DISPLACEMENT-BASED DESIGN OF TALL BUILDINGS STIFFENED WITH A SYSTEM OF BUCKLING RESTRAINED UNBONDED BRACES

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ABSTRACT :

A displacement-based methodology for the preliminary design of a system of buckling-restrained braces is introduced. The methodology applies to the case of tall buildings, whose dynamic response is significantly influenced by global flexural behavior. The methodology is applied to the preliminary design of a twenty four-story building located in the Lake Zone of Mexico City. From the evaluation of the global mechanical characteristics of the building and of its seismic performance when subjected to ground motions generated in that zone, it is concluded that the proposed methodology yields an adequate level of seismic design.

KEYWORDS: buckling-restrained braces, displacement-based design, capacity curve

1. INTRODUCTION

Innovation in earthquake-resistant design has been directed towards the conception of structural systems, either traditional or innovative, that are capable of adequately limiting their level of structural and non-structural damage through the explicit control of their lateral deformation. An attractive option for response control is the development of passive energy dissipating systems. Within this context, the use of buckling-restrained (BR) braces is an attractive and fairly inexpensive solution to earthquake resistance. The idea behind a buckling-restrained brace is to fabricate a structural element that is able to work in a stable manner when subjected to compressive deformations. Because braces are normally able to behave in a stable manner when subjected to tensile forces, a buckling-restrained brace is capable of dissipating large amounts of energy in the presence of multiple yield reversals (Uang and Nakashima 2004).

This paper introduces, within the framework for the design of "damage-tolerant" structures (Wada et al. 2004), a displacement-based design methodology for the preliminary design of a system of BR braces for tall buildings.

2. DESIGN METHODOLOGY

The methodology offered in this paper is based on the conception of a building whose gravity forces are carried by steel frames with standard detailing (as opposed to ductile), and whose earthquake-resistance is provided by a system of buckling-restrained braces that provides lateral stiffness and a large energy dissipation capacity.

2.1 Design Scope

Under the effect of low intensity ground motion, the building exhibits adequate performance if it satisfies the immediate operation performance level. This implies that the gravitational and bracing systems should not exhibit significant structural damage, and that the non-structural system should remain undamaged. Regarding performance for severe ground motion, the gravitational system should satisfy the immediate operation performance level while the bracing system develops significant plastic behavior that allows it to dissipate a large percentage of the input energy; partial or total non-structural collapse should be avoided.

In terms of modeling, the methodology assumes that the total lateral stiffness of the building can be estimated by adding the lateral stiffnesses provided by the gravitational and bracing systems, and that the lateral deformations due to the overall shear and flexural behaviors of the bracing system are independent and produced, respectively, by the axial deformation of braces and their support columns (the latter should remain elastic). Under these two assumptions, it is possible to formulate a simple model that considers that the structural system of the building can be modeled by means of two parallel earthquake-resistant systems. In turn, the bracing system can be modeled as two systems working in series: one that represents the global shear stiffness provided by the braces, and another one that represents the global bending stiffness provided by their support columns. This implies that the overall shear and flexural deformation in the bracing system can be controlled through the adequate dimensioning of the BR braces and support columns, respectively.

In what follows, several subscripts are used to address the different sub-systems that integrate the total structural system. Subscripts *GS* and *BR* refer to the gravitational and bracing systems, respectively. Subscripts *S* and *B* will refer to the overall shear and bending properties of the bracing system, respectively; and subscript *T* will refer to the entire structural system of the building.

2.2 Preliminary Design

The methodology introduced herein, applicable to standard occupation buildings and schematically shown in Figure 1, considers two performance levels: immediate operation (IO) and life safety (LS). Its first step implies establishing a qualitative definition of adequate performance. The second step consists of the quantification of adequate performance through establishing response thresholds. During the third step, the methodology establishes, through the use of displacement spectra, the design value for the fundamental period of vibration of the building, which quantifies the design lateral stiffness. The sizing of the BR braces and its support columns is established according to the value of this period.

