QUANTIFYING EARTHQUAKE COLLAPSE SAFETY OF DUAL FRICTION-SLIDING BRACED FRAME BUILDINGS

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Abstract

The development of modern damage-resistant buildings has received greater attention in the aftermath of recent damaging earthquakes. It is envisioned that installing fusses for input energy dissipation, the structural system could mitigate weak-storey response, minimize residual drift, while having structural members remain free of damage.

This study presents the seismic performance of dual friction-sliding braced frames (D-FSBF) versus friction-sliding braced frames (FSBF). Although previous studies conducted on D-FSBF have proposed R_d values between 4 and 6, a comprehensive characterization has yet to be conducted. Herein, R_d is the ductility-related force modification factor. Since friction devices slide at lower interstorey drifts, a backup moment resisting frame (MRF) is recommended to be installed in parallel with the FSBF, to provide an elastic frame action while friction devices are in the sliding phase. Similar to other conventional steel frame systems, the backup MRF is proportioned to sustain 25% of the building's base shear while the FSBF is designed to resist 100%.

To investigate the collapse mechanism of the D-FSBF system, the effect of modelling assumptions and the simulation of members' behaviour are considered. A detailed friction device model should be able to simulate the slipping and bearing phases. When full-scale experimental tests were conducted on friction-sliding braces using Pall friction dampers, it was observed that friction devices resisted, in bearing, about two times their slip force, and the adjacent structural members were subjected to higher demand than assumed in the design. To mitigate this drawback, a limit for the increased slip force should be considered. The proposed numerical model has been calibrated against experimental test results.

The effect of long duration ground motions, on the seismic response of the structures, are also investigated through a case study consisting of an 8-storey prototype D-FSBF building located in Vancouver, B.C., Canada; which is within close proximity of the Cascadia subduction fault. The studied building is designed in reference to the provisions provided in the National Building Code of Canada (2015) and the Steel Design Standard (2014). Time-history analyses are performed using a detailed numerical model developed in the framework of the OpenSees environment. Incremental dynamic analysis was employed, and a multi-level seismic performance assessment was conducted to verify the margin of collapse safety. The investigated parameters are: interstorey drift, residual interstorey drift, and floor acceleration.

Thus, the purpose of this study is three-fold: i) to quantify the seismic coefficients (e.g. R_dR_0) of the D-FSBF system using FEMA P-695 procedure, ii) to emphasize the effects of the modelling assumptions by the verification of collapse safety using OpenSees and iii) to investigate the effect of long-duration earthquake shaking on the building's response.

The innovative features of this study are as follow: the characterization of D-FSBF system, the effect of long duration subduction earthquakes versus short duration crustal earthquakes on the proposed building system response, as well as, the modelling aspects to simulate the nonlinear behaviour and capture the collapse mechanisms.

Keywords: steel braced frame; friction damper; earthquake; nonlinear time-history analysis; collapse safety

1. Introduction

Concentrically Braced Frames (CBF) are popular earthquake-resistant systems possessing high stiffness and moderate ductility. Despite its easy installation, CBFs are prone to weak-storey response after braces buckle and lose their compressive strength while the mirrored braces act in tension. To overcome this asymmetric response and limit the magnitude of tensile forces developed in the braces, researchers have proposed ductile fuses to dissipate the input energy. Although friction dampers have been installed as ductile fuses in new and existing steel buildings, no extensive research was conducted to characterize the structural system. Decades ago, friction dampers were installed in steel moment frames in order to reduce the interstorey drift. Due to lack of guidance, friction dampers were also installed in-line with braces of CBFs to increase the system's ductility. However, the CBFs with friction dampers installed in-line with braces, labeled herein as friction sliding braced frames (FSBF), are prone to large residual interstorey drift and dynamic instability when subjected to seismic excitations. A feasible solution could be to design the FSBF as a dual system, labelled herein D-FSBF. However, design provisions for such structural systems are not available.

After the Christchurch earthquake in New Zealand, insurance policies have shaped the post-earthquake decisions of building structures. As a response, to reduce the economic loss, Pettinga et al. [1] concluded that residual interstorey drifts should be reduced or completely eliminated. To do this, low-damage systems with self-centering capabilities are recommended. In a study conducted by McCormick et al. [2], the authors concluded that it is less expensive to demolish and rebuild the building than to repair it if the structure exhibited residual interstorey drifts greater than 0.5% h_s, where h_s is the storey height.

