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Lateral-resisting systems capable of multiple seismic performances

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ABSTRACT: This paper aims at presenting an innovative approach for an optimised/full-controlled seismic design of structures which combines recent contributions in the field of earthquake engineering and overcomes the traditional design approach leading to the identification of the characteristics of the lateral-resisting system capable of satisfying multiple seismic performance objectives. In this respect, it is fundamental the total conceptual separation between the structural systems resisting to vertical and horizontal loads. With reference to both (1) a braced pendular frame structure and (2) a shear-type frame system coupled with a lateral-resisting element (such as a reinforced concrete core or a bracing system), the approach here presented identifies the characteristics (strength, stiffness, ductility, energy-absorption) of the system resisting to horizontal loads which enables to satisfy prescribed seismic performance objectives. This is achieved through the identification of an objectives curve, in the Force-Displacement diagram, of the mechanical characteristics of the structure. The lateral-resisting system is obtained by means of (1) special braces in the case of the braced pendular frame structure and (2) special connection elements in the case of the shear-type system coupled with a lateral-resisting element.

INTRODUCTION

The design of building structures capable of providing given seismic performances represents a difficult task due to the complex characterization of the seismic action (not a single action but a set of possible actions of different strength and probability of occurrence) and of the structural response. Many recent contributions in the field of seismic engineering have opened up new possibilities for the structural engineer in terms of conceiving and dimensioning (i.e. designing) a structural system which offers predetermined seismic performances. Skipping all details, these recent contributions may be summarized as follows: (i) the PBSD approach (SEAOC Vision 2000 1995, Bertero and Bertero 2002) formalized the need of satisfying a multiplicity of performance objectives, (ii) the Direct Displacement-Based Design (DDBD) (Priestley et al. 2007) introduced the displacement analysis as a tool for seismic design of structures, (iii) the wide use of protection devices and techniques (e.g. unbounded braces, dampers (Christopoulos and Filiatrault 2006), seismic isolators adopted for the mitigation of the seismic effects upon the structure, which allows the conceptual separation between the structural systems resisting to vertical and horizontal loads. This paper (i) presents a design approach for a full-controlled seismic design of structures which combines these recent contributions and overcomes the traditional one, and (ii) proposes a special solution for lateral resisting systems capable of multiple seismic performances constituted by specifically developed "calibrated-shape devices".

1. IDEA AND NEW ASPECTS OF THE PROPOSED DESIGN APPROACH

The basic idea behind the design approach here proposed lies in the identification of the characteristics (strength, stiffness, ductility, energy-absorption) of the structural system resisting to horizontal loads which enables to satisfy a multiplicity of seismic performance objectives, as required by the PBSD and as already faced in other research works, by adapting and exploiting the complete DDBD approach (Xue and Chen 2003). In general, the horizontal-resisting system (hereafter referred to as HRS) of a given building structure can be seen as composed of a series of single horizontal-resisting elements (hereafter referred to as “horizontal-resisting components”, HRC), working together. The mechanical characterization of each component (being either a shear wall, a bracing system or other) of the horizontal-resisting system necessarily requires to capture both its elastic and inelastic behaviour. Without loss of generality, the mechanical characterization of each elementary component can be assumed to be an elastic-perfectly plastic one or a bilinear one with hardening. The independent parameters, which are necessary to fully characterize the HRC behaviour, are only four and they are stiffness, strength, ductility and strain hardening ratio. For sake of simplicity, in order to present the basic ideas of the design approach here proposed, in the following parts of this paper, the elastic-perfectly plastic model (strain hardening ratio = 0) has been assumed. The mechanical characterization of the whole horizontal-resisting system, as composed of the n horizontal-resisting components working in parallel, can be directly obtained by adding the mechanical characterization of each single component. The new aspects of this design approach lie:

- in a sound and active combination of the most recent contributions in the field of the earthquake engineering (PBSD, DDBD and protection systems),
- in the total “separation” between the structural system resisting to vertical loads (“vertical-resisting system”) and the structural system resisting to horizontal loads (“horizontal-resisting system”),
- in the development of a special solution for the horizontal-resisting system based upon the smart use of peculiar devices which are specifically calibrated in their shape to satisfy multiple performance objectives.

