Comment on European Codes and Comparison of Brazilian, Italian, Romanian Codes – Concerning the Approach of Robustness

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Summary

During the last decades the structural robustness theme has preoccupied the civil engineering experts. Unpleasant, costly and very often tragic events draw the attention of the entire society. The experts in the construction domain try to define and to assess this phenomenon unitary so that the possible events to have no significant consequences.

The paper aims to join in the present day studies regarding the structural resistance with a new research theme proposed and drawn up by some experts in three countries. Here the way that structural robustness notion is found in the national codes is presented, together with a case study based on these codes.

The authors are members of the Working Group Earthquake Resistant Structures belonging to The International Association for Bridge and Structural Engineering.

Keywords: comparative analysis, robustness, scenario, seismic analysis, seismic standards.

1 Introduction

During the last 10-15 years the civil engineering literature has got more substantial in works focused on structural resistance. The starting event for these preoccupations was the Ronan Point tower block gas explosion, in London, on 1968.

The next events that boosted the research in this field were the terrorist attacks against The Alfred Murrah Federal Building in Oklahoma on 1995 and the Khobar Towers in Saudi Arabia on 1996. The third important moment was the September 11 terrorist attacks of 2001 that destroyed the World Trade Center twin towers in New York.

At this moment there are several groups of experts which are studying intensely and organizing scientific events about this topic. However, there is no mutual approach of the phenomenon and there is no consensus about the nomenclature and philosophy of analysis.

The causes of the failure are multiple: natural phenomena, design and, construction mistakes, improper use, etc. Lately, the terrorism can be added.

Every event is individualized by its social, economic and emotional impact.

The paper aims to join, in the present day, studies regarding the structural resistance with a new research theme proposed and drawn up by some experts in three countries. Here the way the structural robustness notion is found in the national codes is presented, together with a case study based on these codes.

2 About Robustness

2.1 History

Following the Ronan Point event – London, 1968 – the experts in the structural design domain started to

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study the peculiar ways of a building collapse. So, the present day English codes contain rules regarding the structural robustness and ways of preventing a disproportionate failure.

In 2005 the Joint Committee on Structural Safety (JCSS) and the International Association for Bridge and Structural Engineering (IABSE) proposed a meeting where more than 60 professionals participated. They started from the idea that "robustness is still an issue of controversy and poses difficulties with regard to interpretation as well as regulation".

During 2007 – 2010 a practical activity took place focused on: COST Actions TU0601, "Robustness of Structures" to which specialists in 25 European countries took part, among which from Romania and Italy as well. The scientific documents of this forum represent the theoretical and practical basis of the moment for the structural robustness notion, [13, 15].

Table 1 – Domain and correspondent robustness definition.

Bayesian Decision Making	By introducing a wide class of priors and loss functions, the elements of subjectivity and sensitivity to a narrow class of choices, are both reduced, [2].
Control Theory	The degree to which a system is insensitive to effects that are not considered in the design, [18].
Design Optimization	A robust solution in an optimization problem is one that has the best performance under its worst case (max-min rule), [3].
Ecosystems	The ability of a system to maintain function even with changes in internal structure or external environment, [12].
Language	The robustness of language... is a measure of the ability of human speakers to communicate despite incomplete information, ambiguity, and the constant element of surprise, [6].
Product Development and Quality Control	The measure of the capacity of a production process to remain unaffected by small but deliberate variations of internal parameters so as to provide an indication of the reliability during normal use.
Software Engineering	The ability... to react appropriately to abnormal circumstances (i.e., circumstances "outside of specifications"). A system may be correct without being robust, [9].
Specialists in structures	Robustness is a desirable property of structural systems which mitigates their susceptibility to progressive disproportionate collapse. It is a property of the structure alone and independent of the possible causes and probabilities of the initial local failure, [20]. Structural robustness can be viewed as the ability of the system to suffer an amount of damage not disproportionate with respect to the causes of the damage itself, [15]. The notion of robustness is that a structure should not be too sensitive to local damage, whatever the source of damage..., [22]. The robustness of a system is defined as the ratio between the direct risks and the total risks (total risks is equal to the sum of direct and indirect risks), for a specified time frame and considering all relevant exposure events and all relevant damage states for the constituents of the system, [28].
Statistics	A robust statistical technique is insensitive against small deviations in the assumptions, [17].
Structural Standards	Robustness, the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause, [25].

In 2011 it appeared a material drawn up by the firm Arup where the structural robustness concept is presented. This material contains a number of 28 recommendations, divided into five categories – Terminology, Approved Document A, Forms of construction, Structural behavior, and Knowledge transfer.

The last scientific event dedicated exclusively to this subject was held in 2013 in Finland, where the Workshop "Safety, Failures and Robustness of Large Structures" took place, also organized by the International Association for Bridge and Structural Engineering (IABSE).

We especially want to mention the scientific works on robustness drawn up by Professor Uwe Starossek and by his collective.

2.2 Definitions

When you say that a thing or a being is robust you imagine an entity that is complete from the point of view of its component elements ensuring force and safety. The notion of robustness is associated to a lot of domains of activity. Consequently, there are a lot of definitions individualizing the robustness notion, depending on the study domain.

In Table 1 there are summarized some definitions for robustness indicating the domain where they are presented.

2.3 Design methods

In almost all scientific works dealing with the structural robustness notion, specific designing methods are presented. In the same time, there are quite few scientific works giving examples of structures designed from this point of view. For this subchapter of this article we used as reference the works from STAROSSEK [10, 5, 21], and the following scientific materials: COST Action TU0601 [26, 13, 15], CORMIE *et al.* [7], KNOLL [8].

The designing methods can be classified from two perspectives:

A. Either by proposing the prevention of a local initial failure of the key element – where it may be taken into consideration the method of the specific local resistance and non-structural protection measures to ensure a high level for the local failure. These methods aim to increase the structural robustness.

The following methods belong to this category:

A.1 Key element design (Increased Local Resistance):
This method helps to verify if certain zones of the construction have enough collapse resistance after losing some structural elements. For this, they have first to establish the hierarchical order of the importance of the structural elements as compared with the failure phenomenon and to identify the key element / elements. When applying this method, the problem of increasing of the design loads acting on the structure raises.

A.2 Tie-force design:
The method consists in introducing some supplementary ties on the directions considered vulnerable, with the role of meeting the robustness requirements through minimum levels of ductility, continuity and tying.

A.3 Protection:
Through this method there are proposed/designed protection rules or elements aiming to reduce the vulnerability of the structure in case a failure is considered to happen.

B. Or by assuming the possibility of a local initial failure – where it may be taken into consideration the method of the alternative way and the segmentation method. These methods aim to increase the structural robustness and to limit the incipient failure for an acceptable extension. In case of applying these methods the structural resistance increases.

The following methods belong to this category:

B.1 Alternative load path method:
The method consists in determining if the whole structure is able to take over and to transmit the forces and to preserve this quality even after losing some components. The relative amount of the lost components when compared with the entire structure shall be determined or limited. In the structural elements, that distribute the forces, other types of loads / strains may appear – e.g.: inversion of the strained fiber in case of bending or the passing from the stress of the bending moment type to the stress of the axial force type (catenary action).

B.2 Isolation by Segmentation:
In this approach, a spreading of failure following an initial damage is prevented or limited by isolating the failing part of a structure from the remaining structure by so-called segment borders.

The method is introduced and exemplified by Professor Starossek in the work: "Disproportionate collapse: a pragmatic approach", [19].

Table 2 – Design methods where a local initial failure possibility is assumed - comparison.

Method "Alternative load path"	It is applied for the entire structure: the necessary changes / modifications that may appear are for the entire structure.	The structure behaviour is influenced by the extent of the initial failure. It is applied for small initial failures.	The structural continuity increases. It is mainly used for buildings.
Method "Isolation by segmentation"	It is applied in individual points of the structure: the changes / modifications that may appear are for every sub-structure separately.	It is not influenced by the size of the initial failure. It is applied for large initial failures.	The structural continuity decreases. It is mainly used for bridges. At buildings it is used for fire protection

In Table 2 the comparison of the two design methods used for the case where possibility to appear an initial failure is taken into consideration is presented.

3 The structural robustness in codes – Eurocodes, Brazilian, Italian and Romanian Codes

Many experts draw the attention that presently the design codes are based predominantly on the design of structural members or on the consideration of member failure modes. Furthermore design codes and their users may not always include all relevant design situations of relevance for the integrity of the overall structural performance, [15].

In COST papers, [26, 13, 15] it is shown that many codes of practice contain some robustness rules, only they are not formulated in a consistent way on a rational basis. In countries where structures are designed for seismic loads, the requirements to obtain earthquake resistant structures include many aspects belonging to robustness too, like redundancy and ductility.

