SEISMIC FORCE REDUCTION FACTOR FOR EQUIVALENT STATIC DESIGN OF CHEVRON-BRACED STEEL FRAMES

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ABSTRACT :

The Japanese seismic design code adopts a factor Ds for chevron-braced frames to consider the seismic force reduction due to the ductility. The Ds value varies from 0.25 to 0.5 according to the type of backup frame, the slenderness ratio of braces λ and the participation ratio β of braces. The selection of Ds in the design process is deemed rather complex. This study examines the ductility of some representative multi-story braced frames designed with different Ds, λ and β, and finds that the braced frames have a smaller deformation than the pure moment frames with the same Ds value. It is possible to use a unique value of Ds for braced frames.

KEYWORDS: Chevron-braced frame, Seismic force reduction factor, Participation ratio of brace, Slenderness ratio of brace, Equivalent single-degree-of-freedom model

1. INTRODUCTION

The chevron-braced steel frame is a typical seismic-resistant structure, in which a pair of braces is pinned to a moment frame to provide lateral stiffness and strength through the axial deformation coupling with the lateral story displacement. However, the brace subjected to the compressive load tends to buckle, making the strength decreased significantly in the post-buckling stage. In consideration of this, the Japanese seismic code (BCJ) [1] stipulates a higher strength or a smaller ductility demand for the chevron-braced frames than the moment resisting frames, where a factor Ds is defined as the reduction factor from the elastic response spectrum. The Ds value is given as 0.25 for the most ductile moment resisting frames, while varies between 0.25 and 0.5 for the chevron-braced frames in accordance with the type of the backup moment resisting frame, the participation ratio β (a factor that equals the ratio of the shear sustained by the braces to the total shear strength for each story) and the slenderness ratio λ of braces. However, if taking the contribution of the tensile brace into account, the ductility of braced frames may not be necessarily small.

In this study, the ductility of braced frames is examined by model-based studies using multi-story structures which are designed following the typical Japanese design procedure using different Ds, β and λ. To overcome the influence from the uncertainties introduced during the seismic design, such as the over-strength ratio and the overturning-introduced secondary axial force, an equivalent model with a single degree of freedom (ESDOF) is constructed for each multi-story structure by assuming the first vibration mode domination. Time history analyses are conducted for each ESDOF model and multi-story braced frames. The ESDOF model is first validated by comparing the roof drift angles of three-story braced frames with those of the corresponding ESDOF models. Then the maximum drift angles of braced frames are compared with that of the pure moment frame with the same Ds value. Finally, the observation from ESDOF models is further demonstrated by a series of six-story braced frames.

2. TYPICAL DESIGN OF PROCEDURE OF CHEVRON-BRACED FRAME IN JAPAN

In this section, Japanese design procedure for a typical chevron-braced frame is introduced using a three-story chevron-braced frame as an example. The three-story chevron-braced frame is shown in Fig.1, which contains five bays. The height of each story is designated as h, and each bay has a width of L. The gravity is sustained by each column, while the horizontal seismic force is resisted by the two exterior pieces of braced frames, as shown in Fig.1.

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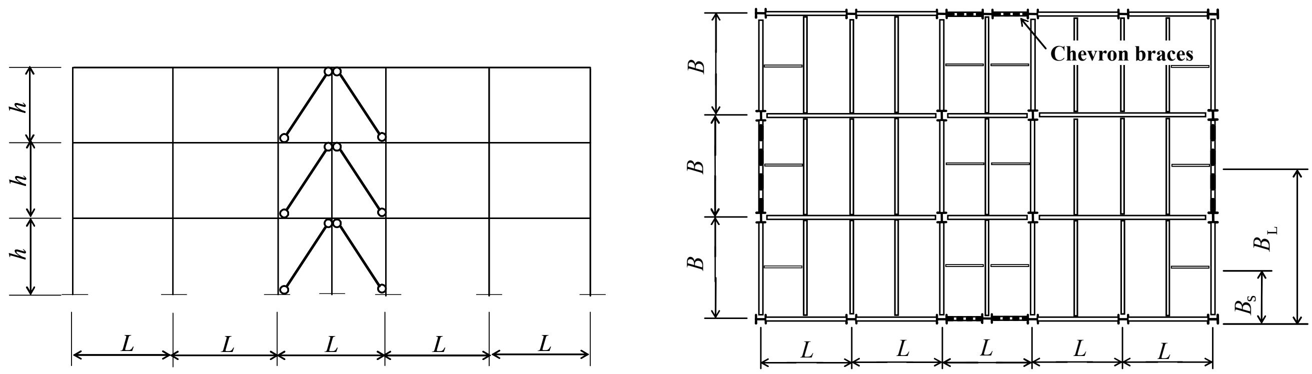
The gravity load of the selected piece of braced frame is calculated by the weight in the range of width B_S for permanent loading, while the horizontal seismic force is computed using the weight in the range of width B_L. A vertical column is inserted in the middle span of each story to sustain the vertical load transferred from the tensile brace due to the buckling of the compressive brace. The same seismic force reduction factors Ds, participation ratios β and slenderness ratios λ for braces are adopted for all stories.