To mitigate the storey mechanism response of conventional braced frames, researchers [3-4] proposed that an option could be to design the braced frames as a Dual system that embeds backup moment resisting frames (MRF). The addition of MRFs is justified by their capability to undergo large deformability in the elastic range while possessing the elastic frame action which can serve as a restoring force mechanism to partially re-center the building after an earthquake event. In [4] it was reported that designing Buckling Restrained Braced Frame (BRBF), as a Dual system, resulted in a small decrease in the braces ductility demand, as well as the system's peak interstorey drift. However, the addition of the backup MRF reduced significantly the residual interstorey drift. These findings were reported by analyzing a 3-storey and 6-storey building configurations. Similarly, in [4] it was shown that the amount of shear force assigned to the elastic MRF did not display differences in terms of its interstorey drifts. Nevertheless, the specific design provisions for a dual system are still limited.

The current National Building Code of Canada (NBCC) [5] allows a variety of seismic force resisting systems derived from a combination of two conventional systems. For example, in the Structural Commentaries of the NBCC [5], it is noted that Dual structural systems may be designed so that 100% of the seismic load is carried by the system having the higher value of R_dR_0, where R_d and R_0 are the ductility-related and overstrength-related force modification factor, respectively. If this design approach is followed, the other system, which is not considered to be part of the seismic force resisting system, SFRS, must be designed to support its gravity loads while undergoing earthquake-induced deformations.

The objective of this research is three-fold: i) to quantify the seismic coefficients (e.g. R_dR_0) of the D-FSBF system using the methodology presented in FEMA P-695 [6], ii) to emphasize the effects of the modelling assumptions by the verification of collapse safety using OpenSees [7] and iii) to investigate the effect of long duration earthquake shaking on the building's response.

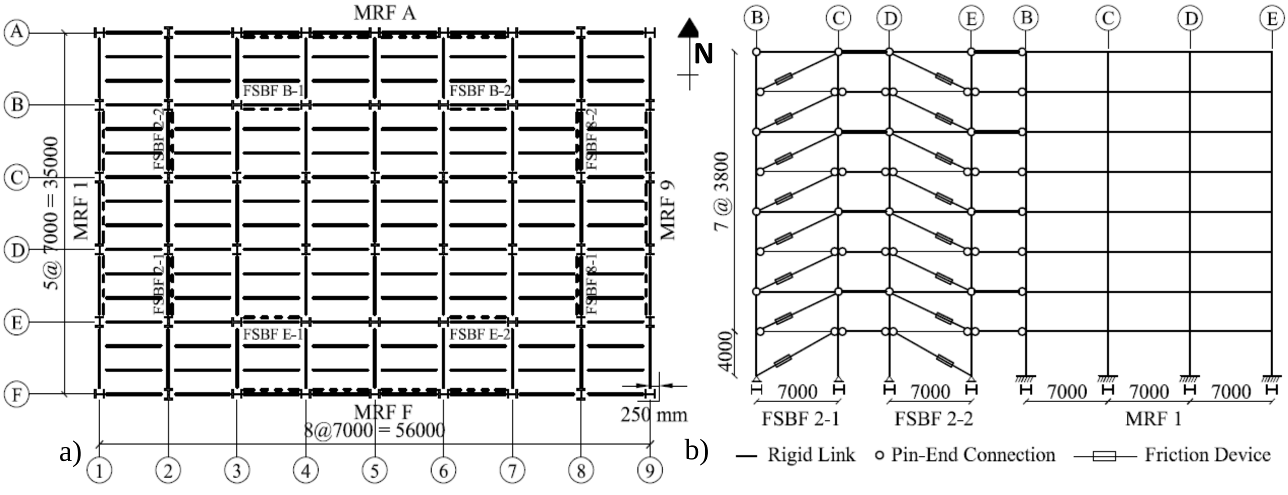
The innovative features of this study are as follow: the characterization of D-FSBF system, the effect of long duration subduction earthquakes versus short duration crustal earthquakes on the proposed building system response, as well as, the modelling aspects to simulate the nonlinear behaviour and capture the collapse mechanisms.

