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Performance-based seismic design of multi-storey frame structures equipped with Crescent-Shaped Brace

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ABSTRACT

10 The primary objective of the Performance-Based Seismic Design (PBSD) is to provide stipulated seismic 11 performances for building structures. However, a certain degree of design freedom is needed for matching a 12 specific seismic response. This design freedomis not obtainable bythe conventional lateral resisting systems 13 because their stiffness and strength are coupled. Here, weput emphasis on the role of the unconventional 14 lateral resisting systems in adding more flexibility to the design. In this paper, we seek to explore the seismic 15 design of moment resisting frame structuresequipped with an innovative hysteretic device, known as 16 Crescent-Shaped Brace (CSB). One conspicuous feature of this device is itsdistinctive geometrical 17 configuration, which is responsible for theenhanced nonlinear force-displacement behavior exhibited by the 18 device. A new performance-based approach for the seismic design of the CSB is proposed. The performance 19 of the deviceisevaluated and its application in multi-storey shear-type structures is investigated. Two case 20 studies were established to illustrate the design methodology. The first is a new two-storey RC structure and 21 the second is an existing three-storeyRC structure. Nonlinear time history and pushover analyses are 22 performed to evaluate the behavior of the controlled structures. The analyses show that for each of the two 23 case studies the acceleration-displacement capacity spectrum conforms to the performance objectives curve. 24 This finding confirms the validity of the proposed design approachandthe effectiveness of the new hysteretic 25 device in resisting lateral forces.

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Keywords:*Crescent Shaped Brace, Design method, Dynamic analysis, PerformanceBased Seismic Design.*

29 1 INTRODUCTION

30 Recent development in earthquake engineering has resulted in the emergence of new 31 structural design approaches such as the Performance-Based Seismic Design (PBSD)[1].PBSD is 32 still deemed as a new approacheven thoughits origin can be traced back as far as the late 20th 33 century. The design efficiency of PBSD is the main reason behind its emergence[2]. The 34 Performance-Based Design specifies the mainobjectives that should be attained by the structure and 35 gives the standards for accepting a specified performance[3]. Today, structures are designed with the 36 goal of achieving a predefined functionality. This is because the challenge is no longer limited to 37 protecting humanlives, butextended to minimizing damages and disruption down to reasonable 38 levels. Nevertheless, matching a defined seismic response necessitates additional design freedom 39 that is unable to be achieved by the traditional structural components, such as beams and columns. 40 Here, it isnecessary to emphasize the role of the unconventional lateral resisting systems in making 41 thedesign more flexibleand thus allowingto reach specific seismic performances.

42 Lately, several efforts in the earthquake engineering discipline could find their ways into 43 various advanced lateral resisting systems. These systems can provide enhanced performances to the 44 structure underparticular ground motion levels. Examples of such systems include: (a) seismic 45 isolation systems, which disengage the superstructure from its substructure, thereby giving rise to a 46 "conceptual separation between the horizontal and vertical resisting systems" [4]; (b) tuned mass 47 damping systems, which are practically employed to reduce the vibration level of the 48 structureresulted from high lateral excitations[5]; (c) active and semi-active systems, which use the 49 actual seismic vibration to modify the mechanical properties of the structure accordingly[6]; (d) 50 dissipative systems, which are integrated into he superstructure to reduce the damage in the 51 structure through their energy dissipation capability[7]. Whilst the listed systems have been nicely

incorporated into practice and literature, none of them could completely fulfil the intended seismicobjectives of structures as outlined by the PBSD.

54 In this paper, we focus on a new innovative lateral resisting device, the Crescent Shaped 55 Brace (CSB). CSB is a hysteretic device that is grouped under the 'energy dissipation devices' 56 classification.Thedevice enablesthe structure to have prescribed multiple seismic 57 performances through its passive resisting capability[8]. Up to the present time, the design of multi 58 storey buildings equipped with Crescent Shaped Braces has not been exposed to wide-ranging 59 research. The application of the CSBs is restricted to a single case study of a steel structure in which 60 the braces were inserted at the ground floor. The objective of that study was toobtain a controlled soft-storey response. The upper storeys were braced with conventional concentric steel diagonal 61 62 braces in order to conceptually model the system as a single degree of freedom (SDOF) system [4].

63 The work presented in this study proposes a comprehensivemethod for the seismic design of 64 multi storey shear-type-structures strengthened withCSB devices. In this study, the geometrical and 65 mechanical properties of the controlled structure are assumed to be given, as in the case of existing 66 structures; therefore, there is no control on the structure's stiffness and strength. This implies that the CSB system is the only variable in the design. In the case of designing new structures, more 67 68 design freedom is added as the properties of the structure can be chosen in accordance with the 69 desired performance objectives. The design method proposed in the study involves: (i)sizing the 70 CSB devices in the elastic field; (ii) verifying the behavior of the braces in the plastic field. The first 71 part of the method is to design the braces in the elastic field with reference to a predefined 72 performance point. Then, the post yielding behavior of the CSB is determined numerically using the 73 FEM software 'SeismoStruct V.7.0.6' [9]. In the second part of the method, the post yielding behavior of the controlled system (i.e. structure equipped with the designed braces) is verified by 74 75 means of nonlinear pushover and time history analyses.