Regarding the quantification of performance for IO, the gravitational system satisfies its structural performance criteria if it remains elastic. In the case of the bracing system, it may develop incipient plastic behavior. Non-structural performance is satisfied if the maximum inter-story drift index (IDI^{IO}) does not exceed the threshold associated to initiation of damage in the non-structural system (IDI_{NS}^{IO}). LS is satisfied if the maximum inter-story drift index (IDI^{LS}) is limited according to: 1) Immediate operation of the gravitational system (IDI_{GS}^{IO}), and 2) Prevention of non-structural local collapse (IDI_{NS}^{LS}).

Numerical design starts with the conception and design of the gravitational system. The system is designed to exclusively resist the gravitational loads. Standard detailing should be used for the gravitational frames. Once the gravitational system is established and designed, a nonlinear static (pushover) analysis is carried out to estimate IDI_{GS}^{IO} and its fundamental period of vibration (T_{GS}). The next step establishes design thresholds for the total roof displacement. This implies, as shown in Figure 1, establishing independent thresholds for the roof displacements produced by overall shear and bending behaviors of the bracing system. Part of this step is to define how the total roof displacement will be accommodated by these behaviors through the definition of parameter η, which establishes, for each performance level under consideration, the ratio between the roof displacements due to overall shear and bending.

Regarding the thresholds of roof displacement due to overall shear for IO and LS:

$$\delta_S^{IO} = \frac{IDI_S^{IO} H}{COD^{IO}} \quad and \quad \delta_S^{LS} = \frac{IDI_S^{LS} H}{COD^{LS}} \tag{2.1}$$

where δ_S is the maximum roof displacement in the bracing system due to overall shear, IDI_S the corresponding design threshold for inter-story drift, H the total height of the building, and Table 2.1 summarizes suggested values for *COD*.

