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1           **DEFINING STRUCTURAL ROBUSTNESS UNDER SEISMIC AND**  
2           **SIMULTANEOUS ACTIONS: AN APPLICATION TO PRECAST RC**  
3           **BUILDINGS**

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8  
9           **ABSTRACT**

10          The increasing complexity of urban systems is making robustness a crucial requirement for  
11          structural design. The paper deals with the concept of robustness of civil structures against extreme  
12          events. After a brief literature survey, a novel point of view to robustness assessment is proposed,  
13          fitting the most accepted robustness definition. The proposed approach is discussed and compared  
14          with other methodologies for quantifying structural robustness. Thus, the methodology is  
15          developed and applied to an existing precast industrial building case study, assumed to be prone to  
16          seismic and wind hazards. In particular, the case study is assumed to be located in Emilia, Italy,  
17          where a significant earthquake occurred in 2012, causing relevant damage to gravity load designed  
18          industrial buildings. Three structural options are discussed, namely a simple supported beam-  
19          column connection (gravity load designed solution) and two pinned connections (seismic designed  
20          solution), where only one of them satisfies the current structural code requirements. The results are  
21          discussed in terms of robustness quantification, by means of a robustness matrix. The authors  
22          envisage that this approach can be effectively adopted for portfolios of existing structures, to  
23          prioritize retrofitting interventions, aimed at maximizing the overall risk mitigation with limited  
24          economic resources.

25          **Keywords:** structural robustness, robustness quantification, extreme events, industrial buildings

# 1 INTRODUCTION

2 Structural retrofitting of existing structures is one of the most common operations in construction  
3 industry, especially in countries where a large amount of the built environment is composed of old  
4 structures. Limited structural capacity, changes in the use of structures or increased complexity of  
5 the urban networks are some of the reasons why structural retrofitting is needed.

6 At the urban scale, structural retrofitting involves prioritization of interventions, aimed at  
7 maximizing the overall risk mitigation with limited economic resources. This is still an unsolved  
8 issue, especially nowadays, when the complexity of urban structures and physical systems is  
9 rapidly increasing, as a result of the need for advanced performances and services in contemporary  
10 cities.

11 Thus, an approach to rank the most urgent retrofit operations in portfolios of structures would be  
12 desirable. Such an approach should take into account two aspects:

- 13 • Existing structures have not been designed to face the risks we currently consider for the  
14 design of new structures (i.e. design actions according to up-to-date codes) and their  
15 “nominal” structural capacity could be very low; on the contrary, they could still have an  
16 unexpected robust behavior, due to capacity reserves, beyond the conventional failure  
17 assessed by structural codes; not considering these reserves could be costly and ineffective.
- 18 • The possibility that extreme events with extreme consequences can take place should also  
19 be considered; indeed, extreme event occurrence can be under estimated due to lack of  
20 knowledge of their past occurrence frequencies, increased cascading effects induced by  
21 urban system interconnections, or exacerbated natural phenomena (e.g. climate change).

22 These issues have stimulated a growing discussion in risk engineering community, about topics  
23 related to robustness of physical systems and structures against extreme events. Structural  
24 robustness concept is herein adopted to propose a methodology to rank the need of retrofitting  
25 operations to existing structures, accounting for both the points mentioned above.

## 1 **Structural robustness**

2 A unique definition for structural robustness is still not available in the literature. Starossek and  
3 Haberland (2010) provided a large review on the definition of robustness; it is often introduced as  
4 the capability of a structure to withstand accidental actions without suffering disproportionate  
5 collapse (Fib 2010, CEN 2002). According to the *fib* Model Code 2010 “*Robustness is a specific*  
6 *aspect of structural safety that refers to the ability of a system subject to accidental or exceptional*  
7 *loadings (such as fire, explosions, impact or consequences of human errors) to sustain local*  
8 *damage to some structural components without experiencing a disproportionate degree of overall*  
9 *distress or collapse*” (Fib 2010). It is evoked as a qualitative property to be achieved in order to  
10 face unexpected events and consequences that are difficult to predict with the existing analytical  
11 methodologies. Many authors also evidenced how robustness improvements could be in contrast  
12 with seismic design strategies (De Biagi and Chiaia 2013).

13 In order to make structural robustness a measurable property to be practically used for design and  
14 assessment purposes, a number of works have been developed, aiming at proposing new  
15 methodologies to quantify robustness (canisius et al. 2007); however an accepted approach in the  
16 literature has not yet been agreed upon. Currently, most of the proposed methods aim at computing  
17 structural performance once an event has taken place and has damaged a limited part of the  
18 structure. The scope is to appreciate how much the damaged structure is capable to withstand the  
19 remaining loads, avoiding much larger damage, such as progressive collapse.

20 More in detail, two approaches can be pointed out: (a) capacity and demand robustness approaches  
21 (CDRA) and (b) capacity robustness approaches (CRA).