(a) Elevation view (b) Plan view

Figure 1 Three-story chevron-braced frame

The BCJ employs a two-level design approach, i.e. Level I for serviceability and Level II for safety. The Level II earthquakes have an exceedance probability of 10% in 50 years, with a return period approximately 475 years. The Level II seismic force is often reduced by the factor Ds. Under the reduced seismic forces, the structure is allowed to enter plastic range, and all stories shall have enough resistance larger than the seismic demand. The seismic demand is given as:

$$Q_{un}^i = D_s^i \, F_{es}^i \, Q_{ud}^i \tag{2.1}$$

Where, Q_{un}^i is the seismic demand for the i th story; D_s^i is the seismic force reduction factor at the story i; F_{es}^i is the shape factor at the i th story, which is chosen as 1.0 for the multi-story frames with a regular and uniform stiffness distribution; Q_{ud}^i is the Level Ⅱ story shear at the i th story, given by the following formula:

$$Q_{un}^i = C_i \sum_{j=i}^{n} W_j \tag{2.2}$$

Where, n is total number of stories above the base; C_i is the Level Ⅱ story shear factor at the i th story. The backup frame and the braces combine to provide the horizontal resistance to the seismic force, which sustain $(1-\beta)Q_{un}$ and βQ_{un}, respectively.

In this study, the tensile strength provided by one of the braces is considered, and the ultimate strength can be treated as the sum of the yielding strength of the tensile brace and the post-buckling strength of the compressive brace. The post-buckling strength of the compressive brace is approximated as 0.3 time of the yielding strength as discussed by Marino et al. [2]. Thus, the ultimate strength is given as Eqn.2.3:

$$\beta Q_{un} = (N_y + N_u) = 1.3 A_b f_y \cos\theta \tag{2.3}$$

Where, N_u is the post-buckling strength; N_y is the yielding strength; A_b is the sectional area of tubular steel for braces; f_y is the yielding strength of steel braces; and θ is the angle of inclination of the brace.

The frame is supposed to deform in a shear mode and the inflection point of one column is assumed to be located at the mid-height. Provided that the exterior columns have a half contribution as that of interior columns, the strength capacity for one column in story i can be formulated as:

$$M_{p,c}^{i}=\frac{(1-\beta)Q_{un}}{m}\cdot\frac{h}{2} \quad (2.4)$$

where, m is the number of columns at each story. At one connection, the beams are supposed to have the same strength capacity as the associated columns and all beams are equally treated so that the strength capacity of one beam at story i should be:

$$M_{p,b}^{i}=\max\left(\frac{M_{p,c}^{i}+M_{p,c}^{i-1}}{2},M_{p,q}^{i}\right) \quad (2.5)$$

Where, $M_{p,b}^{i}$ and $M_{p,c}^{i}$ are the plastic moments for the beam and column at the i th story, respectively; and $M_{p,q}^{i}$ is the end moment introduced by the permanent gravity load. The strength of the column need be enlarged to be $1.5M_{p,c}^{n}$ to ensure all plastic hinges formed only at the two ends of beams.

Finally, with the selected sections, the designed structure should be examined to ensure all members elastic and the story drift angle lower than 1/200 under the Level I seismic force, which is 20% of the Level II unreduced design forces.