2. Prototype Building Models

An 8-story office building with a rectangular footprint, illustrated in Fig. 1a, was considered for investigation. The prototype building is located in Vancouver, British Colombia, Canada, on Site Class C (stiff soil). In this study, FSBFs and Dual FSBFs are analyzed from yielding to failure. Building Model #1 is braced in both orthogonal directions by four FSBFs and the elevation is plotted in Fig. 1b. The FSBF system is derived from the moderately ductile CBF system. The latter was designed to withstand seismic loads reduced by R_dR_0 =3x1.3 as per [5]. As depicted, each braced frame is located in one quadrant of building's plan and is placed symmetrically with respect to the center of mass. The braced frame configuration shows single diagonal braces per bay. All braces are made of hollow structural sections (HSS) and are designed to respond elastically in tension-compression. The upper end of each brace is connected to the frame through a traditional gusset plate and the lower end of each brace is connected to the Pall friction damper which is bolted to a gusset plate connected to the frame. As illustrated in Fig. 1d, the Pall friction damper is made of a middle plate that slides between two external channels bolted together by pretention bolts. The end of the middle plate is welded along the slotted holes of the HSS brace, and both channels are bolted to the gusset plate that is welded to the frame. It is noted that the ductility of CBFs, which is based on the capability of steel braces to yield in tension, is lower than that of friction damper devices. Nevertheless, the ductility of friction dampers is at least equal to that of buckling restrained braces (R_d = 4) [5], but they do not possess overstrength (R_0 =1.0). Thus, to design the FSBF system, the seismic loads are calculated based on R_d =4.0 and R_0 =1.0. It is noted that a value of R_d =4.0 can be accommodated if dampers possess adequate slip length.

To overcome the FSBF's drawbacks, presented later in this paper, the D-FSBF system is proposed to brace the Building Models #2 and #3. As depicted in Fig.1a, the MRFs are displaced symmetrically in both orthogonal directions. The design approach, used to proportion the D-FSBF system, is similar to that presented in [8] for ductile (steel) plate walls. Accordingly, friction dampers are designed to resist 100% of the applied factored storey shear forces and the backup MRFs are designed to resist at least 25% of the factored storey shears. The backup MRF is detailed as per the requirements of the moderately ductile MRF characterized by R_0 = 1.5 [5]. Thus, for the D-FSBF system, an overstrength factor of at least 1.125 can be proposed leading to R_dR_0 = 4.5. For comparison purpose, the design of the D-FSBFs, employed in Building Model #2, were labelled as D-FSBF(4) and is based on the seismic loads computed with R_dR_0 =4. In addition, the D-FSBF(5) system designed with R_dR_0 = 5 was also considered and used to brace the Building Model #3.

To quantify the building's seismic performance factors, the methodology presented in FEMA P695 [6]



DL_{roof} = 3.8 kPa	DL_{floor} = 4.0 kPa
LL_{roof} = 1.0 kPa	LL_{floor} = 2.4 kPa
SL_{roof} =1.64 kPa	$DL_{cladding}$ =1.0kPa

c)

d)

Fig. 1 - Building model geometry: a) plan view, b) elevation of FSBF and MRF, c) loads considered in design, d) detail of Pall friction damper from testing setup [9]

is applied to all studied Building Models, which have the same floor plan. Considering the building's symmetry in both orthogonal directions, the analysis is conducted for half of the building. In this paper, calculation is conducted in the N-S direction only. According to Fig. 1b, there are two FSBFs designed to carry half of the building's base shear and an associated three-span MRF. In the numerical model, described in a section below, both FSBFs and the MRF are connected with rigid links to simulate the rigid diaphragm effect of the floors. The length of the typical bay is 7.0 m, the height of ground floor and typical floors are 4.0 m and 3.8 m, respectively. The total building height, h_n is 30.6 m. All FSBF beams and braces are pinned connected at both ends and the FSBF's columns are pinned connected at their base and continuous over two storeys. All columns and beams are made of W-shapes with a nominal yield strength F_y= 345 MPa and tensile strength F_u= 450 MPa. All braces are made of hollow structural sections (HSS) and the friction dampers are illustrated in Fig. 1. The dead load and live load at typical floor and roof level, as well as, the snow load and cladding's dead load are given in Fig. 1c. The seismic weight (W) of each building including 25% of the snow load at the roof level is the same for all Building Models and equal to W= 62847 kN.