To illustrate the procedure in all the details, the methodology has been applied to two case studystructures. The controlled structures designed to satisfy the 'Essential Objectives' shown in Figure 1[1]. Non-linear pushover and time-history analyses are performed to verify the performance of the controlled system under a given seismic input. The outcome of the study proved the validity of the proposed design method and the efficiency of the hysteretic device.



Figure 1. Performance-based seismic design goals. Adopted from [1]

84 2 THE CRESCENT SHAPED BRACES

85 2.1 Overview

The Crescent-Shaped brace (CSB) (Figure 2) is a unique hysteretic lateral resisting device thatprovides additional design freedom to frame structures. Its geometrical configuration, as shown in Figure 3, permits the structure to have predefined multiple seismic performances[8]. The CSB enables the designer to have full control over the design because its yielding strength and lateral stiffness are not coupled.



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Figure 2. A sample of the Crescent Shaped Brace

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94 2.2 Analytic model of the CSB

95 Previous work conducted on the Crescent-Shaped Braces by Palermo et al. (2015)led tothe 96 derivation of analytical formulations for sizing the device based on a target stiffness and a target 97 yielding strength. Eqs.(1) and (2) represent a simplified version of the original equations developed in [8]. Strength and stiffness are initially imposed according to the predefined performance 98 99 objectives that to be achieved. The process involves a consideration of the structural and nonstructural responses of the studied system. Equation (1) allowsobtaining the arm ratio of these 100 devices, which is the ratio between the arm of the deviced and the diagonal length L (see Figure 3). 101 This ratio canbe assumed as 0.1 forpreliminary designs. The arm ratio is subsequently replaced in 102 103 Eq. (2) to get the target moment of inertia of the CSB device.

$$\xi = \frac{M_{pl}}{F.L} \tag{1}$$

105 where $\xi = d / L$ represents the arm ratio of the device, d is the device arm, $M_{pl} = W_{pl} \cdot f_y$ is the 106 plastic bending resisting moment of the cross section, W_{pl} is the plastic section modulus, f_y is the 107 yield strength, $\overline{F_y}$ is the target yield strength, *L* is the diagonal length(i.e. the line connecting both 108 extremities of the device).

104

$$J = \frac{L^3 \cdot \overline{K} \cdot \xi^2}{3 \cdot E \cdot \cos^2 \theta}$$
(2)

110 where J represents the cross-section inertia, \overline{K} is the target initial lateral stiffness, E is the 111 modulus of elasticity of the steel section, θ is the angle formed between the applied force and the 112 device diagonal (i.e. $\theta = 0$).

113



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Figure 3. The geometric configuration of the studied device. Adopted from[8]

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117 **2.3 Mechanical behavior of the CSB**

118 The post-yieldingbehavior of a random CSB device has been numerically studied using the 119 fiber-based software 'SeismoStruct V.7.0.6', which considers both geometric nonlinearities and material inelasticity. First, a sample of the bracing device 'HEB200 European profile' was subjected 120 121 to a monotonic rising tension load. The result of the numerical analysis is displayed inFigure 4(the solid segment of the curve). At the beginning, the CSB responds in flexure, acting linearly until first 122 123 yielding is reached at the knee section. Then, the deviceencounters a plastic behaviordue to the 124 spread of plasticity (pseudo-horizontal part). This is followed by a secondremarkable hardening behavior as the device's armd decreases. At this stage, the device mainly reacts through its axial 125 stiffness capacity, like a conventional brace or a truss in a tensile layout. 126

127 The same specimen was subjected to a monotonically increasing compressive loading. Figure 128 4(the dotted segment of the curve) is a graphical representation of the constitutive law of the device in compression. It is very important to note that unlike traditional concentric braces, the CSB 129 devicedoes not suffer from sudden Eulerian in-plane buckling when exposed to a compressive 130 131 force, and this is due to its unique shape. Regarding the out-of-plane buckling, the appropriate selection of the cross section is highly effective in preventing such a problem[8] (e.g. choosing 132 133 balanced inertias along weak and strong axes). Another solution is to include longitudinal ribs in 134 correspondence to the neutral axis fiber.





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Figure 4. Monotonic behavior of a single CSB in tension and compression

The hysteretic behavior of the CSB is that of typical steel bracings given that the device is nothing more than a steel member having a curved configuration. The numerical studies conducted on the device has demonstrated a good hysteretic response [8]. The simulated hysteretic responses have been also confirmed by experimental tests conducted by some of the authors (the test results will be available soon [10]) and by other researchers [11].

The hysteretic force-displacement response of the single CSB device is strongly asymmetric due to the non-linear geometrical effects[8][10]: significant hardening response under lateral loads inducing tension in the brace, and softening response under lateral loads inducing compression in the braces (Figure 5a). On the contrary, when two CSB devices inserted in a two-span frame structure, the overall behavior becomes symmetric given that one works in compression while the other one works in tension (Figure 5b).



(a) (b)
 Figure 5. (a) A bilinear CSB device inserted in a frame and its asymmetric force-displacement
 response; (b) two mirrored disposed bilinear CSB devices inserted in two frames and their
 symmetric force-displacement response. Adopted from[10].

