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Research Paper

Parametric study of the seismic vulnerability of steel structures and their vulnerability curves

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ABSTRACT

The use of metal structures is increasingly used. This is due to the fact that these structures are quick to erect (saving time and money) and make it possible to obtain interesting technical characteristics (spans and heights). These structures can be built in seismic zones and therefore despite their ductility can be damaged. This aspect has been very little addressed. Therefore, it is proposed within the framework of this study to investigate the seismic vulnerability of steel structures using the "vulnerability index" method. Parameters having an influence on the seismic behavior of steel frames were identified and then weighting coefficients for these parameters were calculated using the "Push-over" method. To do this, finite element models were developed and vulnerability classes were defined. Damage probability matrices and seismic vulnerability curves as well as a classification of metallic structures according to their vulnerability were developed. Validation and application cases have been processed and the obtained results are in adequacy what observations made in situ.

1 Introduction

Although steel-frame structures represent a smaller percentage of buildings than masonry or reinforced concrete structures, they are nevertheless very prevalent in industrial parks [1-2]. The interest in this type of buildings is due to their speed of construction, flexibility and large spans [3]. These installations, which represent a heavy investment, can be located in seismic zones. In the event of earthquakes, these structures can be severely damaged, resulting in direct losses that are easily quantifiable and indirect losses that are very difficult to assess, as well as loss of human life [4-5].

A few seismic vulnerability studies on steel structures have been carried out [6-7]. They are mainly large-scale studies and therefore only consider this type of construction roughly [8-11] or detailed studies (for selected structures) and therefore require skilled personnel and specific tools, which make them expensive [12]. Between these two ways of doing things, there

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are intermediate methods that combine simplicity and precision, among them is the vulnerability index method. This method was first used for masonry constructions in Italy [13-14] and then extended to the southern European countries [15]. Finally, a guide for the evaluation of the seismic vulnerability of buildings has been elaborated by the AFPS group [16]. Other studies using the same principle have emerged, such as the RISK-EU method [9], the modified vulnerability index method developed by Vicenté et al [16] and the ReLUIS method [17]. Studies on the use of this method to reinforced concrete and masonry structures applied to the Algerian context do exist [18-19]. The development of vulnerability curves has also been undertaken in several studies [9-13, 18-19].

In the present work, the vulnerability index method is used to identify the parameters having an impact on the seismic response of steel structures and then calculate a vulnerability index for these ones. The quantification of weightage coefficients of the various parameters identified according to the classes of constructions considered is carried out using push-over analyses on finite element models developed for this purpose. Probability damage matrices are also developed to allow the construction of seismic vulnerability functions for steel structures.

2 Vulnerability Index Method

The method consists of assigning to a structure a vulnerability index indicating its capacity to be damaged by an earthquake. To do this, a number of structural and non-structural parameters are considered. These parameters are assigned a numerical value according to their vulnerability class and the sum of these numerical values will constitute the vulnerability index (VI) of the studied structure [20-22].

The main steps of the method are:

1) Identification of structural and non-structural parameters
2) Definition of the vulnerability classes of these parameters
3) Determination of weightage coefficients for each parameter and for each vulnerability class
4) Calculation of the vulnerability index (VI) and classification of the studied structure

3 Parameters Identification and Class Vulnerability Definition

First of all, the different parameters affecting the seismic vulnerability of steel constructions are identified through post-seismic observations and seismic experience feedback. A database containing 297 steel-frame structures that have been damaged in various earthquakes around the world is used [01-02]. On this basis fourteen (14) parameters have been identified. They are given in Table 1:

Table 1 – Identified parameters

N°	Parameters	N°	Parameters
1	Ductility	8	Plan regularity
2	Bearing capacity	9	Modifications
3	Assemblages	10	Elevation regularity
4	General maintenance conditions	11	Ground conditions
5	Type of soils	12	Pounding effect
6	Horizontal diaphragms	13	Roof
7	Buckling	14	Details

Three classes of vulnerability have been defined, namely classes A, B and C.

Class A expresses a parameter having or leading to a good behaviour of the structure during an earthquake and therefore no or little damage is recorded.

Class C, expresses a parameter having or causing a bad behaviour of the structure during an earthquake and therefore important damages or even collapse are observed.

Class B represents an intermediate situation between the two classes mentioned above.

