 1998-1, the calculated value was limited to $1.3 < \overline{\lambda} \leq 2.0$.
- Check that the maximum overstrength $\Omega_i = N_{pl,Rd,i} / N_{Ed,i}$, defined in § 6.7.4(1) of EN 1998-1, did not differ from the minimum value Ω_{min} by more than 25% in order to satisfy the homogeneous dissipative behaviour of the diagonals. $N_{Ed,i}$ is the design value of the axial force in the diagonal i in the seismic design situation.

The non-dissipative connections of the diagonal members made using a fillet weld and bolts were designed to satisfy the following criterion:

$$R_d \geq 1.1\, \gamma_{ov}\, R_{fy}$$

where R_d is the resistance of the connection in accordance with EN 1993-1-1; R_{fy} is the plastic resistance of the connected dissipative member based on the design yield stress of the material as defined in EN 1993-1-1 (= $N_{pl,Rd}$); and γ_{ov} is the overstrength factor taken as equal to 1.15 for the diagonals made from S 355 (according to NF EN 1998-1/NA).

The beams and columns related to each bracing system are the non-dissipative elements, and they have to be designed according to

$$N_{pl,Rd}\,(M_{Ed}) \geq N_{Ed,G} + 1.1\, \gamma_{ov}\, \Omega\, N_{Ed,E}$$

where $N_{pl,Rd}(M_{Ed})$ is the design buckling resistance of the beam or the column in accordance with EN 1993-1-1, taking into account the interaction of the buckling resistance with the bending moment M_{Ed}, defined as its design value in a seismic design situation,

$N_{Ed,G}$ is the axial force in the beam or in the column due to the non-seismic actions included in the combination of actions for the seismic design situation;

$N_{Ed,E}$ is the axial force in the beam or in the column due to the design seismic action;

γ_{ov} is the overstrength factor and defined above;

Ω is the minimum value of the overstrength Ω_i, both defined above.

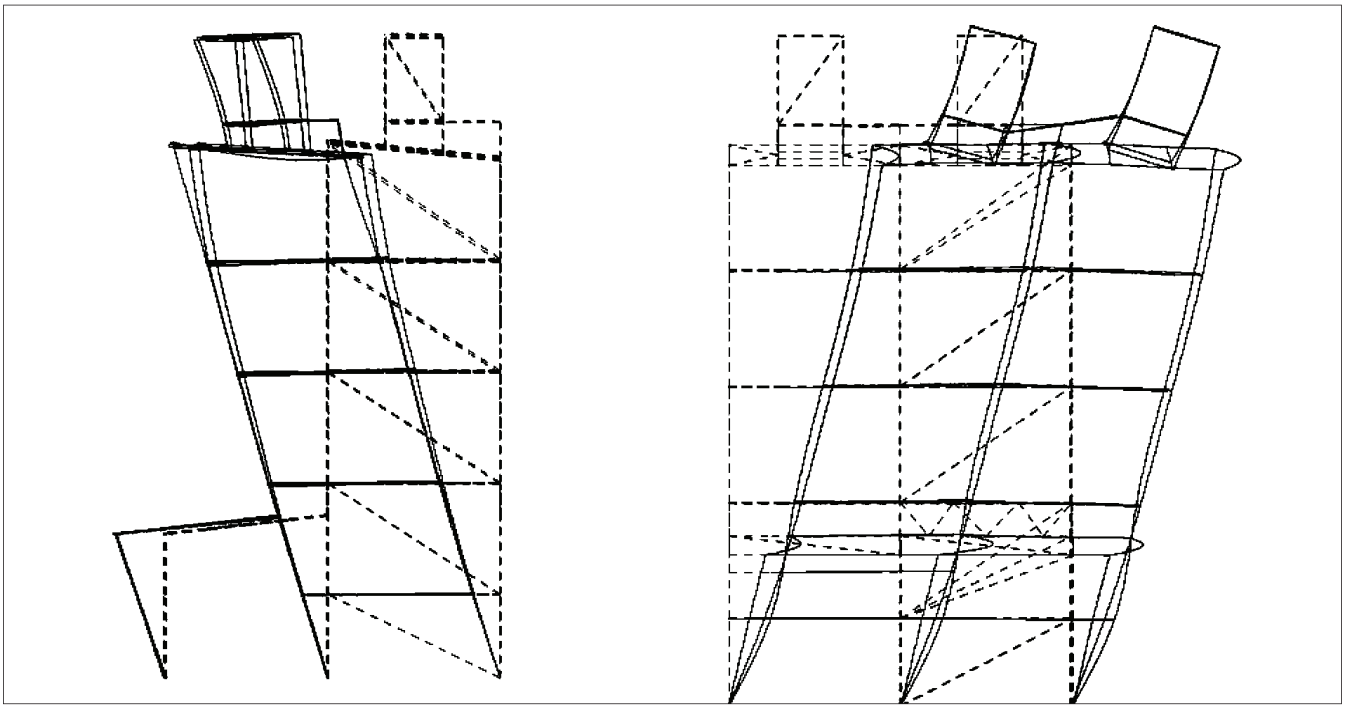

Fig. 5 *Graphic representation of the first two vibration modes.*

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According to §6.4 (1) of EN 1998-1, each intermediate floor as well as the roof were considered as active **horizontal diaphragms,** which should have conformed to §4.4.2.5 of the same code. This means that in the seismic design situation, **all the primary floor and roof elements,** such as all the primary beams which are not related to the vertical bracing systems but to the horizontal diagonals, were designed to the value of the axial force obtained from the calculations and multiplied by an overstrength factor γ_d greater than 1.0. For the given project, the axial resistance of each diaphragm element was taken as

$$N_{pl,Rd}\ (M_{Ed}) \geq N_{Ed} = N_{Ed,G} + 1.3\ N_{Ed,E}$$

4.4 Some comments on the design results

The first design step was to analyze the different bracing systems because they have to transfer all the horizontal action applied on the structure to the column anchorage system. Three design situations: ULS – standard, ULS – seismic, and ULS – accidental (extreme climatic actions) were separately analyzed.

