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Review of Resilience-Based Design

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Abstract. The reliability of structures is affected by various impacts that generally have a negative effect, from extreme weather conditions, due to climate change to natural or man-made hazards. In recent years, extreme loading has had an enormous impact on the resilience of structures as one of the most important characteristics of the sound design of structures, besides the structural integrity and robustness. Resilience can be defined as the ability of the structure to absorb or avoid damage without suffering complete failure, and it can be chosen as the main objective of design, maintenance and restoration for structures and infrastructure. The latter needs further clarification (which is done in this paper), to achieve the clarity of goals compared to robustness which is defined in Eurocode EN 1991-1-7 as: “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”. Many existing structures are more vulnerable to the natural or man-made hazards due to their material deterioration, and a further decrease of its load-bearing capacity, modifying the structural performance and functionality and, subsequently, the system resilience. Due to currently frequent extreme events, the design philosophy is shifting from Performance-Based Design to Resilience-Based Design and from unit to system (community) resilience. The paper provides an overview of such design evolution with indicative needs for Resilience-Based Design giving few conducted examples.

Keywords: resilience; extreme loads; Resilient-Based Design; earthquakes; Performance-Based Design

1. Introduction

In recent years the effects of climate change and man-made and natural disasters (Ibrahimbegovic and Ademović 2019) in several scenarios have led to major damage and in some cases to collapse of structures. The effects are more visible at present in the existing buildings and infrastructure facilities, especially in bridges, due to the increased freight transportation (Ademović *et al*. 2019). The norm ISO 2394 (ISO 2394) is the basis for most national design standards in Europe and worldwide, which defines the basic requirements for the design, construction, and maintenance of structures during the working life. The question which arises is how to apply the norm to existing structures in the presence of unknowns: the lack of design documentation, unknown properties of built-in material, degradation of a material due to different exposures, etc.. Of special concern in this paper are existing structures, either residential or industrial, and infrastructure. This being connected

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to the change of the initial use of structures to the increase of loading, extreme loading conditions, the influence of climate change, pollution and natural and anthropogenic effects. In most cases, it is the combination of all the above-mentioned influences. A catastrophic event that woke up and shook the engineering profession regarding the safety of structure was the collapse of the Moradi bridge in Genova in August of 2018 killing 43 people and leaving 600 homeless. This event as well had economic and social consequences which were manifested by the closure of the neighboring streets for the safety of the people and the fear of new collapses (Rania *et al*. 2019). At the time of its opening in 1967, it was one of the longest concrete bridges in the world and a novel structure having new specific features like four cables per tower covered in pre-stressed concrete. The deck was completely made of reinforced concrete, with minimum use of steel, as Italy at that time was under international sanctions and had no facilities for internal production of steel. The structure was lighter and stronger, and at the time believed that: "The bridge's concrete structure won't need any maintenance", as stated in La Stampa newspaper before its opening. "Neither will its stayed cables, which are protected from atmospheric agents by their concrete vest". At the time of construction, this was an innovation and the issue of durability and life assessment of structure was not a major focus. The engineering community is divided regarding what has caused the failure of the bridge. On one side are engineers who believe lack of maintenance was the major issue while others believe there were fundamental design or construction flaws. As stated by Paul Jackson: "The bridge doesn't have a lot of redundancies, so if one cable goes it could be enough to take the whole bridge down" (Newscientist 2019). Redundancy is one of the four R components (robustness, redundancy, resourcefulness, and rapidity) of resilience (Bruneau *et al.* 2003). Fifteen months later another bridge in the same region of Italy collapsed, due to intensive rain that triggered an enormous landslide from a nearby hillside. Once again it was proven that no structure is "absolutely" safe, making monitoring and maintenance of structures inevitable and crucial. The resilience cannot be looked at a structure level but on a broader scenario, on the city (community).

On the other hand, natural disasters usually cause damage to wider society and not only to one or several structures. Drastic effects on communities were noted in recent history due to earthquake actions, some examples are L'Aquila earthquake in 2009 with the total destruction of the city of Onna - basically wiped off from the map, or the 2011 Christchurch earthquake in New Zealand, which revealed the existence of a hidden fault within about six kilometers of the city: along the southern edge of the city. This near-field earthquake caused significant vertical ground motions that were not taken into account in the calculations. A set of earthquakes that hit Albania in November 2019 were the strongest to hit Albania in more than 40 years, and the one on 26th November was the deadliest in 99 years and the world's deadliest earthquake in 2019. This region was shaking for several days and by December 1st, 1.300 aftershocks were registered. The magnitude of the aftershocks was rather high, twenty-six were between M4 and 5, while four were larger than M5, until December 15th. Due to the massive destruction of hotels and residential buildings, in total more than 14.000 buildings were damaged, prosecutors issued 17 arrest warrants for builders, engineers, and officials suspected of breaking safety standards. This kind of damage has a wider impact on the entire society from the economy, cultural to social and may reduce to a high degree the functionality of the entire community. The lack of resilience can lead to complete loss of performance which can be complete destress. In this sense, resilience, as the capacity to recover quickly from difficulties is extremely important.

This all has affected the change in the design philosophy of structures from Force-Based Design towards Performance-Based Design and finally moving to Resilient-Based Design, which is reviewed in this paper. The outline of the paper is as follows. In Section 2, we briefly recall different design philosophies. In Section 3, we discuss different current design practices. In Section 4, we summarize the basic requirements of the Resilience-Based Design. The conclusions are drawn in Section 5.