In this article the European [23, 24, 25], Brazilian [35], Italian [30, 31] and Romanian [32, 33, 34] technical regulations in force will be presented.

3.1 In Eurocodes

There are more provisions aiming to increase the structural robustness and to limit any possible local failures.

In EN 1990:2002-Basis of Structural Design, Section 2 – 2.1 (4) is stipulated: "A structure shall be designed and executed in such a way that it will not be damaged by events like explosion, impact and the consequences of human errors, to an extent disproportionate to the original cause. In the case of fire, the structural resistance shall be adequate for the required period of time."

Practically, in EN 1990:2002-Basis of structural design, Section 2 – 2.1 (5)P it is also presented the way the structural robustness shall be checked: "Potential damage shall be avoided or limited by appropriate choice of one or more of the following: avoiding, eliminating or reducing the hazards to which the structure can be subjected; selecting a structural form which has low sensitivity to the hazards considered; selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localized damage; avoiding as far as possible structural systems that can collapse without warning; tying the structural members together."

The practical designing measures required to attain this aim are mainly provided in EN 1991-1-7:2006 Eurocode 1: Part 1-7 Accidental Actions.

In spite of all these, the word "robustness" is explicitly used in few paragraphs. So:

(i) in EN 1990:2002-Basis of structural design, in 2.2.(5) letter e: other measures relating to the following "other design matters: the basic requirements; the degree of robustness (structural integrity); durability, including the choice of the design working life; the extent and quality of preliminary investigations of soils and possible environmental influences; the accuracy of the mechanical models used; the detailing."

(ii) in EN 1991-1-7:2006-Accidental actions, section 1 paragraph 1.5.14 where the robustness notion is defined (see table 1);

(iii) EN 1991-1-7:2006 section 3 paragraph 1.(2) where the designing strategies for accidental situations are defined – Figure 3 in code. In Figure 1 is presented the scheme in the code.

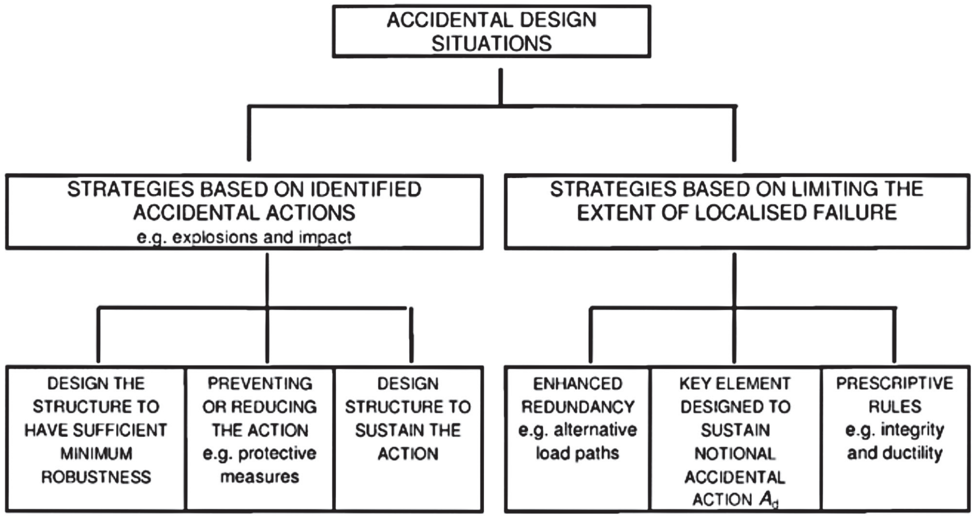


Figure 1 *– The scheme in the EN 1991-1-7:2006, code section 3 [25]*

Here in note 3 is also presented: "Strategies based on unidentified accidental actions cover a wide range of possible events and are related to strategies based on limiting the extent of localized failure. The adoption of strategies for limiting the extent of localized failure may provide adequate robustness against those accidental actions, or any other action resulting from an unspecified cause.

(iv) EN 1991-1-7:2006 Paragraph 3.2 (3) letter (c), "ensuring that the structure has sufficient robustness by adopting one or more of the following approaches: 1) by designing certain components of the structure upon which stability depends as key elements to increase the likelihood of the structure's survival following an accidental event. 2) designing structural members, and selecting materials, to have sufficient ductility capable of absorbing significant strain energy without rupture. 3) incorporating sufficient redundancy in the structure to facilitate the transfer of actions to alternative load paths following an accidental event.

(v) EN 1991-1-7:2006 Paragraph 3.3 (2) letter b, NOTE 2 The National Annex may state the acceptable limit of "localized failure". The indicative limit for building structures is 100 m2 or 15 % of the floor area, whichever is less, on two adjacent floors caused by the removal of any supporting column, pier or wall. This is likely to provide the structure with sufficient robustness regardless of whether an identified accidental action has been taken into account.

(vi) EN 1991-1-7:2006 Paragraph 3.3 (2) letter (c) – applying prescriptive design/detailing rules that provide acceptable robustness for the structure (e.g. three-dimensional tying for additional integrity, or a minimum level of ductility of structural members subject to impact).

(vii) EN 1991-1-7:2006 Paragraph 3.4 (2) for class CC 1 (Low consequences of failure) no specific consideration is necessary for accidental actions except to ensure that the robustness and stability rules given in EN 1990 to EN1999, as applicable, are met"

(viii) EN 1991-1-7:2006 Annex A2. (1) "Designing a building such that neither the whole building nor a significant part of it will collapse if localized failure were sustained, is an acceptable strategy, in accordance with Section 3 of this part. Adopting this strategy should provide a building with sufficient robustness to survive a reasonable range of undefined accidental actions."

(ix) EN 1991-1-7:2006 Annex A4. (1) "Adoption of the following recommended strategies should provide a building with an acceptable level of robustness to sustain localized failure without a disproportionate level of collapse."

(x) EN 1991-1-7:2006 Annex 5.2. (1) For Class 2 buildings (Lower Risk Group), see Table A.1:

Appropriate robustness should be provided by adopting a cellular form of construction designed to facilitate interaction of all components including an appropriate means of anchoring the floor to the walls.

(xi) EN 1991-1-7:2006 Annex B.6.(1) letter (d) overcome the hazard by providing, for example, increased reserves of strength or robustness, availability of alternative load paths through structural redundancy, or resistance to degradation, etc.

(xii) EN 1991-1-7:2006 Annex B.9.4 – Guidance for application of risk analysis related to impact from rail traffic:

(1) the static system (structural configuration) of the structure and the robustness of the supports.

(2) provision of robustness to the supports of the structure to withstand the glancing impact from a derailed train to reduce the likelihood of collapse of the structure.

(xiii) EN 1991-4:2006-Silos and tanks, in Paragraph 2.5.(5) Note 2 – The above differentiation has been made in relation to the uncertainty in determining actions with appropriate precision. Rules for small silos are simple and conservative because they have an inherent robustness and the high cost of materials testing of stored solids is not justifiable. The consequences of structural failure and the risk to life and property are covered by the Action Assessment Classification of EN 1992 and EN 1993.

(xiv) In EN 1993-1-1:2005-Design of steel structures – Part 1-1: General rules and rules for buildings, in paragraph 2.1.3 titled Design working life, durability and robustness are also reminded from Code EN 1990:2002 and EN 1991-1-1-7.

(xv) In EN 1993-1-11:2006-Design of steel structures – Part 1-11: Design of structures with tension components, Annex A paragraph 3.(3) it is shown that: The above requirements should be met by:

– appropriate choice of materials for wires, strands, steels and protective coatings;
– choice of the form of construction in respect of strength, stiffness, ductility, durability and robustness for manufacturing, transport, handling and installation;
– quality control of accurate fitting of the end termination to ensure the correct alignment of tension component in service.

(xvi) In EN 1993-2:2006-Steel bridges, one can find the same paragraph as in EN 1993-1-1:2005, namely: 2.1.3 Design working life, durability and robustness. Then in paragraph 2.1.3.4: "Robustness and structural integrity" it is shown that: The National Annex may define components that are subject to accidental design situations and also details for assessments. Examples of such components are hangers, cables, bearings.

(xvii) The same in EN 1993-6:2006-Crane supporting structures, one can find the paragraph 2.1.3 Design working life, durability and robustness

(xviii) In EN 1996-1-1:2005-Design of masonry structures, section 5, paragraph 5.1. (2)P it is shown: The general arrangement of the structure and the interaction and connection of its various parts shall be such as to give appropriate stability and robustness during construction and use.