22 In the CDRA the scope is to manage robustness through the quantification of the reliability index  
23 or the probability of failure of the structures in different damaged configurations. For instance,  
24 Frangopol and Curly (1987) firstly proposed a robustness index  $\beta_r$  as

$$\beta_r = \frac{\beta_i}{\beta_i - \beta_d} \quad (1)$$

1  
2 being  $\beta_i$  and  $\beta_d$  the reliability index of the intact and the damaged structure, respectively. This  
3 indicator approaches infinity as  $\beta_d$  approaches  $\beta_i$ , that is when the damaged structure has the same  
4 reliability of the intact one. This approach has been applied in (Ribeiro et al . 2014) and similar  
5 indicators have been proposed by Lind (1995) and Baker et al. (2008). In particular, Lind  
6 introduced the Vulnerability index  $V$  as:

$$V = \frac{P(r_d)}{P(r_i)} \quad (2)$$

7  
8 being  $P(r_d)$  and  $P(r_i)$  the probability of failure of the damaged and the intact structure, respectively.  
9 On the contrary, Baker introduced the robustness index  $I_{rob}$  as an extension from the failure  
10 probability to risk:

$$I_{rob} = \frac{R_{DIR}}{R_{DIR} + R_{IND}} \quad (3)$$

11 being  $R_{DIR}$  and  $R_{IND}$  the risk associated with the direct damage and the indirect (caused by the  
12 disproportionate collapse) damage, respectively. Within the computation of  $R_{DIR}$  and  $R_{IND}$ , a direct  
13 quantitative approach to the risk holds. In these methods, all dealing with the hazards the structure  
14 is prone to, the computation of risk or reliability indices is needed, both for the intact and the  
15 damaged structural configuration.

16 On the other side, the CRA defines some indices, aimed at measuring the robustness as an intrinsic  
17 characteristic of the structural system, not depending on the hazards the structure is prone to. The  
18 objective of these methods is to quantify the capability of the investigated structure to withstand  
19 extreme damage, whatever the event causing it. Different authors proposed various indices based  
20 on the load carrying capacity or the stiffness of the structures, e.g. the degree of indeterminacy or  
21 the stiffness matrix, to be evaluated in different damaged configurations and compared with the  
22 intact configuration (Starossek and Huberland 2008, Wisniewski et al. 2006, Smith 2006, Ciaia

1 and Masoero 2008). For example, Starossek and Haberland (2011) proposed the measure of  
2 robustness  $R_S$  as

$$R_S = \min \frac{\det \mathbf{K}_j}{\det \mathbf{K}_0} \quad (4)$$

3  
4 being  $\mathbf{K}_0$  the stiffness matrix of the undamaged structure and  $\mathbf{K}_j$  the stiffness matrix of the structure  
5 after removal the  $j^{th}$  element or connection.

6

### 7 **ROBUSTNESS ASSESSMENT**

8 A novel point of view to robustness computation is herein introduced. It links the relationship  
9 between local and global damage, used in the accepted robustness definitions, with the relationship  
10 between the conventional and the ultimate failures, adopted in the structural failure assessment.

11 In fact, the conventional failure, that is assessed by structural codes analysis, generally consists in  
12 a local failure occurring to a limited portion of the structure (e.g. one section, one element, one  
13 sub-assembly). Once this failure is achieved, the structure is conventionally failed according to  
14 structural codes. Beyond this local failure the structure is able to provide capacity reserves able to  
15 delay the global failure mechanisms involving large portions of the structure. For instance, the  
16 conventional failure of a reinforced concrete frame structure due to seismic actions can occur when  
17 one element achieves its ultimate shear capacity or its ultimate rotation capacity, that is a local  
18 damage. Instead, the structure exhibits a robust behavior, if, beyond this state, it can still sustain  
19 seismic actions avoiding that this local failure propagates up to a global failure mechanism, e.g. a  
20 soft storey mechanism.

21 Thus, the accepted robustness definition, such as that provided by the *fib* Model Code 2010 (Fib  
22 2010) previously mentioned, is herein applied to describe the behavior of a structure, that, once  
23 achieved a local failure (that structural codes would consider the conventional failure of the

1 structure), goes beyond this state and is capable to sustain the local damage delaying its  
2 propagation to a global collapse mechanism.

3 Furthermore, this point of view to robustness helps to overcome two issues. Initially, it can be  
4 argued that most of the methods, both CDRA and CRA, need to distinguish between the actions  
5 causing local/limited damage and the global mechanisms causing the extension of this damage to  
6 large and disproportionate collapse. In other words, these approaches need to manage an  
7 intermediate structural “stage”, that corresponds to limited and localized damage, at the onset of  
8 larger damage involving the whole structure. Identifying these intermediate “stages” is not  
9 straightforward. CDRA can address this issue by means of event tree methods (Baker et al. 2008),  
10 but investigating all the local damage that could likely take place on a structure and their  
11 probability of occurrence can be unpractical. A detailed review on these approaches and an  
12 exhaustive discussion on this issue is present in Starossek and Huberland (2011). Thus, the point of  
13 view here proposed tries to overcome the issue related to the identification of these intermediate  
14 structural “stages”, that could result in large computational efforts.

15 A second motivation comes from the fact that CRA and CDRA methodologies disagree on  
16 considering hazard intensity into robustness quantification. In fact, two identical structures, prone  
17 to different hazard levels, result equally robust, according to CRA methodologies, whilst they  
18 present different robustness values, according to CDRA methodologies. The authors believe that a  
19 procedure aimed at quantifying structural robustness cannot disregard the hazard intensities the  
20 investigated structure is prone to. In particular, in the authors’ opinion, robustness quantification  
21 should also help appreciating the capability of the structure to withstand exceptional actions,  
22 compared with the capability to withstand normally expected actions, as those usually defined by  
23 codes and guidelines for design or assessment. In other words, robustness should quantify the  
24 capability to guarantee a structural capacity level, which is higher than that requested by structural  
25 codes.

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## **A novel approach**

The proposed approach moves from a comparison between what can be considered as a “normal event” and what is an “extreme or exceptional event”. The idea is that a “normal event” is an event that is “expected” with a non-negligible probability (although severe), that is about 10%, in the structural lifetime. As a consequence of this, structural design and assessment are conducted with respect to these events, that represent the reference events of current structural codes. On the contrary, “extreme events”, according to the current knowledge, are typically “expected” with a very low probability in structural life time and structural codes do not require to take them into account explicitly into structural analyses.