2. Different design philosophies

Many existing bridges and structures in Europe located in seismic areas have been designed based on the old codes which are based on the prescriptive approach. This is the case of the reliability methods that are used for the assessment of structures, whereas the capacity and demand are represented as resistance and actions of the structures. The codes usually define the actions through a response spectrum. In this way, the probability of earthquake occurrence is not explicitly taken into account but implicitly through the response spectrum combined with a behavior factor, which reduces the actions on the structure taking into account the nonlinear material behavior and energy dissipation. On the other hand, performance-based assessment methods, either pushover analysis or nonlinear time history analysis, take into account the frequent occurrence of seismic hazards. The pushover can be carried out to estimate the target displacement until the structure reaches collapse, or for estimating the seismic demand until the target displacement. Two methods can be used for the determination of the target displacement: the Coefficient Method (FEMA 356) or the Capacity Spectrum Method (CSM). In the Coefficient Method, the target displacement represents the maximum displacement which occurs at the top of structures during a chosen earthquake; while the CSM, the pushover curve (Freeman 1978), is plotted in acceleration–displacement response spectrum (ADRS) format defined as the capacity spectrum. FEMA 440 (2005) gives clear instructions for the CSM and displacement coefficient method (DCM). Chopra and Goel (2002) developed an improved procedure to calculate the target displacement. In the classical pushover analysis, the forcing function is kept constant however, several modifications and upgrading have been done in the adaptive pushover analysis developed by various researchers (Bracci *et al*. 1997, Papanikolaou *et al*. 2005, Rofooei *et al.* 2007, Amini and Poursha 2018). The time history method is the most time consuming and accurate method for seismic analysis. The codes prescribe the type of earthquake motion for the nonlinear calculation that will be selected for the analysis. For example, the Eurocode 8 (EC8) (CEN, 2003) allows the application of three kinds of accelerograms as the main input for the structural analysis based on the time history method. The average spectral ordinates of the selected recording set have to be matched with the target code-based spectral shape. The set has to contain at least seven recordings (each of which includes both horizontal components of a recorded motion if the spatial analysis is concerned) to consider the mean of the response. For unbiased determination of the seismic demand, the real accelerograms are becoming very popular. To be able to predict the possible damages and structure response improved knowledge regarding earthquake occurrence and ground motion is of the utmost importance on one side and on the other the structural response characteristics.

3. Current practices of structural design

3.1 Force-Based Design and Performance-Based Design

Looking at the oldest codes it is seen that the building mass (weight) was taken as an input value for determining the forces that are to be taken in the seismic design. Further, the base shear force resulting from the earthquake dynamic motion were calculated using the acceleration response spectrum and the expected elastic period of the building, forming the basis of the Force-Based Design (FBD). The aftermath of the devastating earthquakes in the 1960s showed that some ductile structures resisted higher acceleration than the ones required for the yielding of the material. This led to the development of the equivalent lateral force method which is used in most of the current codes, where the seismic forces are a function of the seismic weight and the seismic response coefficient which depends on several factors (seismic zone, the structure period, the inelastic performance of the lateral revisiting system, etc.). The forces were presented as a fraction of the weight, usually 5%, 10%, and 20%. In this process, a multi-degree of freedom structure is equated to a structure with one degree of freedom. In this case, the period of the structure represents the fundamental characteristic of the structure. In Eurocode 8 (CEN 2003), for structures up to 40 meters the value of the fundamental period may be estimated as

$$T_1 = C_t \cdot H^{3/4} \tag{1}$$

where the value C_t depends on the structural typology and material and is in the range from 0.050 to 0.085, and H is the height of the structure in meters. Alternatively, it can be determined by taking into account the lateral elastic displacement of the top of the building denoted with d, and expressed in meters, due to the gravity loads applied in the horizontal direction. The expression reads

$$T_1 = 2 \cdot \sqrt{d} \tag{2}$$

This represents a static linear elastic procedure, where the static forces are applied to the structure with the intensities and direction that in a good manner estimates the earthquake activities. The later forces are placed at each floor of the structure due to the mass concentration and assumption of the floor being rigid in their plane, vertical components possess stiffness only in their plane, axial deformation of the elements are neglected, movement of the supports is equal and the only possible movement is horizontal. This is generally applicable for symmetric structure and structures having the fundamental period $T_1 \leq 2$ s.

Further for structures that do not satisfy these conditions, modal response spectrum analysis is applied. It is a linear-dynamic analysis which takes into account the contributions from each natural mode of vibrations and per Eurocode 8 (CEN 2003) the sum of the effective modal masses for the modes that are taken into account have to be at least 90% of the total mass of the structure, and each mode which has an effective modal mass greater than 5% of the total mass has to be taken into account. Then, the base shear force of the corresponding mode k is determined as

$$F_{\mathrm{bk}} = S_d \cdot \left(T_k\right) \cdot m_k \tag{3}$$

where m is the effective modal mass, T is the period and S_d is the design spectral acceleration value for the corresponding mode k. In this way information regarding the dynamic behavior is obtained in relation to the structural period and level of damping. A combination of modes can be done either by the Square Root of Summation of Square (SRSS) (e.g., Clough and Penzien 2016) method if the modes are not close to each other so there is no interaction between the modes or Complete Quadratic Combination (CQC) (Wilson *et al.* 1981) where the interaction of the modes is taken into account.

The shift from linear to nonlinear analysis was conducted allowing an acceptable amount of damage and nonlinear behavior of the material and energy dissipation. This was covered by the behavior factor (q) in Eurocode 8. This factor is equal for the whole group of structures and gives a rough estimation of its real behavior (Nikolić *et al*. 2017). Going into the nonlinear range methods that can be used are non-linear static (pushover) analysis and time history analysis (Ademović 2011, Ademović 2012). Pushover analysis is a non-linear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads (Ademović *et al.* 2013, Salihovic and Ademović 2017). The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity. As a result, a capacity curve is plotted showing the dependency of the base shear force and displacement. By this analysis weakness of the structure can be identified. From this analysis, some important features can be assessed like inter-story and global drift, inelastic element deformations, connection forces between elements and deformations between elements. The nature of this method is approximative and is based on the static loading, meaning that it cannot take into account the real dynamic phenomena of the earthquake ground motion. In the case of severe earthquakes, this method is not applicable as it cannot detect some very important modes as well when the influence of the higher modes becomes significant.