(xix) In paragraph 5.1.(3) in the note "Where the structure is made of separately designed components the overall stability and robustness should be ensured."

(xx) In EN 1998-1:2004-Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings, Paragraph 9.2.1.(1) - Masonry units should have sufficient robustness in order to avoid local brittle failure.

(xxi) In EN 1998-2:2005-Design of structures for earthquake resistance – Part 2: Bridges, annex A paragraph A.2.(3) – The robustness of all partial bridge structures should be ensured during the construction phases independently of the design seismic actions.

(xxii) In EN 1999-1-1:2007 – Design of aluminium structures-Part 1-1: General structural rules, one can find the paragraph 2.1.3 Design working life, durability and robustness.

In Eurocodes the buildings are subdivided into three classes – with four levels – for which there are stipulated minimal requirements regarding the robustness, as follows:

C 1– Low level – Buildings ≤ 3 stories. It is not necessary to be taken into consideration for accidents.

C2a – Intermediate level – Buildings from 3 to 6 floors and Offices with less than 4 floors: Only the robustness and stability recommendations of the Eurocode 1-9 are to be considered.

C2b – High level – Buildings with 7 to 10 stories or less, public buildings with less than 200 sqm: Simplified calculation using the equivalent static loads or design and construction regulations that can be applied.

C3 – Severe level – Buildings with more than 10 stories or with more than 200 sqm: dynamic, nonlinear or the load-structure interaction analysis can be applied.

3.2 In Brazil

There is nothing at all in Brazilian codes [35] regarding robustness, excepting for one provision in flat slabs, near the columns, in which an amount of positive reinforcement is required, corresponding to the horizontal pin resistance necessary if a diagonal tension rupture would occur in the concrete near the support.

3.3 In Italy

In the Italian Code [30] there are some provisions where the word robustness is used directly:

(i) in paragraph 2.1 "Safety and expected performance – Fundamental principles":

Particularly, according to what is stated into the specific chapters, structures and the structural typology itself should have the following requisites:

– robustness against accidental actions: ability to avoid damages which are not proportionate with the causing events such as fire, explosions, impacts.

(ii) in paragraph 3.1.1 "Constructions Actions – Civil and Industrial structures":

During the design, the structure robustness shall be verified applying conventional nominal actions, together with the other actions (not coming from seismic nor wind), applied along the two orthogonal horizontal directions and consisting into the 1% part of the loads, in order to verify the general behaviour.

(iii) in paragraph 4.2.2 "Steel Structures – Safety evaluation":

The safety evaluation is made following the fundamental principia.

The requirements of resistance, functionality, durability, and robustness are there if ultimate and service limit states are verified for the structure and for the connections which are described into this code.

(iv) in paragraph 4.2.2.1 "Steel Structures – Limit States":

Vibration Limit State, with the aim of keeping a minimum comfort level for the people using the structure, is required in order to not have low robustness and/or possible damages into secondary elements.

(v) in paragraph 4.2.6 "Steel Structures – Accidental Actions Verification":

For accidental situation, the project shall demonstrate construction robustness by using damage scenarios procedures for which the material reduction factor can be taken equal to one.

(vi) in paragraph 4.3 "Steel-Concrete composite structures"

The following regulations apply to steel-concrete composite civil and industrial constructions regarding the needing of resistance, functionality, durability, robustness and construction.

(vii) in paragraph 4.3.1 "Steel-Concrete composite structures – Security assessment"

Resistance, functionality, durability and robustness requirements are satisfied if the ultimate limit states and the service limit states of the structure, of the structural components and of the connections are verified accordingly to the present code.

(viii) in paragraph 4.3.8 "Steel-Concrete composite structures – Accidental loads verification"

For accidental events, the design shall demonstrate the robustness of the construction by using damage scenarios where the safety coefficients for the materials γ_M can be assumed equal to one.

(ix) in paragraph 4.4.1 "Wooden structures – Security assessment"

Resistance, functionality, durability and robustness requirements are satisfied if the ultimate limit states and the service limit states of the structure, of the structural components and of the connections are verified according to the present code.

(x) in paragraph 4.4.12 "Wooden structures – Robustness"

Structural robustness requirements can be achieved also by having specific design choices and construction attentions that, for wooden elements, shall regard at least:

– protection from humidity of the structure and of the structural components;
– use of intrinsically ductile connections or ductile connection systems;
– use of composite elements with ductile global behaviour;
– limitation of tension perpendicular to the fibres, in particular when shear stress is present and, in general, when high humidity gradient is forecasted in life time.

(xi) in paragraph 4.5.9 "Masonry structures – Accidental events verification"

For accidental design situations, the design shall verify the construction robustness by adopting damage scenarios where the material safety factors γ_M are half of the ones for normal situation.

In the Italian Code Commentary [31] the word robustness is used directly:

(xii) in paragraph C2 "Safety and expected performance"

The code requires safety and performance of a structure or part of a structure shall be evaluated according to the limit states that may occur in lifetime. The code requires also the robustness against accidental actions.

(xiii) in paragraph C3.6 "Constructions Actions – Accidental actions"

Accidental actions, which only sometimes must be considered into the design, have to be known well in order to evaluate the correct robustness.

(xiv) in paragraph C3.6.1.4 "Constructions Actions – Design criteria"

Structure design in fire situation shall demonstrate a sufficient robustness against the fire in order to get damage proportional to its cause, and to obtain required performance levels.

(xv) in paragraph C4.4.12 "Wooden structures – Robustness"

Any provision with the aim of reducing the sensibility of the structure against accidental actions or events that are unexpected by the current code (earthquake, fire, weather events, etc.) has to be adopted.

In defining the design choices, it has to be defined:

– protection from humidity of the structure and of the structural components;
– use of intrinsically ductile connections or ductile connection systems;
– use of composite elements with ductile global behaviour;
– limitation of tension perpendicular to the fibres, in particular when shear stress is present and, in general, when high humidity gradient are forecasted in life time;
– use of structural systems not prone to partial collapse;
– correct disposition of brace systems;
– use of connection elements not sensible to the fire;
– use in parallel of more than one functional element or of connections with a high number of simple connection elements with non-fragile behaviour.

(xvi) in paragraph C8A.5.3 “Criteria for consolidation operations of masonry buildings – Reduction of the high deformability of the floors”.

During seismic actions, in masonry building, floors transfer horizontal actions to the walls that are parallel to the seismic action; furthermore, they have to constrain the walls that are excited by actions perpendicular to their plane. The need of a particular stiffener to distribute the seismic action among the vertical elements is not so frequent. For these reasons it is sometime necessary a floor stiffener, localised, whose behaviour has to be evaluated; consequently, the elements resistance is increased, which benefits the structure robustness.

3.4 In Romania

There are in force both the Eurocodes and the National Annexes. In the cases some other technical regulations exist, they are in line with the Eurocodes. Thus, for the seismic action design they have drawn up the *Seismic Design Code, Part 1 – P100-1/2006, Earthquake Resistant Design of Buildings* [32].

In the paragraphs 4.4.1.1 and 4.4.1.2 concepts like the following can be found:

(1) The structural simplicity implies the existence of continuous and strong enough structural system able to ensure a clear path, as direct as possible, non-interrupted for the seismic forces, irrespective their direction, up to the foundations ground. The seismic forces that appear in all the elements of a building, as mass forces, are taken over by the floors – horizontal diaphragms – and sent to the vertical structures, that transfer them to the foundations and then to the ground. The design shall ensure that no discontinuities will appear along this path. For instance, a large hole in the floor or the absence, in the floor, of the reinforcing bars collecting the inertia forces to be transmitted to the vertical structure – also represent discontinuities.

(2) The seismic design aims to provide the structure of the building with the appropriate redundancy. This will ensure that:

* the failing of one single element or of one single structural tie does not expose the structure to the loss of its stability;
* a plasticized mechanism is obtained with sufficient plastic zones, able to allow the exploitation of the resistance reserves of the structure and an advantageous dissipation of the seismic energy.

These measures aim to increase the structural robustness.

In the CR – 6 – Design Code for Masonry Structures [34] – there are some provisions where the word robustness is used directly:

(i) in paragraph C3.1.2.2. letter C(4) they specify: “The provisions in the *National Annex* have in view to ensure the robustness of the masonry elements as it is required by standard SR EN 1998-1 [33], robustness that is first determined by the characteristics of the inner geometry of the elements.” ... and by “The minimum thickness of the walls as it is established by ASTM (a code in USA) is much higher than that given in the Romanian Code compatible with EN 1996-1-1 which ensures a higher level of robustness for the elements and the avoidance of the fragile ruptures by forcing out the exterior faces.”