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157 3 METHOD: PERFORMANCE-BASED DESIGN OF A MULTI-STOREY SHEAR-TYPE 158 FRAME EQUIPPED WITH CSB DEVICES

159 The design philosophy behind the use of CSBs as enhanced bracings is grounded on the 160 concept of actively designing a structure behaving according to a so called "Building-Target Capacity (B-TC) curve'' that is then translated into a "Building-Actual Capacity (B-AC) 161 162 curve''[4]. The B-TC curve is the graphical representation of the idealized seismic behavior of the 163 building that we expect to achieve by imposing preselected multiple performance objectives, while the B-AC curve is the graphical representation of the effective seismic behavior of the building, 164 165 onceall structural members are designed. The use of CSBs at all storey levels is the design strategy here adopted to achieve the performance design objectives. 166

Given that CSBscan be used in different configurations, several design strategies can be 167 identified to achieve the desired performance objectives. In the literature, the behavior of an SDOF 168 169 steel structure equipped with this device has been investigated [4]. In this section, we propose a general procedure for the seismic design of multi-storey shear-type frame structures equipped with 170 171 Crescent-Shaped Braces (CSB). The proposed method can be used to design or strengthen structures that do not satisfy particular performanceobjectives. The design method proposed in the 172 study involves: (i) designing (sizing) the CSB devices in the elastic field; (ii) verifying the behavior 173 174 of the braces within the global system in the plastic field.

175 (i) Designing the CSB devices is done with reference to the performance point corresponding 176 to the earthquake level occasional (EQ2) and the performance level fully operational (IO) (Figure 1). This point belongs to the Essential Objective performance line, not the ordinary Basic Objective 177 178 performance line. The reason to choose a high seismic demand is to show the capability of the 179 braces in achieving a predefined performance level. The sizing method comprises 6 steps. In the first 180 step, an initial global stiffness matrix for the controlled structure (i.e.structure equipped with braces) is imposed based on certain criteria, which are described in section 4. The global stiffness 181 182 matrix is refined as more iterations are executed. In the second step, a modal analysis for the system 183 is performed. The building's drift obtained from the modal analysis is compared to the design drift that is set according to the desired performance point(i.e.EQ2-IO). The global stiffness matrix is 184 continuouslymodified through several iterations until the structure's drift meets the target drift. 185 Once the actual drift matches the design drift, we move to step four and we compute the stiffness of 186 187 the CSB bracing system. This is done by subtracting the stiffness matrix of the naked structure from 188 the global stiffness matrix. In step five, the structural configuration (i.e.position and number of 189 braces) of the CSB system is defined and hence the stiffness of each brace is computed. Finally, by

190 knowing the stiffness of each device, the moment of inertia and the arm of the devices are evaluated 191 in step 6, and this allows choosing a cross-section for the device from a wide range of cross-192 sections that satisfy the inertia demand. Once the cross-sectionis known, the post-yieldingbehavior 193 of the brace is obtained by means of a static nonlinear pushover analysis using the fiber-based FEM software "SeismoStruct V.7.0.6". SeismoStruct considers the geometric nonlinearity of the model 194 195 based on the corotational formula [12], and the material nonlinearity in accordance to Menegotto 196 Pinto law, with adequate focus on the isotropic hardening as given in [13]. The stiffness of the 197 device is computed at each step of analysis, and then updated automatically in the following analysis step. Generally, the post yielding behavior of the device is greatly affected by its section 198 199 profile; therefore, different section profiles must be compared and the one that conforms most to the 200 predefined performance is chosen.

(ii) The behavior of the CSB system within the global system is obtained by means of 201 nonlinear static pushover (PO) and dynamic time-history (TH) analyses using the FEM software 202 SAP2000 [14]. The behavior of the equipped structure is verified against the performance points 203 'EQ3-O' and 'EQ4-LS' shown in Figure 1. The CSB devices are introduced in the model as multi 204 205 linear links (NL) by importing the force-displacement curves (backbone curves) of the braces 206 obtained from SeismoStruct software. Using the backbone curves of the braces, SAP2000 updates the stiffness of the device at each analysis step according to the displacement exhibited by the 207 208 device. The force-displacement curves obtained from SeismoStruct are calibrated in order to 209 account for the structural configuration (inclination) of the devices in the structure. Moreover, the 210 kinematic hysteresis model, which is the default hysteresis model for all metal materials in the 211 program, is considered in the analysis as it is very appropriate for ductile materials. The above 212 mentioned implies that the actual nonlinear stiffness of each device is effectively considered in the 213 analysis. The nonlinearity of the structure is considered using concentrated plastic hinges. The 214 results of both PO and THanalyses are plotted together in order toverify the analysis performed. 215 Finally, the nonlinear pushover curve (i.e.capacity curve) is compared with the predefined 216 performance curve, according to which the devices were initially designed, to check if the target 217 performances are met. Although the nonlinear behavior of the structure equipped with the CSB 218 braces is not designed for 'automatic', previous studies suggested that the system would perform in 219 a good way with respect to severe earthquakes [4][15][16][17]. This is mainly due to the shape of 220 the brace (the peculiar mechanical behavior) (Figure 2) and to its hysteretic dissipation properties. In 221 the following section, we introduce the first part of the methodology(i.e. the design of the CSB 222 system), and in section 5 we cover the second partby means of a case study (i.e. the post yielding 223 verification of the braces within the global system).