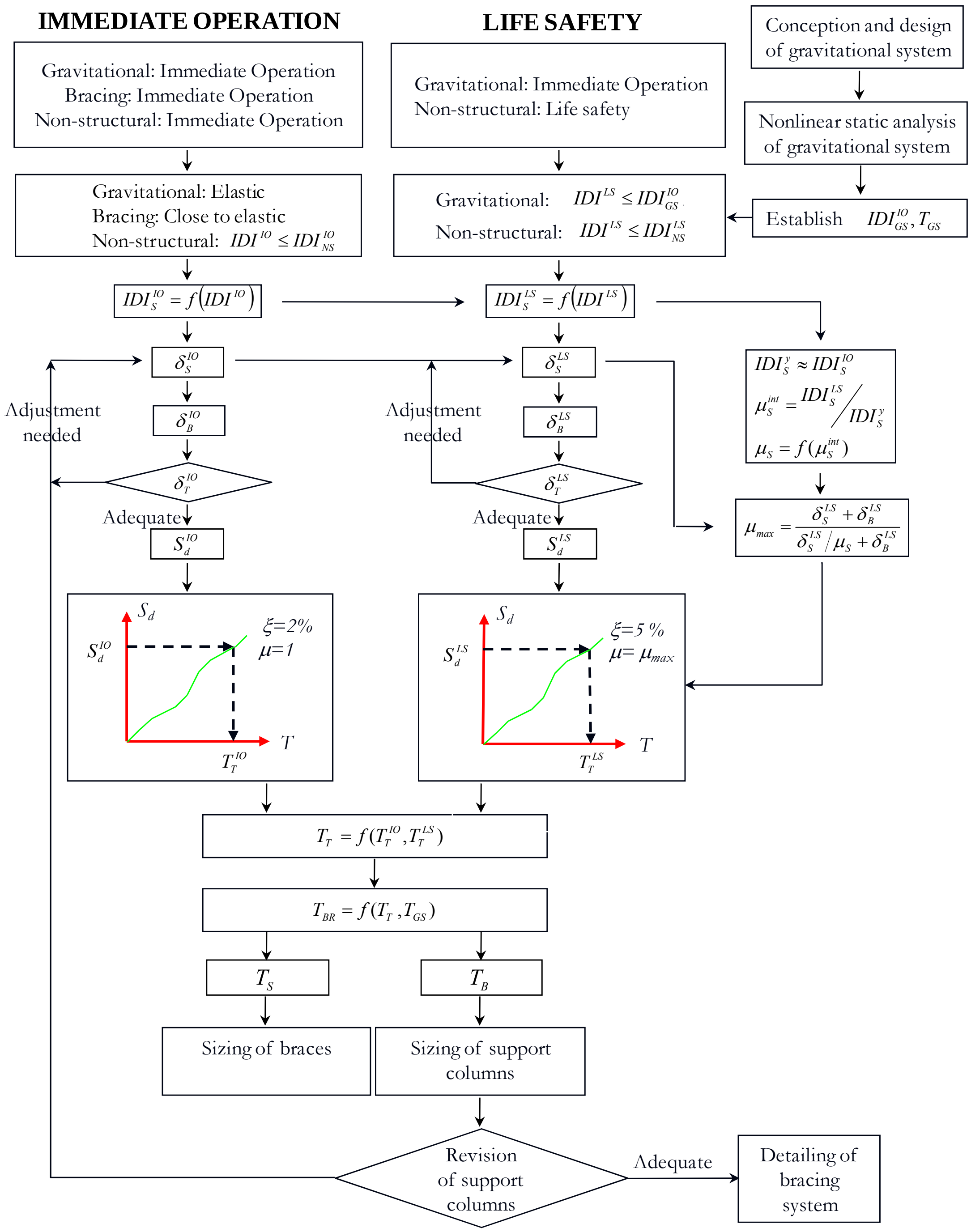


Figure 1. Preliminary design methodology

Table 2.1 Values of *COD*

Global Ductility	Stiffness Distribution through Height		
	Regular	Irregular	Highly Irregular
1	1.2	1.5	> 1.5
2+	1.5	2.0	> 2.0

The design thresholds IDI_S^{IO} and IDI_S^{LS} in Eqn. 2.1 are equal to IDI^{IO} and IDI^{LS}, respectively, in cases in which the global bending behavior of the gravitational and bracing systems are coupled. In case these systems are uncoupled in terms of overall bending, the columns of the gravitational systems exhibit negligible increase in their axial deformations, in such manner that the gravitational system accommodates its lateral deformation entirely through inter-story shear deformation. Because the bracing system accommodates the total lateral displacement through overall shear and bending behaviors, its thresholds for inter-story drift due to shear behavior are given by:

$$IDI_S^{IO} = \frac{\eta^{IO} IDI^{IO}}{\eta^{IO}+1} \quad and \quad IDI_S^{LS} = \frac{\eta^{LS} IDI^{LS}}{\eta^{LS}+1} \tag{2.2}$$

The displacement thresholds due to overall bending are established as a function of their respective thresholds due to overall shear:

$$\delta_B^{IO} = \frac{\delta_S^{IO}}{\eta^{IO}} \quad and \quad \delta_B^{LS} = \frac{\delta_S^{LS}}{\eta^{LS}} \tag{2.3}$$

and the total roof displacement thresholds are estimated by adding both components:

$$\delta_T^{IO} = \delta_S^{IO} + \delta_B^{IO} \quad and \quad \delta_T^{LS} = \delta_S^{LS} + \delta_B^{LS} \tag{2.4}$$

Once the thresholds for total roof displacement are established, they are checked according to: A) The possibility of impact against neighboring structures, and B) The comfort level of occupants. In case the values of the total roof displacements are considered adequate, the methodology proceeds to the next step. The fundamental period of vibration of the building can be estimated through the use of the total displacement thresholds and displacement spectra corresponding to the performance levels under consideration. For this purpose, δ_{IO} and δ_{LS} should take into consideration multi-degree-of-freedom effects:

$$S_d^{IO} = S_{dS}^{IO} + S_{dB}^{IO} = \frac{\delta_S^{IO}}{\alpha_S} + \frac{\delta_B^{IO}}{\alpha_B} \quad and \quad S_d^{LS} = S_{dS}^{LS} + S_{dB}^{LS} = \frac{\delta_S^{LS}}{\alpha_S} + \frac{\delta_B^{LS}}{\alpha_B} \tag{2.5}$$

where δ represents the total roof displacement; S_d, the pseudo-displacement; and the subscripts S and B denote overall shear and bending behavior, respectively. Table 2.2 presents suggested values for α.