(ii) In paragraph C5.3.1 (3) they specify: “The rigidity in the horizontal plan of the floors made up of tile floor elements reinforce-over concrete depends on the elements robustness and on the details of the floor configuration (including the details of their fastening to the vertical structure).”

(iii) In paragraph C7.3.1 (4) – The masonry of the ventilated facades – they specify that “The requirements for the economic efficiency (reducing the cost and/or the execution period), mainly are materialized by introducing the masonry elements lacking the resistance and robustness and by renouncing to fill up with mortar the completely the vertical joints.”

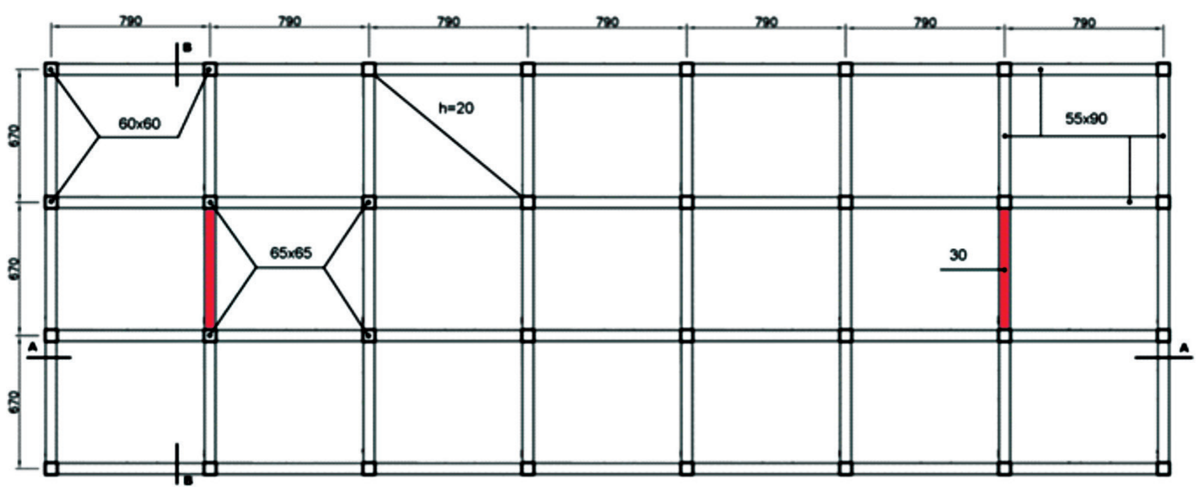
4 Case study

4.1 Structure description

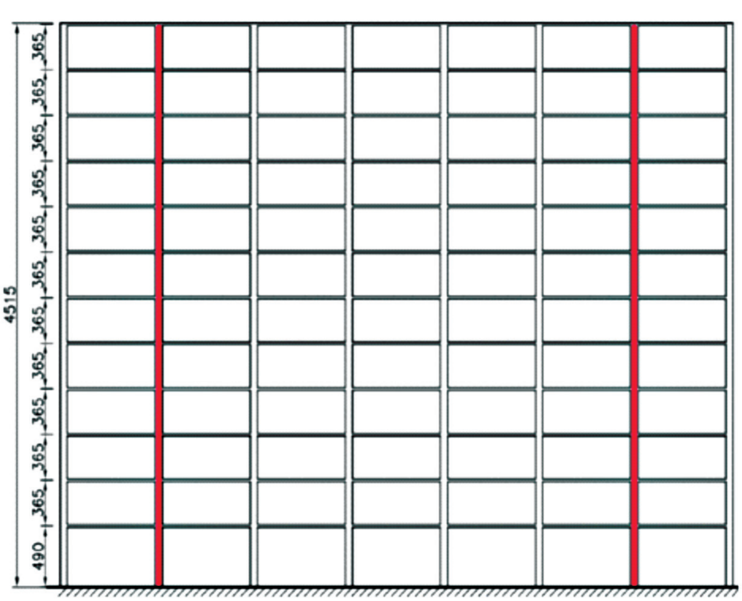
The analysed structure is the same as that in the article: “Comparative Study of Codes for the Seismic Design of Structures” [4]. Choosing the response elastic spectra, as well as the comments of them in the context of codes comparing are to be found in the same article. The calculations are made for the elastic domain. A simple and symmetrical building structure (the "Model Building") has been chosen as an example for illustrating the comparison among the seismic standards. This model is an adaptation of the one already analysed by [16]. Formwork drawings and a longitudinal section of the Model Building are presented in Figure 2.

The main data of the building are:

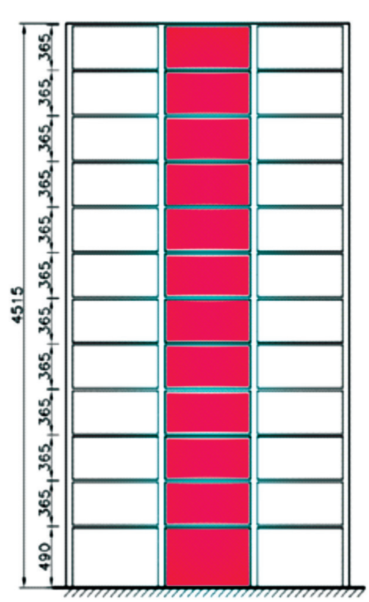
- Nominal concrete strength: f_{ck} = 28 MPa.
- Young modulus of concrete: E_c = 32 GPa.
- Concrete specific weight: γ_c = 25 kN/m³.
- Non-structural finishing weight, typical floors: 1.5 kN/m².
- Non-structural finishing weight, top floor: 0.5 kN/m² (distributed) plus four concentrated loads of 900 kN.
- Plan dimensions: 20.1 m x 55.3 m (between axes of columns).
- Total building height: 45.05 m, in 12 floors, (4.9 m + 11x3.65 m)
- Dimensions of the exterior columns: 60 cm x 60 cm
- Dimensions of the interior columns: 65 cm x 65 cm
- Dimensions of the beams: 55 cm x 90 cm
- Thickness of the slabs: 20 cm
- Thickness of the two shear-walls: 30 cm
- Total weight of the building: 154143 kN



a. Typical floor plan



b. Longitudinal section (A-A)



c. Transversal section (B-B)

Figure 2 *– Model Building.*

4.2 Modelling the structure and the loads

The design model is made up of one-dimensional elements for columns and beams and of two-dimensional elements for floors.

Schematic perspectives of the building, drawn by program SAP2000 [36], are shown in Figure 3.

Due to the variety of the recommendations in the technical regulations about loads, we took the following decision: (i) the load to be only the dead-weight of the structure; (ii) mechanical values of the structural responses used for the comparisons to be taken only from the seismic action; (iii) the seismic action to be modelled under the form of the response elastic spectrum; (iv) the combination relations and the multiplying coefficients shall not be used.

In conclusion, the only variable is the seismic load – the elastic response spectra are presented in Figure 4.

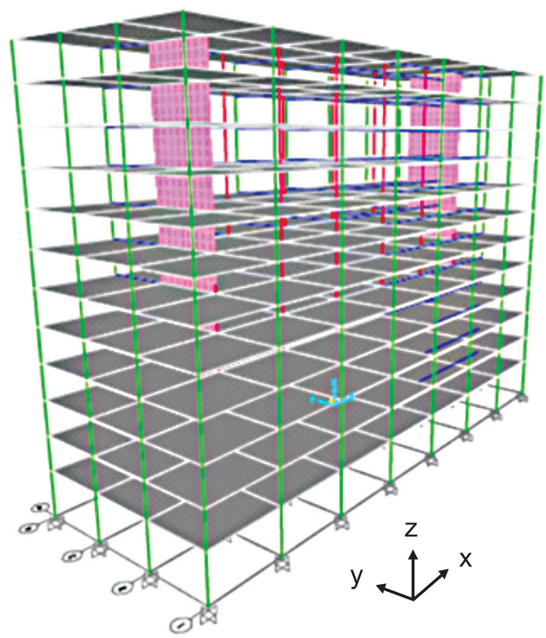


Figure 3 *– Model Building – Perspective generated by SAP2000.*

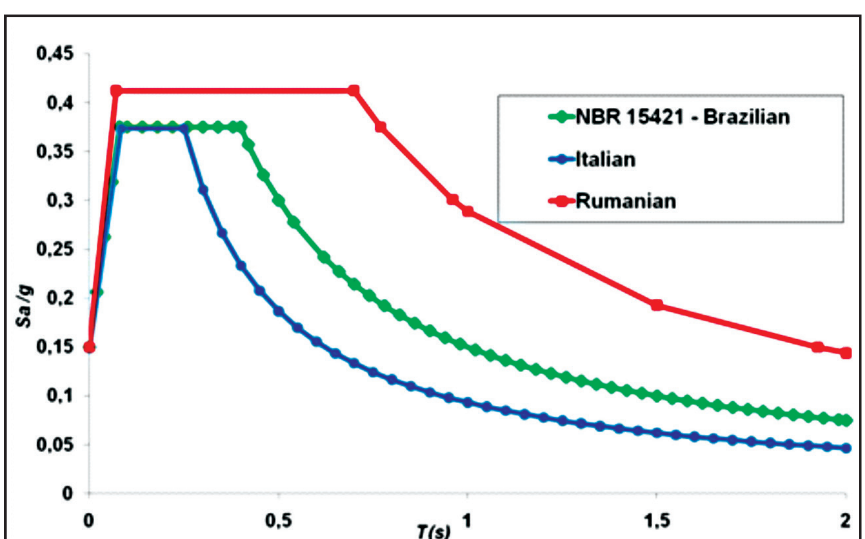


Figure 4 *– Elastic response spectra according to the three used standards (Brazilian and Italian: 475-year return period; Romanian: 100 years return period).*

