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Asprone, D., de Risi, R., & Manfredi, G. (2016). Defining structural robustness under seismic and simultaneous actions: an application to precast RC buildings. Bulletin of Earthquake Engineering, 14(2), 485-499. DOI: 10.1007/s10518-015-9820-4

Peer reviewed version

Link to published version (if available):
10.1007/s10518-015-9820-4

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The final publication is available at Springer via 10.1007/s10518-015-9820-4

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DEFINING STRUCTURAL ROBUSTNESS UNDER SEISMIC AND SIMULTANEOUS ACTIONS: AN APPLICATION TO PRECAST RC BUILDINGS

Domenico ASPRONE [1*], Raffaele DE RISI[1], Gaetano MANFREDI[1]

[1]Department of Structures for Engineering and Architecture - University of Naples Federico II, Naples, Italy

[*]Corresponding author: email: d.asprone@unina.it - tel: +39 081 7683672 - fax: +39 081 7685921

ABSTRACT

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

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

INTRODUCTION

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

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

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

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

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

Structural robustness

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

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

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

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

$$\beta_r = \frac{\beta_i}{\beta_i - \beta_d} \tag{1}$$

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

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

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

$$I_{rob} = \frac{R_{DIR}}{R_{DIR} + R_{IND}} \tag{3}$$

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

On the other side, the CRA defines some indices, aimed at measuring the robustness as an intrinsic characteristic of the structural system, not depending on the hazards the structure is prone to. The objective of these methods is to quantify the capability of the investigated structure to withstand extreme damage, whatever the event causing it. Different authors proposed various indices based on the load carrying capacity or the stiffness of the structures, e.g. the degree of indeterminacy or the stiffness matrix, to be evaluated in different damaged configurations and compared with the intact configuration (Starossek and Huberland 2008, Wisniewski et al. 2006, Smith 2006, Ciaia and Masoero 2008). For example, Starossek and Haberland (2011) proposed the measure of robustness R_S as

$$R_S = \min \frac{\det \boldsymbol{K}_j}{\det \boldsymbol{K}_0} \tag{4}$$

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

ROBUSTNESS ASSESSMENT

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

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

Thus, the accepted robustness definition, such as that provided by the *fib* Model Code 2010 (Fib 2010) previously mentioned, is herein applied to describe the behavior of a structure, that, once achieved a local failure (that structural codes would consider the conventional failure of the structure), goes beyond this state and is capable to sustain the local damage delaying its propagation to a global collapse mechanism.

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

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

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

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

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

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

Methodology

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

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

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

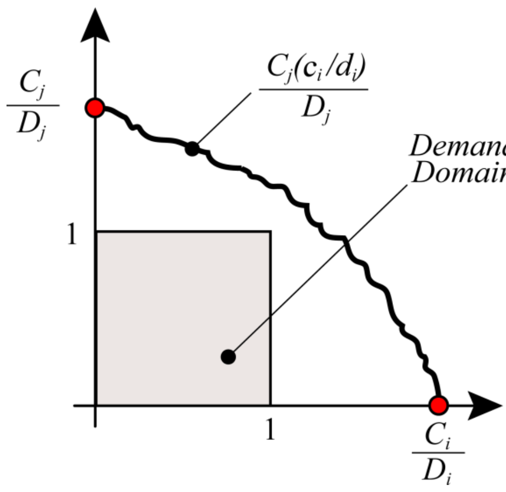

Figure 1. Robustness domain

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

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

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

These values can be arranged in a symmetric matrix, namely the *robustness matrix* **R**, of dimension n, whose generic element i,j represents:

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

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

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

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

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

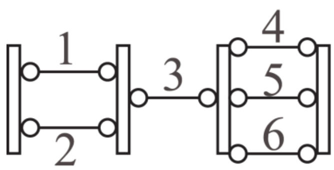

Figure 2. Schematic representation of a cutest

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

It could be argued that the available methodologies, computing robustness as the capability to avoid global collapse once local damage has occurred, try to measure only this second contribution. In these cases, robustness is interpreted as the contribution to the overstrength provided by the capability to avoid the activation of a global mechanism, once the conventional (according to structural codes) failure has taken place. On the contrary, in the methodology here proposed, also the first contribution is considered and, once both contributions are summed up, the total structural overstrength is normalized with respect to the conventional hazard intensity.