Focusing on structural systems, “exceptional events” can be divided into four categories, whose definition depends on the investigated structure:

- Type 1: events whose typology is already considered in the current structural design, but whose intensity is larger than the maximum value structural codes would consider for design (at the most severe limit state condition). For example, this is the case of a particularly severe earthquake, hitting a certain structure, whose intensity is larger than the reference intensity, which would have been used to design that structure (collapse limit state intensity);
- Type 2: events that are not considered by structural codes for the investigated structure, (since it is not expected that they could significantly affect it), but that are considered for other structures in the same site or hazard conditions (e.g. fire actions, that are considered in structural design only for some kind of structures). Also earthquakes can belong to this category, in case of structures located in areas characterized by comparatively low-seismicity, according to the current geophysical knowledge of that area. However, it is underlined that, in case of existing structures located in seismic areas, but designed only for



1 gravity loads (e.g. because of recent modification in structural codes), earthquakes should  
2 be considered as Type 1 events; in fact, in this case, earthquakes have now become “normal  
3 events”.

- 4 • Type 3: more than one event occurring simultaneously, characterized by intensities that are  
5 larger than those considered by structural codes in load combinations (e.g. an earthquake  
6 hitting a structure when it is loaded by particularly severe vertical live loads).
- 7 • Type 4: totally unexpected events that are not typically considered by structural codes for  
8 any structural typology (e.g. meteor impact or soil liquefaction).

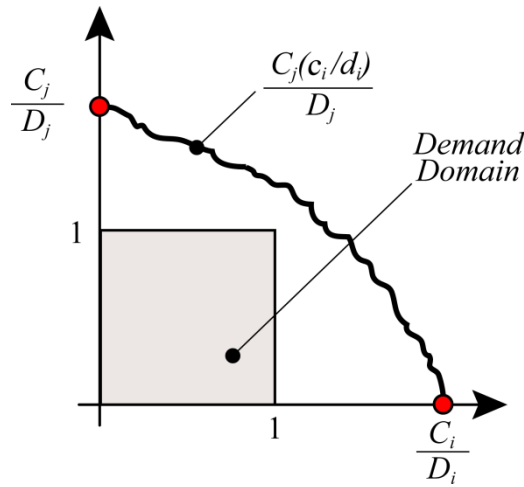
9 The methodology here presented can be used to quantify structural robustness with reference to  
10 extreme events of Type 1, 2 and 3. Therefore, the idea is to quantify the capability of the  
11 investigated structure to withstand events or combination of events, whose intensity exceed the  
12 design threshold, that is posed by structural guidelines and codes.

## 14 **Methodology**

15 Let us consider a structure prone to  $n$  hazard types. Each hazard will be characterized by an  
16 intensity measure  $IM$ , and an intensity demand value  $D$  (expressed in terms of  $IM$ ), posed by  
17 structural codes to design the structural capacity of new structures.

18 Let us compute, by means of proper structural models, the structural capacity  $C_i$  (in terms of  $IM$ )  
19 for the  $i$ -th event type, taking into account all the potential resisting mechanisms, that is setting all  
20 the capacity safety factors to unity (i.e. for material strength, member resisting mechanisms and  
21 global resisting mechanisms). The aim is to compute the maximum available structural capacity,  
22 including any resistance mechanism that would make the structure capable to withstand event  
23 intensities beyond the demand intensity posed by structural codes. Let us compute also, for each  
24 event type couple  $i$ - $j$ , the structural capacity function  $C_j(d_i)$  against the hazard  $j$ , in case the hazard  
25 type  $i$  is applied on the structure with intensity  $d_i$ .

1 Finally, let us compute the capacity over demand ratios ( $C/D$ ) for all the considered hazards and  
 2 hazard combinations. These can be plotted in a  $(C_i/D_i-C_j/D_j)$  plan, as depicted in **Figure 1**.



3  
 4 **Figure 1.** Robustness domain

5  
 6 The resulting curve represents a sort of a capacity domain. The points inside the smaller square  
 7 domain, namely the “demand domain”, represent intensity values, that are lower than the reference  
 8 values  $D_i$  and  $D_j$ , for the  $i$  and  $j$  event types, respectively. Hence, robustness can be quantified as  
 9 follow:

- 10 • Robustness against extreme events of Types 1 and 2 is represented by the intersection of  
 11 the capacity domain on the axes, namely  $R_i=C_i/D_i$ . This represents the capability of the  
 12 structure to withstand events, overcoming the reference intensity used by structural codes  
 13 for design purposes.
- 14 • Robustness against extreme events of Type 3 is represented by the ratio of the area below  
 15 the capacity domain, namely  $R_{i,j}$ , and the area of the demand domain, equal to unity. This  
 16 would represent an integral measure of the capability of the structure to withstand  
 17 simultaneous events, overcoming the reference intensity used by structural codes. The  
 18 perimeter of the demand domain does not need to represent exactly the points of the  
 19 demand actions, especially in case of simultaneous actions. Although this would be

1 desirable, this would complicate its definition and the computation of its area. Thus, since it  
 2 is only used as a reference domain, whose area is adopted only to normalize the area of the  
 3 robustness domain, to simplify the computation, it is assumed to be square. This  
 4 assumption does not affect the framework of the methodology.