4.3 Choosing the scenarios to assume the initial damages

According to the domain literature and to the previous own research [27, 1], the authors propose to study a series of 9 failure scenarios, Figure 5 for the initial damages, namely: (**S1**) a column as close as possible to the middle of the long side, at the ground floor; (**S2**) a column as close as possible to the middle of the short side, at the ground floor; (**S3**) a column in the corner, at the ground floor; (**S4**) three columns in the same corner, at the ground floor; (**S5**) a column on two floors, at the ground floor and on the 1st floor + the beams and the afferent plates, at the middle of the long side; (**S6**) a column on two floors + the beams and the afferent plates, at the middle of the short side at the ground floor and on the 1st floor; (**S7**) a column at the ground floor, as close as possible to the middle of the structure; (**S8**) a column on two floors (at the ground floor and on the 1st floor) + the beams and the afferent plates, as close as possible to the middle of the structure; (**S9**) the wall + the 2 adjacent columns on the ground floor. To this the scenario of the undamaged structure shall be added (**S0**).

The authors consider that a first evaluation of the robustness can be made, from the qualitative point of view, from the study for the own dynamic characteristics of the damaged structure compared with the undamaged structure. Thus, one can determine, from this simple study, if the elimination of a certain structural element may bring about local or global effects on the structure.

4.4 Comments and conclusions

Displacements under the gravity load – Table 3

From the study of the displacements values resulted from their dead load we have chosen to make evident the vertical displacements in two points of the

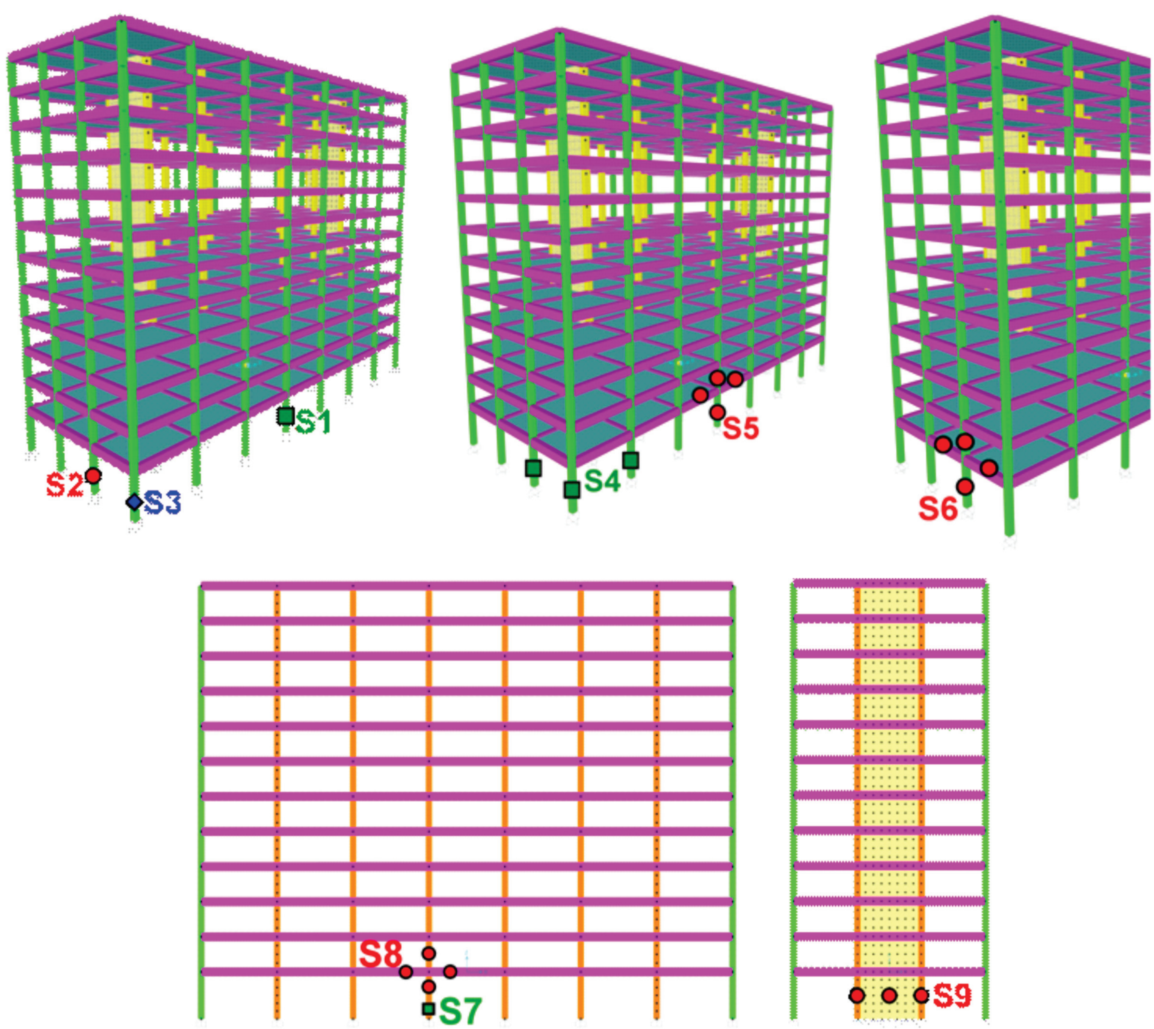


Figure 5 *– Scenarios to test the robustness.*

structures. They are: (1) one point – with fixed position – situated at the maximum level of the structure, placed as close as possible to the centre of the horizontal plan and in the frame where the structural elements are eliminated – marked with number 369 in the design model, Figure 6; (2) a point – with variable position in the structure – placed above the structural element eliminated in every scenario.

For the vertical displacement read at joint 369, scenario **S8** (elimination of 2 columns on two floors – ground floor and 1st floor – exactly on the vertical, joint 369) brings the highest increase, namely 34%. In case of the reading of the displacement in the joints above the eliminated elements, the most disturbing scenario is **S4** (the elimination of 3 corner columns), the increase being of 2440%. In point of importance the next scenario is **S9**, (elimination of the adjacent wall and columns) where the increase is 1533%.

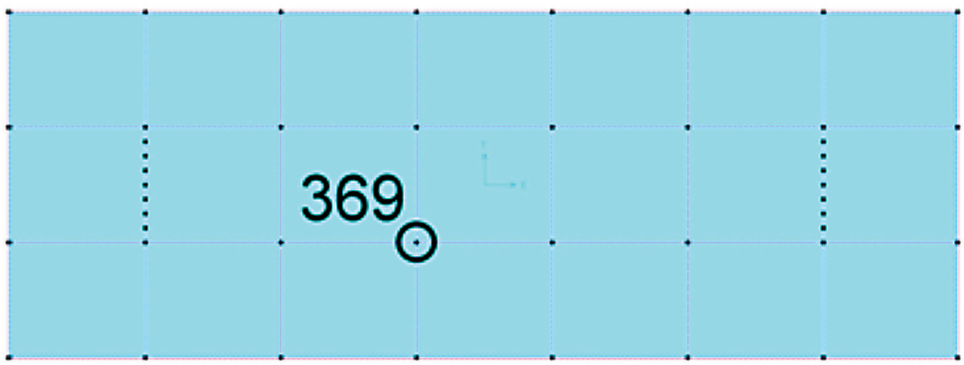


***Figure 6** – Joint placed as close as possible to the centre of the structure – maximum level, toward the area where the structural elements are eliminated.*

Table 3 – Displacements on the vertical direction resulted from the gravity load – in the joint 369 (Figure 6) and in the joint placed above the eliminated structural element.