2. **THE PROPOSED DESIGN APPROACH**

The previous considerations, which show how a backbone F-δ curve can be developed and controlled acting on each single HRC, can be collected and formalized in a 3-Phases seismic design approach which is aimed at identifying the characteristics of the structural system resisting to horizontal loads which enables to directly satisfy given seismic performance objectives (without recurring to any trial-error processes). The approach is composed of the following 3-phases:

- Phase 1: starting from selected seismic performance objectives, identification of the F-δ objectives curve of the structural system to be designed;
- Phase 2: development of “calibrated-shape devices” which are capable of satisfying the selected performance objectives when used either (i) as special braces in the case of a pendular frame structure or (ii) as special connections in the case of a frame coupled with a lateral-resisting element;
- Phase 3: verification, by means of appropriate time-history analyses, of the seismic performances achieved.

The approach is illustrated in the following part of the paper with reference to two applicative examples, as developed with reference to:

1. a braced pendular frame structure, and
2. a shear-type frame system coupled with a lateral-resisting element (such as a reinforced concrete core or a steel bracing system).

3. **FIRST APPLICATIVE EXAMPLE**

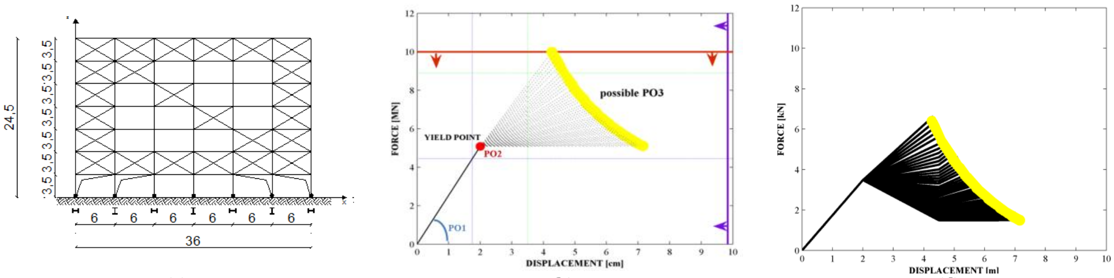
The first applicative example is carried out with reference to a building structure composed of seven-storey pendular steel frames (for sake of simplicity it is here assumed that there is no transmission of bending moments at the column-beam connections). The building plan is 36 x 18 m, inter-storey height is 3.5 m and the total building mass is $4.54 \cdot 10^6$ kg. The building is assumed to be located in Bologna (Italy) on D.M. 14/01/2008 soil type C and on topographic surface S1. It is designed to meet the D.M. 14/01/2008 provisions. The structure is characterized by the separation between the vertical-resisting system (beams and columns) and the horizontal-resisting system (special bracing system). The vertical-resisting system is sized to support just the vertical loads. The horizontal-resisting system is designed to display a controlled inelastic behaviour at the ground level and to behave elastically from the second storey up. It is composed of: (1) special bracing elements, named “calibrated-shape devices” placed between the ground storey and the first storey, (2) traditional diagonal bracing elements from the second storey up. The horizontal-resisting system, placed between the ground and the first storey, is calibrated, within a Performance-Based Seismic Design approach, to satisfy a multiplicity of performance objectives through the identification of a "objectives curve", in the Force-Displacement diagram, of the mechanical characteristics of the structure. The horizontal-resisting elements from the second storey up can be designed through a capacity design approach and will not be considered in the following analyses. Figure 1a shows the geometry of one of the two perimeter pendular steel frames in both the North-South (NS) and the East-West (EW) directions. The seismic behaviour of the building along each direction may be schematised as the one of a SDOF system characterized by a mass corresponding to that of the whole superstructure (second storey up) and by the lateral force-displacement relationship controlled by the HRSs composed of 8 HRCs along both the NS and the EW directions, respectively, together with the little contribution to the lateral resistance provided by the vertical-resisting systems. In the following part of the paper, for sake of conciseness, only the seismic behaviour of the building along the NS direction will be considered.

3.1 Identification of the F-δ Objectives curve (Phase 1)

3.1.1 *The selected performance objectives*

With the aim of obtaining a desired behaviour for the structure at hand and designing the horizontal-resisting system, we imposed the following Basic Objectives as defined in the Vision 2000 document:

1. First Performance Objective (PO1): defined as a coupling of the Fully Operational performance level with the Frequent Earthquake Design Level;
2. Second Performance Objective (PO2): defined as a coupling of the Operational performance level with the Occasional Earthquake Design Level;
3. Third Performance Objective (PO3): defined as a coupling of the Life Safe performance level with the Rare Earthquake Design Level.