224

225 **4 DESIGN OF THE CSB SYSTEM**

The dimensioning procedure of the braces is illustrated in Figure 6. The purpose of this design procedure is to obtain a target lateral stiffness for the single CSB device. The stiffnessoutput is then used in the previously delivered design formulas (Eqs.(1) and (2)) to get the inertia demand of the brace. Once securing the moment of inertia, the cross-section profile of the device can be selected from a broad range of cross-sections. In the following, the design procedure of the CSB is described in all details.

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Figure 6. Flowchart of the CSB design scheme

237 4.1 Step 1: Global stiffness matrix

The global stiffness matrix defines the rigidity of the controlled system. This matrix is determined by summing (as they act in parallel) the stiffness matrices of the bare structure and the bracing system.

 $\frac{241}{242}$

243
$$[K^*] = [K] + [K_b] = \begin{pmatrix} k_1^* + k_2^* & -k_2^* & & \\ -k_2^* & k_2^* + k_3^* & \ddots & \\ & \ddots & \ddots & -k_{N-1}^* & \\ & & -k_{N-1}^* & k_{N-1}^* + k_N^* & -k_N^* \\ & & & -k_N^* & k_N^* \end{pmatrix}$$
(3)

244

245 where $[K^*]$ denotes the stiffness matrix of the controlled system, k_1^* , k_2^* , ..., k_N^* represent the 246 stiffness terms of the controlled system at the different storey levels. These stiffness terms 247 aremathematically represented as follows:

249

$$k_i^* = k_i + k_{bi} \tag{4}$$

where k_i^* is the stiffness of the controlled system at storey *i*, k_i is the stiffness of the uncontrolled system at storey *i*, k_{bi} is the stiffness of the bracing system at storey *i*. From the mathematical illustrations above, we see that the global stiffness matrix $[K^*]$ consists of *N* unknowns, denoted as k_1^* , k_2^* , ..., k_N^* . The number of unknowns, however, can be reduced by enforcing a certain storey-stiffness distribution along the building height. In this work, the storey stiffness distribution is assumed to be proportional to the storey height and mass. The new expressions of the global stiffness matrix components can be obtained using the following formula, where m_j represents the mass of the j^{th} storey level, z_j is the height of the j^{th} storey level.

$$k_{i}^{*} = \frac{\sum_{j=i}^{N} (z_{j} \cdot m_{j})}{\sum_{i=1}^{N} (z_{j} \cdot m_{j})} k_{1}^{*}$$
(5)

259 The global stiffness matrix can be rewritten in a different form by substituting k_1^* , k_2^* , ..., 260 k_N^* in Eq.(3). The new global stiffness matrix becomes as follows:

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The mathematical illustration in Eq. (6)indicates that the global stiffness matrix isnow dependent on just one term (k_1^*) . For the first iteration, we can set the numerical value of k_1^* to be the same as k_1 . Alternatively, k_1^* can be kept as an unknown in the analysis, whichmakes the method non-iterative. However, modal analyses of systemsconsisting ofmore than 3-DOFswould be analytically difficult to deal with if there are many unknowns.

278 4.2 Step 2: Modal analysis

Amodal analysis of the controlled system is executed using the initial global stiffness matrix and the mass matrix of the system. The modal analysis enables obtaining the elastic displacements of each respective storey for the different modes. The SRSS rule is then used to combine the elastic displacements, as shown in Eq. (7). Afterwards, we compute the inter-storey drifts for each storey level using Eq.(8).

284
$$u_i = \sqrt{\sum_{n=1}^{N} \left(u_{i,n}^2\right)}$$
(7)

$$\delta_i = \left| u_i - u_{i-1} \right| \tag{8}$$

where *i* represents the storey number, u_i is the storey displacement at the *i*th storey, δ_i denotes the 287 storey drift between two successive storey levels i-1 and i, n is the mode's number, N is 288 thenumber of modes. 289

290

291 4.3 Step 3: Matching the design drifts

292 To achieve the predefined design objective, it is essential that the actual and the design inter-293 storey drifts match. Any discrepancy between the two drifts entails adjustment of the global stiffness 294 matrix. This adjustment is accomplished by adding an increment to the stiffness matrix, as shown in Eq.(9), and then re-running the modal analysis. This increment is illustrated in Eq. (10). It 295 isimportant to note that either of the global stiffness matrices introduced in Eq. (3) and Eq.(6) can be 296 used in the analysis. Moreover, the designer mustverify that the design drift of the structure is less 297 than its yielding drift. This is because we are conducting a linear analysis, andtherefore the elastic 298 299 range should not be exceeded.