Evidently, Force-Based Design (FBD) is faced with several drawbacks, like the ambiguous determination of structural elements' stiffness, the difficulties of determining the reduction factor (*R*) adequately, and the absence of a physical basis for such an analysis, as the earthquake generates displacement and impart energy and it does not produce forces. So, it may be said, that lateral displacement with the gravity load brings down the structure and not the lateral forces. Some of these problems were solved with the development of Performance-Based Design (PBD) (Pettinga and Priestley 2005). Performance-Based Design is a process that enables the development of structures that will have predictable performance when subjected to defined loading (ASCE 2018). In this concept, displacements are taken into account at the beginning of the design process, as they can be connected directly with the damage, and in that respect adequate limit states are determined. The performance levels such as Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP) are used to quantify the performance objective (FEMA 356 2000). Different levels of limit states are connected to different levels of earthquake activity and different types of structures. The earthquakes are defined into groups: frequent earthquake (serviceability seismic action), design-basis earthquake, and maximum considered earthquake. Immediate occupancy is connected to the frequent earthquake (serviceability seismic action). On the other hand, in the case of design-basis earthquake (DBE), an earthquake for which the safety systems are designed to remain functional both during and after the event, thus assuring the ability to shut down and maintain a safe configuration a Life Safety (LS) state will be appropriate. In the case that the structure needs to remain functional after the earthquake (hospitals, schools, hotels, etc.), the performance level has to be increased leading to the Immediate Occupancy for the design-basis earthquake, and life safety for maximum considered earthquake (MCE). In this method, a target displacement is assumed matching to the required performance level. Eurocode 8 (CEN 2003), FEMA 356 (2000) and ATC 40 (1996) give limitations for inter-story drift in relation to different performance levels for better estimation of the target displacements. Various displacement-based design methods have been developed in recent years (Sullivan *et al*. 2003). Freeman (1978, 1998) developed the Capacity Spectrum method (CASPECP). Chopra and Goel (1999) developed a Direct displacement-based design which was updated in 2001 (Chopra and Goel 2001) where an inelastic displacement spectrum for various ductility levels is defined. A year later an Initial Stiffness Deformation Control method was developed by Panagiotakos and Fardis (1999) for particular structure types and with specific limit states that have to be checked during the analysis based on the response spectra by utilizing the initial stiffness, while "Method A" from SEAOC recommended lateral force (1999) is direct deformation-specification based and uses either initial or secant stiffness. Aschheim and Black

(2000) developed a Yield Point Spectrum (YPS) method for various ductility levels needed for the determination of the design base shear. Priestley and Kowalsky (2000) formulated a new Direct Displacement Based Design (DDBD) method based on the secant stiffness being direct deformation-specification based (DDSB), which was further upgraded by Priestley *et al*. (2007). Browning (2001) proposed an Initial Stiffness Iterative Proportioning (ISIP) method based on the iterative deformation specifications (IDSB), while Kappos and Manafpour (2001) developed an Advanced Analytical Techniques (T-HIST) procedure which is based on the time-history analysis. The application of this method in the beginning phase is for the “serviceability” type earthquake. This initial stage is used for determination of the initial strength and then an inelastic model is formulated that will be further used in the inelastic time-history analysis to design for other limit states. The utilization of the energy-based methods was proposed by Goel *et al*. (2010) called Performance-Based Plastic Design (PBPD). It takes into account the target drift as the design constraint which enables it to be a part of the DBD method even thought for determination of the base shear design it uses the work-energy principle. All of these methods have their pros and cons and limitations that have to be taken into account during investigations.

Moving towards the probabilistic seismic performance assessment of structures (Cornell and Krawinkler 2000) represents a step forward to a more realistic scenario conducted by the Pacific Earthquake Engineering Research (PEER) center (Krawinkler and Miranda 2004). In this respect an Incremental Dynamic Analysis (IDA) has been developed, which takes into account a set of accelerograms, each scaled to several intensity levels, pushing the structure through various stages from the linear elastic stage to the global dynamic instability (Vamvatsikos and Cornell 2004). So, the Performance-Based Design is based on seismic hazard denoted by the Peak Ground Acceleration (PGA), damage measure and capacity measure. Determination of quantities is the specific aim of this assessment analysis, which is represented by the mean annual frequency of collapse λ. The anticipated strategy contains the expansion of the mean annual frequency in terms of the structural Damage Measures (DM) and ground motion Intensity Measures (IM) which can be presented as

$$\lambda(\mathrm{DV}) = \iint G(\mathrm{DV}|\mathrm{DM})dG(\mathrm{DM}|\mathrm{IM})d\lambda(\mathrm{IM}) \tag{4}$$

Where, $G(\mathrm{DV}|\mathrm{DM})$ is the probability that the Decision variable (DV) exceeds specified values assumed that the engineering damage measures (e.g., the maximum storey-drift index) are equal to particular values (fragility curves). $G(\mathrm{DM}|\mathrm{IM})$ is the probability that the Damage measure (DM) exceeds these values given that the PGA defined as the intensity measure equals particular values. $\lambda(\mathrm{IM})$ is the mean annual frequency of the intensity measure.

According to PBEE, damage caused by a certain seismic event is acceptable if this demonstrates to be the most acceptable solution in the economic sense. Evidently, these kinds of designs are more scientifically oriented and require more precise characterization and predictions.

Eq. (4) has been updated by (Krawinkler and Miranda 2004) and can be written as

$$\underbrace{\lambda\ (\mathrm{DV})}_{\textit{Seismic risk}} = \iiint G\underbrace{(\mathrm{DV}|\mathrm{DM})}_{\textit{Loss Analysis}}\underbrace{dG(\mathrm{DM}|\mathrm{EDP})}_{\textit{Damage Analysis}}\underbrace{dG(\mathrm{EDP}|\mathrm{IM})}_{\textit{Response Analysis}}\left|\underbrace{d\lambda(\mathrm{IM})}_{\textit{PSHA}}\right| \tag{5}$$

where DV is the Decision variable, DM is the Damage measure, EDP is the Demand parameter, IM is the Intensity measure, $G(a|b)$ represents the probability of exceedance where $a>a_0$ given b, and $\lambda(\mathrm{DV})$ is the mean annual frequency of a Decision variable (DV). Different components are presented in Table 1 (Krawinkler and Miranda 2004).