(a) (b) (d)

Figure 1. (a) Schematic representation of the building structure considered. (b) Objectives curve in the Force-Displacements diagram. (c) “Objectives curve” of each couple of HRCs.

3.1.2 *Objectives Curve in the force-displacements diagram*

Imposing on the considered structure the previous performance objectives (making use of the tools of the typical Priestley’s DDBD approach, such as the identification of the seismic demands by equating equivalent viscous damping and reading off an effective period from highly-damper displacement spectra), we have obtained the objectives curve in the Force-Displacements diagram, for the city of Bologna, for D.M. 14/01/2008 soil type C and for topographic surface S1. For sake of conciseness, it is here only represented in graphical form in Figure 1b, which illustrates the target points for an optimised/controlled seismic behaviour of the structure (performance objectives PO1, PO2, PO3). As illustrative example, as far as PO1 is concerned, F_{PO1} = 4.41 MN, δ_{PO1} = 1.75 cm and $K_{PO1} = K_{target}$ =

253790kN/m.

3.2 **Identification of the characteristics of each single horizontal-resisting component (Phase 2)**

In this section, the physical characteristics of the horizontal-resisting system are obtained taking into account also the mechanical properties along the horizontal direction of the vertical-resisting system.

3.2.1 *Lateral stiffness of the vertical-resisting system*

For the case-study at hand, the lateral stiffness (initial inclination of the force-displacement relationship) of the vertical-resisting system, as composed by 28 equal HEA300 columns (14 of which placed to act in weak direction and 14 of which in strong direction), is computed as:

$$K_{\mathrm{VRS}} = \sum_{i=1}^{28} k_{\mathrm{i}} = \sum_{i=1}^{14} 1.6 \frac{EJ_y}{h^3} + \sum_{i=1}^{14} 1.6 \frac{EJ_x}{h^3} = 80060 \frac{\mathrm{kN}}{\mathrm{m}} \tag{1}$$

where E = 210000 MPa (Young modulus), J is the moment of inertia along the considered direction (J_y =19380cm^4 and J_x =58220cm^4), and h = 3.5 m (inter-storey height). The stiffness coefficient 1.6 derives from the specific static scheme corresponding to pendular columns which are constrained to move together, along the X-direction, in the upper stories due to the presence of traditional diagonal bracing elements. The vertical-resisting system alone is not able to satisfy the performance objects imposed.

3.2.2 *Lateral stiffness of the horizontal-resisting system*

Without modifying the lateral stiffness of the vertical-resisting system, we assign the part of the lateral stiffness, required for satisfying the performance objectives, to the horizontal-resisting system placed between the ground and the first storey. Figure 1c shows the “objectives curve” of each HRC. It is obtained subtracting from the structure “objectives curve” the lateral contribution of the vertical-resisting system and dividing by the total numbers of horizontal-resisting components which compose the horizontal-resisting system along the considered direction. Let us indicate the lateral stiffness of the horizontal-resisting system with ΔK or K_{HRS}:

$$\Delta K = K_{\mathrm{HRS}} = K_{\mathrm{Target}} - K_{\mathrm{VRS}} = 173730 \ \frac{\mathrm{kN}}{\mathrm{m}} \tag{2}$$

3.2.3 *Design of the linear mechanical/geometrical characteristics of the calibrated-shape device*

The lateral stiffness of each couple of horizontal-resisting components (one brace in compression + one brace in tension) is 1/4 of the lateral stiffness of the horizontal-resisting system (there are four Calibrated-shape devices on each face of the building in North-South direction): $K_{\mathrm{2HRC}} = K_{\mathrm{HRS}}/4 = 43440\ \mathrm{kN/m}$. Figure 2a shows the generic couple of calibrated-shape devices. The Virtual Works Principle gives the lateral stiffness of the single crescent shaped brace as follows:

$$\frac{K_{\mathrm{2HRC}}}{2} = \frac{3EJ\cos^2\alpha}{d^2 \cdot \ a_1 + a_2} \tag{3}$$

Where E is the steel Young Modulus; J is the inertia moment of the HRC’s cross section; α is the inclination of the portal’s diagonal; *d* is the distance of the knee point, *P*, from the portal diagonal; $a_1 + a_2$ is the length of the calibrated-shape device. The first equation for sizing the single crescent shaped brace can be found imposing the equality between Eq. (2) and Eq. (4):

$$\frac{J}{d^2} = \frac{K_{\mathrm{2HRC}}/2 \ \cdot \ a_1 + a_2}{3E\cos^2\alpha} \tag{4}$$