5 These values can be arranged in a symmetric matrix, namely the *robustness matrix*  $\mathbf{R}$ , of  
 6 dimension  $n$ , whose generic element  $i,j$  represents:

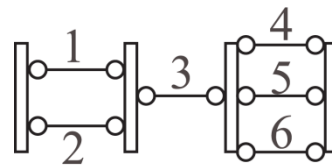
- 7 - in case  $i$  is equal to  $j$ , the robustness  $R_i$  against the single event  $i$  (extreme event Type 1 and  
 8 2);
- 9 - in case  $i$  is not equal to  $j$ , the robustness  $R_{i,j}$  against the simultaneous events  $i$  and  $j$  (extreme  
 10 event Type 3):

$$\mathbf{R} = \begin{pmatrix} R_i & R_{i,j} & \cdots & R_{i,(n+m)} \\ R_{i,j} & R_j & & \vdots \\ \vdots & & \ddots & \\ R_{i,(n+m)} & \cdots & & R_{(n+m)} \end{pmatrix} \quad (5)$$

11  
 12 The methodology here proposed can be interpreted as a way to measure how much the demand  
 13 intensity can be scaled, up to the maximum event intensity the structure can withstand (the  
 14 maximum capacity intensity). According to this interpretation, the methodology would be aimed at  
 15 measuring a sort of safety factors against the “likely” hazard levels, based on the elaboration of  
 16 capacity-demand ratios. In fact, a structure that is designed according to current structural codes  
 17 (satisfying demand intensities) would exhibit a robustness that depends on its overstrength. Indeed,  
 18 according to this procedure, structural capacity is computed neglecting all the safety  
 19 factors/mechanisms, typically taken into account by structural codes. This overstrength can be  
 20 interpreted as composed by two contributions:

1 1. Capacity safety factors. When dealing with code-based design/assessment, design material  
 2 properties and mechanism capacity values are much lower than their expected values (which are  
 3 used in the proposed robustness quantification), due to partial safety factors;

4 2. Conventional failure. When dealing with code-based design/assessment, conventional  
 5 structural failure is typically achieved when structural capacity is overcome in the first structural  
 6 element. In robustness quantification, failure is achieved only when a global collapse mechanism is  
 7 activated, or the final global equilibrium/stability configuration is overcome, or the structure  
 8 becomes unstable. Structural redundancy largely influences this contribution to robustness,  
 9 especially due to alternate load paths and resisting systems. The conventional failure can be easily  
 10 recognized, as illustrated by Jalayer et al. (2011), defining the mechanism into a cut-set  
 11 formulation (Ditlevsen and Masden 1996).



12  
 13 **Figure 2.** Schematic representation of a cutset  
 14

15 A cut set (schematically represented in **Figure 2**) is a series of parallel systems. The system  
 16 collapses only if all the elements of the generic parallel system reach the crisis. Only in this way,  
 17 as said before, it is possible to take into account all the possible collapse mechanisms. A typical  
 18 example is the case of the soft storey mechanism. In this case, the parallel system is composed by  
 19 all the hinges of the same storey and the system only collapses if all the hinges are activated.

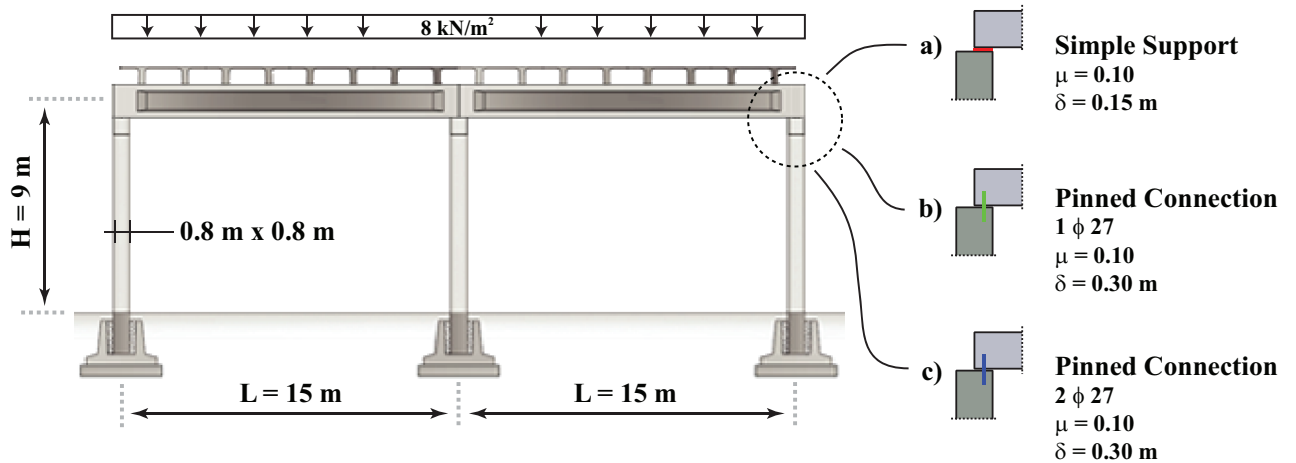
20 It could be argued that the available methodologies, computing robustness as the capability to  
 21 avoid global collapse once local damage has occurred, try to measure only this second  
 22 contribution. In these cases, robustness is interpreted as the contribution to the overstrength  
 23 provided by the capability to avoid the activation of a global mechanism, once the conventional

1 (according to structural codes) failure has taken place. On the contrary, in the methodology here  
2 proposed, also the first contribution is considered and, once both contributions are summed up, the  
3 total structural overstrength is normalized with respect to the conventional hazard intensity.