$$k_{i,r+1}^* = k_{i,r}^* + C_{i,r}$$
(9)

$$\begin{array}{l}
300\\
301\\
302\\
303\\
\end{array}
\qquad k_{i,r+1}^{*} = k_{i,r}^{*} + C_{i,r} \tag{9}$$

$$\begin{array}{l}
C_{i,r} = k_{i,r}^{*} \cdot \frac{\delta_{i,r} - d_{i,r}}{d_{i,r}} \ge 1 \tag{10}$$

In the above equations, r represents the iteration step, C is the modification coefficient, δ is 304 the actual drift, d is the design drift, which is obtained from the predefined performance objective. 305

307 4.4 Step 4: Stiffness of the CSB system

Thetarget stiffness matrix of the bracing system isacquired by subtracting the stiffness matrix of 308 309 the uncontrolled structure from the global stiffness matrix, which isobtained in the final iteration of step 3. The mathematical equation is given below: 310

311
$$\begin{bmatrix} K_b \end{bmatrix} = \begin{bmatrix} K^* \end{bmatrix} - \begin{bmatrix} K \end{bmatrix} = \begin{pmatrix} k_{b1} + k_{b2} & -k_{b2} & & \\ -k_{b2} & k_{b2} + k_{b3} & \ddots & \\ & \ddots & \ddots & -k_{b(N-1)} & \\ & & -k_{b(N-1)} & k_{b(N-1)} + k_{bN} & -k_{bN} \\ & & & -k_{bN} & k_{bN} \end{pmatrix}$$
(11)

312

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313 4.5 Step 5: Stiffness of the single CSB device:

In order to obtain the target stiffness of each CSB device, the target stiffness components of 314 the CSB system $(k_{b1}, k_{b2}, ..., k_{bN})$ are divided over the total number of devices that exist at the 315 correspondingstorey level, as indicated by Eq. (12). It is the sole responsibility of the professional 316 designer to assign the number of devicestaking into account the architectural constraints in the 317 318 building structure.

319

 $K_{CSB.i} = K_{b.i} / N_{CSB,i}$ (12)

where $K_{CSB,i}$ represents the stiffness of the single CSB device at the i^{th} storey, $N_{CSB,i}$ is the 320 321 number of devices at the i^{th} storey.

323 4.6 Step 6: Moment of inertia of the CSB

324 At this stage, Eqs. (1) and (2) are used to calculate the moment of inertia of the devices. In these two formulas, \overline{K} is set equal to K_{CSB_i} , which is the target stiffness that we seek to 325 achieve, while \overline{F} represents the target yielding strength at which the device goes inelastic. Once 326 securing amoment of inertia for each CSB unit, across-section profile for the CSB is chosen from a 327 broad range of cross sections satisfying the target inertia. It is important to note that the cross-328 section profile choice may dominate the post yielding behavior of the bracing device. This can have 329 a significant impact on the post yielding behavior of the whole structure[8]. Thus, it isnecessary to 330 331 evaluate different cross-section profiles n order to fulfil the inelastic performance objectives 332 (i.e.performance pointscorresponding to EQ3-O and EQ4-LS shown in Figure 1).

333

334 5 POSTYIELDING VERIFICATION OF THE CSB SYSTEM: CASE STUDIES 335 5.1 The reference structures

The first case study structure (CS1) is a newcommercial building situated in Gubbio city. 336 Italy. Gubbio is a city located in the far north-eastern area of the Italian province of Perugia, which 337 is in a comparatively high seismic zone. The building was designed according the Italian seismic 338 339 standard[18]. Therefore, the building satisfies the operational and the life safety seismic objectives under occasional and rareearthquakelevels, respectively. Figure 7showsthe geometry of the building 340 structure. The building is rectangular with dimensions equal to 34.11m x 19.10m. It consists of two 341 342 storey levels with 4.1m height each. The backbone forming the structureconsistsofthree bays in the 343 y-direction (Elevation 1) and two bays in the x-direction (Elevation 2).

344 The second case studystructure (CS2) is an existing elementary school built in 1983. It is located inBisignano city, Italy, which is also a high seismic zone. As shown inFigure 8, the building 345 346 structure has a rectangular planar geometry with dimensions equal to 21.39m x 15.00m. It is made 347 up of three storey levels with a roof pavilion on the top. The backbone forming the structure consists of four bays in the y-directionand three bays in the x-direction. The mechanical properties 348 349 of the concrete were determined by the presidency of the council of ministers and the department of 350 civil protection in Italy, who performed ultrasonic and rebound hammer tests on a set of columns 351 and beams. The mechanical and geometrical properties of the concrete elements of both case 352 studies are listed inTable 1.

353





355 356



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360

361 Table 1at the end of the paper

362 5.2 Types of analysis

363 Two types of non-linear analysis are performed to verify the performance of the case study 364 structures. A three-dimensional model was built using the commercial software SAP2000in order to perform the analysis.As recommended by the Italian seismic standard, the loads applied to the 365 366 structure are:(a) the live loads multiplied by a combination factor (Ψ_E);(b) the dead loads without 367 any combination factor. The P- Δ effect was neglected given the small height and the high regularity 368 of the structures. The nonlinearbehavior of the frames is modelled using concentrated plastic 369 hinges.Flexural Hinges (type Moment M3) were applied to the beam elements, while flexural hinges (type P-M2-M3) were applied to the columns. The hinge force-deformation relationship, 370 371 also known as the 'backbone curve', isobtained using the concentrated plasticity model indicated by 372 FEMA 356 [19].