CASE STUDY

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

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

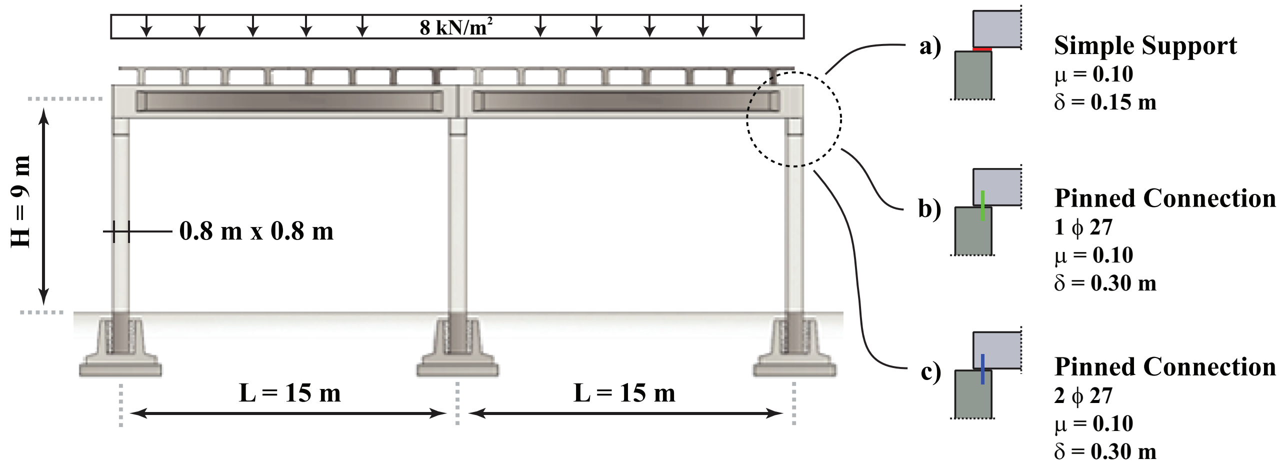

Figure 3. Case study structural model

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

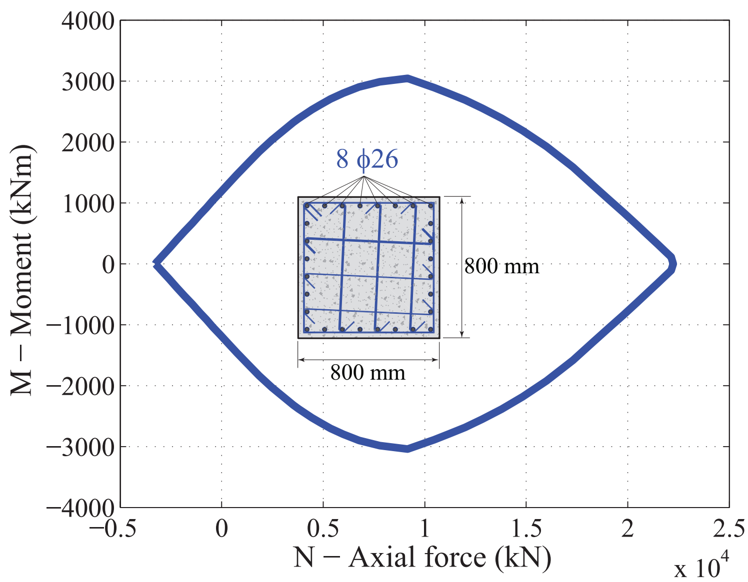

Figure 4. Column cross section and failure domain

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

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

a) simply supported connection, assuming a Teflon-concrete friction coefficient μ equal to 0.1 and a support length of 15cm;