#### 4 **CASE STUDY**

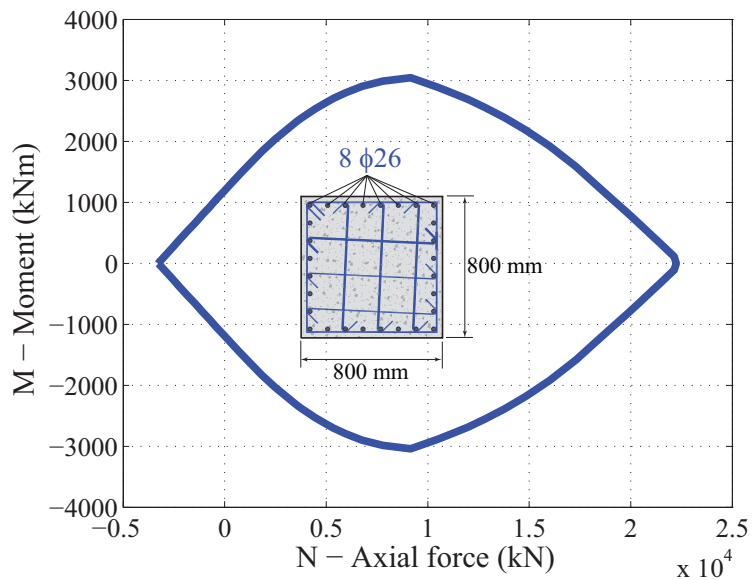
6 In 2012, a significant earthquake event hit the Emilia region in Italy (Iervolino et al. 2012,  
7 Scognamiglio et al. 2012, Meletti et al. 2012), causing significant damage, in particular, to  
8 numerous industrial buildings (Liberatore et al. 2013). Before this event, the general perception of  
9 the seismicity of that zone was not supported by a strong cultural memory of earthquakes, since the  
10 previous significant seismic event only occurred in the 16<sup>th</sup> century. Many buildings, built in past  
11 decades, were only designed for gravity loads, even if structural codes have recently introduced the  
12 need for seismic design for new structures. This event triggered a wide discussion between Italian  
13 scientific and practitioner communities on structural robustness, especially for industrial buildings  
14 (Liberatore et al. 2013, Savoia et al. 2012).

15 Hence, in order to contribute to this issue, the proposed methodology is here applied to a typical  
16 industrial precast reinforced concrete building, assumed to be located in Mirandola, in the  
17 epicentre region of the Mw 6.1 mainshock. The investigated structural model consists of a 2D  
18 frame, composed by two spans and one floor, with a height of 9m and a span length of 15m  
19 **(Figure 3)**.



1  
2 **Figure 3.** Case study structural model

3  
4 The three columns have a square section of 80cm of width. **Figure 4** depicts the column cross  
5 section internal reinforcement and the relative axial force-bending moment failure domain. The  
6 concrete compression strength  $f_c$  is assumed equal to 30MPa, whereas the steel reinforcement  
7 yielding tensile stress  $f_y$  is assumed equal to 450MPa. The Young modulus  $E$  adopted for concrete  
8 is equal to 30GPa.



9  
10 **Figure 4.** Column cross section and failure domain

1 The most critical failure mechanism experienced for this type of structures, especially during the  
2 Emilia earthquake, was the beam-column joint failure. In particular, in case of a simply supported  
3 connection, largely adopted in gravity load designed structures, beam collapse was widely  
4 experienced.

5 Hence, three different typologies of column-beam connection have been taken into account in this  
6 case study analysis:

7 a) simply supported connection, assuming a Teflon-concrete friction coefficient  $\mu$  equal to 0.1 and  
8 a support length of 15cm;

9 b) pinned connection with one dowel of 27mm of diameter and 640MPa of failure stress, for each  
10 beam support, assuming a Teflon-concrete friction coefficient  $\mu$  equal to 0.1 and a support length  
11 of 30cm;

12 c) pinned solution with two dowels of 27mm of diameter and 640MPa of failure stress, for each  
13 beam support, assuming a Teflon-concrete friction coefficient  $\mu$  equal to 0.1 and a support length  
14 of 30cm.

15 In particular, solution c) fits the requirements of the current structural codes, for the industrial  
16 buildings located in Emilia. Solution b) represents an intermediate option between the gravity load  
17 designed structure, represented by solution a), and the seismic load designed structure.

18 In order to apply the procedure and quantify structural robustness, three action types have been  
19 considered:

20 a) horizontal seismic actions, whose intensity was identified by the horizontal Peak Ground  
21 Acceleration (PGA);

22 b) vertical seismic actions, whose intensity was identified by the vertical PGA; and

23 c) wind actions, whose intensity was identified by the wind velocity.

24 Horizontal seismic actions and wind actions can be treated as extreme event Type 1, since current  
25 structural codes requires that these types of actions are considered as design loading conditions. On

1 the contrary, vertical seismic action can be treated as extreme event Type 2, since it is not required  
2 to be considered (in case the beam span is less than 20m), among design loading conditions. The  
3 combination of the three action types are treated as extreme event Type 3.

4 As demand intensity values D, 0.21g (CSLP 2008) was used for both the horizontal and vertical  
5 PGA (considering soil type C) and 23m/s was used for the wind velocity (CSLP 2008), since these  
6 are the reference values for the design collapse limit state.

7 The model has been reduced to an equivalent single degree of freedom (SDOF), and non-linear  
8 dynamic analyses have been conducted, for all the combination loading conditions. The SDOF  
9 parameters are:

$$k = \sum_{i=1}^3 3 \cdot \frac{EI}{H^3} \quad \xi = 5\% \quad m = 250 t \quad (6)$$

10 Here,  $k$  is the SDOF stiffness, that is the summation of the shear stiffness of the three columns,  $E$  is  
11 the Young modulus,  $I$  is the inertia modulus of the column cross section,  $H$  is the column height,  $\xi$   
12 is the equivalent viscous damping, and  $m$  is the seismic mass, calculated assuming a distributed  
13 mass of  $800 \text{ kg/m}^2$ . These values resulted in a vibration period of the SDOF system equal to 0.87s.