After designing the CSB devices as introduced in section 4,the force-displacement curve of eachdevice is obtained using SeismoStruct software by performing a nonlinear static pushover analysis. The Braces are then introduced in the SAP model as multi linear links (NL) by importing the force-displacement curves of the braces. The kinematic hysteresis model is considered in the analysis as it is very appropriate for ductile materials (Figure 9).



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381382

Figure 9. Nonlinear plastic link with kinematic hysteresis type to model the behavior of the CSB in SAP2000.

383 The first type of analysis is the static pushover analysis, which yields the capacity curve of the structure starting from the failure point [20]. In this analysis, two displacement shapes were 384 applied 'linear' and 'uniform', whose average is considered. The pushover curve was obtained in 385 386 terms of the base shear and the roof (top) displacement. The second type of analysis isthedynamic 387 time-history analysis, which was performed using the non-linear direct integration method witha damping ratio of 5%. The analysis was conducted by scaling a set of seven accelerograms to the 388 389 four design values of PGA at the fundamental period of the structure. The ground motion 390 accelerograms needed for the time-history analysis have been obtained using the software 391 SIMQKE_GR[21]. The accelerograms are consistent with the design spectra of the structure given 392 by the Italian seismic standard. The Earthquake design levels and the corresponding response spectra parameters indicated in Table 2. In the table, T_{y} represents the return period of the design 393

arthquake, *PGA* is the peak ground acceleration, F_0 is the maximum spectral dynamic amplification,

395 T_c^* is the characteristic period at the beginning of the constant velocity branch of the design 396 spectrum. As shown in the table, the design requirements of the school (CS2) are more stringent 397 than the commercial structure (CS1). The reason is that schools are generally more vulnerable than 398 other types of structures.

399

400 Table 2at the end of the paper

401

402 **5.3** Structural configurations and local optimization of the CSB devices

The structural configuration of the bracing devices defines their effectiveness level. A proper arrangement of the bracing devices in the structure would maximize the lateral resistance capacity while decrease the internal forces in the structural elements. This also leads to a reductionin thedevices' cross sections[22]. In addition, high axial force levels can dramatically decrease the moment capacity of columns; therefore, large axial forces should be avoided.

408 Choosing the right configuration depends on several factors, such as the architectural 409 constraints, the beam span length, and the axial and moment capacities of the columns and 410 foundations. The latter is very important especially if the structure is an existing structure where the 411 structural elements capacities are predetermined. In the design case, on the other hand, the designer 412 can design the columns to stand the additional axial forces coming from the bracing system, and 413 thus this problem can be prevented.

In this section, three possible configurations of the bracing devices (see Figure 10) are investigated by performing a time-history analysis.





418 Figure 11shows the results of the time history analysis in terms of the axial force transmitted 419 into column (C1) and foundation (F1) for each of the configurations. Config.A indicates the highest axial forces in C1 and F1 compared to the other twoconfigurations, whereasConfig. B shows small 420 421 axial forces in columns and foundations. The third configuration Config. C induces almost no axial force in column C1, while it causes the least amount of forces in foundation F1. Among all three 422 423 configurations, Config. C is the best configuration regarding theinternal stresses in columns and foundations: however, this comes at the cost of the resistance efficiency. On the other hand, 424 425 although Config. A produces the highest amount of forces in the columns and foundations, the 426 resistance efficiency is very high. Finally, Config. B seems to be less problematic in the 427 architectural point of view, as it leaves sufficient area in the facade for windows installation; 428 nevertheless, it is less resistant than the previous two configurations and it causes concentrated 429 stress in the mid span of the beam.



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- Figure 11.(a) Maximum axial force in column (C1) for each of the three configurations;(b) maximum axial force in foundation (F1) for each of the three configurations
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439 **5.4 Performance Objectives**

As we mentioned earlier, the first case study (commercial structure)has been designed in compliance with the Italian seismic standard;therefore,the building satisfies the basic design objectivescorresponding to the two earthquake design levels 'occasional' and 'rare' indicated in Figure 1 and Table 2. The second case study (school), on the other hand, is an existing structure; thus, we need first to verify its performance. This is done byperforming a pushover analysis to capture the base shear level at which the building yields.

In this work, higher demands are set to be attained by the structures. The *Essential Objectives* specified in Figure 1 are considered instead of the *Basic Objectives* according to which the structures were designed in the first place. The 'Essential Objectives' require the structure to remain in a fully operational condition under *occasional* earthquake design level (EQ-2), to stay in an operational condition with limited yielding and damages under *rare* earthquake design level (EQ-3), and to have some degree of damage while preventing life losses under *very rare* earthquake design level (EQ-4).