4 Quantification

The calculation of the weightage coefficients for each parameter and for each vulnerability class is carried out by numerical modelling procedures. Finite element models were developed for each parameter and then push-over analyses were performed to determine the weightage coefficients. The quantification of these coefficients was determined on the basis of the top displacement of the studied structures.

4.1 Principle of the Pushover analysis

It is a static calculation, the method is based up on the establishment of a single force-displacement curve to characterize the behaviour of the structure by pushing until a state of plastic damage is reached that is considered to represent the limit of what is acceptable for safety. Taking into account a non-linear model, and the horizontal forces applied to the mass level of the model structure. This in order to reproduce the inertial forces representative of the seismic action. These forces have a distribution generally similar to that of the displacements of the fundamental mode of vibration of the structure [12].

The curve (Fig. 1) showing the behaviour of the structure is plotted with the top displacement delta on the abscissa and the basic shear force V (the sum of the horizontal forces) on the ordinate. It is an intrinsic characteristic of the structure from the point of view of the effect of horizontal actions, whether static or dynamic in nature. It provides an estimate of the plasticization mechanisms and the distribution of the progressive damage, as a function of the force intensity and horizontal displacements [23].

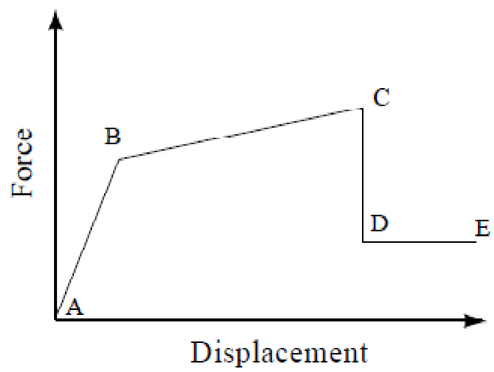


Fig. 1 – Push over curve.

The curve given in 1 figure 1 is composed of four segments and each segment corresponds to a damage stage. This is described hereafter.

"Segment AB" or First level called Immediate Occupancy (IO): it corresponds to the elastic behaviour of the structure and represents the usual seismic design level. It therefore indicates a state of superficial damage (or lack of damage) because the structure retains a large part of its initial stiffness and resistance, in our case it is referred to as class A.

"Segment BC" or Second level called Life Safety (LS): corresponds to a controlled level of damage. The stability of the structure is not in danger, but however minor damage is likely to develop which could lead to a significant loss of stiffness, in our case it is referred to as class B.

"Segment CD" or Third level called Collapse Prevention (CP): It represents an advanced state of damage; the stability of the structure is being at risk.

"Segment DE": It is above the third level and the structure is prone to collapse, as it has no resistance capacity. It may become unstable and collapse. In our case Segments CD and DE are referred to as class C.

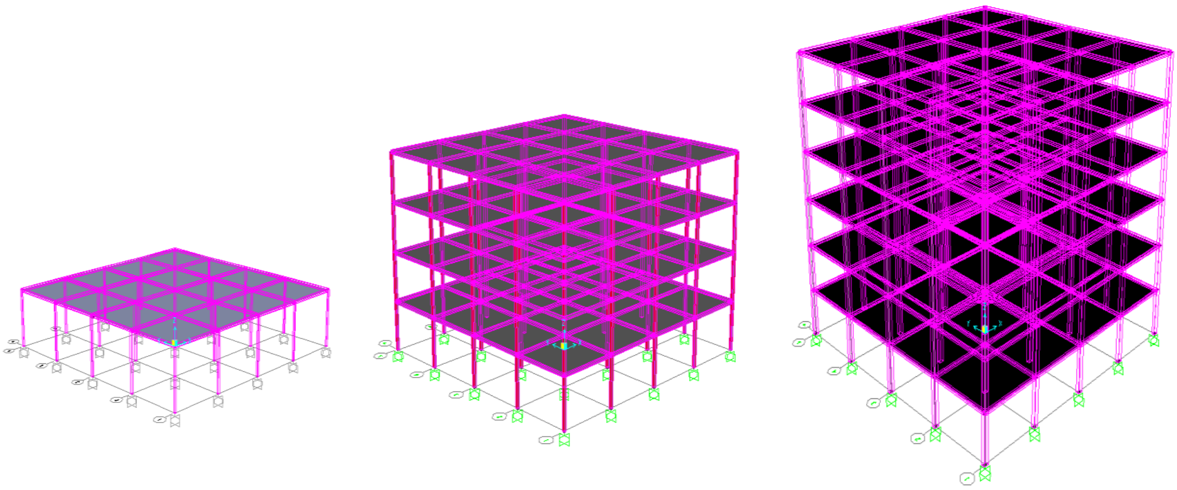
4.2 Used Models

Three steel buildings will be considered. The first one is a low-rise building (less than 6 m high with 1 to 2 levels). The second one is a mid-rise building from 6 to 15 m high with 3 to 5 levels and finally a high-rise building of more than 15 m and more than 5 levels [9].