Table 2.2 Suggested values for α for regular buildings

Number of stories	Shear (α_S)		Bending (α_B)
	$\mu = 1$	$\mu = 2+$	$\mu = 1$
1	1.00	1.00	1.00
2	1.20	1.10	1.20
3	1.30	1.20	1.30
5	1.40	1.20	1.40
10	1.40	1.20	1.50
15	1.40	1.20	1.55
20+	1.40	1.20	1.60

In the case of LS, the methodology requires an approximate estimate of the total maximum ductility demand (μ_{max}) of the building. For this purpose, it is necessary to establish the inter-story drift due to overall shear at which the bracing system yields: IDI_S^y. The yield stress of the braces should be designed in such a manner that IDI_S^y is as close as possible to IDI_S^{IO}. The inter-story ductility for the braces (μ_S^{int}) is established by normalizing IDI_S^{LS} by IDI_S^y. Then a global value of ductility due to overall shear behavior, μ_S, is assigned to the bracing system as a function of μ_S^{int}, the structural regularity of this system and the number of stories of the building. To estimate the total maximum ductility demand in the building, it is necessary to consider that the support columns are designed to remain elastic:

$$\mu_{max} = \frac{\delta_S^{LS} + \delta_B^{LS}}{\delta_S^{LS}/\mu_S + \delta_B^{LS}} \tag{2.6}$$

As illustrated in Figure 1, with the pseudo-displacement threshold for a given performance level, a value of period is determined through the use of the corresponding displacement spectrum. The values of period established for IO and LS are denoted T_T^{IO} and T_T^{LS}, respectively. Percentages of critical damping of 2 and 5% are considered for the IO and LS spectra, respectively. The design value for the fundamental period of vibration of the building (T_T) is that that leads to the most favorable condition in terms of lateral displacement demands. Under the assumption that the gravitational and bracing systems work in parallel, the fundamental period of vibration for the bracing system (T_{BR}) can be determined as:

$$\frac{1}{T_{BR}^2} = \frac{1}{T_T^2} - \frac{1}{T_{GS}^2} \tag{2.7}$$

Through the consideration that the lateral stiffnesses that the braces and support columns should provide is inversely proportional to the lateral displacements that the bracing system can accommodate due to overall shear and bending, respectively, the periods for which the braces and support columns should be sized (T_S and T_B, respectively) are:

$$T_S = \min\left(\frac{T_{BR}}{\sqrt{1 + \frac{S_{dB}^{IO}}{S_{dS}^{IO}}}}, \frac{T_{BR}}{\sqrt{1 + \frac{S_{dB}^{LS}}{S_{dS}^{LS}/\mu_S}}} \right) \quad and \quad T_B = \min\left(\frac{T_{BR}}{\sqrt{1 + \frac{S_{dS}^{IO}}{S_{dB}^{IO}}}}, \frac{T_{BR}}{\sqrt{1 + \frac{S_{dS}^{LS}/\mu_S}{S_{dB}^{LS}}}} \right) \tag{2.8}$$

The stiffness-based sizing of the braces and support columns, which should follow as closely as possible the story shear and overturning moment distributions through height, respectively, should result in that the actual fundamental periods of vibration of the bracing system due to overall shear and bending behaviors are equal or slightly smaller than T_S and T_B, respectively. The methodology considers the revision of the axial strength of the support columns. A capacity design approach should be followed to check if the columns are capable to accommodate within their elastic range of behavior the vertical components of the axial forces acting on the braces.

3. GRAVITATIONAL SYSTEM

The building used to illustrate the application of the methodology (Figure 2) has twenty four stories, and a 45 by 45 meter plan. The inter-story heights are 4.5 meters, except for the four lower stories, which exhibit heights of 4.0, 5.65, 5.65 and 6.0 meters, and for the two top stories, which exhibit heights of 6.0 and 6.5 meters. Overall, the building has a total height of 114.8 meters. The building has four central bays of 9 meters, two lateral bays of 4.5 meters, and seven frames in each one of its principal directions. As shown in Figure 5, the two central bays of the three internal frames are braced. The building will be assumed to be located in the Lake Zone of Mexico City. The frames of the building were designed to accommodate the gravitational loads that act on them according to the Mexico City Building Code. A50 steel was used (f_y = 3515 kg/cm^2). Table 3.1 shows the structural shapes selected for the structural elements of the gravitational system.