Scenario	in joint 369 (m)	% Com-pared to S0	in joint above the eliminated element (m)		
			before elimination	after elimination	% after / before
S0 – undamaged structure	(-) $9.3x10^{-3}$	----	-----	-----	-----
S1 - 1 column in the middle of long side	(-) $10.6x10^{-3}$	114	(-) $1.5x10^{-3}$	(-) $10.1x10^{-3}$	673
S2 – 1 column in the middle of short side	(-) $8.3x10^{-3}$	89	(-) $1.2x10^{-3}$	(-) $7.1x10^{-3}$	592
S3 – 1 corner column	(-) $7.6x10^{-3}$	82	(-) $1.0x10^{-3}$	(-) $9.6x10^{-3}$	900
S4 – 3 column in the same corner	(-) $8.5x10^{-3}$	91	(-) $1.0x10^{-3}$	(-) $25.4x10^{-3}$	2540
S5 – 1 column on 2 floors in the middle of long side	(-) $10.6x10^{-3}$	114	(-) $2.5x10^{-3}$	(-) $10.7x10^{-3}$	428
S6 – 1 column on 2 floors in the middle of short side	(-) $9.3\ x10^{-3}$	100	(-) $2.0x10^{-3}$	(-) $7.6x10^{-3}$	380
S7 – 1 central column	(-) $12.3x10^{-3}$	132	(-) $9.0x10^{-3}$	(-) $10.0x10^{-3}$	111
S8 – 1 column on 2 floors to the centre of the structure	(-) $12.5x10^{-3}$	134	(-) $3.1x10^{-3}$	(-) $10.8x10^{-3}$	348
S9 – wall + the 2 adjacent columns	(-) $9.6\ x10^{-3}$	103	(-) $0.9x10^{-3}$	(-) $14.7x10^{-3}$	1633

Another observation refers to the difference between the vertical displacement of the central joint 369 and that in the adjoining joints (positioned both in the longitudinal plan and in the transversal one) and the same difference calculated for the displacements of the joints above the eliminated element and that of the adjoining joints. Figure 7 is an example of scenario **S3** (elimination of a corner column). As it can be seen, the relative displacements are significant in the area where the structural element is eliminated. So, the major effects remain local

Table 4 – Eigenfrequencies, eigenvectors.

Scenario	eigenfrequencies (s) // eigenvectors		
	Fundamental mode	Mode 2	Mode 3
S0 – undamaged structure	1.514 – 1/4 wave longitudinal	1.078 – 1/4 wave transversal	0.938 torsion
S1 - 1 column at the middle of long side	1.519 – 1/4 wave longitudinal	1.084 – 1/4 wave transversal	0.938 torsion
S2 – 1 column at the middle of short side	1.525 – 1/4 wave longitudinal	1.079 – 1/4 wave transversal	0.938 torsion
S3 – 1 corner column	1.527 – 1/4 wave longitudinal	1.090 – 1/4 wave transversal	0.946 torsion
S4 – 3 columns in the same corner	1.563 – 1/4 wave longitudinal	1.146 – 1/4 wave transversal	0.967 torsion
S5 – 1 column on 2 floors at the middle of long side	1.533 – 1/4 wave longitudinal	1.094 – 1/4 wave transversal	0.942 torsion
S6 – 1 column on 2 floors at the middle of short side	1.529 – 1/4 wave longitudinal	1.080 – 1/4 wave transversal	0.940 torsion
S7 – 1 central column	1.520 – 1/4 wave longitudinal	1.078 – 1/4 wave transversal	0.938 torsion
S8 – 1 column on 2 floor in the center	1.528 – 1/4 wave longitudinal	1.079 – 1/4 wave transversal	0.938 torsion
S9 – wall + the 2 adjacent columns	1.531 – 1/4 wave longitudinal	1.369 – rotation round the undamaged wall	0.991 torsion

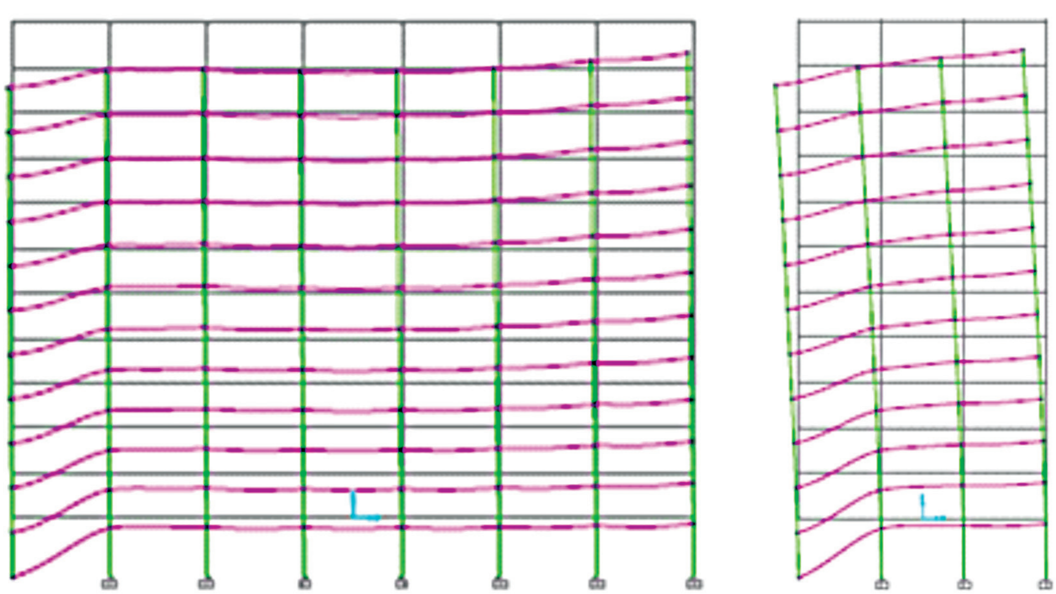


***Figure** 7 – Examples of relative displacements at adjoining joints – scenario **S3** – longitudinal and cross section.*

Comments on the proper dynamic response – Table 4

Period of the fundamental mode varies from 1.514s for the undamaged structure (**S0**) and 1.563s for the scenario **S4** (elimination of 3 columns in the same corner), the increase being 3%. From the point of view of the shape of the eigenvector itself it is for all proposed scenarios, a bending in the longitudinal plan with quarter wave. In conclusion, neither their eigenvectors nor the values of their eigenfrequencies in the fundamental mode are significantly influenced by the structural eliminations proposed in the study.

For the *transversal vibration mode* the period of undamaged structure (S0) is 1,078s and its eigenvector is of the bending type with a quarter wave. From the point of view of the values of the eigenfrequencies in the mode 2, the majority of the scenarios do not have significant variations. The eigenvector also keeps, for the majority of the scenarios, the transversal bending shape. An exception is the scenario S9 (elimination of a wall and the adjoining columns on the ground-floor), which brings about an increase of 27% of the proper transversal vibration period (1,368s) and the shape of the eigenvector changes, being a rotation round the wall that was not damaged.

The eigenvector mode 3 is torsion for all scenarios as compared with the vertical symmetric axe. The value of the period for the undamaged structure is 0,938s. Also, the scenario **S9** gives a maximum increase of the value of the eigenperiod with 6%.

Comments regarding the dynamic response at the spectral action – Tables 5, 6, 7 and 8

Having in view the first two eigenvectors representing bending of the two perpendicular planes – longitudinally and transversally – the spectral response is studied separately by successively loading the structure, with the same spectral values, along the two perpendicular directions. Three spectra were considered, according to three compared codes – Brazilian code, Italian code and Romanian code.

The following comments refers to the spectral displacements, Table 5, of the same *joint placed at the maximum level and near the vertical axis of the structure – joint 369*, Figure 6. The longitudinal displacements of the joint under the spectral action

Table 5 – Displacements of joint 369 (Figure 6) from the longitudinal (L) and transversal (T) spectral actions, read on the longitudinal and on the transversal directions respectively.