3. SIMPLIFICATION TO ESDOF SYSTEM

The uncertainties introduced during the design procedure, such as the gravity-governed over-strength of column and the axial force of columns induced by the overturning moment, would result in difficulties in the parameter study. To focus on the study of the three basic design parameters Ds, β and λ, a simplified model is proposed, as shown in Fig.2. It is a one-story one-bay chevron-braced frame which dynamically contains a single degree of freedom, and is statically equivalent to the corresponding multi-story chevron-braced frame.

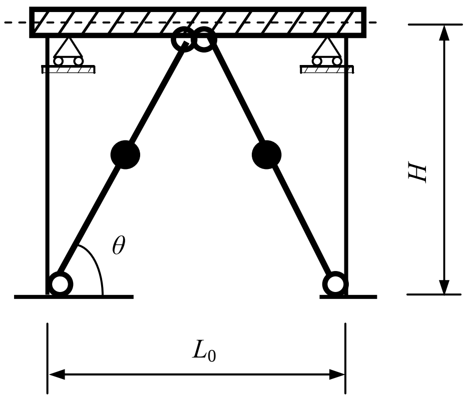


Figure 2 ESDOF system

Several assumptions are adopted to achieve the similitude between the ESDOF model and the multi-story chevron-braced frame, given as follows: (1) The weight of the ESDOF system, evaluated for the seismic design situation, is the total weight of the multi-story frame; (2) The dynamics of the multi-story frame is dominated by the first vibration mode, and the natural period of the ESDOF system is identical as the natural period of the multi-story frame; (3) The ESDOF model has the same design parameters: Ds, β and λ, as those adopted for the corresponding multi-story frame; (4) The beam of ESDOF system is rigid so that the seismic force is sustained by the two columns and the pair of braces; (5) The varying axial forces in the columns are not considered in the ESDOF model by assigning the vertical restraints at the both ends of the beam, because of the small P-Δ effect associated with multi-story braced frames.

3.1. Sectional Design of ESDOF System

A rectangle section is adopted for both columns with the width of B_{col} and the depth of h_{col}, and the braces have a pipe section with the diameter of D_b and the thickness of t_b. Other two unknowns to be determined are the height H and the inclination angle of braces θ, as shown in Fig.2.

Given the design parameters Ds and β, the strength of the two columns and the pair of braces should be calculated by the following equations:

$$Q_{f,un}=\frac{4M_c}{H}=(1-\beta)\cdot Ds\cdot W \quad (3.1)$$

$$Q_{b,un} = 1.3 f_y A_b \cos\theta = \beta \cdot Ds \cdot W \tag{3.2}$$

where $Q_{f,un}$ and $Q_{b,un}$ are the seismic resistance of two columns and a pair of braces in the ESDOF system, respectively.

Using the stiffness assumption (2), the stiffness of the two columns and the pair of braces can be determined as follows:

$$K_f = \frac{24EI_c}{H^3} = \frac{4\pi^2}{T_f^2} \cdot \left(\frac{W}{g}\right) \tag{3.3}$$

$$K_b = \frac{2E_b A_b}{H} \sin\theta \cos^2\theta = \frac{4\pi^2}{T_b^2} \cdot \left(\frac{W}{g}\right) \tag{3.4}$$

where, K_f and K_b are the lateral stiffness of the two columns and the pair of braces, respectively; T_f and T_b are separately the fundamental periods of the backup frame and the pure chevron braces in the multi-story braced frame; W/g is the total seismic mass of the ESDOF system, which is located on the top of braces.

The slenderness ratio of braces λ is a given design parameter, which is defined by

$$\lambda = \frac{H}{\sin\theta \sqrt{\dfrac{I_b}{A_b}}} \tag{3.5}$$

where, I_b is the moment of inertia of brace.