A nonlinear static analysis of the gravitational system was carried out using DRAIN 2DX (Prakash et al. 1993). A fundamental period of vibration of the gravitational system (T_{GS}) equal to 8.42 seconds was estimated. A plastic rotation of 0.005 (IO threshold) was reached simultaneously in several beams of the frame for a roof displacement of 72 cm. The corresponding inter-story drift (IDI_{GS}^{IO}) was close to 0.010.

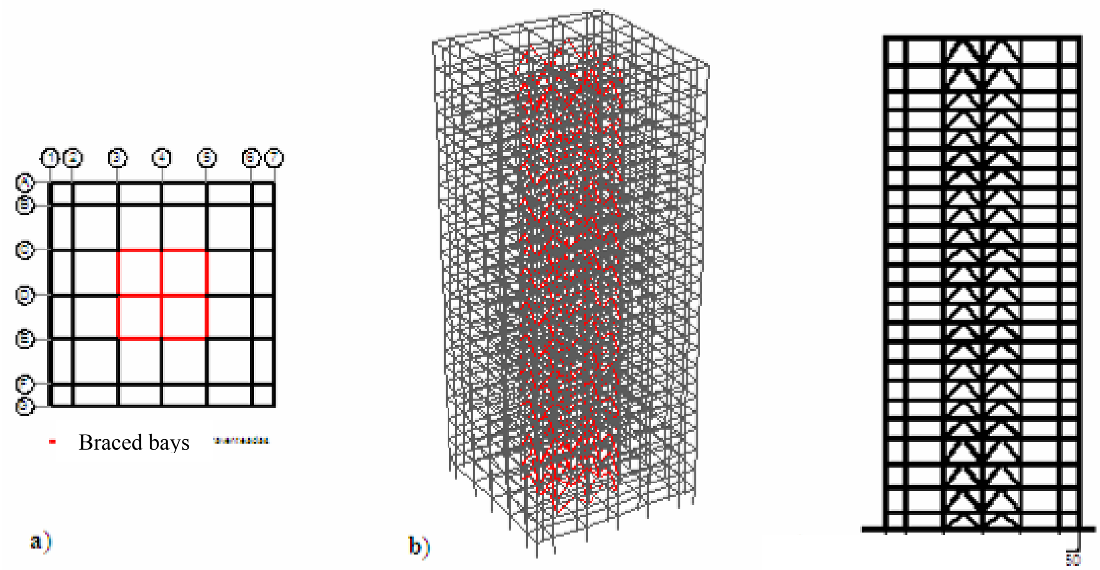


Figure 2. Geometry and structural layout of 24-story building. a) Plan, b) 3D model, c) Elevation

Table 3.1 Structural shapes of gravitational system

Stories	End columns	Intermediate Columns	Central Columns	Beams (9 m bays)	Beams (4.5 m bays)
17-24	W14x68	W14x176	W14x211	W16x50	W12x22
9-16	W14x132	W14x342	W14x426	W16x50	W12x22
5-8	W14x257	W14x550	W14x665	W16x50	W12x22
1-4	W14x257	W14x550	W14x665	W18x65	W12x22

4. BRACING SYSTEM

Once the gravitational system is established and its structural properties are estimated, the methodology proceeds to the sizing of the bracing system (BR braces and support columns). The design spectra for IO and LS are shown in Figure 3. While for IO the spectrum corresponds to elastic behavior and 2% of critical damping, the design spectrum for LS corresponds to a ductility of two and 5% of critical damping. The design spectrum for each performance level corresponds to the mean and one standard deviation spectrum derived from a set of motions recorded in the Lake Zone of Mexico City and having an intensity that is consistent with the performance level under consideration.