The structure first yield displacement, δ_{y1}, is also the single horizontal-resisting component first yield displacement. So, when each crescent shaped brace has reached its first yield displacement, the bending moment in the most stressed section reaches the first yielding moment, M_y. The maximum bending moment at point *P* and the first yielding moment are given by:

$$M_{\mathrm{P}} = \frac{F}{\cos\alpha} \cdot d; \qquad M_{\mathrm{y}} = f_{\mathrm{y}} \cdot W_{\mathrm{el}} \tag{5}$$

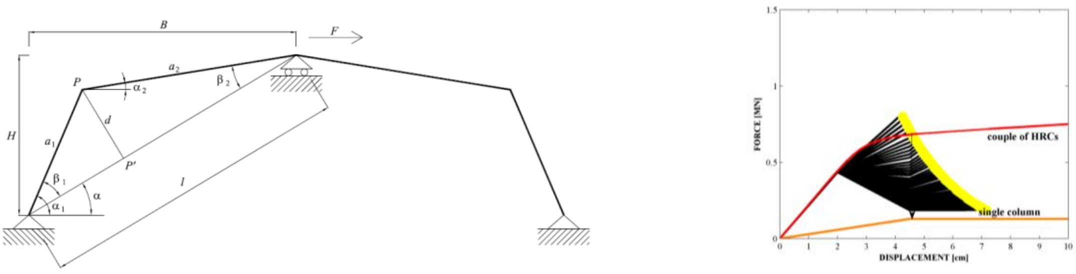
where f_{y} is the steel yielding tension ($f_{\mathrm{y}} = 275\ \mathrm{MPa}$) and W_{el} the section modulus of the HRC's cross section. The second equation for sizing the single crescent shaped brace can be found imposing the equality between the equations (5):

$$\frac{W_{el}}{d} = \frac{F_{\mathrm{y1,\,2HRC}}/2}{\cos\alpha} \cdot \frac{1}{f_{\mathrm{y}}} \tag{6}$$

where *h* is the height of the cross section and $F_{\mathrm{y1,\,2HRC}}/2$ is the first yielding force of each couple of HRCs. The number of the unknown quantities (*J*, W_{el}, *d*) is greater than the number of the equation, hence it is necessary to fix one of the three unknown quantity. Fixing d = 1 m, we obtain $J = 33414\ \mathrm{cm}^4$ and $W_{el} = 1789\ \mathrm{cm}^3$. At this point, several cross-sections may be found with these prescribed values of *J* and W_{el}. The following cross-sections can be taken into account: rectangular cross-section characterised by *h* = 35 cm and *b* = 9.5 cm ($J = 33943\ \mathrm{cm}^4$ and $W_{el} = 1940\ \mathrm{cm}^3$).

3.2.4 *Design of the non-linear mechanical characteristics of the calibrated-shape device*

Provided that there are different possibilities regarding the second non-linear branch of the objectives curve (the satisfaction of the PO3 does not lead to a unique objectives curve, but many different solutions may be feasible), the constitutive law of each couple of HRCs has been numerically obtained (Figure 2b) with a non-linear static push-over analysis (displacement control) and then checked with respect to the possibilities of the objectives curve. The couple of braces has been modelled in the STRAUS7 version 2.2.3 package using 2D plane-stress plate elements with the typical stress-strain curve for mild steel material (S275), considering both mechanical and geometrical non-linearities. Figure 2b shows the constitutive law of each couple of HRCs as reported above the objectives curve, together with the contribution of the single column of the vertical-resisting system, which may be considered negligible. Inspection of Figure 2b also indicates that the system is far from the ultimate displacement capacities of both the columns (128 cm) and the calibrated-shape devices (larger than 40 cm). It is clear that the single couple is able to satisfy the performance objective, consequently, also the structure should be able to satisfy the imposed performance objectives. This will be verified in next Phase 3. From Figure 2b, it can be noted that the imposed (designed) displacement under the Rare Earthquake is equal to 4.6 cm.



(a) (b)

Figure 2. (a) Generic couple of two calibrated-shape devices. (b) The constitutive law and the objectives curve of each couple of HRCs.