Table 1 Performance assessment approach (Krawinkler and Miranda 2004)

Performance Targets	Decision Variables (DV)	Damage Measures (DM)	Demands (EDP)	Seismic Hazard (IM)
Collapse and Life safety P_f<*y*	Collapse Number of causalities	Fragilities for failure states	Engineering analysis (story drift, floor acceleration, etc.)	Hazard analysis Ground motion
Losses<*x*	Dollar Losses	Structural		
Downtime<*z*	Length of downtime	Nonstructural Content	Soil-Foundation Structure system	
λ(DV)	G(DV/DM)	G(DM/EDP)	G(EDP/IM)	λ(IM)

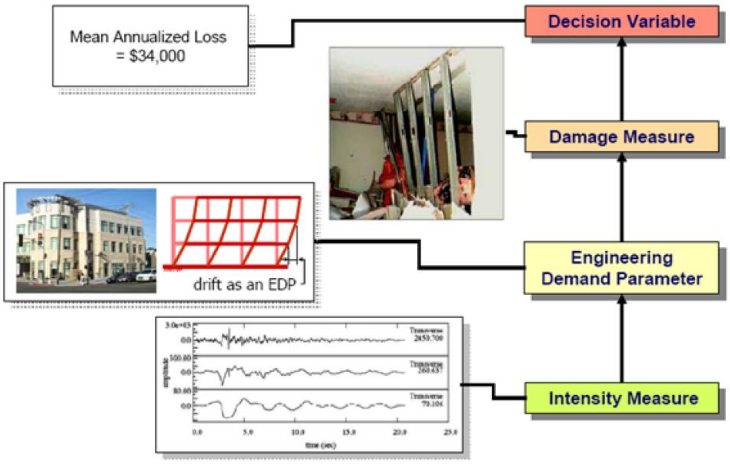


Fig. 1 PEER center methodology for Performance-Based Design (PBD)

The basis of PBEE is the achievement of desired performance targets. Achievement of desired performance targets can be seen on two levels, either it can be a concept of individuals or a specific group on one side, or the entire society. Eq. (5) can have various forms in the function of the purpose and the decision variable of interest. Generally, the seismic risk assessment problem is decomposed into four basic elements: 1) hazard analysis; 2) modeling of damage state; 3) demand parameters and 4) loss predictions, by introducing the three intermediate variables IM, EDP, and DM. Once this is done, these elements are again recoupled by the integration taking into account all levels of the intermediate variables. This means that the conditional probabilities (EDP|IM), *G*(EDM|DP) and *G*(DV|DM) have to be assessed employing parameters over a suitable range of DM, EDP and IM levels.

The idea of Performance Based Earthquake Engineering (PBEE) is illustrated in Fig. 1 and this method is implemented already in the USA through the ATC-63 (2007). The safety of individual structures is a necessary but not sufficient requirement for the functionality of a particular structure and the community as a whole. The resilience of structures is extremely important. A key component of a resilient building/structure is a robust structural system, which limits the progression of failure under extreme natural and man-made hazards. In that respect, Resilience-Based Design is inevitable.

4. Resilience based design

In the Resilience-Based Design (RBD), the structure is considered in a wider scenario, as a component of a coupled multi structural system, taking into account not only the interaction with the community but also a potential multi-risk scenario, to be able to evaluate the regional loss analysis. This means that the structure has to be regarded in the interaction with other structures and the community as a whole. In this way, the structure is not considered as a unit but a group or blocks of buildings. The concept was taken from finances where Modern portfolio theory was developed by Harry Markowitz in 1952. Modern portfolio theory (MPT) is a theory on how risk-averse investors can construct portfolios to optimize or maximize probable return based on a given level of market risk, highlighting that risk is an inherent part of higher reward. Markowitz's theories emphasized the importance of portfolios, risk, the correlations between securities, and diversification. As stated by Cimellaro (2013), "MPT is a mathematical formulation of the concept of diversification in investing, with the aim of selecting a collection of investment assets that has collectively lower risk than any individual assets". The concept of diversification can be applied in disaster resilience, in a way that diversification in retrofitting and strengthening concepts of various buildings in a specific area can increase resilience collectively than any individual strengthening or rehabilitation. The concept from the financial field is translated to the engineering concept in a way that the portfolio asset's return in MPT is regarded as the weight aggregated losses over the building

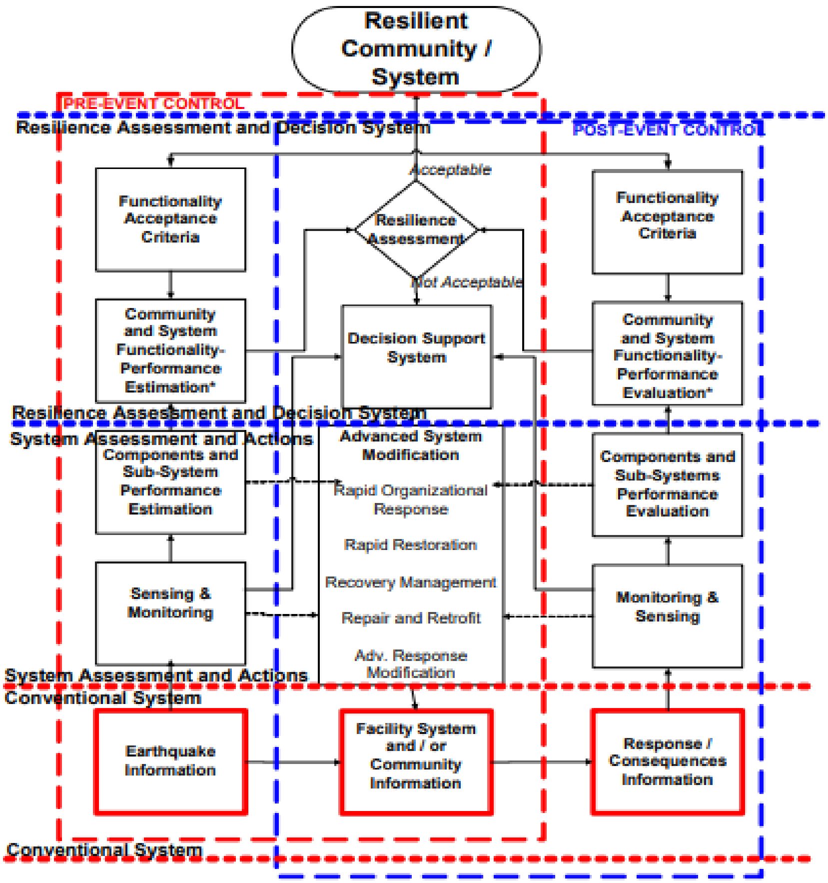


Fig. 2 System (community) resilience (Bruneau *et al.* 2003)