Scenario	Displacements (m)		
	Brazilian Spectrum	Italian Spectrum	Romanian Spectrum
S0 – undamaged structure	L 73.5 x 10^{-3} T 57.8 x 10^{-3}	L 45.6 x 10^{-3} T 35.8 x 10^{-3}	L 141.8 x 10^{-3} T 112.5 x 10^{-3}
S1 – 1 column in the middle of long side	L 73.7 x 10^{-3} T 58.3 x 10^{-3}	L 45.7 x 10^{-3} T 36.1 x 10^{-3}	L 142.2 x 10^{-3} T 114.7 x 10^{-3}
S2 – 1 column in the middle of short side	L 74.1 x 10^{-3} T 57.4 x 10^{-3}	L 46.0 x 10^{-3} T 36.1 x 10^{-3}	L 143.0 x 10^{-3} T 113.3 x 10^{-3}
S3 – 1 corner column	L 73.4 x 10^{-3} T 62.8 x 10^{-3}	L 45.5 x 10^{-3} T 39.2 x 10^{-3}	L 141.8 x 10^{-3} T 123.7 x 10^{-3}
S4 – 3 columns in the same corner	L 72.8 x 10^{-3} T 79.0 x 10^{-3}	L 45.1 x 10^{-3} T 49.0 x 10^{-3}	L 141.2 x 10^{-3} T 157.4 x 10^{-3}
S5 – 1 column on 2 floors in the middle of long side	L 74.2 x 10^{-3} T 57.8 x 10^{-3}	L 46.0 x 10^{-3} T 35.9 x 10^{-3}	L 143.4 x 10^{-3} T 113.1 x 10^{-3}
S6 – 1 column on 2 floors in the middle of short side	L 74.1 x 10^{-3} T 57.5 x 10^{-3}	L 46.0 x 10^{-3} T 35.9 x 10^{-3}	L 143.2 x 10^{-3} T 112.9 x 10^{-3}
S7 – 1 central column	L 73.8 x 10^{-3} T 57.3 x 10^{-3}	L 45.7 x 10^{-3} T 35.8 x 10^{-3}	L 142.4 x 10^{-3} T 112.5 x 10^{-3}
S8 – 1 column on 2 floors to the centre of the structure	L 73.9 x 10^{-3} T 57.3 x 10^{-3}	L 45.8 x 10^{-3} T 35.8 x 10^{-3}	L 142.7 x 10^{-3} T 112.6 x 10^{-3}
S9 – wall + the 2 adjacent poles	L 74.1 x 10^{-3} T 58.6 x 10^{-3}	L 46.0 x 10^{-3} T 36.5 x 10^{-3}	L 143.3 x 10^{-3} T 116.2 x 10^{-3}

acting on the longitudinal direction are higher than the transversal displacements for the spectral action on the transversal direction, with only one single exception, namely the scenario **S4** (elimination of 3 columns in the same corner). This notice is valid for all three considered spectra.

The values of the longitudinal displacements do not differ significantly for all scenarios considered compared with undamaged structure. The highest difference is of about 1% for the scenario **S5** (elimination of a column on 2 floors in the middle of long side) and for the three types of spectra. For the spectral action considered on the transversal direction and for the scenario **S4**, the increase, as compared to **S0**, is about 36.7% (Brazilian spectrum) and 39.9% (Romanian spectrum).

The spectral action in the Italian code gives the lowest displacements. The highest displacements are obtained from the loads in the Romanian spectrum. **S0**, the values of the response from the action of the Italian spectrum represent about 62% from the Brazilian spectral response and 32% from the Romanian one. The Brazilian spectral response represents about 52% from the Romanian one.

In conclusion, the global spectral response is not significantly influenced by elimination of the structural elements proposed in the studied scenarios.

Comments regarding the spectral joints displacements above the remote structural element, Tables 6, 7 and 8

In Table 6 there are presented the displacements obtained from the seismic action at the level of the first and second floor as well as level relative displacements for the case of the undamaged structure (**S0**). These floors were chosen because the eliminated structural elements are placed on these two levels – ground floor and level 1. So, for the case where the floor above the ground floor is checked, comparisons can be made with the scenarios **S1**, **S2**, **S3**, **S4**, **S7** and **S9**; for the case of the floor above the 1st floor is checked, comparisons can be made with the scenarios **S5**, **S6** and **S8**.

In ***the Brazilian code*** [35], to design complying with the seismic action there are limitations only for the service ultimate limit state, namely:

Between two consecutive floors:

– Category I - usual buildings: 0,020
– Category II - important buildings: 0,015
– Category III - essential buildings: 0,010

For the service limit state the limitations are only those for the design for wind action.

Italian codes [30, 31] requirements for concrete frame buildings while doing a seismic design:

Table 6 – Study case: undamaged structure **S0** Displacements of the joints at the level of the first and second floors resulted from the longitudinal (L) and transversal (T) seismic actions, appropriately read on longitudinal and transversal directions respectively plus the inter-story drift ratio.

Scenario	Displacements in the joints above the structural elements considered to be eliminated – scenario **S0** (m)					
	Brazilian spectrum		Italian spectrum		Romanian spectrum	
	displacement	dr/h	displacement	dr/h	displacement	dr/h
Displacement of the first floor (above ground-floor) – [for comparisons with scenarios: **S1**, **S2**, **S3**, **S4**, **S7** and **S9**]	L 13.8×10^{-3} T 3.3×10^{-3}	L 2.8×10^{-3} T 0.7×10^{-3}	L 8.6×10^{-3} T 2.4×10^{-3}	L 1.7×10^{-3} T 0.5×10^{-3}	L 25.9×10^{-3} T 6.6×10^{-3}	L 5.3×10^{-3} T 1.4×10^{-3}
Displacement of the 2nd floor (above 1st floor) – [for comparisons with scenarios: **S5**, **S6** and **S8**]	L 22.4×10^{-3} T 7.4×10^{-3}	L 2.4×10^{-3} T 1.1×10^{-3}	L 14.0×10^{-3} T 4.8×10^{-3}	L 1.1×10^{-3} T 0.7×10^{-3}	L $42.4 \times 10\text{-}^{-3}$ T 13.8×10^{-3}	L 4.5×10^{-3} T 2.0×10^{-3}
dr/h (accepted) (h1 = 4.9m; h2 = 3.65m)	**SLS: NA** **ULS: dr/h ≤ 20.0 × 10^{-3}**		**SLS: dr/h ≤ 5.0 × 10^{-3}** **ULS: NA**		**SLS: dr/h ≤ 5.0 × 10^{-3}** **ULS: dr/h ≤ 25.0 × 10^{-3}**	

Where dr = dr(i+1) – dr(i); i = floor position; h = floor height

For use class I (low occupancy) and II (normal occupancy that is the case for this Model Building) DLS (Damage Limit State) Spectrum is used with the following inter-story drift ratio limitations:

- dr ≤ 0.005 h generally (where h is the level height)
- dr ≤ 0.01 h when non structural elements do not influence the structure

For use class III (high occupancy) and IV (strategic structure) OLS (Occupancy Limit State) Spectrum is used with the following inter-story drift ratio limitations:

- dr ≤ 0.0033 h generally
- dr ≤0 .0067 h when non structural elements do not influence the structure

If the service life of the structure is 50 years and the Use Class is II (as it is for normal structures like the Model Building treated into this paper), DLS Spectrum has a return period T_R of 50 years. (For completeness, even if it is not the case of the analyzed structure, if the service life of the structure is always 50 years and the Use Class is III, OLS Spectrum has a T_R of 64.5 years, while if the Use Class is IV, OLS Spectrum has a T_R of 96 years.)

Italian codes have no specific inter story drift requirement either for ULS seismic design and for non-seismic design; other relevant codes might be used to avoid this regulations lack.

In the ***Romanian code*** – Seismic Design Code, Part 1 – P100-1/2006, Earthquake Resistant Design of Buildings, Annex E – Method to check the Structures Lateral Displacement [32] the accepted values are:

- for service limit state (SLS): dr ≤ 0,005 h;
- for ultimate limit state (ULS): dr ≤ 0,025 h (where h is the level height)

In case they use the Brazilian and Italian spectra the relative level displacements are not over-passed for either of the studied scenarios.

For loads complying with the Romanian code, the accepted relative level displacement for the service limit state is surpassed only by the spectral action along the longitudinal direction with 6%.

In table 7 there are presented the displacements of the joints above the remote structural element.

The longitudinal displacements of the joints above the eliminated elements under the spectral action acting on the longitudinal direction are higher than the transversal displacements under the spectral action

Table 7 – Displacements of the joints above the remote structural element, resulted from the longitudinal (L) and the transversal (T) seismic actions, appropriately read on longitudinal and transversal directions respectively.