The characteristics of the three modelled buildings are given in Table 2 and the finite element models are shown in Figure 2.

Table 2 – Models dimensions and used profiles sections.

	Length(m)	Width (m)	Total Height (m)	levels	Section beams	Section columns
Low rise	24	24	5	1	IPE 270	HEA 180
Middle rise	24	24	15	4	IPE 270	HEA 260
High rise	24	24	21	6	IPE 270	HEA 450



(a) low rise (b) middle-rise (c) high rise

Fig. 2 –Finite elements models.

Each identified parameter will be modelled in order to determine its weight in each of the three defined vulnerability classes. The maximum displacement at the top is used as a tool to quantify the weighting factors. To do so, non-linear static analyses will be performed.

4.3 Modeling Parameters

The definitions and characteristics of the various parameters that were identified are given in the following paragraphs. The purpose of this is to provide the necessary resources for their design and classification.

4.3.1 Ductility parameter

Under the effect of high-intensity seismic action, structures undergo deformations in the post-elastic range. They withstand a level of stress higher than that for which they were designed, because of their ability to dissipate energy. This is due to their non-linear behaviour provided by the ductility of steel [24-26].

In order to take these incursions into account, in the post-elastic domain, a behavioural coefficient is considered. In seismic codes this coefficient depends on the bracing system. It varies from 2 to 6 for the different typologies existing in the Algerian earthquake regulations [27]. According to the behaviour factor, the variation of the ductility from one vulnerability class to another is a function of the deformation of the material. In class A, the material remains in the elastic range, in class C, the material reaches the limit of rupture, whereas in class B, the material reaches the plastic range figure 3.

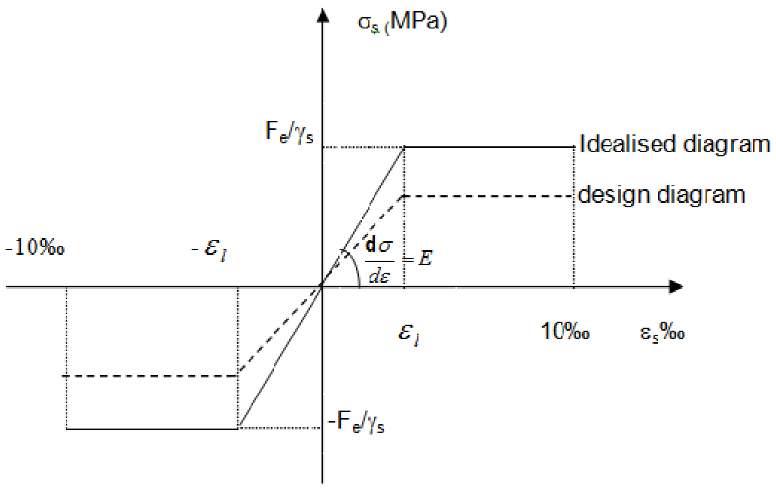


Fig. 3 – Stress - deformation diagram of steel.

4.3.2 Bearing capacity parameter

This parameter represents the comparison between the shear force at the base V_s induced by the earthquake, calculated with the Algerian regulations RPA 99 and CCM97, and the maximum shear force of columns V_t supported by the structure (plastic shear calculation) [24-26]. The calculation of the shear force V_t is done according equation. 1:

$$Vt = A\left(\frac{fy}{\sqrt{3}}\right)\gamma_{M0} \tag{1}$$

Where : A, γ_{Mo}, fy are respectively, shear area in the base of column, partial safety coefficient and yield strength.

This parameter is considered in class A, if the shear cross-section of the columns resists the shear force at the base induced by the earthquake and remains within the elastic range. On the other hand, if the shear cross-section of the columns does not resist the shear force at the base, the parameter is classified in class C. Finally, class B represents the intermediate situation.