The equivalent height H of the ESDOF system is given for a multi-story frame as [3]

$$\left(\frac{H}{Nh}\right)^2 = \frac{(N+1)(2N+1)}{6N^2} \tag{3.6}$$

where, N is total number of stories above the base in the prototype of three-story braced frame

3.2. Determination of T_f and T_b

In order to correlate the stiffness of the two columns and the pair of braces in the ESDOF model with the counterparts in the multi-story braced frame, the periods of the backup frame and the braces of the multi-story braced frame shall be provided, as T_f and T_b used in Eqn.3.3 and Eqn.3.4. To relate the periods with the concerned parameters, particularly Ds and β, the relationships are constructed using some specific multi-story braced frames, and plotted as the curves of periods with respect to the effective seismic force reduction factor $\bar{D}_{s,f}$ which is defined as Eqn. 3.8 for the backup frames and $\bar{D}_{s,b}$ as Eqn. 3.9 for the braces, respectively.

$$\bar{D}_{s,f} = (1-\beta) \cdot Ds \tag{3.8}$$

$$\bar{D}_{s,b} = \beta \cdot Ds \tag{3.9}$$

Following the ultimate state design procedures, some pure moment frames are designed with Ds ranging from 0.0 to 0.5. They sustain the same story weight as the braced frames to be explored. Then the periods of these pure moment frames can be obtained and plotted with respect to $\bar{D}_{s,f}$, as shown in Fig.3, where $\bar{D}_{s,f}$ equals to Ds for these pure frames. Note that when the seismic force reduction factor Ds of a braced frames is small, the member sections are governed by the long-term gravity, so that the same model is used for these cases with small $\bar{D}_{s,f}$, and the period curve keeps constant.

The braces are implemented as the braced frame with the backup frame removed, resulting in the following equation:

$$\frac{1}{T_b^2}=\frac{1}{T^2}-\frac{1}{T_f^2} \tag{3.10}$$

where, T is the fundamental period of the braced frame. Firstly, the pure braces are designed with the anticipated effective seismic force reduction factor $\bar{D}_{s,b}$ ranging from 0.025 to 0.5 regularly. Secondly, the same backup frame with the effective seismic force reduction factor $\bar{D}_{s,f}$ of 0.2 is given. Absolutely, other value of $\bar{D}_{s,f}$ can be selected from 0.1 to 0.5, which can be testified to has a same analysis result for the braces. Then, a group of braced frames can be designed with Ds ranging from 0.225 to 0.7 and the corresponding β obtained by Eqn.3.8 and Eqn.3.9. The slenderness ratio of braces λ is 100. Conducting eigen-analyses for the braced frames and the backup frame and using Eqn.3.10, the periods of braces can be calculated and plotted in Fig.3.

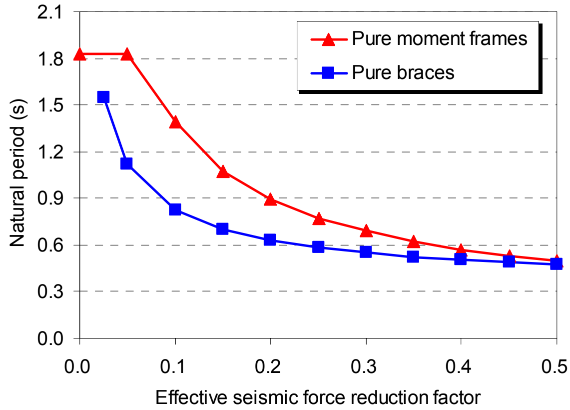


Figure 3 Natural periods of pure moment frames and pure

4. TIME HISTORY ANALYSES FOR ESDOF SYSTEM

Time history analyses are conducted for each ESDOF model and multi-story braced frames. 20 ground motions are adopted, which are developed by the FEMA/SAC project [4] with the exceedance probability of 10% in 50 years. The software called Open System for Earthquake Engineering Simulation (OpenSEES) developed by Pacific Earthquake Engineering Research Center [5] is employed. Each beam and column in the ESDOF models and multi-story braced frames is modeled by a nonlinear beam element with concentrated plasticity at both ends. The beam of the ESDOF model is treated to be rigid. Each brace is cut into two elements and one plastic hinge is inserted at the mid-length of the brace. The brace is pinned to the backup frame. An initial imperfection of 1/1000 of the brace length is assigned at the mid-length of the brace, and a small mass representing half weight of the brace is also attached at this point to provide a small lateral force during vibration. Stiffness proportional damping, with 2% for the first mode, is assigned for the fundamental vibration mode.