3.3 **Analysis and verification (Phase 3)**

In this phase, the analysis of the structure so obtained is carried out to verify if the actual structural behaviour is congruent with the expected/imposed performances. A plane model of the structure has been realized using the SAP2000 v14 package, and each crescent shaped brace has been modelled with a synthetic non-linear link element, characterised by a reasonable schematization of the sophisticated constitutive law obtained in the previous Phase 2. Non-linear time-history dynamic analysis have then been developed on the structural model, using as earthquake ground motions, two groups of seven accelerograms. The first group is composed of seven accelerograms which are overall compatible with the design spectrum of the Italian code corresponding to the Frequent Earthquake. The second group is composed of seven accelerograms which are overall compatible with the design spectrum of the Italian code corresponding to the Rare Earthquake. The accelerograms have been obtained using the program REXEL v 2.2 (beta) (Iervolino 2008). The average values of the maximum displacements of the first storey caused by the seven accelerograms corresponding to the Frequent Earthquake and by the seven accelerograms corresponding to the Rare Earthquake (which represent the average displacement demand required by the earthquakes), are compared with the value of the displacement demand imposed in Phase 1 for each performance objective. These values are almost the same in fact we obtain 1.76 cm vs. 1.75 cm, and 5.34 cm vs. 5 cm.

4. **SECOND APPLICATIVE EXAMPLE**

The second applicative example is carried out with reference to a building structure composed of five-storey steel shear-type frames coupled with a lateral-resisting element (i.e. a r.c. wall). The connection between the frame and the lateral-resisting element is realized through the spread placement of hysteretic dissipative devices (therefore referred as HDDs) at each floor of the structure (Figure 3a). The building plan is 36 x 18 m, inter-storey height is 3.5 m and the total building mass is $5.40 \cdot 10^6$ kg. The building is assumed to be located in Bologna (Italy) on D.M. 14/01/2008 soil type C and on topographic surface S1. It is designed to meet the D.M. 14/01/2008 provisions. The structure is characterized by the separation between the vertical-resisting system (beams and columns) and the horizontal-resisting system (r.c. walls of the core). The vertical-resisting system is sized to support just the vertical loads whilst the lateral-resisting element is sized to support just the horizontal loads. The HDDs are calibrated, within a Performance-Based Seismic Design approach, to satisfy a multiplicity of performance objectives through the identification of a "objectives curve", in the Force-Displacement diagram, of the mechanical characteristics of the structure. In this second case, the seismic behaviour of the building along each direction may be schematised as the one of a MDOF system.

4.1 **Identification of the F-δ Objectives curve (Phase 1)**

As far as the identification of the F-δ objectives curve (Phase 1) is concerned, the same Basic Objectives have been imposed on the considered structure to obtain the objectives curve in the Force-Displacements diagram.

4.2 **Identification of the characteristics of each single horizontal-resisting component (Phase 2)**

Since the structure at hand is a MDOF system, the HDDs should be calibrated along the height of the building. A tri-linear constitutive law is imposed for each HDD according to the following procedure:

1. Step 1: identification of the total stiffness which the lateral-resisting system should provide to the structure for satisfying the imposed performance objectives;
2. Step 2: distribution of the stiffness of each HDD along the height of the structure;
3. Step 3: identification of the yielding force of each HDD;
4. Step 4: selection of the slope of the second post-yielding branch, of the slope of the third hardening branch, and of the ultimate displacement.

4.2.1 *Step 1: identification of the total stiffness for the HDD*

The following criterion has been envisaged to obtain a reasonable total stiffness. The total stiffness is evaluated through the equation (9) accounting for the stiffness of the device placed on the first floor, $k_{d,floor1,new}$. This latter is evaluated multiplying the floor stiffness of the shear-type frame, $k_{p,floor,old}$, by a coefficient corresponding to the maximum ratio between the maximum inter-storey drift calculated from the linear analysis ($\max\ id_{floor}$) and the displacement corresponding to PO1, δ_{PO1}, or the elastic displacement of the structure, δ_{el}:

$$k_{d,floor1,new} = k_{p,floor,old} \cdot \max\left(\frac{\max\ id_{floor}}{\delta_{PO1}}, \frac{\max\ id_{floor}}{\delta_{el}}\right) \quad (7)$$

4.2.2 *Step 2: distribution of the stiffness of each HDD along the height of the structure*

For the distribution of the stiffness of each HDD along the height of the structure different criteria have been envisaged and analysed:

- Criterion 1: transformation of a n-DOF system (where n is the number of floors) in one made up of n unrelated SDOF systems characterized by the same stiffness:

  $$k_d = \frac{4\pi^2 m_{floor}}{T^2_{Frequent}} \quad (8)$$

  where $k_{d,i}$ is the stiffness of the i-th device (where $k_{d,1} = k_{d,2} = ... = k_{d,i} = ... = k_{d,N}$), m_{floor} is the mass of the floor (which is assumed to be the same for each floor), $T_{Frequent}$ is the period of the whole structure corresponding to the initial imposed stiffness.