14 The different loading conditions have been treated using the procedure here described. Firstly,  
15 seven horizontal-vertical ground motions have been identified to have an average spectrum  
16 matching the code acceleration spectrum for the specific site of Mirandola. These ground motions  
17 resulted in 14 possible couples, since for each event, two horizontal time histories were considered,  
18 one for each of the horizontal directions. Each ground motion has been incrementally scaled, so to  
19 assume the same PGA value. These input ground motions have been applied to the SDOF system  
20 and the spectral acceleration time histories  $a(t)$  at the top of the structure have been derived.

21 The selection of the accelerograms has been carried out through the software REXEL (Iervolino et  
22 al. 2010). Horizontal-vertical ground motions have been identified to have an average spectrum  
23 matching the code acceleration spectrum for the specific site of Mirandola (Lat. 44.878, Long.  
24 11.062). The accelerograms have been chosen from the European Strong Motion Database; they



1 are natural signals not scaled that have an epicentral distance between 0 and 50 km, a magnitude  
 2 between 4 and 7 km, and are recorded in free filed stations located on soli type C according  
 3 Eurocode 8. The table 1 below reports the main characteristics of the signals, whereas figure 5  
 4 shows the code spectrum and the mean spectrum of all the used accelerograms.

5

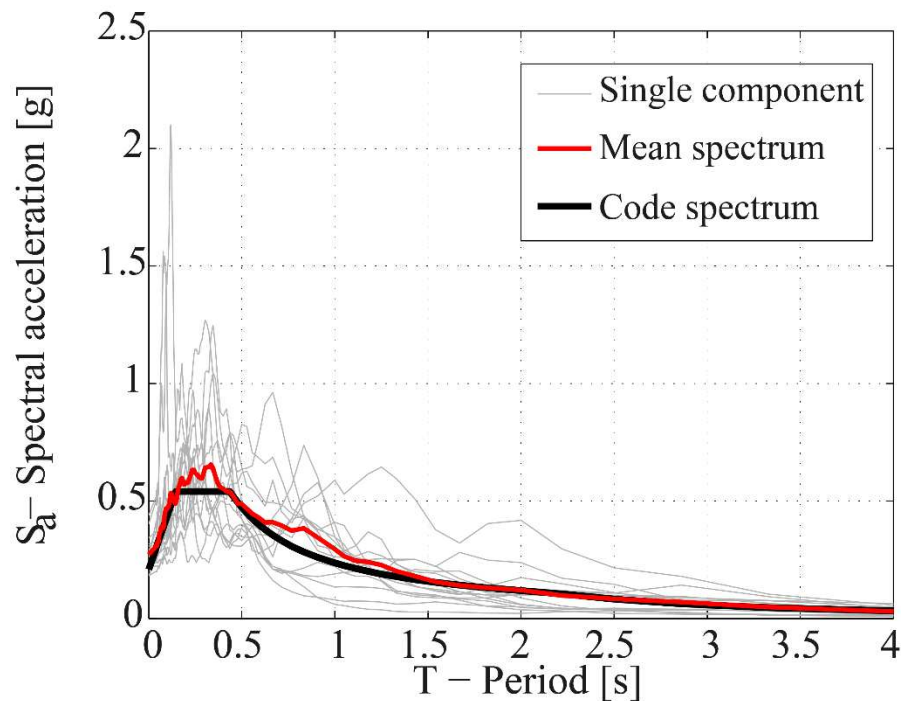
Earthquake Name	Date	Mw	Fault Mechanism	Epicentral Distance [km]
Alkion	24/02/1981	6.6	normal	20
Spitak	07/12/1988	6.7	thrust	36
NE of Banja Luka	13/08/1981	5.7	oblique	7
Chenoua	29/10/1989	5.9	thrust	29
Umbria Marche	26/09/1997	5.7	normal	3
Adana	27/06/1998	6.3	strike slip	30
Dinar	01/10/1995	6.4	normal	8

6

**Table 1** - Used records

7

8



9

10

**Figure 5.** Code spectrum and mean spectrum of all the used accelerograms.

1 Is important to observe that the Mirandola event is not taken into account in the selected event in  
2 order to maintain the application of the procedure as much as possible general, avoiding event  
3 specific effects.

4 The following phase of the analysis is different for the case of simply supported connection and  
5 pinned connection.

6 In the first case, the potentially sliding horizontal acceleration has been computed as:

$$a^*(t) = |a(t) - \mu N(t) \text{sign}[a(t)]|/m \quad (7)$$

7  
8 where  $N(t)$  is the vertical axial force, equal to the gravity load. This was also reduced by means of  
9 the wind uplift action, induced by the internal overpressure, and the vertical seismic acceleration;  $\mu$   
10 is the friction coefficient, assumed equal to 0.1. The potentially sliding horizontal acceleration  $a^*(t)$   
11 was double integrated over time to derive the support displacement  $d(t)$ . In particular, a stick-slip  
12 model was applied (Andreaus and Casini 2001) and two stages were considered:

- 13 a) the connection is effective (when the friction force is larger than the inertia force) and
- 14 b) the connection is not effective (when the friction force is exceeded).