3. Design Procedure

The design procedure used for the FSBF system is not covered in the building code provisions [5]. To design the FSBF system of Building Model #1, a proposed force based design method [9], similar with that presented for CBFs is applied. According to [5] the seismic base shear is calculated as:

$$V= S(T_a)M_vI_EW/(R_dR_o) \tag{1}$$

where $S(T_a)$ is the 5% damped spectral response acceleration expressed in units of gravitational acceleration corresponding to $2T_a$, M_v is a factor accounting for higher modes effect on base shear, I_E is the importance factor for earthquake loads associated to building importance category, W is the seismic weight and R_dR_0 =4.0. For normal importance category buildings, I_E = 1.0 and from calculation M_v = 1.0. The fundamental lateral period in the direction of consideration is determine as $T_a=0.025h_n$ for braced frames, where h_n is the building height. For the case studied, h_n = 30.6 m and T_a = 0.765 s. When dynamic analysis is employed, T_a shall not be taken greater than $2T_a$ which in this example becomes 1.53 s.

The site-specific response spectral accelerations for Vancouver (Site Class C) at the specified periods of 0.2, 0.5, 1.0, 2.0 and 5.0 s are 0.848, 0.751, 0.425, 0.257 and 0.08 g, respectively. These values are based on a 2% probability of exceedance in 50 years (2475 year return period). The effect of torsion caused by accidental eccentricity and notional loads are neglected, but the P-Δ effect was considered. From Eq. (1), the base shear is equal to V = 5279 kN. However, the design base shear, V_d, resulting from dynamic analysis by means of modal response spectrum method shall not be taken less than $0.8V$ for regular buildings or as resulting from dynamic analysis if $V_d > V$ computed for $2T_a$. The first and second mode period evaluated from eigenvalue analysis in the N-S direction are given in Table 1. Using a 3-D model and ETABS software, T_1 is slightly lower than $2T_a$ and the base shear is about 10% greater than that computed with Eq. (1).

Table 1 - Characteristics of Building Models

Building Model	**R_dR_o**	**Static Equiv. Method**		**ETABS** Linear analysis			**OpenSees** Nonlinear analysis	
		$2T_a$ (s)	**V** (kN)	**T_1** (s)	**T_2** (s)	**V_d** (kN)	**T_1** (s)	**T_2** (s)
Building Model #1: FSBF	4x1.0=4.0	1.53	5279	1.40	0.44	5907	1.45	0.482
Building Model #2: D-FSBF(4)	4.0			1.34	0.44	5945	1.36	0.465
Building Model #3: D-FSBF(5)	5.0		4223	1.55	0.51	4349	1.54	0.508

The first step in the preliminary design is to compute the axial slip force, F_{slip}, triggered in each friction-sliding brace. The procedure is similar to that used to compute axial forces in traditional CBF braces. The friction damper is installed in-line with the brace and is only activated when F_{slip} is reached; otherwise the

FSBF system behaves as a traditional CBF. To ensure continuous and uniform sliding within the damper, braces should behave elastically without undergoing buckling. To guarantee elastic response of braces, a safety coefficient of 1.3 is applied in design. This safety coefficient was also recommended in ASCE/SEI 41 [10]. Hence, all HSS braces should be designed to carry a factored axial force of $1.3F_{slip}$ of the attached friction damper, while responding elastically in tension-compression. Thus, the brace's compressive resistance C_r should be larger or equal to the factored axial load, C_f. To size the beams and columns of FSBF and to ensure their elastic response, the capacity design method is applied. However, the process is not straightforward. Based on an experimental test conducted on a full scale Pall friction damper of 700 kN slip force explained in [9], the friction damper was able to resist a slip force greater than $2F_{slip}$. When friction damper was loaded further than its available stroke, the end bolts hit the end of the damper's slotted holes and the friction damper behaved in bearing. The stroke parameter results from design. To ensure safety seismic response, the stroke was multiplied by 1.3 safety factor and the value was provided to manufacturer.