4.3.3 Assemblage parameter

This is the point of passage for stresses, which must be transmitted from one component to another in a structure. Their failure directly affects the functioning of the whole structure. The modelling of this parameter was done by dividing the frame elements at the nodal zone [24, 29]. Thus, in class A, one finds the assemblies which work in the elastic domain. In class C, one finds the connections which reach the breaking limit, while in class B one finds the connections which reach the plastic domain.

4.3.4 General maintenance conditions

The main inconvenience of steel constructions is their corrodibility. Regular maintenance is necessary to maintain the resistance of buildings over time [30]. In the first class (A), the columns deform only in the elastic range, while in the second class (B), the columns reach the plastic range, while in the third class (C), the columns reach the limit of rupture, according to the curve shown in figure 3. In order to evaluate the maintenance parameter the modulus of elasticity E and the deformation (ε) of the steel were taken as variables in non-linear cases.

4.3.5 Type of soil

The soil type parameter is defined by its rigidity. According to the Algerian regulation soils are classified into four categories. These are rocky soils with a mean shear wave velocity exceeding 800 m/s, firm soils with a mean shear wave velocity exceeding 400 m/s, as well as soft soils with a mean shear wave velocity exceeding 200 m/s, and finally very soft soils with a mean shear wave velocity not exceeding 200 m/s [25]. The soil is modelled according to its stiffness factor [28]. This one is calculated according to equations (2) to (7):

Translation according to Z :

$$Kz = \frac{GB}{1-\gamma}\left[3.1\left(\frac{L}{B}\right)^{0.75} + 1.6\right] \tag{2}$$

Translation according to Y :

$$Ky = \frac{GB}{1-\gamma}\left[6.8\left(\frac{L}{B}\right)^{0.65} + 0.8\left(\frac{L}{B}\right) + 1.6\right] \tag{3}$$

Translation according to X :

$$Kx = \frac{GB}{1-\gamma}\left[6.8\left(\frac{L}{B}\right)^{0.65} + 2.4\right] \tag{4}$$

Torsion around Z:

$$Kzz = GB^2\left[4.25\left(\frac{L}{B}\right)^{2.45} + 4.06\right] \tag{5}$$

Torsion around Y:

$$Kyy = \frac{GB^2}{1-\gamma}\left[3.73\left(\frac{L}{B}\right)^{2.4} + 0.27\right] \tag{6}$$

Torsion around X:

$$Kxx = \frac{GB^2}{1-\gamma}\left[3.2\left(\frac{L}{B}\right) + 0.8\right] \tag{7}$$

Where: G : Shear modulus; L : Foundation length; B : Width of the foundation; γ : Poisson's ratio.

4.3.6 Horizontal Diaphragm

The diaphragm is a rigid horizontal plane, providing three main functions: Transmitting horizontal seismic loads on vertical bracing elements, stiffening buildings, and coupling vertical elements [25]. In the class A, all the floor nodes are selected and assigned a diaphragm. In the class C, the floor nodes are not connected to the vertical bracing elements, while in the class B half of the nodes are connected to the bracing system.

4.3.7 Buckling

This parameter consists in checking the resistance of the load-bearing elements "columns" to buckling [24, 31]. This is determined by equation 8, where once the P_{CR} load is exceeded, stability is lost.

$$P_{CR} = \frac{EL\pi^2}{L_{eff}} \tag{8}$$

With: E : longitudinal modulus of elasticity; I : moment of inertia of the profile; L_{eff} : length of buckling.

In the first class A, the basic compressive force is in the elastic domain. In the class C the compressive force is not supported by the section of columns at the base and the class B is dedicated to intermediate situation.

4.3.8 Plan regularity

The aim is to verify the criteria imposed by the regulation, which concern the distribution of mass, rigidity and geometric form in planes [25]. In class (A), buildings are modelled with equal load distribution. In class (B) a modification is made to the permanent loads or to the operating loads, finally in class (C) a modification is made for both permanent and operating loads.

4.3.9 Modification parameter

A change in the structural system or in the operating load or live load can have a negative impact on the behaviour of a structure under seismic action [30]. In class (A), the use of the construction and the bracing system is not modified compared to the initial state. In class (C), the modification is done in the use of the structure (increase in mass) and in bracing elements. In class (B), the modification is in made either in the use of the construction or in the bracing system.

4.3.10 Elevation regularity

This involves verifying two essential aspects related to [25]:

- The variation of the distribution of the mass, between two successive levels
- The variation of the rigidity of the bracing system, between two successive floors

The same modelling principle will be used as for the plan regularity parameter, except that the change of masses is done from one floor compared to another.