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

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

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

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

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

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

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

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

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

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

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

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

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

The selection of the accelerograms has been carried out through the software REXEL (Iervolino et al. 2010). Horizontal-vertical ground motions have been identified to have an average spectrum matching the code acceleration spectrum for the specific site of Mirandola (Lat. 44.878, Long. 11.062). The accelerograms have been chosen from the European Strong Motion Database; they are natural signals not scaled that have an epicentral distance between 0 and 50 km, a magnitude between 4 and 7 km, and are recorded in free filed stations located on soli type C according Eurocode 8. The table 1 below reports the main characteristics of the signals, whereas figure 5 shows the code spectrum and the mean spectrum of all the used accelerograms.

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

Table 1 - Used records

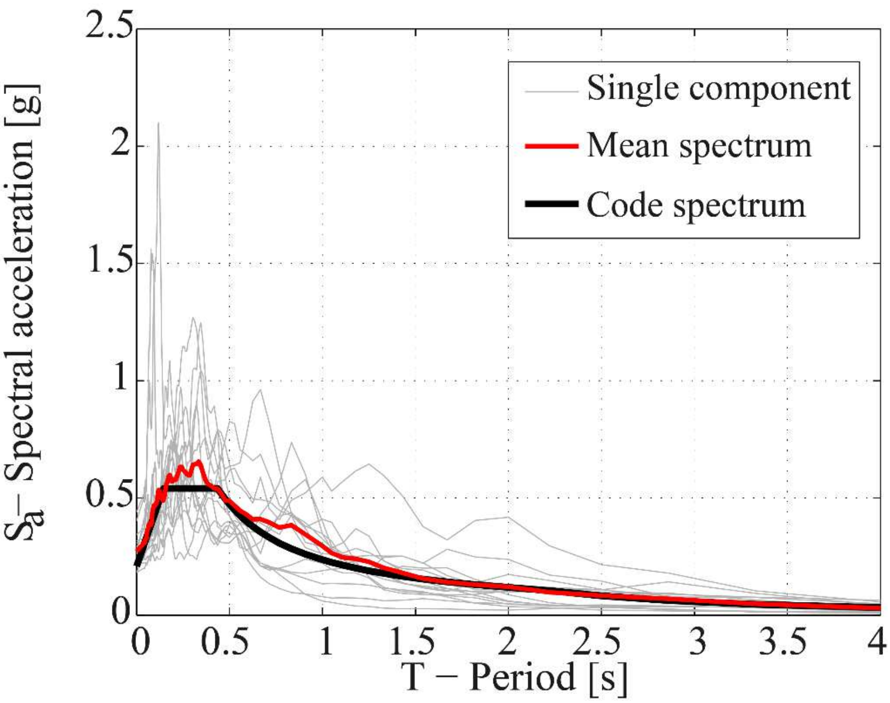

Figure 5. Code spectrum and mean spectrum of all the used accelerograms.

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

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

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

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

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

a) the connection is effective (when the friction force is larger than the inertia force) and

b) the connection is not effective (when the friction force is exceeded).

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

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

a) time dependent axial force and bending moment values acting on the column were derived and checked against the column failure domain in **Figure 4**;

b) negative axial force (i.e. acting downwards) on the beam-column connection was checked against the compression failure value of the support, computed as $0.6 f_c A$ (being A the support area, equal to the beam width times the support length). This failure criterion governed in case of very large vertical acceleration values.

For each horizontal PGA, the median value of the vertical PGA causing collapse, within the 14 possible horizontal-vertical ground motions, was computed, for each value of wind velocity.