4.1. Three-story braced frames for model-based study

Four types of three-story chevron-braced frames (prototypes) are adopted, all having the basic configuration as shown in Fig.1. The configuration parameters are summarized in Table 4.1. Because of the length of the paper, the results are only shown for the third model, namely Model 3, hereinafter. The natural periods of the backup frames and the pure braces in Model 3 are shown in Fig.3 as well.

Table 4.1 Configuration parameters of four models

	B (m)	B_S (m)	B_L (m)
Model-1	10	5.0	15.0
Model-2	10	5.0	7.5
Model-3	7	3.5	10.5
Model-4	8	4.0	6.0

4.2. Validation of ESDOF models

The ESDOF model is validated in this section by comparing the natural periods and the roof drift with those obtained from multi-story braced frames. The natural periods of the three-story braced frames are shown in Table 4.2 compared with those obtained from the corresponding ESDOF models. Close similarity can be observed, and the largest difference of 3.5% occurs for the case with values $\beta = 0.9$ and $Ds = 0.5$ primarily due to the less accuracy of the curves in Fig.3 for larger β values. The roof drift angles are compared for the two models in Fig.4, where the drift angles with 84% possibility are drawn with respect to β. The two curves in each graph are basically similar only with the largest difference of 14%. The difference is found increasing with the β value primarily due to larger uncertainties introduced by the buckling of braces. Based on these two observations, the ESDOF model is believed to be able to represent the basic performance of the corresponding prototype.

Table 4.2 Natural periods of 3-story chevron-braced frames and ESDOF system

β	3-story braced frames (*Ds*)			ESDOF system (*Ds*)		
	0.3	0.4	0.5	0.3	0.4	0.5
0.0	0.689	0.565	0.498	0.690	0.566	0.498
0.1	0.647	0.549	0.477	0.653	0.550	0.476
0.3	0.608	0.516	0.461	0.609	0.525	0.467
0.5	0.584	0.505	0.451	0.589	0.515	0.466
0.7	0.572	0.508	0.462	0.574	0.501	0.472
0.9	0.557	0.516	0.490	0.546	0.502	0.473

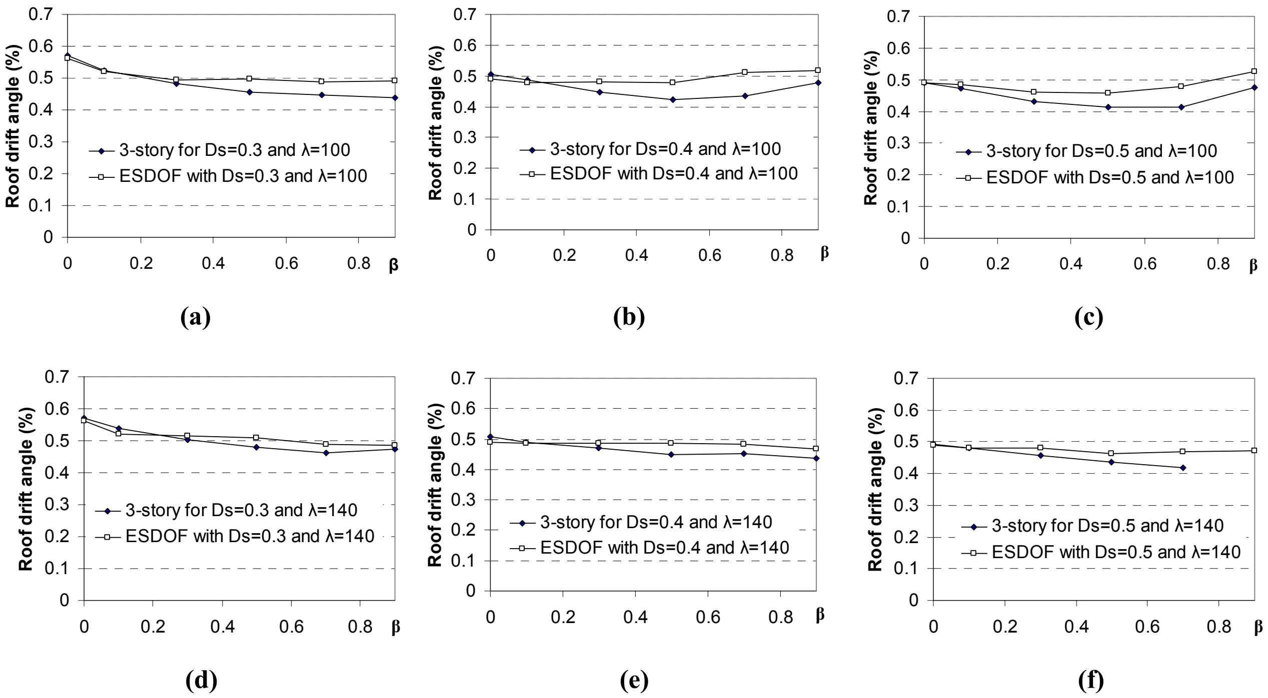
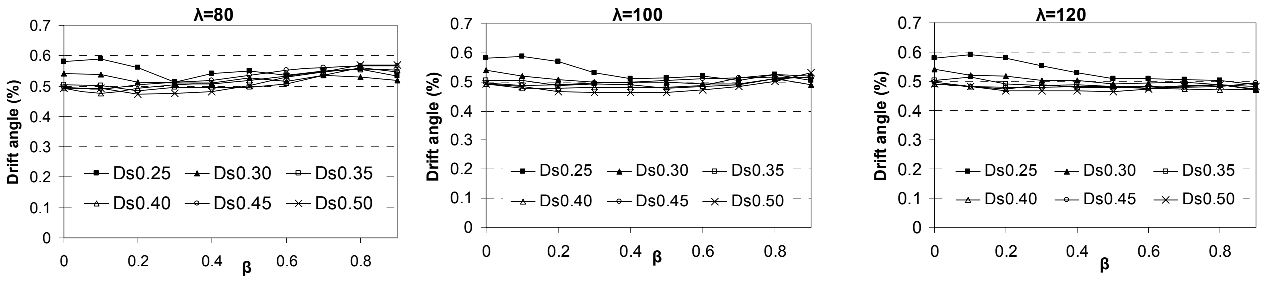