4.3.11 Ground conditions

This parameter deals with cases where the structure is located and is likely to aggravate the seismic action. These cases are constructions located: i) on unstable ground, ii) at the end of a cliff, iii) at the top or bottom of a hill, iv) on the banks of a river, v) on uneven ground [25]. In Class A, all supports are restrained. In class C, all supports are simply supported, while in class B, half of the supports are restrained.

4.3.12 Pounding effect

Two structures built in close proximity to each other must be separated by a seismic joint in accordance with the regulation in use. The absence of a seismic joint could cause damage to the structure due to the hammering effect. This parameter was verified according to equation (9) given by the Algerian seismic code RPA99.v.2003. This formula gives the minimum distance separating between two adjacent buildings.

In Class A, the structure is isolated or the distance between two adjacent buildings is greater than 40 mm. In this case, the weighting factor is zero. In class B, the structure has one or more adjacent buildings with d $\leqslant$ 40 mm. In this case, the weighting factor is calculated according to the difference in joint thickness (40 mm - d). The thickness (d) is calculated for each building according Eq. 9. In class C, the structure has one or more adjacent buildings with d = 0 mm. In this case, the weighting factor is taken equal to one..

$$d_{\min} = 15mm + (\delta_1 + \delta_2) \geq 40mm \tag{9}$$

Where: $d_{\min}$: calculated joint thickness; δ_1: maximum displacement of building 1; and δ_2: maximum displacement of building 2.

4.3.13 Roof parameter

The roof is the upper part of the structure and its function is on the one hand to ensure the absorption of charges and on the other hand to ensure the closure of the building (protective function). The principle of modelling this parameter is the same as the diaphragm parameter. For the roof parameter, the dead and live loads are considered to be different (greater) with respect to the weight of common floors.

4.3.14 Details

The detail parameter refers to the state and quality of non-structural elements, which can influence the behaviour of the structure during an earthquake; as well as the state of the various networks that influence the functionality of the structure. These are: balconies, partition walls, handrails, acroteria, staircases, etc. For modelling purposes, the "balconies" have been taken as an example. The deformation of these will be studied. In class A, there are balconies whose deformation remains in the elastic range, in class B, the deformations reach the plastic range, while in class C, the deformation of the balconies reaches the failure limit.

4.4 Determination of Weighting Coefficients

Each parameter is modelled for each type of building (low, medium and high) and for the three classes of vulnerability, this means 14 parameters multiplied by 3 buildings multiplied by 3 classes of vulnerability, which gives 126 models so that the modelling took us about 600 hours of analysis. Weighting factors are calculated from nonlinear static analyses using the maximum displacements at the top of the structures. These weighting factors are obtained according to the following formula:

$$K_i = \frac{d_{\max}}{\sum_{i=1}^{i=3} d_{\max i}} \tag{10}$$

$d_{\max}$: maximum displacement for each vulnerability class (A, B and C)

$\sum_{i=1}^{i=3} d_{\max i}$: sum of the maximum displacements of the three vulnerability classes.

Table 3 -Displacement values for the three models (low, middle and high rise)

	Low			***Middle***			***High***		
	A	B	C	A	B	C	A	B	**C**
1- Ductility	0,060	0,346	0,468	0,090	0,445	0,620	0,142	0,440	0,717
2- Assemblage	0,082	0,316	0,518	0,104	0,417	0,577	0,125	0,353	0,717
3- maintenance	0,099	0,467	0,778	0,080	0,446	0,645	0,111	0,427	0,616
4- Bearing capacity	0,099	0,369	0,530	0,080	0,476	0,529	0,111	0,357	0,471
5- Type of soils	0,057	0,366	0,528	0,108	0,371	0,562	0,135	0,346	0,522
6- Horizontal diaphragm	0,148	0,369	0,498	0,196	0,336	0,512	0,164	0,356	0,417
7- Plan regularity	0,099	0,359	0,477	0,130	0,361	0,463	0,142	0,359	0,475
8- Buckling	0,120	0,380	0,499	0,160	0,390	0,450	0,125	0,357	0,472
9- Modifications	0,133	0,341	0,484	0,159	0,376	0,457	0,129	0,372	0,471
10- Elevation regularity	0,133	0,359	0,477	0,159	0,409	0,431	0,129	0,374	0,459
11- Pounding effect	0,133	0,356	0,553	0,159	0,364	0,432	0,129	0,362	0,459
12- Ground conditions	0,133	0,402	0,763	0,159	0,486	0,720	0,129	0,438	0,754
13- Roof	0,196	0,290	0,390	0,189	0,308	0,389	0,158	0,359	0,424
14- Details	0,196	0,290	0,390	0,190	0,325	0,389	0,158	0,324	0,417