As an example, different diagonal parameters for two bracing systems are presented in Tab. 6. The diagonal in the lowest position is marked as the first one; the diagonal on the top of the system is called the fifth. The second column of the table gives the diagonal cross-section. As mentioned in §4.1, in order to satisfy the cross-sectional class, any angles which usually are in class 3 were avoided, and circular tubes were chosen. For practical construction reasons, the minimal section used for this project was defined as Ø 60.3 mm (external diameter) / 3.2 mm (tube thickness). Considering the geometry of each storey, the slenderness $\bar{\lambda}$ was verified.

In the two next columns of Tab. 6, the calculated utility ratio is presented separately for standard or accidental situations and a seismic situation. The results clearly show the design character of the wind load, which is more important than the seismic action. While the maximum utility ratio of the first diagonal is over 88 % under a wind load, it is only 21 % in a seismic situation. For this reason, the minimal value of the overstrength Ω_{min} is rather high (more than 5 for the northern bracing system), and it is bigger than the value of the behaviour factor $q = 3.2$. Such a result means that no bracing diagonal will yield and that the whole system will conserve its elastic behaviour in the given design seismic situation.

With regard to these results, it was decided to respect the earthquake design criteria for beams and columns according to §4.3 of this article, but the conditional term 1.1 $\gamma_{ov}\ \Omega_{min}$ was limited to 1.1 $\gamma_{ov}\ q$, which gives a value of 1.1 x 1.15 x 3.2 = 4.048. So all the calculated axial forces in the beams or in the columns due to the design seismic action were multiplied by 4.048. In practice, this corresponds to the same approach as designing all the beams and columns for a seismic event considered with an elastic spectrum and $q = 1$ without considering any plastic behaviour.

For this structure (the only exception from the other buildings designed for the same project), it was decided not to satisfy the last design criteria for the dissipative bracing system ($\Omega_{max}\ /\ \Omega_{min} \leq 1.25$) and to conserve all the cross-sections as given in Tab. 6. The bracing system's capacity to dissipate the seismic energy would not be homogeneous from one floor to another; however, while the structure remains only in an elastic domain, there is no meaningful application of this design criterion. Moreover, if it were decided to satisfy this design condition, the correct calculation of five successive diagonal

Tab. 6 *Calculation results for two of the vertical bracing systems.*

Bracing system	Tubular cross-section	Slenderness $\bar{\lambda}$	Utility ratio at ULS		Ω_{min} (by system)
			SDT / ACC	SEI	
BRACING SYSTEM AT THE SOUTHERN FACE					
1st diag. element	60.3 mm / 5.0 mm	1.67	0.88	0.21	4.76
2nd diag. element	60.3 mm / 5.0 mm	1.80	0.86	0.21	
3nd diag. element	60.3 mm / 5.0 mm	1.80	0.76	0.18	
4th diag. element	60.3 mm / 3.2 mm	1.75	0.77	0.17	
5th diag. element	60.3 mm / 3.2 mm	1.78	0.49	0.11	
BRACING SYSTEM AT THE NORTHERN FACE					
1st diag. element	60.3 mm / 5.0 mm	1.67	0.83	0.20	5.10
2nd diag. element	60.3 mm / 5.0 mm	1.80	0.81	0.20	
3nd diag. element	60.3 mm / 5.0 mm	1.80	0.71	0.17	
4th diag. element	60.3 mm / 3.2 mm	1.75	0.73	0.16	
5th diag. element	60.3 mm / 3.2 mm	1.78	0.47	0.10	

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levels would lead to highly overdesigned cross-sections of the diagonal members on the first three floors at least and also to a very significant overdesign of all the diagonal connections. All the other design conditions according to EN 1998-1 were respected and satisfied.

5. CONCLUSIONS

Through the article presented, we can see how the choice of the ductility class of a steel structure acts in the different stages of a construction design.

At the beginning of each project the design engineer is often free to choose the ductility class. A non-dissipative or low-ductility class DCL structure is designed according to Eurocode 3, with checks for the usual resistance to service loads, wind loads, etc. The Eurocode 8 only defines the seismic action under which the structure is expected to behave elastically.

A dissipative structure designed in DCM or DCH is calculated for a seismic action which is lower than that used in a DCL design, because the value of the behaviour factor q is greater. As a result, the weight of the structural elements can be substantially reduced, although the design method following Eurocode 8 is longer, and there are restrictions on the classes of the sections, the connections, the materials and the control of the material properties.

However, the practical example of a dissipative structure presented here does not result in a more competitive solution, because the seismic checks were not critical.

In every case, a seismic design has to respect classical design (SLS criteria, resistance to service loads, wind loads, etc.), but this may govern the size of the sections needed. In such a case, the capacity design according to Eurocode 8 may result in dissipative sections which have a greater overstrength, which then leads to an overstrength and more weight for the other structural elements. This situation can occur in areas where the seismic activity is lower than other design actions. It can also occur for light and flexible structures, because it is important to show that the choice of a ductility class for a given design should also depend on the mass/volume ratio of the structure. If the structure is an empty envelope, for example, an industrial building, the sum of the wind load can be greater than the earthquake shear, so designing for a higher ductility is pointless.

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