Figure 4 Comparison of roof drift angle between 3-story & ESDOF

4.3. Results of Time History Analysis for ESDOF System

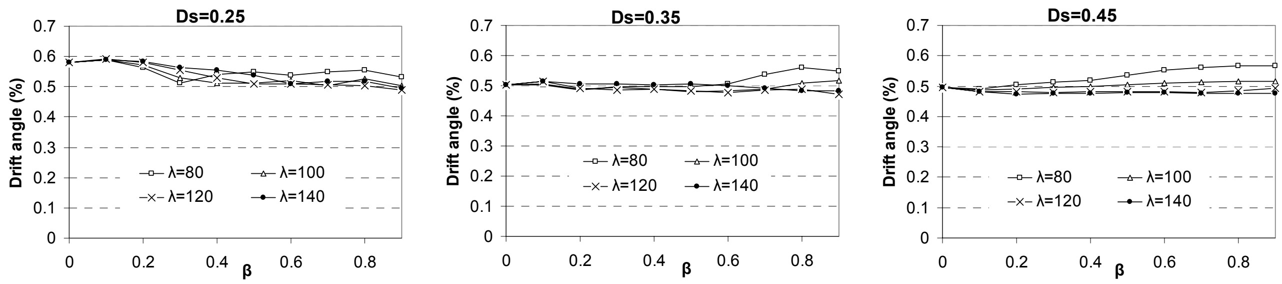
The results of time history analyses for ESDOF models corresponding to Model 3 are given in this section. The maximum inter-story drift angles are shown in Fig.5 with respect to the participation ratio λ of braces. It is observed that the inter-story drift angles of ESDOF systems with $\beta = 0$ (implying pure frame) are larger than those of ESDOF systems with $\beta \neq 0$.



(a) (b) (c)

Figure 5 Roof drift angle for ESDOF system with different Ds

It is also observed that the slenderness ratio basically has little effect on the performance of structures, as shown in Fig.6, where the curves representing different slenderness ratios λ are quite similar, in spite of having the maximal difference of 16% for ESDOF system with Ds =0.45 and $\beta = 0.8$. Therefore, it is possible to select the same Ds value for braced frames as for pure moment frames.