The elements that dissipate the input energy are the friction dampers installed in-line with HSS braces subjected to tension-compression that should be Class 1 or 2. According to [8], the probable compression resistance of HSS brace, C_u, is $1.2C_r/\phi$, where C_r is computed using R_yF_y and ϕ=0.9. For HSS sections, the probable yield stress R_yF_y shall be taken not less than 460 MPa [8]. From calculation, it results that a force of $2F_{slip}$ corresponds approximately to the probable compression resistance of HSS brace. To preserve columns in the elastic range, they should be proportioned to carry the factored gravity loads and the brace forces corresponding to the brace's probable compressive resistance. Columns shall be continuous and of constant cross-section over a minimum of two storeys. Columns of braced bays shall meet the requirements of Class 1 or 2 beam-columns. For columns design, an additional bending moment in the direction of the braced bay of $0.2ZF_y$ in combination with the axial loads shall be considered, where Z is the plastic section modulus of the column's cross-section. Columns and beams are made of W-shape sections.

The D-FSBF system employed in Building Models #2, #3 was similarly designed. As aforementioned, the SFRS is composed of FBSFs and backup MRFs. The beams of the MRF were designed to possess sufficient flexural resistance such that 25% of the applied factored storey shear force is resisted by the MRF members. Both beams and columns of the MRFs are made of W-shape sections. Although a slightly larger overstrength-related force modification factor can be used (e.g. R_0 = 1.125), R_dR_0 = 4.0 was considered for Building Model #2. The D-FSBF of Building Model #3 was designed for seismic loads based on R_dR_0 = 5.0.

4. Analyse Seismic Response of Building Models

4.1 Numerical model

To investigate the nonlinear response of studied buildings, numerical models were developed in OpenSees [7]. Each model was built for half of building's floor area and the direction of earthquake application was considered in the N-S direction only. The FSBF system of Building Model #1 was geometrically modelled as two-dimensional. To simulate the behaviour of friction-sliding brace, with incorporated friction damper, the *twoNodeLink* element object was used and defined with a non-zero length and three degrees of freedom (translations along local *x, y,* axes and rotation about local *z* axis) for the *2D*-case. The *twoNodeLink* element has the length of the brace plus damper and each of its ends were connected to a rigid link that replicates the length of gusset plate connecting the sliding-brace member to the frame. Thus, the friction damper properties were assigned to a translational spring inserted in the *twoNodeLink* element. This translational spring is made of the uniaxial *BoucWen* material, which was selected to replicate the smooth hysteresis behavior of friction damper and is able to accommodate the development of high nonlinear Coulomb friction. The input parameters used in the definition of *BoucWen* material are: the initial elastic stiffness k_0, exponent n that influences the sharpness of the model in the transition zones, α which is the ratio of the post-yield stiffness to the initial elastic stiffness, and γ and β parameters controlling the shape of the hysteresis cycle. Other parameters such as A_0, A, ν and η control the stiffness and strength degradation. Herein, the above parameters were taken as $A_0 = 1$, $\alpha = A = \nu = \eta = 0$, while γ and β parameters were considered equal and calculated based on the equation: $\gamma + \beta = 1/(\Delta_y)^n$, where n =10. To simulate the bearing phase that occurs when the demand exceeds the damper's stroke, three parallel springs made of the *Elastic-Perfectly-Plastic Gap* material were added in the *twoNodeLink* element. The numerical model proposed for the Pall friction damper installed in-line with an HSS brace was calibrated against experimental tests results reported in [9]. Columns of FSBF were modelled as nonlinear *Force-Based Beam-Column* elements with distributed plasticity and fiber cross-section discretization. Each column was divided into 8 elements having an initial out-of-straightness equal to 1/1000 of its length and the W-shape column section was made of 120 fibers. Beams were modelled as elastic members. It is noted that a 2% Rayleigh damping was specified in the first and the third mode of vibration of studied buildings.