portfolio, meaning that the variation of the aggregated loss of the building portfolio is seen as the risk of losses at the community level. In this case, the building portfolio can be defined as a weight combination of the performance index of each housing unit (Cimellaro 2013). Attributes defining resilience are robustness, redundancy, resourcefulness, and rapidity (Bruneau *et al.* 2003), meaning that, as robustness is an attribute of resilience, it can have a direct implication on the reduction of structure's performance. The process is clearly defined in Fig. 2, where the resilience of the system is addressed through two phases before and after the event, with its implications, in this specific case due to prior and post of earthquake activity. Fig. 2 presents the stepwise development of the RBD from the PBD. The PBD is defined by the second and third row, while the enhancement to the RBD is obtained by the addition of the first row which takes into account the resistance of the entire system (community) which may be exposed to some kind of natural hazard (e.g., earthquake). Once resilience parameters are identified and evaluated as depicted in the first row, this information is used as an input for decision making regarding the selection of adequate remedial actions. In contrast with the PBD, RBD takes into account several dimensions like social, economic, functionality parameters as well as different features of the population (age, gender, etc.). This is required for the functionality of a certain community. Some researchers have identified a set of resilience parameters that have to be considered for a community which has been placed in a model shortly called PEOPLES, which accounts for Population and Demographics, Environmental/Ecosystem, Organized Governmental Services, Physical Infrastructure, Lifestyle and Community Competences and Social-Cultural Capital (Renschler *et al*. 2010).

The key parameter for measurement of resilience is the functionality of the whole system (Henry and Ramirez-Marquez 2012). The basis for the formulation of the resilience factor and uncertainty-weighted resilience metric are three resilience capacities: adaptive capacity, absorptive capacity, and recoverability. These three capacities (absorptive, restorative, and adaptive capacity) also form the basis of the proposed resilience factor and uncertainty-weighted resilience metric given by Royce and Bekera (2014). Vugrin *et al.* (2011) defined the absorptive capacity as the degree to which a system can absorb the impacts of system perturbations and minimize consequences with little effort, defined as F_r/F_0. The restorative capacity of a resilient system can be characterized by rapidity to return to normal or improved operations and system reliability, defined by S_p. Whereas, the adaptive capacity is the ability of a system to adjust to undesirable situations by undergoing some changes, defined by F_d/F_0. Royce and Bekera (2014) propose a new Resilience factor defined as

$$\rho_i = \mathrm{S_p} \frac{F_r}{F_0} \cdot \frac{F_d}{F_0} \tag{6}$$

where S_p defined in Eq. (7), is the speed recover factor which takes into account the time needed for the system to recover, F_0 is the performance level of the original stable system, F_d performance level immediately after disruption, F_r is the performance level after recovery. Time to the final recovery is denoted as t_r, t_r^* is the time required to finalize the initial recovery activities and t_δ defines the slack time, and a is a numerical parameter that controls the decay in resilience attributable to the time of the new equilibrium (Fig. 3).

$$\mathrm{S_p} = \begin{cases} \left(\dfrac{t_\delta}{t_r^*}\right) \exp\left[-a\left(t_r - t_r^*\right)\right], \text{for } t_r \geq t_r^*, \\ \left(\dfrac{t_\delta}{t_r^*}\right), \qquad\qquad \text{otherwise.} \end{cases} \tag{7}$$

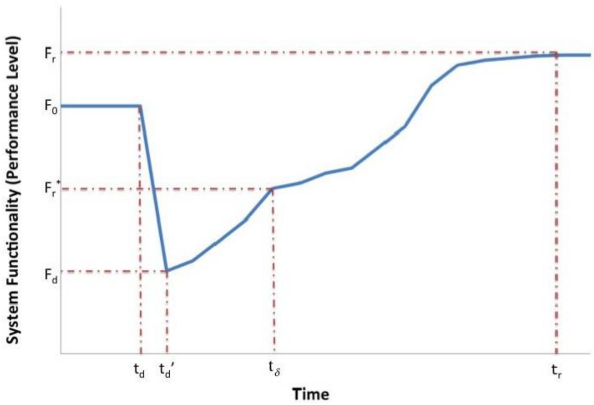


Fig. 3 The relationship between time and system functionality

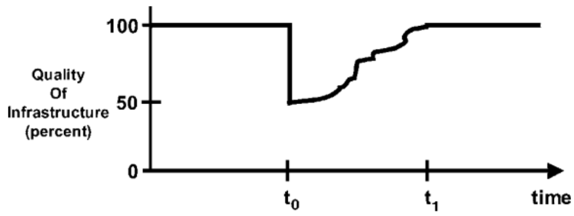


Fig. 4 Schematic representation of seismic resilience concept (Bruneau *et al*. 2003)

Resilience, R, is defined graphically as the normalized shaded area underneath the function describing the functionality of a system, defined as $Q(t)$. A quality function $Q(t)$ was introduced by Reed *et al*. (2009) while evaluating the resilience of networked infrastructure. The value of $Q(t)$ has boundary values, 0 when the system failed and 1 if the system is fully operational. If an earthquake occurs at the time t_0, the structure or infrastructure may experience major damage causing the reduction of the quality measure by 50 % (Fig. 4). A certain time is needed for the infrastructure to be repaired and get to its full recovery (t_1) and to reach the initial state.

Hence community earthquake loss of resilience, R, concerning the specific earthquake, can be measured by the size of the expected degradation in quality (probability of failure), over time (that is, time to recovery) (Bruneau and Reinhorn 2007), as a sudden drop characteristic for earthquakes.

Mathematically, it represents an area of the upper surface in the form of a triangle between t_0 and t_1 and can be calculated as

$$R = \int_{t_0}^{t_1} \left[100 - Q(t)\right] dt \tag{8}$$

The time is not fixed here indicating the increase of resilience as the time passes (longer period of time=higher resilience). $Q(t)$ is a nonstationary stochastic process, and each ensemble is a piecewise continuous function (Fig. 4), where $Q(t)$ is the functionality of the region considered. The community functionality is an aggregation of all functionalities related to different facilities, lifelines, etc. Renschler *et al.* (2010). For communities, the drop can be gradual, as represented in Fig. 5. The proposed solution was given by Renschler *et al.* (2010) where the area under the functional term was normalized, defining resilience as