It was assumed that the inter-story drift indexes associated to initiation of damage and complete damage of nonstructural elements (IDI_{NS}^{IO} and IDI_{NS}^{LS}, respectively) are equal to 0.003 and 0.010, respectively. If the performance requirements for the gravitational system (IDI_{GS}^{IO}) are superimposed to those of the non-structural system (IDI_{NS}^{IO} and IDI_{NS}^{LS}), the design inter-story drifts are $IDI^{IO} = 0.003$ and $IDI^{LS} = 0.010$. If preliminary design values of one and three are considered for η^{IO} y η^{LS}, respectively; values of 0.0015 and 0.008 are obtained from Eqn. 2.2 for IDI_S^{IO} and IDI_S^{LS}, respectively. The minimum values for *COD* in Table 2.1 and Eqns. 2.1, 2.3 and 2.4 estimate values of 0.30 and 0.80 for δ_T^{IO} and δ_T^{LS}, respectively. This results according to Eqn. 2.5 and the values of α recommended in Table 2.2 in S_d thresholds of 0.20 and 0.63 m, respectively, for IO and LS.

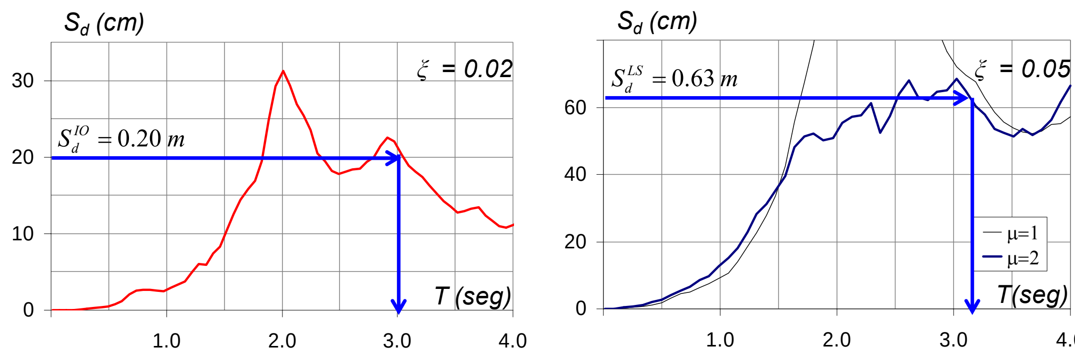


Figure 3. Design spectra for performance levels under consideration

Considering the geometry of the bracing system, an f_y of 2250 kg/cm^2 for the BR braces corresponds to an IDI_y^S of 0.015 and an inter-story shear ductility (μ_S^{ent}) of $IDI_S^{LS}/IDI_y^S =$ 0.008/0.0015 = 5.3. Considering the regularity of the bracing system and that the braces and support columns are sized to follow the story shear and overturning distributions through a height, an overall shear ductility (μ_S) of four is assigned to the bracing system. According to Eqn. 2.6, an overall maximum global ductility (μ_{max}) of 2.3 is estimated. According to Figure 3, a fundamental period of vibration (T_T) of 3.20 seconds is established. While Eqn. 2.7 yields a T_{BR} of 3.46 seconds, Eqn. 2.8 results in values of 2.45 and 2.37 seconds, respectively, for T_S and T_B. The final areas for braces and support columns that satisfy the stiffness requirements imposed by T_S and T_B are summarized in Table 4.1 together with the structural shapes selected for the support columns and beams.

Table 4.1 Areas of BR braces and support columns (cm^2)

Story	Area of BR braces	Area of support columns	Structural shapes for support columns	Structural shapes for support beams
24-21	37	190	2(W14x68)	W24x76
17-20	62	560	2(W14x176)	W24x84
13-16	77	1060	2(W14x342)	W24x94
9-12	86	1630	2(W14x550)	W24x104
5-8	89	2250	2(W14x730)	W24x117
1-4	90	2980	2(W14x730)+2(PL700x50)	W24x131

A nonlinear static analysis of a two-dimensional model of the braced building was carried out to evaluate its mechanical characteristics. As shown in Figure 4, a bilinear idealization of the capacity curve results in a roof displacement at yield of 37 centimeters, and a global ductility of 2.2 for a roof displacement of 80 centimeters (μ_{max} = 80/37 = 2.2). The fundamental period of vibration estimated for the building is 3.0 seconds. The actual values of μ_{max} and period show reasonable correspondence with the values of 2.0 and 3.2 contemplated for them, respectively, during the preliminary design of the bracing system.