The OpenSees model developed for the MRF of D-FSBF system of Building Models #2 and #3 is similar to that presented in [11]. Thus, each MRF beam is made of the B*eamWith* H*inges* element with fiber-based cross-section discretization within the plastic hinge zone to which the *modified Gauss-Radau integration* scheme was assigned. The strength and stiffness deterioration caused by flange local buckling is simulated by assigning a calibrated low-cycle fatigue material model assigned to the flange fibers as per [11].

4.2 Ground motions

For Western Canada, the important contributions to hazard are moderate to large earthquakes of magnitudes M7 – M7.5 which are compatible with the design spectrum developed for a 2% probability of exceedance in 50 years. An additional contributor to hazard is the megathrust M9 earthquake that may occur along the Cascadia subduction fault. Although it is recommended that a minimum of 11 ground motion records be used for each suite, using fewer than 11 records for a suite is permitted provided that no less than five records are used for each suite and the total number of records in all suites is not less than 11 [5]. In this study, two suites of seven ground motions were selected to comply with the reference ground conditions of Site Class C, characterized by time-averaged shear wave velocity, V_{s30}, between 360 m/s and 760 m/s. The first suite contains crustal ground motions selected from the PEER-NGA ground motion database and the second contains subduction ground motions recorded during Tohoku earthquake in Japan (March, 2011). The characteristics of selected records are presented in Table 2, where NGA is the record identification. The magnitude of the earthquake events, the peak ground acceleration (PGA), peak ground velocity (PGV), the Trifunac duration (t_D), the principal period of the ground motion (T_p) and the main period of the ground motion (T_m) are also provided.

According to [5], ground motions are scaled such that the mean response spectrum of the suite does not fall more than 10% below the code design spectrum across the period of interest $0.2T_1 – 2T_1$, where T_1 is the first-mode period of the building. The response spectrum of all scaled records, their mean and the code design spectrum are plotted in Fig. 2, while the scaling factors, *SF* are given in Table 2. To match the code design spectrum across the period of interest, some subduction records should be scaled with a factor greater than 1.0. These records selected from the M_w9 earthquake are already very severe. For this reason, the subduction records were scaled carefully and it was accepted to let the mean response spectrum to drop slightly more than 10% below the code design spectrum for T > 2.0s. It is noted that the subduction records were selected to match the geotechnical profile for Site Class C in Vancouver and the distance to the Cascadia subduction fault.

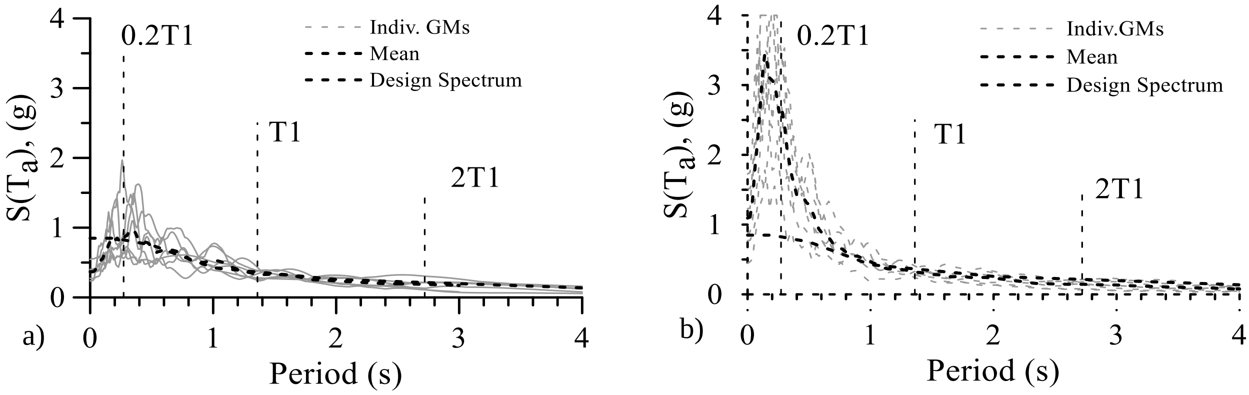


Fig. 2 - Response spectrum of ground motions scaled to match the design level: a) crustal, b) subduction