15 When the maximum of  $d(t)$  was larger than the support length, the beam was considered collapsed.

16 In the case of pinned connection, the term  $a(t)$  was multiplied by the mass to obtain the horizontal  
17 force at the top of the structure. This model has been adopted for seek of simplicity although a  
18 more refined model could also consider the change of the dynamic properties of the system once  
19 sliding has initiated. The criterion proposed by Vintzēleou and Tassios (1986) and adopted by the  
20 *fib* Model Code 2010 (Fib 2010) was used to check the failure of the dowels. It also accounts for  
21 the tensile axial load on the dowel that was computed as the absolute value of  $N(t)$ . Once the  
22 dowels failed, it was assumed that the connection reduced to a simply supported connection; then,  
23 for the remaining time history of  $a(t)$ , the support sliding failure was checked, as in the previous  
24 case.

1 In both cases, two more failure criteria were also checked:

2 a) time dependent axial force and bending moment values acting on the column were derived and

3 checked against the column failure domain in **Figure 4**;

4 b) negative axial force (i.e. acting downwards) on the beam-column connection was checked

5 against the compression failure value of the support, computed as  $0.6f_cA$  (being  $A$  the support area,

6 equal to the beam width times the support length). This failure criterion governed in case of very

7 large vertical acceleration values.

8 For each horizontal PGA, the median value of the vertical PGA causing collapse, within the 14

9 possible horizontal-vertical ground motions, was computed, for each value of wind velocity.

10 In the investigated case study, it is clear how the novel approach to robustness applies. The local

11 damage consists in the overcoming of the friction threshold, in case  $a$ ), and in the dowel failure, in

12 cases  $b$ ) and  $c$ ); it represents the conventional failure according to structural codes. Thus, the

13 analysis is aimed at assessing the capability of the structure to sustain this local damage and delay

14 its propagation to a global failure mechanism, that is the support loss of the beam.

15 **Figure 6** depicts the robustness domain for the horizontal seismic-vertical seismic actions (a),

16 horizontal seismic-wind actions (b), vertical seismic-wind actions (c), for the three investigated

17 connection types.

18 The robustness domains were also elaborated in terms of robustness matrices for the three

19 investigated cases, namely the simply supported connection  $a$ ), the one dowel pinned connection

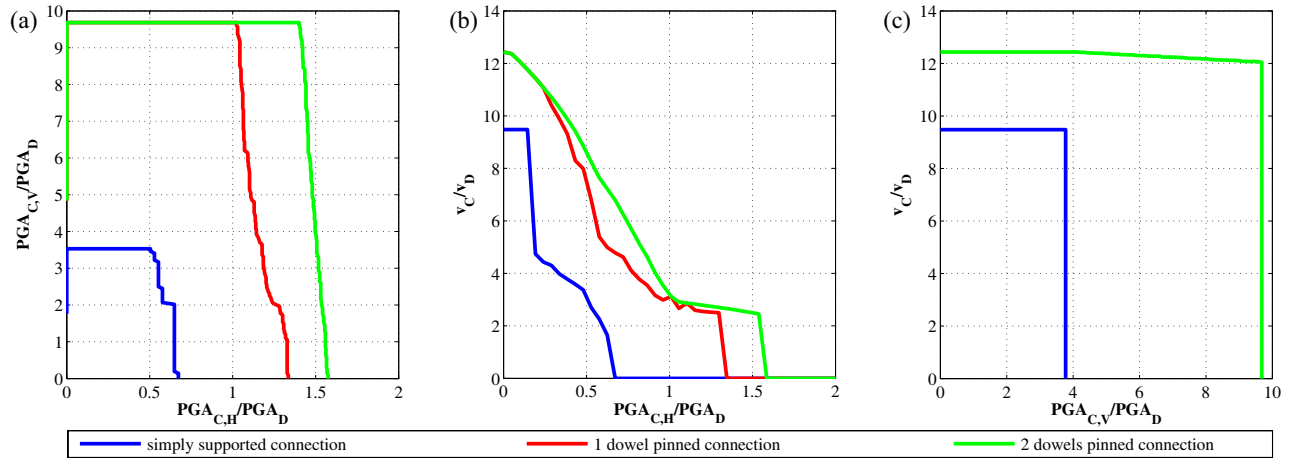
20  $b$ ), and the two dowels pinned connection  $c$ ). The results are reported in Relation 8, where the

21 row/column matrix indices 1, 2 and 3 refer to horizontal seismic action, vertical seismic action and

22 wind action, respectively.

$$R_{(a)} = \begin{pmatrix} 0.67 & 1.45 & 1.74 \\ 1.45 & 3.53 & 5.98 \\ 1.74 & 5.98 & 9.84 \end{pmatrix} \quad R_{(b)} = \begin{pmatrix} 1.34 & 3.37 & 2.89 \\ 3.37 & 9.68 & 10.87 \\ 2.89 & 10.87 & 12.44 \end{pmatrix} \quad R_{(c)} = \begin{pmatrix} 1.57 & 3.79 & 3.13 \\ 3.79 & 9.68 & 10.87 \\ 3.13 & 10.87 & 12.44 \end{pmatrix} \quad (8)$$

23



**Figure 6.** Robustness domains for the cases horizontal seismic-vertical seismic actions (a), horizontal seismic-wind actions (b), and vertical seismic-wind actions (the red curve and the green curve are overlapped) (c). Subscripts “C” and “D” refer to “Capacity” and “Demand”, respectively. Subscripts “H” and “V” refer to “Horizontal” and “Vertical”, respectively.