453 The Performance Objectives are usually set depending on the client's requirements, building's 454 destination, building's importance, and building's typology[15]. A study conducted by Bertero et al. 455 established applicable performance limits on the basis of some structural and non-structural damage criteria, such as structural damage indexes (DM), storey drift indexes (IDI), and rate of 456 457 deformations (floor velocity, acceleration)[1]. Those performance objectives, however, correspond 458 to the Basic Objectives (Figure 1); therefore, they cannot be used in our design because our desire is 459 to fulfil higher requirements. Table 3 reveals the basic objectives corresponding to each of the four earthquake levels, as proposed by Bertero et al. (2002). The table also showstwo proposed sets of 460 performance limits (for the two case studies) belonging to the EssentialObjectives. Selecting the 461 new performance limits was done by firstly setting the inter-storey drift indexcorresponding to EQ-3 462 463 (PO-3) to a value that insures no structural or nonstructural damage in the structure. The IDI corresponding to PO-3 of the first case study structure is 0.005while it is 0.0045for the second one. 464 465 The second case study structure was found to yield at a low IDI and this is the reason we set a more stringent performance demand (i.e. IDI=0.0045). Other objective points (PO-1, PO-2, and PO-4) 466 were set proportionally to the corresponding values of PGA at the fundamental period of the 467 468 structure.

- 469
- 470 Table 3at the end of the paper
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472 **5.5 Design of the CSB devicein the x-direction**

Following the CSB design methodology presented in section 4,Table 4 shows the methodology applied to the two case study structures. The reason of considering two case studies is to show the stability of the design method when applied to structures with different occupancies and different seismic demands. Another reason is to stress that existing structures do not always satisfy the seismic standards. For instance, the second case study structure (existing school) yielded at an inter-storey drift index of 0.0045 (PO-3), which does not comply with the Italian seismic standard that requires the building to yield at a higher drift ratio.

480 Table 4at the end of the paper

482 **5.6 Numerical verification**

In this section, the fulfilment of the pre-defined seismic performance objectives is verified. This was done through a numerical simulation of the seismic behavior of the two case studies. With this purpose, a finite element model for each case study has been developed using SAP2000. The fiberbased software "SeismoStruct V.7.0.6" was used to obtain the constitutive laws of the designed CSB bracing elements, which were then imported to SAP2000 as non-linear links (NL).

488 First, a non-linear pushover analysis was conducted using two displacement shapes (linear 489 and uniform), whose average was considered. The base shear and the roof (top) displacement were 490 used to signify the force and displacement respectively. Figure 12 and Figure 13show the 491 capacityspectra of the controlled and uncontrolled structures with their correspondingobjective 492 curves in S_{ad} format for the case studies 1 and 2 respectively. Investigation of the graphs reveals 493 that the for each of the two case studies the capacity spectrum (i.e. pushover curve) of the 494 controlled structure matches the corresponding predefined target curve (i.e. objective curve). On the 495 other hand, the capacity spectrum of the uncontrolledstructure was not able to match the 496 correspondingobjective curve.



Figure 12. The performance objectives and the results of the pushover analyses in S_{ad} format of the controlled and uncontrolled structures (Case study 1)

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503 Figure 13. The performance objectives and the results of the pushover analyses in S_{ad} format 504 of the controlled and uncontrolled structures (case study 2)

505 Another type of analysis, nonlinear time-history, was performed to assess the seismic performance of the structure. Four groups of spectrum-compatible accelerograms were considered 506 507 in agreement with the EQ levels reported in Table 2. Each group consists of seven ground motion 508 records scaled to the PGA of the corresponding EQ level at the fundamental period of the structure. 509 The results of the time-history analyses for the two case studies are plotted in Figure 14 and Figure 15 respectively, where each point represents the maximum base shear and ultimate displacement of 510 511 the corresponding time-history analysis. Investigation of the graph allows observing that the 512 seismic response of the uncontrolled structure fails to achieve the predefined performances, unlike 513 the controlled structure whose time-history analyses results show a large agreement with the 514 prescribed objectives.

515 It is important to note that the nonlinear behavior of the structure equipped with the CSB 516 braces is not designed for in this study 'automatic'; however, this good behavior is expected due to 517 the shape of the brace (the peculiar mechanical behavior) (Figure 2) and to its hysteretic dissipation 518 properties, and this is verified in this study.



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Figure 14. The performance objectives and the results of the time-history analyses in S_{ad} format of the controlled and uncontrolled structures (case study 1)





526 6 CONCLUSION

527 In this paper, a comprehensive procedure for the seismic design of multi-storey frame 528 structures equipped with an energy dissipation device "Crescent Shaped Brace" is proposed. The 529 procedure falls within the Performance-Based Seismic Design (PBSD) approach. The first part of the method is to design the braces in the elastic field with reference to the performance point IO-530 531 EQ2. Then, the post yielding behavior of the CSB is determined numerically using the FEM 532 software SeismoStruct. In the second part of the method, the post yielding behavior of the controlled system (i.e. structure equipped with the designed braces) is verified by means of 533 534 nonlinear pushover and time history analyses.