In the investigated case study, it is clear how the novel approach to robustness applies. The local damage consists in the overcoming of the friction threshold, in case *a)*, and in the dowel failure, in cases *b)* and *c)*; it represents the conventional failure according to structural codes. Thus, the analysis is aimed at assessing the capability of the structure to sustain this local damage and delay its propagation to a global failure mechanism, that is the support loss of the beam.

Figure 6 depicts the robustness domain for the horizontal seismic-vertical seismic actions (a), horizontal seismic-wind actions (b), vertical seismic-wind actions (c), for the three investigated connection types.

The robustness domains were also elaborated in terms of robustness matrices for the three investigated cases, namely the simply supported connection *a)*, the one dowel pinned connection *b)*, and the two dowels pinned connection *c).* The results are reported in Relation 8, where the row/column matrix indices 1, 2 and 3 refer to horizontal seismic action, vertical seismic action and wind action, respectively.

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

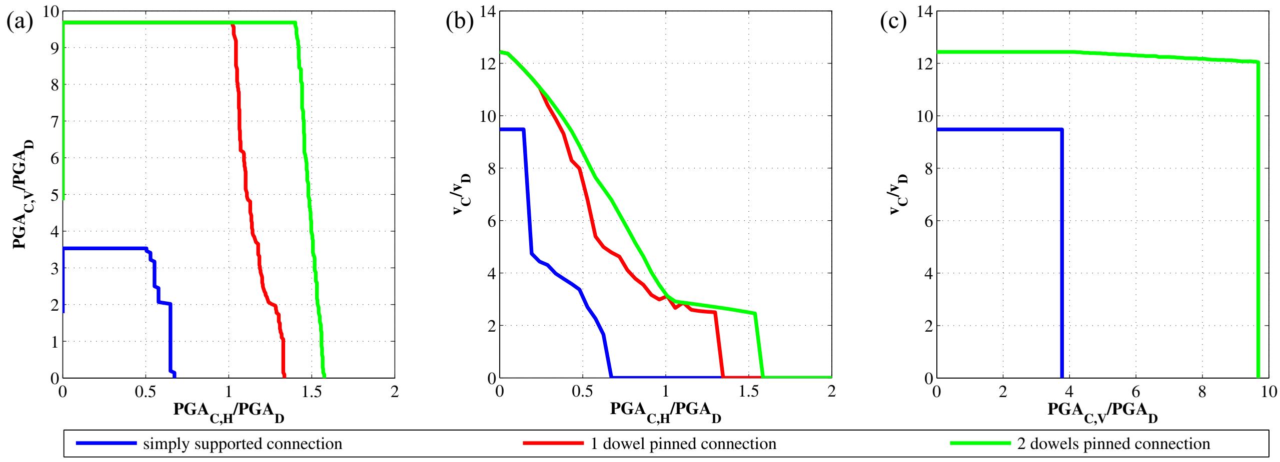

Figure 6. Robustness domains for the cases horizontal seismic-vertical seismic actions (a), horizontal seismic-wind actions (b), and vertical seismic-wind actions (the red curve and the green curve are overlapped) (c). Subscripts "C" and "D" refer to "Capacity" and "Demand", respectively. Subscripts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

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

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

POSSIBLE IMPROVEMENTS

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

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

The structural analysis for extreme events Type 3 (simultaneous events) can presents some difficulties. Depending on the couple of events considered, the definition of the structural model and the identification of the failure mechanisms could be not straightforward.

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

CONCLUSIONS

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

The methodology is easily applicable in practical cases, being strictly related to structural code design requirements. In fact, such an approach can be potentially implemented in a structural code framework, in order to practically perform robustness assessment of existing structures, but also to design new structures for robustness.

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

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

REFERENCES

Andreaus U, Casini P. Dynamics of friction oscillators excited by a moving base and/or driving force. Journal of Sound and Vibration. 2001;245:685-99.

Baker JW, Schubert M, Faber MH. On the assessment of robustness. Struct Saf. 2008;30:253-67.