To evaluate the seismic performance of the braced building, a series of nonlinear time-history analyses were carried out. The motions used for this purpose are those included in the sets used to establish the design spectra shown in Figure 3. Figure 4a shows, for each motion under consideration, envelopes for the inter-story drift distributions through height for IO. The wide line shows the mean plus one standard deviation (σ) of these envelopes. Regarding roof displacement demands, the nonlinear dynamic analyses estimate a mean + σ value of 28 centimeters, which is slightly smaller than the design threshold of 30 centimeters. As shown in the figure, the required performance is practically satisfied in terms of inter-story drift demands. All structural elements of the gravitational and bracing systems remained elastic. Figure 4b shows envelopes for the inter-story drift distributions through height for LS. Regarding roof displacement demands, the nonlinear dynamic analyses estimate a mean + σ value of 69 centimeters, which is smaller than the design threshold of 80 centimeters. As shown in the figure, the required performance is satisfied in terms of inter-story drift demands. The mean + σ value of inter-story ductility in the BR braces located at the critical story is 5.0, which is reasonably close to the value of 5.3 contemplated during preliminary design. Regarding the plastic rotation demands, the mean + σ value of the critical maximum plastic rotation for the gravitational system is 0.0039, which is less than the design threshold of 0.005 contemplated during preliminary design.

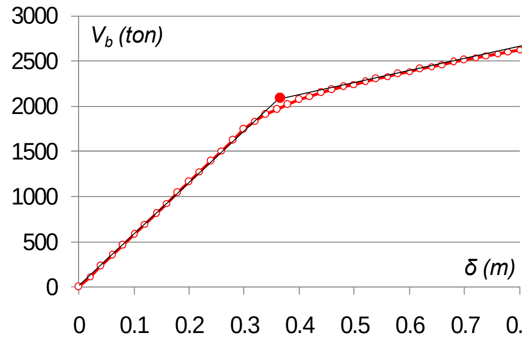
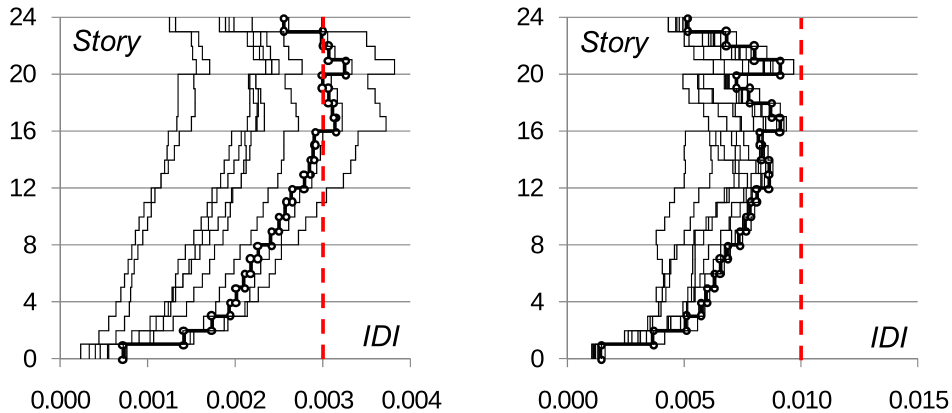


Figure 4. Capacity curve of braced building, nonlinear static analysis



a) Immediate operation b) Life safety

Figure 5. Distributions of inter-story drift for braced building

5. CONCLUSIONS

Within the context of a displacement-based seismic design methodology, the area of braces and support columns required for lateral stiffness should be determined as a function of the fundamental period of vibration required by the structure to control the level of damage in the gravitational and non-structural systems. The application of a displacement-based methodology to a twenty four-story building has given place to an adequate level of seismic design for immediate operation and life safety.

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