Scenario	Displacements (m)		
	Brazilian Spectrum	Italian Spectrum	Romanian Spectrum
S0 – undamaged structure	----------	----------	----------
S1 – 1 column in the middle of long side	L 14.2×10^{-3} T 3.6×10^{-3}	L 8.9×10^{-3} T 3.0×10^{-3}	L 26.7×10^{-3} T 6.6×10^{-3}
S2 – 1 column in the middle of short side	L 14.1×10^{-3} T 3.3×10^{-3}	L 8.8×10^{-3} T 2.2×10^{-3}	L 26.5×10^{-3} T 6.2×10^{-3}
S3 – 1 corner column	L 14.1×10^{-3} T 3.5×10^{-3}	L 8.8×10^{-3} T 2.3×10^{-3}	L 26.6×10^{-3} T 15.6×10^{-3}
S4 – 3 columns in the same corner	L 14.5×10^{-3} T 4.1×10^{-3}	L 9.1×10^{-3} T 2.7×10^{-3}	L 27.5×10^{-3} T 7.7×10^{-3}
S5 – 1 column on 2 floors in the middle of long side	L 23.7×10^{-3} T 7.4×10^{-3}	L 14.8×10^{-3} T 4.3×10^{-3}	L 45.4×10^{-3} T 13.9×10^{-3}
S6 – 1 column on 2 floors in the middle of short side	L 23.0×10^{-3} T 7.4×10^{-3}	L 14.3×10^{-3} T 4.8×10^{-3}	L 43.6×10^{-3} T 14.1×10^{-3}
S7 – 1 central column	L 14.3×10^{-3} T 3.7×10^{-3}	L 8.9×10^{-3} T 2.4×10^{-3}	L 26.8×10^{-3} T 6.7×10^{-3}
S8 – 1 column on 2 floors to the centre of the structure	L 23.3×10^{-3} T 7.5×10^{-3}	L 14.5×10^{-3} T 4.9×10^{-3}	L 44.2×10^{-3} T 14.1×10^{-3}
S9 – wall + the 2 adjacent poles	L 15.1×10^{-3} T 21.9×10^{-3}	L 9.5×10^{-3} T 13.7×10^{-3}	L 28.6×10^{-3} T 41.2×10^{-3}

on the transversal direction with a single exception, namely at the scenario **S9** (elimination of the wall and of the two adjacent columns) and for all considered spectra. The bigger longitudinal displacements resulted from the longitudinal seismic action are for the scenarios **S5**, **S6**, and **S8**, namely those where a column on two floors as well as the beams and the adjacent floors are to be eliminated, Figure 5. Comparing it with the longitudinal displacements of the undamaged structure, the increase is not significant (6%). From the point of view of the transversal displacement under the spectral action acting on the transversal direction, only scenario **S9** (elimination of a wall and of the adjoining columns at the ground-floor) brings a significant increase compared to the displacement of an undamaged structure – more than 500%. In the same time, it is the highest from all scenarios proposed to be studied.

We shall draw the attention on the remark referring to the displacements on the vertical direction in two of the studied scenarios, namely: (1) the values of the displacements on the vertical direction are almost equal with to those on the longitudinal in scenario **S4** and of the same value with the displacements on the transversal direction, in scenario **S5**; (2) the displacements on the vertical direction are significantly higher than those on the transversal direction, in case of the scenario **S4** (ratio between the two displacements being of about 4,3).

Regarding the comparison between the values of the displacements for the three codes, those presented in the previous sub-chapter remain valid.

For the scenarios **S5** (one column on two floors, ground-floor and 1st floor + girders and the adjoining plates, at the middle of long side) and **S9** (wall + the 2 adjoining columns at the ground-floor) the level relative displacements were calculated, Table 8.

In case they use the Brazilian and Italian spectra the relative level displacements are not over-passed for either of the studied scenarios.

In case the Romanian spectrum is used, the level relative displacements on the longitudinal direction resulted from the spectral loading on the same direction are higher than the allowed level displacement for the service limit state for both **S5** and **S9** scenarios – the displacement is about 6% – 16% respectively. For the situation of the transversal load the allowed level displacement is exceeded only for the scenario **S9**, is of about 68%.

5 Final comment

The article is structured in two parts, namely:

1) Eurocodes and three national codes were commented – Brazilian, Italian and Romanian.

2) Case study made based on building type structure with a dual static system considered loaded with the elastic spectra corresponding to the three national codes. 9 scenarios were studied considering the initial damages. From these 5 are cases where the vertical elements (columns) were eliminated from the ground-floor level, 3 are cases where one vertical

Table 8 – Displacements and inter-story drift ratios for scenarios **S5** and **S9**.

Scenario	Displacements in the joints above the structural elements considered to be eliminated – scenario **S0** (m)					
	Brazilian spectrum		Italian spectrum		Romanian spectrum	
	displacement	dr/h	displacement	dr/h	displacement	dr/h
Scenario with maximum displacement at the 2nd floor (above the 1st) – **S5 –** (height on 2 floors)	L 23.7×10^{-3} T 7.4×10^{-3}	L 2.8×10^{-3} T 0.9×10^{-3}	L 14.8×10^{-3} T 4.3×10^{-3}	L 1.7×10^{-3} T 0.5×10^{-3}	L 45.1×10^{-3} T 13.9×10^{-3}	L 5.3×10^{-3} T 1.6×10^{-3}
Scenario with maximum displacement at the 1st floor (above the ground-floor) – **S9** (only ground-floor h1)	L 15.1×10^{-3} T 21.9×10^{-3}	L 3.1×10^{-3} T 4.5×10^{-3}	L 9.5×10^{-3} T 13.7×10^{-3}	L 1.9×10^{-3} T 2.8×10^{-3}	L $28.6 \times 10\text{-}^{-3}$ T 41.2×10^{-3}	L 5.8×10^{-3} T 8.4×10^{-3}
dr/h (accepted) (h1 = 4.9 m; h2 = 3.65 m) h1+ h2 = 8.55 m)	**SLS: NA** **ULS: dr/h ≤ 20.0 × 10⁻³**		**SLS: dr/h ≤ 5.0 × 10⁻³** **ULS: NA**		**SLS: dr/h ≤ 5.0 × 10⁻³** **ULS: dr/h ≤ 25.0 × 10⁻³**	

element was eliminated on two floors (ground-floor and 1st floor) accompanied by the afferent beams and floors, 1 case where a wall at the ground-floor level was eliminated and the two adjoining columns. To this the undamaged structure scenario is added.

Codes presentation – the following comments can be made:

- In the Eurocodes they use directly the term "robustness" very frequently.
- Unfortunately, it does not happen in the national codes. In the Italian and Romanian codes the use the term "robustness" directly is met in some cases. In Brazilian code there is nothing at all regarding robustness.

The following comments can be made for the case study:

- The maximum displacements under the gravity action are in the scenario **S4** (elimination of 3 corner columns) where the increase is 2440%.
- The proper dynamic response in scenario **S9** (elimination of a wall and of the adjoining columns at the ground-floor) brings about an increase with 27% of the eigenperiod and the eigenvector shape changes, being a rotation round the wall that has remained undamaged.
- The spectral action in the Italian code brings about the smallest displacements. The amplest displacements are obtained from the loading with the Romanian spectrum. So, the response values from the action of the Italian spectrum represent about 62% from Brazilian spectral response and 32% from the Romanian one. The Brazilian spectral response represents about 52% from the Romanian one.
- For the global structure behaviour one can see that the global spectral response is not significantly influenced by the elimination of the structural elements proposed in the studied scenarios. The only case where the displacements values in the joint taken as the reference mark (joint 369) registers a significant increase is that of scenario **S4** (elimination of 3 columns at the same corner) when loaded with the spectrum on the transversal direction of the structure.
- For the local behaviour in the area of the eliminated structural element the following conclusions can be presented:
  (1) The most ample longitudinal displacements resulted from the longitudinal seismic action are for the scenarios **S5**, **S6**, and **S8**, especially that where one column is be eliminated on two floors as well as the adjoining beams and floors.
  (2) From the transversal displacements point of view, under the spectral action acting on the transversal direction, in scenario **S9** (elimination of a wall and of the adjoining columns at the ground-floor) it is obtained the most significant increase.
  (3) Although the relative level displacement is calculated only from the seismic action (that is without using the formulas of the effects superposing), complying with the Romanian regulations, the accepted value for the displacement for the service limit state is surpassed with 6 – 16% for the longitudinal direction even for the undamaged structure **S0** as well as for the other scenarios. In the transversal direction for scenario S9 the displacement is with 68%.

In the end, we would like to cite the work "Load and Resistance Factor Criteria for Progressive Collapse Design" [14] where he evidences: "Professionalism requires an acknowledgement of the fact that good design involves looking beyond the minimum design requirements in a code or standard... It is essential for structural engineers to understand the issues involved and to think the unthinkable at the conceptual design stage."

The authors wish to continue the activity with a study on the progressive collapse. In Romania, the code "P100-1 Earthquake Resistant Design of Buildings", was reviewed and its stipulations will be applied in the numerical example of the new paper.

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