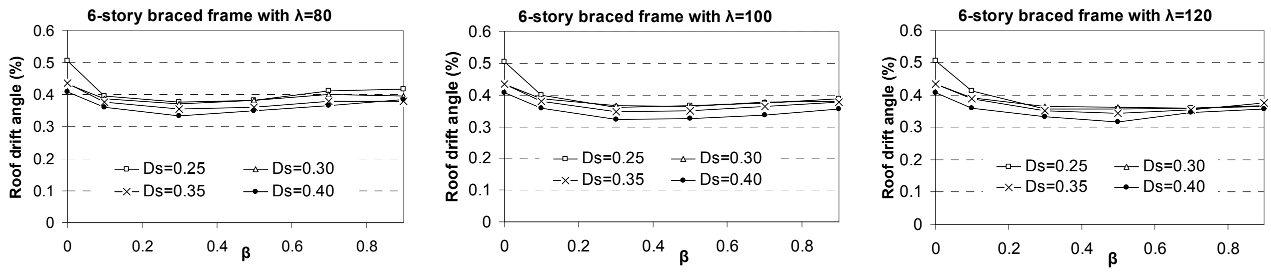


(a) (b) (c)

Figure 6 Roof drift angle for ESDOF system with different λ

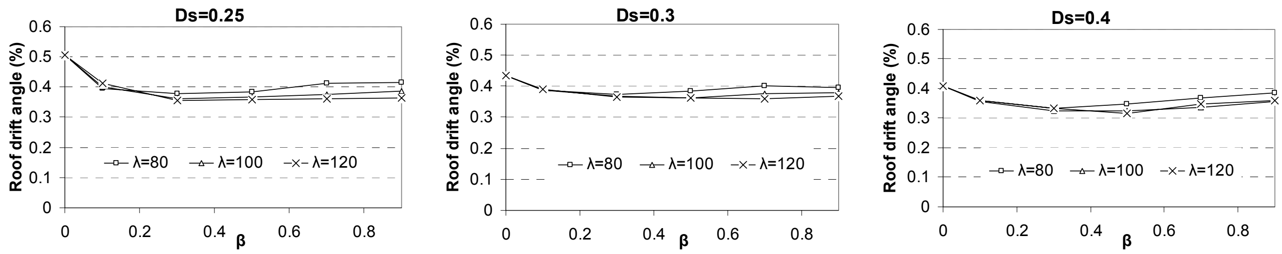
5. VALIDATION BY SIX-STORY CHEVRON-BRACED FRAMES

The above conclusion is further calibrated using a few six-story braced frames that are also designed following the procedures described in Section 2. The same set of ground motions as that used in Section 4 are employed here. The maximum roof drift angle with 84% probability is drawn in Fig.7 with respect to the participation ratio β. Each curve represents a group of frames with the same slenderness ratio λ and seismic force reduction factor Ds, but different participation ratios β. It shows that all pure 6-story frames have larger drifts than the 6-story braced frames with the same Ds, and λ do not affect the performance significantly, as shown in Fig.8.



(a) (b) (c)

Figure 7 Roof drift angle of 6-story braced frame with different Ds



(a) (b) (c)

Figure 8 Roof drift angle of 6-story braced frame with different λ

6. CONCLUSIONS

This paper examines the seismic performance of chevron-braced steel frames using simplified ESDOF models which represent the basic design parameters of the prototypes. The findings are obtained from the time history analyses for ESDOF models originated from some three-story braced frames and validated by the time history analyses for six-story braced frames. The major findings are shown as follows:

(1) An equivalent procedure is proposed to estimate the seismic performance of multiple-story chevron-braced frames. An ESDOF system that reliably represents the basic design parameters concerned in this study, i.e. the seismic force reduction factor, Ds, the participation factor, β, and the slenderness ratio, λ, is adopted. The similarity of seismic performance between the ESDOF model and the corresponding prototype is found acceptable.

(2) It is found that if considering the tensile contribution of the brace, most of the braced frames have smaller drifts than the pure frames that have the same seismic force reduction factor. The slenderness ratio has a neglectable effect on the seismic performance of the braced frames, so that it is possible to use the same seismic force reduction factor for the braced frames as the referenced pure frames.

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