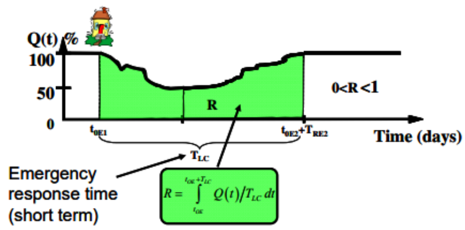


Fig. 5 Functionality curve and resilience (Renschler *et al.* 2010)

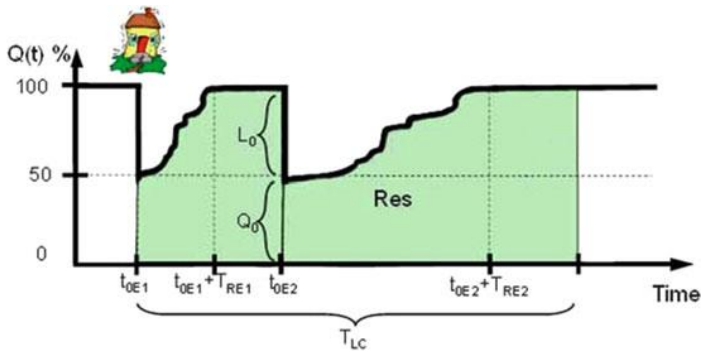


Fig. 6 Functionality curve and resilience (Cimellaro *et al.* 2015)

$$R = \int_{t_0}^{t_n + T_{LC}} \frac{Q(t)}{T_{LC}} dt \tag{9}$$

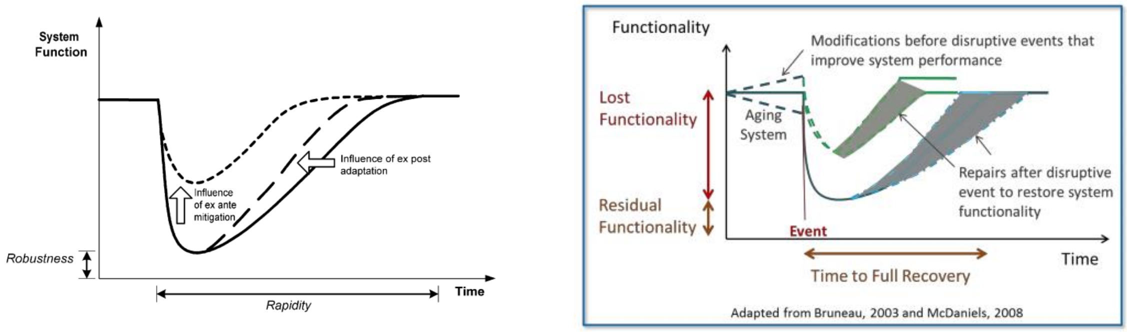
where T_{LC} defines the control time which is usually defined by the building owners or society at large. The Eq. (9) is schematically presented in Fig. 5.

Cimellaro *et al.* (2015) updated the Eq. (9), as given in Eq. (10) and the interpretation is given in Fig. 6

$$R\left(\vec{r}\right) = \int_{t_{OE}}^{t_{OE} + T_{LC}} \frac{Q_{TOT}(t)}{T_{LC}} dt \tag{10}$$

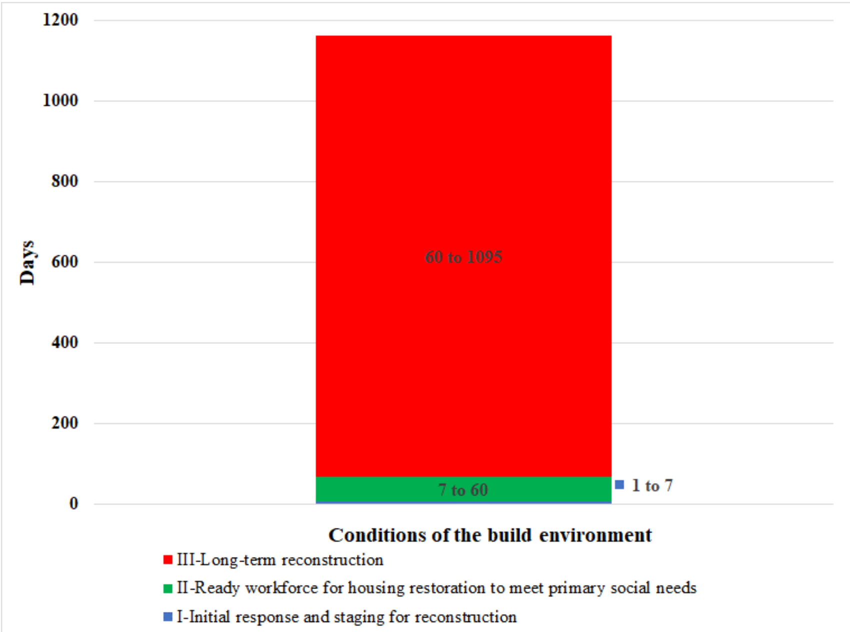
where L_0 is the loss, or the drop of functionality, right after the extreme event (disruption severity), Q_0 is the robustness, Q_{TOT} (t) is the global functionality of the region under consideration, $\vec{r}$ is a position vector that defines the location in the selected region where the resilience index is evaluated Cimellaro *et al.* (2015).

Mattasson and Jenelius (2015) gave a vulnerability and resilience analysis of a transportation network. Enclosing of resilience and vulnerability was done by Reggiani *et al.* (2015). All these ideas can be well presented by the concept formulate by McDaniels *et al.* (2008) regarding the effects of decision-making on infrastructure resilience presented in Fig. 7. Fig. 7(b) indicates the presence of two areas, one before the hazard event and one after. In the case that modifications were done before the event, this will have a beneficial effect on the residual functionality and the time to full recovery will be shorter. If no modifications were done before the event this would lead to longer time recovery.