Then for each parameter, an average is calculated of the K_i coefficients obtained for the three types of models (low, middle and high rise), according to the Eq.11 and the results is given in Table 4.

$$K_j = \frac{\sum_{i=1}^{i=n} K_i}{n} \tag{11}$$

K_j : weighting coefficient obtained for each parameter.

n : number of considered buildings (n = 3).

Table 4 - Average of the "K_j" coefficients

	A	B	C
1- Ductility	0,097	0,410	0,602
2- Assemblage	0,104	0,362	0,604
3- maintenance	0,097	0,447	0,680
4- Bearing capacity	0,097	0,401	0,510
5- Type of soils	0,100	0,361	0,537
6- Horizontal diaphragm	0,169	0,353	0,476
7- Plan regularity	0,124	0,360	0,472
8- Buckling	0,135	0,376	0,474
9- Modifications	0,140	0,363	0,471
10- Elevation regularity	0,140	0,381	0,456
11- Pounding effect	0,140	0,361	0,481
12- Ground conditions	0,140	0,442	0,746
13- Roof	0,181	0,319	0,401
14- Details	0,182	0,313	0,398

The coefficients obtained in Table 4 are not used directly in the classification of steel structures. They have been normalized to be between 0 and 1 using equation (12) by dividing each weighting coefficient by the value 7.30, which represents the sum of the factors in Class C (the largest sum).

$$K_n = \frac{K_j}{7.30} \tag{12}$$

The new coefficients are given in Table 5:

Table 5- Weighting factor K_n

	A	B	C
1- Ductility	0,01	0,06	0,08
2- Assemblage	0,01	0,05	0,08
3- maintenance	0,01	0,06	0,09
4- Bearing capacity	0,01	0,05	0,07
5- Type of soils	0,01	0,05	0,07
6- Horizontal diaphragm	0,02	0,05	0,07
7- Plan regularity	0,02	0,05	0,06
8- Buckling	0,02	0,05	0,06
9- Modifications	0,02	0,05	0,06
10- Elevation regularity	0,02	0,05	0,06
11- Pounding effect	0,02	0,05	0,07
12- Ground conditions	0,02	0,06	0,10
13- Roof	0,02	0,04	0,05
14- Details	0,02	0,04	0,05

These weights express a percentage of the seismic quality of structural and non-structural elements, and can only take one vulnerability value.

5 Calculation of the vulnerability index and classification of steel structures

The sum of the numerical values gives us a value called the "vulnerability index" Eq. (13). It characterizes the seismic quality of the building.

$$I_v = \sum_{1}^{14} K_n \tag{13}$$

According to the value of "Iv" calculated in Eq. (13), the studied structure can be classified within one of the three defined given in Table 6.

Table 6 - Vulnerability index classes

Class	**Green**	**Orange**	**Red**
Iv	[0.25 – 0.48[	[0.48 – 0.86[	[0.86 – 1]

- The first class (green colour), classifies the construction as resistant with no requirement to any repairs.
- The second class (orange colour), classifies the construction as moderately resistant requiring reinforcement.
- The third class (red colour), classifies the construction to be a construction with low resistance requiring replacement.

The quantification of the various parameters is done in situ through the filling in of a specifically developed technical data sheet.

6 Semi empirical vulnerability curves

In order to perform seismic scenarios, vulnerability functions could be used. In the present work, vulnerability curves are developed based on a statistical method. These curves express the statistical correlation between seismic intensity and damage observation during past earthquakes.

6.1 Damage Probability Matrices

The relationship between seismic intensity and damage observation is described through damage probability matrices (DPM). These DPMs describe the probability of a certain level of damage occurring after a given earthquake [09,11,20].

The method is based on five degrees of damage (Table 7). The description of each degree of damage is given in the table 07 below [09]:

Table 7 - Damage Categories

CLASS	**GRADE**	**DESCRIPTION**
Green	1	Negligible to light damage.
Green	2	Light for the structuredelements and moderate for the not structuredelements.
Orange	3	Moderated for the structural elements and heavy for the non -structural.
Orange	4	Heavy for the structural and veryheavy for the non -structural.
Red	5	Veryheavy for the structured, collapse total or close.