The increase in robustness indices moving from case a) to case b) and from case b) to case c) is reported in Relation 9 as the ratio of the homologous indices.

$$\frac{R_{(b)}}{R_{(a)}} = \begin{pmatrix} 2.00 & 2.32 & 1.66 \\ 2.32 & 2.74 & 1.82 \\ 1.66 & 1.82 & 1.31 \end{pmatrix} \frac{R_{(c)}}{R_{(b)}} = \begin{pmatrix} 1.17 & 1.12 & 1.08 \\ 1.12 & 1.00 & 1.00 \\ 1.08 & 1.00 & 1.00 \end{pmatrix} \quad (9)$$

These results underpin some considerations about the performance of these kind of structures and the appropriateness (and the need) of retrofit operations. In fact, it can be observed that the increment in robustness index is more significant moving from the simply supported connection to the one dowel pinned connection, than from the one dowel pinned connection to the two dowels pinned connection. This fact triggers the consideration that introducing a seismic device makes structural robustness considerably increasing, even if all the seismic requirements posed by structural codes are not fully satisfied (case b); then, a further advance in structural safety, resulting by a complete fulfillment of seismic code requirements (case c), would not result in an equivalent increase in structural robustness. This fact evokes a further consideration: among the

1 existing structures that do not satisfy current seismic safety requirements, those designed only for  
2 gravity loads are much less robust than those designed for slight seismic intensities; thus, retrofit  
3 operations of this type of structures can be much more urgent. These considerations can drive  
4 retrofitting strategies of a portfolio of structures in case of limited resources. It can be also noticed  
5 that the robustness indices for vertical earthquake and wind do not increase from case b) to c). This  
6 is because in this case robustness depends on the support compression failure that does not change with the  
7 amount of the tensile reinforcement.

8

### 9 **POSSIBLE IMPROVEMENTS**

10 The scope of this paper is to contribute to scientific discussion by proposing a novel point of view  
11 to structural robustness assessment, that can be applicable to real cases and framed within  
12 structural codes approaches, in order to rank the robustness of structures against extreme  
13 hazardous events. However, a number of points are here highlighted for possible further  
14 improvements:

- 15 • The definition of the demand intensity values  $D_i$  is a crucial point; the idea is that these  
16 values could represent the demand intensity posed by structural codes or, in other words,  
17 the event intensity that is now expected to occur, during the lifetime of the investigated  
18 structure, with a reference (non negligible) probability. Hence the definition of  $D_i$  is not  
19 univocal and depends on the hazard type and on the investigated structure.
- 20 • As previously specified, structural analysis should be conducted minimizing all the safety  
21 factors, in order to take into account all the possible resisting mechanisms the structure is  
22 able to exhibit. Furthermore, the structural collapse should be set when the structure  
23 becomes unstable and a mechanism is activated. Again, this kind of approach is not  
24 univocal and depends on several factors, as the hazard type and the adopted structural  
25 models.

90 The structural analysis for extreme events Type 3 (simultaneous events) can presents

1 some difficulties. Depending on the couple of events considered, the definition of the  
2 structural model and the identification of the failure mechanisms could be not  
3 straightforward.

- 4 • The approach illustrated so far only considers the contemporaneity of two event types. The  
5 contemporaneity of more than two events, although much more unlikely, could be also  
6 considered, especially to account for very severe cascade events.
- 7 • The approach does not take into account extreme event Type 4. It is the case of events that  
8 are not considered by structural codes. For such events, a demand intensity value  $D_i$  cannot  
9 be identified and the definition of a structural model can be particularly critical. In this  
10 case, the results of CRA, e.g. indices related to the structural stiffness, can represent a  
11 proxy of the capability of the structure to withstand this kind of extreme events, and can be  
12 used to enrich the information provided by the robustness matrix.

## 13 **CONCLUSIONS**

14 A novel point of view has been proposed to the measurement of robustness of structural systems,  
15 resulting in the definition of the robustness matrix **R**. The main feature of this methodology is that  
16 robustness measure is based on the actual hazard types and intensities the investigated structure is  
17 prone to, and on a clear classification of the potential extreme events that can hit the structure. In  
18 particular, it is aimed at investigating how the structure is capable to withstand unexpected events  
19 of extreme intensities, or multiple events unexpectedly occurring together. The methodology fits  
20 the definition of robustness by measuring the capacity reserves that the structure is able to exhibit,  
21 beyond the local (conventionally set by structural codes) damage mechanisms, up to their  
22 propagation into a global failure mechanism.

23 The methodology is easily applicable in practical cases, being strictly related to structural code  
24 design requirements. In fact, such an approach can be potentially implemented in a structural code

1 framework, in order to practically perform robustness assessment of existing structures, but also to  
2 design new structures for robustness.

3 The methodology has been deployed and applied to an existing industrial building, as a case study,  
4 assumed to be located in Emilia, Italy, and prone to seismic and wind hazards. The scope of this  
5 application was to test the feasibility of the procedure in estimating structural robustness and  
6 comparing the resulting robustness matrices of three different options, ranging from the gravity  
7 load designed solution to the seismic load designed solutions. The results, in terms of robustness  
8 matrix  $\mathbf{R}$ , are coherent with the starting hypothesis that states that robustness increases  
9 significantly from the gravity load designed structure to the seismic designed structures.

10 The authors envisage that the results of the proposed methodology can be worthwhile when  
11 dealing with a portfolio of existing structures, to obtain a priority ranking, aimed at identifying the  
12 less robust structures, which more urgently need to be retrofitted. The idea behind this concluding  
13 remark is that robustness can be adopted as a quantifiable indicator able to appreciate, more  
14 reliably than vulnerability, the urgency of retrofit operations on existing structures. More than  
15 vulnerability, robustness is expected to measure the capability of the structure to withstand an  
16 exceptional event, even beyond the accepted design event, based on all the contributions to its  
17 structural capacity, up to the collapse mechanism. Accordingly, the attempt made in this work is to  
18 provide a feasible framework for measuring robustness.

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