(a) (b)

Fig. 7 Effects of decision-making on resilience (McDaniels *et al.* 2008) (McAllister 2013)



Category	Performance standard
A	Safe and operational: Essential facilities such as hospitals and emergency operations centers
B	Safe and usable during repair: Shelter-in-place residential buildings needed for emergency operations
C	Safe and usable after repair: Current minimum design standard for new non-essential buildings
D	Safe but not repairable: Below current standards for new buildings, often used for voluntary retroit
E	Partially unsafe or completely collapsed: Damage that will lead to casualties in the event of the expected disaster - the killer buildings

Fig. 8 Performance goals and transparent performance measures for buildings

In this concept performance, goals and performance measures of structures and infrastructure were defined (Khaloo and Mobini 2016) depending on their importance and application. The performance goals for the expected disaster are grouped into three phases. Phase I for the time frame of 1 to 7 days, which represents the initial response and staging for reconstruction; followed by Phase II from 7 up to 2 months and the final Phase III which represents the long-term reconstruction and can last up to 3 years. On the other hand, the transparent performance measures for buildings

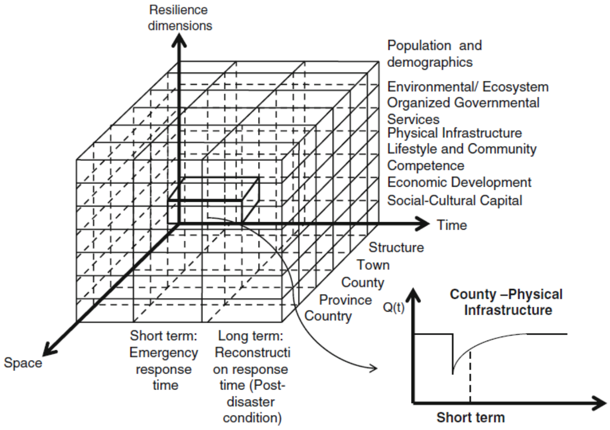


Fig. 9 PEOPLES approach (Cimellaro *et al.* 2015)

are set in five categories from A (safe and operational) to F (partially unsafe or completely collapsed).

Resilience matrices are becoming one of the most important indicators in earthquake engineering. Cimellaro *et al.* (2006) formulated a procedure which defines resilience as a function of losses and loss recovery based on multidimensional system fragility. In their work, they presented a quantitative definition of resilience through the use of an analytical function applicable to hospitals that were suitable for technical and organizational aspects. Further investigation in this domain was done by (Bruneau and Reinhorn 2007, Cimellaro *et al.* 2010).

In this context, the PEOPLES approach can help define the spatial and temporal dimension of Resilience-Based Design, as it is shown in Fig. 9.

This framework connected identified resilience characteristics (technical, organizational, societal, and economic-TOSE) defined by Bruneau *et al.* (2003) and four resilience attributes. The physical systems are covered by technical and economical characteristics, while the community which is affected by the physical systems is concerned by the organizational and social characteristics. The ability of the system to be functional is covered by technical resilience. The ability of the organizers to manage the system is defining organizational resilience. How will society handle the loss of services due to certain causes is defined by social resilience. While the ability to reduce economic losses (direct and indirect) is covered by economic resilience.

When referring to a disaster and its effects on society and for a proper definition of the resilience index it is necessary to define the event in space (building, town, country, etc.) and time (short term response, long term response, etc). If an earthquake hits a largely populated area the recovery process will be longer and to be able to make adequate comparisons the time is normalized as stated previously T_{LC}.

For the community level, all the parameters defined in the “PEOPLES” framework have to be taken into account, and this is done through Eq. (11) (Reinhorn and Cimellaro 2014)

$$Q_{TOT}(t) = Q_{TOT}(Q_P, Q_{Env}, Q_o, Q_{Ph}, Q_L, Q_{Eco}, Q_S) \tag{11}$$

where each of these aspects has to take into account their specific elements that influence their functionality. In this way time-dependent functionality maps are obtained, which are then

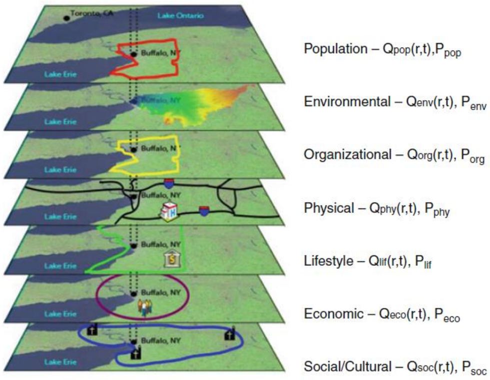


Fig. 10 Layer model of PEOPLES framework (Cimellaro *et al.* 2015)

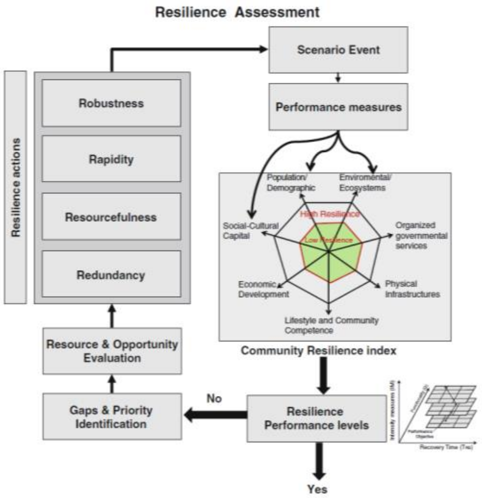


Fig. 11 Resilience scoring using PEOPLES methodology (Cimellaro *et al.* 2015)

transformed into the temporal residence contour scaled maps by the application of time $T_{LC,}$ being time-independent but varying in space. The community resilience index (Cimellaro *et al*. 2015) is then given by an equation

$$R_{com} = \int_{A_c} R(\vec{r}) / A_c dr = \int_{Ac} \int_{t_{OE}}^{t_{OE}+T_{LC}} \frac{Q_{TOT}(t)}{A_c T_{LC}} dtdr \tag{12}$$

where Ac is the total area of the selected region.

Graphically this is presented in Fig. 10 where for each of the seven resilience dimensions a contour plot is obtained.

This information will further be used for the determination of the resilience scoring as defined in the PEOPLES methodology (Cimellaro *et al*. 2015) which will detect gaps and identify priority actions that have to be taken which use as input data in the decision process (Fig. 11).