The quantification of damage corresponding to different MMI intensities for each vulnerability class is shown here after for Algerian steel structures are given in Table 8, 9 and 10.

Table 8 - Green class

			Green		
Damage	**1**	**2**	**3**	**4**	**5**
Intensity					
IV					
V					
VI					
VII	Rare				
VIII	Few	Rare			
IX	Many	Few			
X		Many	Few		
XI			Many	Few	
XII				Many	

Table 9 - Orange class

			Orange		
Damage	**1**	**2**	**3**	**4**	**5**
Intensity					
IV					
V					
VI	Few				
VII	Many	Few			
VIII	Most	Many			
IX		Most	Many		
X			Most	Many	
XI				Most	Many
XII					Most

Table 10 - Red class

			Red		
Damage	1	2	3	4	5
Intensity					
IV					
V	Few				
VI	Many	Few			
VII	Most	Many			
VIII		Most	Many		
IX			Most	Many	
X				Most	Many
XI					Most
XII					

The terms used: rare, Few, Many, Most are defined as follows:

- Rare: The percentage of damaged buildings is between 0 and 5%.
- Few: The percentage of damaged buildings is between 0 and 20%.
- Many: The percentage of damaged buildings is between 20% and 60%.
- Most more than 60% of the buildings were damaged for a given intensity.

6.2 Vulnerability Curves

Damage probability matrices (DPM) can be graphically represented by continuous vulnerability functions graphically represented by vulnerability curves. For each intensity the average damage rate μ_D is calculated to define the degrees of damage observed. The semi-empirical vulnerability curves are expressed in terms of μ_D, which is derived for each vulnerability class of the construction, by considering the vulnerability index I_V, which determines the damage rate with a given intensity.

From the vulnerability index, which is between 0 and 1 and the MMI intensity, we have defined the average damage rate which makes it possible to know the percentage of buildings in a given level of damage.

The empirical vulnerability curves are based on deterministic approaches, expressing the average damage rate as a function of the seismic level which is:

$$\mu_D = 2.55\left[1+\tanh\left(\frac{I+\left(6.25 I_{Vmean}\right)-13.1}{2.8}\right)\right] \tag{14}$$

The Beta distribution was used to calculate the damage distribution (DPM) for each vulnerability class, in order to obtain the vulnerability curves, which are called semi-empirical vulnerability functions and are shown in Fig 4.

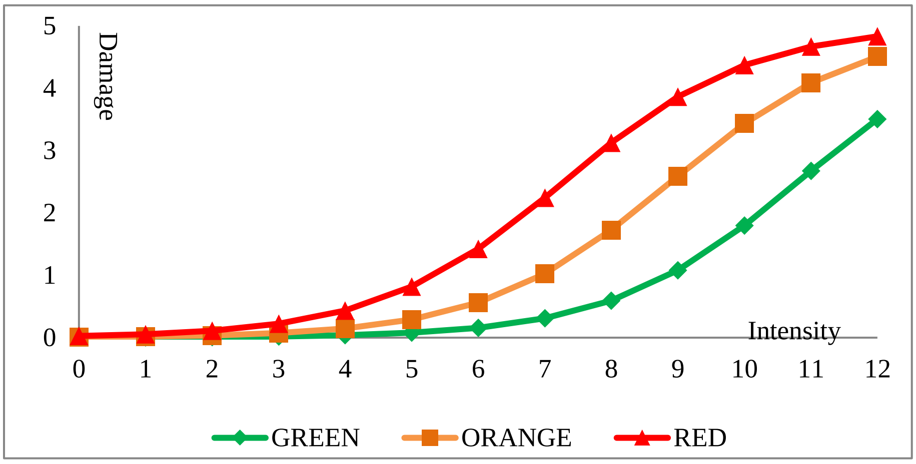


Fig. 4 –Mean semi empirical vulnerability functions

7 Results and analyses

Several constructions have been treated, here after four examples are given below to show the efficiency of the method.

7.1 Studied Examples

7.1.1 Example 1

It is a Zinc production manufacture built in 1949 and located in the west part of Algeria. The following figures show the damage undergone by this structure.