535 The validity of the method was determined by analyzingtwo reinforced concrete frame 536 structures equipped with crescent-shaped braces (CSB). First, the performance objectives are chosen. The performance objectives have been expressed in terms of the storey drift index (IDI), which isa 537 measure of the non-structural damage in the structure. Then, the CSB devices have been designed by 538 539 implementing the proposed design procedure. Static pushover and dynamic time-history analyses 540 were conducted on the case study structures to validate the nonlinear behavior of the CSB within the global system. The analyses performed showed a good behavior of the devices when applied to 541 542 both case studies although the two structures were of different occupancies and different seismic 543 demands. This confirms the validity of the proposed design approach and the effectiveness of the 544 new hysteretic device in resisting lateral forces regardless of structure's mechanical properties and 545 the seismic demands.

546 It isimportant to point out that all priorefforts to design theCSB were majorly based on SDOF 547 structures. The present design procedure applicable to both SDOF and MDOF shear-type 548 structures.Future research will be aimed atgeneralizing the method to be applicable to other types of 549 structures.

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Table 1. Mechanical and geometrical properties of the structural elements

		1	
Characteristics	CS1 (commercial building)	CS2 (school)	
Concrete average cubic strength, R _{ck}	C45/55, R _{ck} =55 Mpa	C20/25, R _{ck} =24.6 MPa	
Steel yield strength, f _y	B540C, f _y =450 Mpa	FeB38K, f _y =375 Mpa	
Modulus of elasticity, E	E=36000 Mpa	E=25150 Mpa	
Columns cross-sections	1 st level 60cmx60xm 2 nd level 50cmx50cm	1 st level 50cmx40cm 2 nd level 50cmx40cm 3 rd level 50cmx40cm	
Beams cross-sections	x-direction 50cmx40cm y-direction 50cmx40cm	x-direction 60cmx40cm y-direction 50cmx40cm	

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Table 2. Earthquake design levels with corresponding response spectra parameters for the two case studies

Earthquake Earthquake design level performance level		$T_r[years]$		<i>PGA</i> [g]		F_0		$T_c^*[s]$	
	p•110111111100 10 + 01	CS1	CS2	CS1	CS2	CS1	CS2	CS1	CS2
EQ1: frequent	Fully operational-IO	30	45	0.071	0.089	2.39	2.27	0.27	0.29
EQ2: occasional	Damage-O	50	75	0.093	0.116	2.34	2.28	0.27	0.32
EQ3: rare	Life safety-LS	475	712	0.230	0.323	2.39	2.45	0.31	0.38
EQ4: very rare	Near collapse-NC	975	1462	0.293	0.426	1.27	2.49	0.32	0.41

Table 3. Quantification of the Basic and the Essential performance objectives

Limit state	<i>IDI</i> [1]	Limit state	IDI	IDI
(Basic objectives)	(Basic	(Essential objectives)	(Essential	(Essential
	objectives)		objectives) CS1	objectives) CS2
EQ1: Fully operational	0.003	EQ1: Fully operational	PO-1 = 0.0015	PO-1 = 0.0013
EQ2: Damage	0.006	EQ2: Fully operational	PO-2 = 0.0020	PO-2 = 0.0018
EQ3: Life safety	0.015	EQ3: Damage	PO-3 = 0.0050	PO-3 = 0.0045
EQ4: Near collapse	0.020	EQ4: Life safety	PO-4 = 0.0067	PO-4 = 0.0055

Table 4. Application of the proposed design methodology to the two case studies

First case study:	Second case study:		
Step 1: Global stiffness matrix			
✤ Mass matrix:	✤ Mass matrix:		
$\begin{bmatrix} M \end{bmatrix} = \begin{pmatrix} m_1 & 0 \\ 0 & m_2 \end{pmatrix} = \begin{pmatrix} 8781.55 & 0 \\ 0 & 7035.165 \end{pmatrix} (kN)$	$\begin{bmatrix} M \end{bmatrix} = \begin{pmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \end{pmatrix} = \begin{pmatrix} 3799.5 & 0 & 0 \\ 0 & 3470.1 & 0 \end{pmatrix} (kN)$		
 Initial stiffness matrix: 	$\begin{pmatrix} 0 & 0 & m_3 \end{pmatrix} \begin{pmatrix} 0 & 0 & 3153.08 \end{pmatrix}$		
$[K] = \begin{pmatrix} 338474 + 163230 & -163230 \\ kN \end{pmatrix}$	 Initial stiffness matrix: 		
$\begin{bmatrix} \mathbf{K} \end{bmatrix} = \begin{pmatrix} -163230 & 163230 \end{pmatrix} \begin{pmatrix} \mathbf{m} \end{pmatrix}$ ★ Initial global stiffness matrix for the first iteration: $\begin{bmatrix} K^* \end{bmatrix} = \begin{pmatrix} 1+0.615 & -0.615 \\ -0.615 & 0.615 \end{pmatrix} k_1 \begin{pmatrix} kN \\ m \end{pmatrix}$ For the first iteration: $k_1^* = k_1 = 338474$ kN/m	$\begin{bmatrix} K \end{bmatrix} = \begin{pmatrix} 362800 + 318810 & -318810 & 0 \\ -318810 & 318810 + 189340 & -189340 \\ 0 & -189340 & 189340 \end{pmatrix} \begin{pmatrix} kN \\ m \end{pmatrix}$		
	For the first iteration: $k_1^* = k_1 = 362800 \text{ kN/m}$		
Step 2: Modal analysis (LS response spectrum)			