Recovery function can be defined by various models using either empirical or analytical functions. Empirical recovery functions are based on test or field data interpretation and engineering judgment, which can use various methods including the Monte Carlo simulations. For earthquake events, the analytical recovery functions can be obtained using various numerical simulations, from response spectral analysis, nonlinear time history analysis, etc. One needs to take into account the time of the recovery process (short term for the emergency phase vs long term for the reconstruction phase). For the reconstruction phase, the simplest recovery model is the uniform cumulative distribution (linear) which has only one parameter-rapidity, two parameters-rapidity and the delay in the recovery process, and robustness. Next in respect of complexity is the Rayleigh probability density function recovery model and then the lognormal probability density recovery function (Cimellaro *et al.* 2010a, b, c). For the emergency phase, more complex models are available for example metamodel (Cimellaro *et al.* 2010a, b, c)

The Resilience-Based Design tries to provide resilient engineering solutions that could be adopted in the practice, according to the resilience requirements derived from resilience models like PEOPLES. These solutions can be adopted at different scales, at the level of the structure or the level of a system of systems. At the scale of a single structure, one example of this kind of resilient design's application was done by Mitoulis and Rodriguez (2016) with the proposal of the new resilient hinge (RH) (Fig. 12) design which has minimal damage during the ground motion and is cost-effective. Several goals are envisaged by this specific hinge formulation, from energy dissipation, reduction of severe effects due to earthquake activity on bridges and minimization of the pier drift. The residual drift of this kind of hinge was reduced for 93% with respect to the classical reinforced concrete piers.

Titirla *et al*. (2017) proposed a versatile foundation. The bridge is isolated by rocking footing which is under-designed on purpose and supported on elastomeric high damping rubber pads enhancing the period and damping and delivering damage-free bridges. The connections are simplified and the settlement is minimized. Dissipation of energy is provided by the pads and minimal residual drift is obtained. The pad controls the fluctuation of the axial force as well. The beneficial effect of this rocking isolation was seen in the enormous reduction of the bending moments up to 45% and the shear actions up to 80% on the bridge piers (Titirla *et al*. 2017, Tubaldi *et al*. 2016).

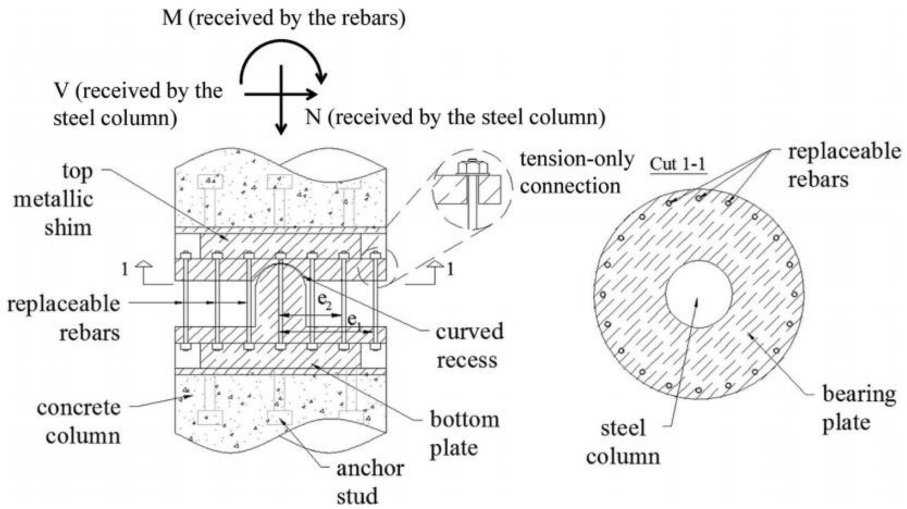


Fig. 12 New resilient hinge (RH) design (Mitoulis and Rodriguez 2016)

Application of the resilience on the community level is relatively a new concept which has been applied on several occasions. Reinhorn and Cimellaro (2014) implemented the concept of resilience design on the hospitals described in Park *et al.* (2004) and elaborated in detail in Cimellaro *et al.* (2009), where two cases were elaborated. In the first case loss estimation study of a specific hospital was conducted applying a nonlinear dynamic analysis for various limit states utilizing the median and log-standard deviation. The goal of the second case study was to determine the economic losses of a hospital network in the city of Memphis, Tennessee. Several numerical models were done from the equivalent SDOF system to MDOF which was used for nonlinear time history analysis, and fragility analysis with the application of 100 synthetic near-fault ground motions. The functionality Q(t) was conducted for four hazard levels taking into account the exponential recovery functions. It was interesting to see that the resilience was almost constant with the increase of earthquake intensity (Cimellaro *et al.* 2009), indicating that the structure has a consistent design for different hazard levels. In respect to four possible strengthening methods (moment resisting frame, buckling restrained braces, shear walls and weakening and damping), the choice of weakening and damping strategy was the best option for resilience improvement. This technique reduces both displacement and acceleration, which is very important for structures with sensitive contents (hospitals), where the response can damage nonstructural components that are acceleration sensitive (Viti *et al*. 2006).

5. Conclusions

To be able to describe earthquake effects on the community and its functionality, it is important to replace the Performance-Based Approach by the Resilience-Based Design (RBD) where resilience is considered as a global characteristic of the whole system (community included) and not only as an index of a single structure. RBD aims to make individual structures and communities as “Resilient” as possible, after the disaster or extreme event, and to be able to regain its full functionality as quickly as possible with the application of certain actions.

The data required for the system resilience is very diverse and encompasses the four segments (technical, organizational, societal, and economic) which are covered with numerous uncertainties that need to be incorporated in a unique function that will give unbiased results regarding the magnitude of risk. The performance level is moved from the structure to the system, with uncertainties being handled within the framework of a multi-scale approach very similar to defining the inelastic response of heterogeneous materials (e.g., Sarfarat *et al.* 2018).

Recovery after simultaneous extreme hazards (like an earthquake and the aftershock) should be as rapid as possible in order for the system to be fully functional as soon as possible. A step in this direction is the resilient bridge design which will enable damage-free or minimum damage to the structure due to extreme earthquake actions. The resilience has moved to a community level, with an illustrative example of hospitals presented. The same concept should be expanded to schools and other facilities as well. In order to make RBD applicable to a community, the metamodel developed for the hospital should be generated to a region that would represent the behavior of an entire community. This kind of methodology should be developed for different types of structures. Until now, no explicit procedure exists for quantifying structure and infrastructure response and resilience in the context of multiple hazards (e.g., Ibrahimbegovic *et al*. 2013, Satterthwaite and Dodman 2014, Ibrahimbegovic *et al*. 2016). It is important to ensure that the recovery process to be quick and that potential consequences in the life of people, cost and social loss will be mitigated.

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