Table 2 - Ground motions

Crustal Ground Motions											
ID	NGA	Event	M_w	Comp (°)	PGA (g)	PGV (m/s)	t_D (s)	T_p (s)	T_m (s)	V_{s30} (m/s)	S.F.
C1	779	1989, Loma Prieta	6.9	000	0.525	0.917	10.15	0.70	0.80	417	0.45
C2	787	1989 Loma Prieta	6.9	360	0.277	0.313	11.61	0.30	0.69	425	1.50
C3	787	1989, Loma Prieta	6.9	270	0.207	0.314	12.66	0.80	0.88	425	1.32
C4	802	1989, Loma Prieta	6.9	090	0.321	0.434	8.02	0.22	0.57	381	1.25
C5	983	1994 Northridge	6.7	022	0.428	0.837	12.49	0.76	1.32	489	0.65
C6	1013	1994 Northridge	6.7	064	0.498	0.674	6.65	0.30	0.88	629	0.71
C7	1039	1994 Northridge	6.7	180	0.272	0.221	14.22	0.26	0.47	405	2.00
Subduction Ground Motions											
ID	Station	Event	M_w	Comp	PGA (g)	PGA (g)	t_D (s)	T_p (s)	T_m (s)	V_{s30} (m/s)	S.F.
S1	FKS005	2011 Tohoku	9.0	EW	0.45	0.35	92	0.15	0.32	469	1.00
S2	FKS009	2011 Tohoku	9.0	EW	0.86	0.56	66	0.18	0.27	409	1.8
S3	FKS010	2011 Tohoku	9.0	EW	0.83	0.44	74	0.20	0.20	387	1.00
S4	MYG001	2011 Tohoku	9.0	EW	0.43	0.23	83	0.26	0.27	441	1.8
S5	MYG004	2011 Tohoku	9.0	EW	1.22	0.48	85	0.25	0.26	430	1.00
S6	IBR004	2011 Tohoku	9.0	EW	1.03	0.38	33	0.15	0.21	382	1.65
S7	IBR006	2011 Tohoku	9.0	EW	0.78	0.30	36	0.12	0.25	406	1.40

4.3 Design level

The nonlinear seismic response, of the studied buildings, is presented in terms of the interstorey drift, residual interstorey drift, and floor acceleration recorded at the code demand level. As presented in Table 1, the first-mode period (T_1=1.36s), computed for the D-FSBF(4) system of Building Model #2, is lower than that resulting from the FSBF system of Building Model #1, which is 1.45s. Hence, for Building Model #1, the spectral acceleration ordinate corresponding to T_1 = 1.45s is $S_a(T_1)$ = 0.346 g and in case of Building Model #2, the spectral acceleration ordinate corresponding to T_1 = 1.36s is $S_a(T_1)$ = 0.365g. The SFRS of Building Model #3 is less stiff than that of Building Model #2. In consequence, the first-mode period of D-FSBF(5) is longer (T_1=1.54s) and requires lower seismic demand ($S_a(T_1)$ = 0.334 g), than the D-FSBF(4) system. The interstorey drift, residual interstorey drift and floor acceleration across the building's height were recorded at design level and are presented for all case studied in Figs. 3, 4, and 5, respectively. The mean and mean + standard deviation (Mean+SD), computed for the seismic response parameters resulting from each suite of scaled ground motions are also plotted in these figures. The seismic response under crustal ground motions is plotted in blue and that under subduction ground motions in red. It is worth noting that the total record duration, as well as the Trifunac duration, is significantly greater for subduction records than for crustal records. Hence, for Buildings Model #2 and Model #3, the peak of mean and (Mean+SD) interstorey drift are within the code limit which is 2.5%h_s. Comparing the interstorey drift configuration along the building's height for D-FSBF(4) versus D-FSBF(5), it appears that the more flexible system tends to show peak demands at upper and lower floors while the more stiff system exhibits higher demand at bottom floors. The peak of the mean residual drift is below 0.5%h_s for both D-FSBF(4) and D-FSBF(5) systems under both suites of ground motions. However, for the FSBF system of Building Model #1, the seismic response is different. At design level, several friction dampers located at the upper and lower floors reached their allowable slip length and exhibited bearing. Under the crustal records, the peak of mean interstorey drift occurred at the 7th floor followed by the bottom 2 floors and is still within the code limit. Conversely, this is not the case when the effects of subduction ground motions are investigated. As depicted in Fig. 3a, the building reached collapse at the design level. Thus, the FSBF system is not recommended to be used in high seismic zones. In the case of floor acceleration, the demand is about two times greater under the subduction than crustal ground motion suites. Under the crustal record suite, the demand is uniformly distributed along the building's height and the peak of the mean is 0.494g for D-FSBF(4) and 0.513g for D-FSBF(5).