Canisius TDG, Sorensen JD, Baker JW. Robustness of structural systems–a new focus for the joint committee on structural safety (JCSS). 2007.

CEN. European Committee for Standardisation, EN 1990: 2002-Basis of Structural Design. 2002.

Chiaia BM, Masoero E. Analogies between progressive collapse of structures and fracture of materials. International journal of fracture. 2008;154:177-93.

CSLP. DM 14 gennaio 2008 Norme tecniche per le costruzioni, Ministero dei Lavori Pubblici,. Suppl. or. n. 30 alla GU n. 29 del 4/2/2008 2008 (In Italian).

V. De Biagi and B. Chiaia, "Complexity and robustness of frame structures", Int. Journal of Solids and Structures, Vol. 50, pp. 3723-3741, 2013.

Ditlevsen O, Masden H. Structural Reliability Methods.: Wiley, New York, 1996; 1996.

Fédération Internationale du Béton. Model Code 2010-final draft: Fédération Internationale du Béton fib/International Federation for Structural Concrete; 2012.

Frangopol DM, Curley JP. Effects of damage and redundancy on structural reliability. Journal of Structural Engineering. 1987;113:1533-49.

Iervolino, I., Galasso, C., & Cosenza, E. (2010). REXEL: computer aided record selection for code-based seismic structural analysis. *Bulletin of Earthquake Engineering*, *8*(2), 339-362.

Iervolino I, De Luca F, Chioccarelli E. Engineering seismic demand in the 2012 Emilia sequence: preliminary analysis and model compatibility assessment. Annals of Geophysics. 2012;55.

Jalayer F, Elefante L, Iervolino I, Manfredi G. Knowledge-based performance assessment of existing RC buildings. Journal of Earthquake Engineering. 2011;15:362-89.

Lind NC. A measure of vulnerability and damage tolerance. Reliability Engineering & System Safety. 1995;48:1-6.

Liberatore, L., Sorrentino, L., Liberatore, D., & Decanini, L. D. (2013). Failure of industrial structures induced by the Emilia (Italy) 2012 earthquakes. Engineering Failure Analysis, 34, 629-647.

Meletti C, D'Amico V, Ameri G, et al. (2012) Seismic hazard in the Po Plain and the 2012 Emilia earthquakes. Ann Geophys. doi: 10.4401/ag-6158

Ribeiro, Filipe LA, André R. Barbosa, and Luís C. Neves. "Application of Reliability-Based Robustness Assessment of Steel Moment Resisting Frame Structures under Post-Mainshock Cascading Events." Journal of Structural Engineering 140.8 (2014).

Savoia, M., Mazzotti, C., Buratti, N., Ferracuti, B., Bovo, M., Ligabue, V., & Vincenzi, L. (2012). Damages and collapses in industrial precast buildings after the Emilia earthquake. Ingegneria Sismica, 29, 120-131.

Scognamiglio L, Margheriti L, Mele FM, et al. (2012) The 2012 Pianura Padana Emiliana seimic sequence: locations, moment tensorsand magnitudes. Ann Geophys. doi: 10.4401/ag-6159

Smith JW. Structural robustness analysis and the fast fracture analogy. Structural engineering international. 2006;16:118-23.

Starossek U, Haberland M. Measures of structural robustness–requirements and applications. ASCE SEI Structures Congress 2008. Crossing Borders, Vancouver, Canada2008.

Starossek U, Haberland M. Disproportionate collapse: terminology and procedures. Journal of Performance of Constructed Facilities. 2010;24:519-28.

Starossek U, Haberland M. Approaches to measures of structural robustness. Structure and Infrastructure Engineering. 2011;7:625-31.

Vintzēleou EN, Tassios TP. Mathematical models for dowel action under monotonic and cyclic conditions. Magazine of concrete research. 1986;38:13-22.

Wisniewski D, Casas JR, Ghosn M. Load capacity evaluation of existing railway bridges based on robustness quantification. Structural engineering international. 2006;16:161-6.