Fig. 5 –Degradation of the bracing system and advanced corrosion

7.1.2 Example 2

It is manufactory inaugurated in 1975. The process of manufacturing zinc liberates H2SO4 which is very harmful to the metal, as it accelerates the corrosion process. This manufactory is built near the sea on a sandy soil and is limited to the south by a high cliff. Photos below were taken on site during our visit.

Fig. 6 –Deformation of the bracing system, lack of bolts in the joints and 45 degree cracks in both directions of the joint

Fig. 7 –Bucking of columns

Fig. 8 –Seismic joint seen from the outside at the top and bottom

7.1.3 Example 3

It is a heavy vehicle manufacturing plant, it was built in 1978. As it can be seen it is an L-shaped building with no rupture joints.

Fig. 9 –Instability phenomenon on the bars of the bracing system

Fig. 10 –Soil subsidence, cracks at the base of walls in pavements and walls

7.1.4 Example 4

The structure dates back to the 1940 and was used as a stable before being transformed in a sports hall after 1962.

Fig. 11 –Advanced corrosion of bearing parts and assemblies

Fig. 12 –cracks in walls and pavements

7.2 Results

In Table 11, results of the studied examples are given.

Table 11 - Results of studied examples

		Example 1		**Example 2**		**Example 3**		**Example 4**	
N	Parameters	Class	Ki	Class	Ki	Class	Ki	Class	Ki
1	Ductility	C	0,08	B	0,06	C	0,08	B	0,06
2	Bearing capacity	C	0,07	B	0,05	A	0,01	B	0,05
3	Assemblage	C	0,08	B	0,05	A	0,01	B	0,05
4	General maintenance conditions	C	0,09	B	0,06	B	0,06	C	0,09
5	Type of soil	B	0,05	C	0,07	B	0,05	B	0,05
6	Horizontal diaphragm	C	0,07	B	0,05	B	0,05	A	0,02
7	Buckling	C	0,06	B	0,05	A	0,02	B	0,05
8	Plan regularity	C	0,06	C	0,06	C	0,06	A	0,02
9	Modifications	A	0,05	A	0,02	C	0,06	C	0,06
10	Elevation regularity	C	0,06	C	0,06	C	0,06	A	0,02
11	Pounding effect	B	0,05	C	0,07	C	0,07	A	0,02
12	Ground conditions	C	0,1	C	0,1	C	0,1	A	0,02
13	Roof	B	0,05	B	0.04	A	0,02	B	0,04
14	Details	C	0.05	C	0,05	A	0,02	A	0,02
	$\sum_{1}^{14} Kn$	**0.87**		**0.75**		**0.67**		**0.57**	

accordance with the proposed classification, the structure in example 1 is classified red while the structures in examples 2, 3 and 4 are classified orange. This is in adequacy with in situ observations and expert reports carried out by national control organisms.

The vulnerability curves for steel structures given above, show that constructions belonging to the green class will only reach damage level 1 at seismic intensity 9. The DPMs developed for these structures show that many constructions classified as green will have a damage level 1 for the same intensity, and that many constructions will reach damage level 4 for an intensity of 12. On the other hand, metal structures belonging to the red class will reach a damage level 1 for an intensity of 6 and will reach a damage level 2 to 3 for an intensity of 8 and most will have a damage level 5 for an intensity of 10.

8 Conclusion

Even if they are not designed to resist earthquakes, steel structures built in accordance with the rules of the art can perform well during an earthquake, provided that certain design (ductility, resilience, resistance) and execution (assembly, maintenance) rules are respected. Nevertheless, the monitoring of these structures must be permanent. This monitoring can only be effective if the elements at risk are correctly identified. Therefore the present study focuses on identifying the parameters that play an important role in seismic behaviour for this type of structure. Thus 14 parameters have been identified including ductility, seismic capacity and buckling. It was shown that the latter play an important role in the seismic behaviour of steel structures, while the detail parameter for instance plays a less important role.

These parameters were quantified using numerical models where each parameter was modelled according to its vulnerability class. The weightage coefficients were determined according to the displacements at the top of the structures according to push over analyses. Based on the value of the vulnerability index, a classification of steel frame structures has been proposed. The results of this classification are in adequacy with the observations made in situ.

Damage probability matrices (DPM) for metal structures have also been developed. These give the percentage of damage as a function of seismic intensity and the vulnerability class of the building. Then, using the continuous form of these DPM and the vulnerability index (IV), vulnerability curves were established.

Seismic scenarios on urban sites can be carried out by considering these vulnerability curves, which can be seismic disaster management tools for the concerned government authorities.

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