- Criterion 2: calibration of the stiffness of the i-th device based of the i-th shear floor (evaluated through a linear modal analysis) of the shear-type frame; in this case the stiffness of the devices varies from floor to floor:

  $$k_{d,i} = \frac{T_i}{\delta_{PO1}} \quad (9)$$

  where $k_{d,i}$ is the stiffness of the *i*-th device, δ_{PO1} is the imposed displacement at PO1.

- Criterion 3: iteration carried out upon Criterion 2 which stops when the variation of the group of stiffness between two consecutive is negligible;
- Criterion 4: transformation of n-DOF system in one made up of n-linked SDOF systems. This conception is able to consider the shear contribute of overhanging floors.
- Criterion 5: optimization of the initial stiffness through Genetic Algorithms.

On the basis of the results obtained, the best methodology has been identified in Criterion 3, and, for design purpose, it has been deduced the following simplified formula: the relative values of the storey stiffnesses are roughly positioned on a line described by:

$$i_r, 2 = \frac{i}{S(n)}; \quad S\ n = \sum_{i=1}^{n} i = \frac{n \cdot\ n+1}{2} \quad (10)$$

where *n* is the number of floors and *i* is the floor index (floors are numbered in a crescent order from the top to the bottom).

4.2.3 *Step 3: identification of the yielding force of each HDD*

Two criteria have been envisaged to identify the yielding force of each device:

1. the first one considers the same yielding force for all the devices (the yielding force of the device placed on the first floor has been chosen);
2. the second one considers the same yielding displacement for all the devices (the yielding displacement corresponding to the PO1 has been chosen).

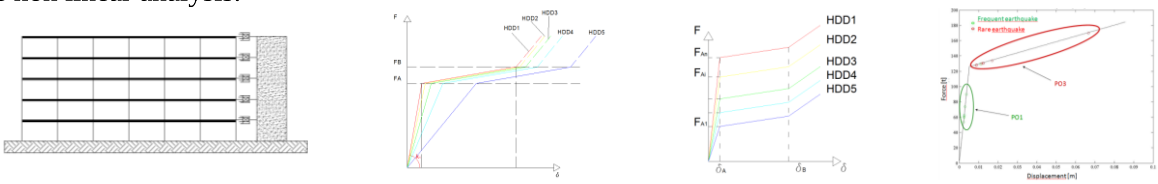
4.2.4 *Step 4: selection of the slope of the second post-yielding branch, of the slope of the third hardening branch, and of the ultimate displacement*

The slope of the second post-yielding branch is defined as the 3% of the initial stiffness of each device, in order to limit the forces on the structural elements. The slope of the third hardening branch is defined as the 50% of the initial stiffness of each device. The ultimate displacement is defined on the basis of the PO3.

In conclusion, the tri-linear constitutive law represented in Figure 3b is obtained.

4.3 **Analysis and verification (Phase 3)**

A plane model of the structure has been realized using the SAP2000 v14 package, and each connection element has been modelled with a synthetic non-linear link element, characterised by a constitutive law obtained in the previous Phase 2. Non-linear time-history dynamic analysis have then been developed on the structural model with the same earthquake ground motions used in the previous example. Figure 3d displays the on the objectives curve of the first floor the results obtained through the non linear analysis.



(a) (b) (c) (d)

Figure 3. (a) Schematic representation of the building structure considered. Constitutive law of HDDs: (b) same yielding force and (c) same yielding displacement. (d) Verification of the seismic performances of the HDDs. (c) Objectives curve for the first floor and results of the non-linear time-history analysis.

5. **CONCLUSIONS**

This paper presents an innovative seismic design approach which allows to exploit at their best all the potentialities offered by both the PBSD framework and the DDBD methodologies. This approach, which has been applied to the case studies of a (1) moment-resisting frame structure and (2) shear-type frame system coupled with a lateral-resisting element, leads to the identification of the characteristics of the (1) structural system resisting to horizontal loads (special calibrated-shape braces) in the first case-study and (2) special connection elements in the second case-study, which enable to satisfy given seismic performance objectives. This is achieved by considering a total conceptual separation between the structural systems resisting to vertical and horizontal loads and by the use of peculiar calibrated-shape devices as horizontal-resisting system.

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