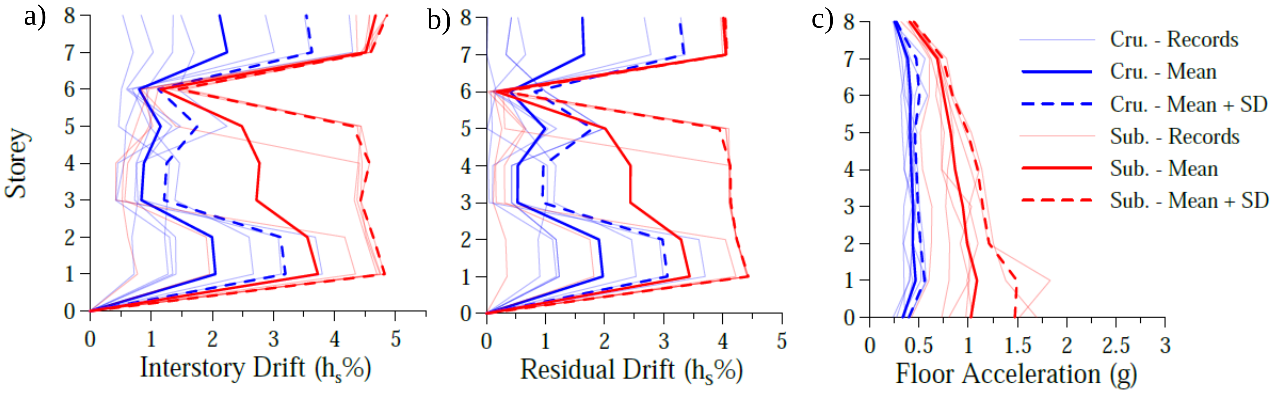


Fig. 3 - Seismic response of FSBF: a) interstorey drift, b) residual interstorey drift, c) floor acceleration

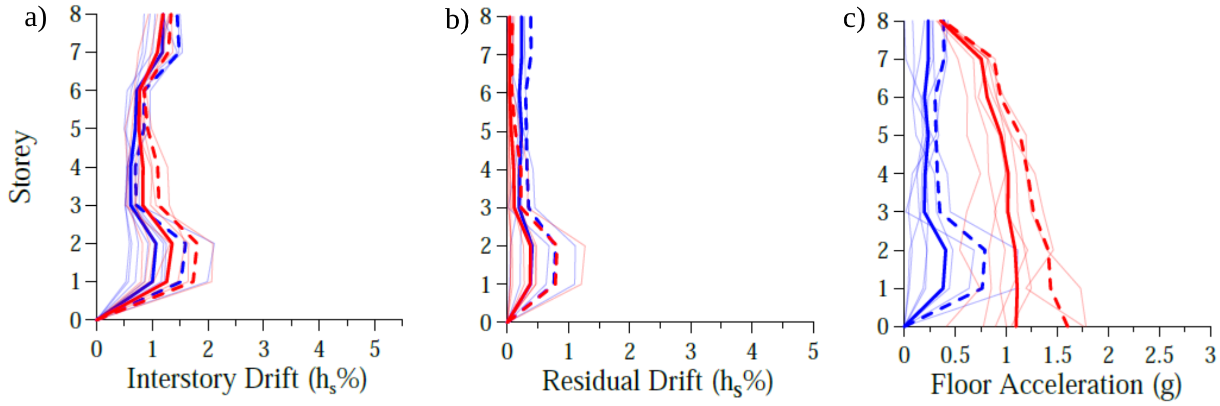


Fig. 4 - Seismic response of D-FSBF(4): a) interstorey drift, b) residual interstorey drift, c) floor acceleration

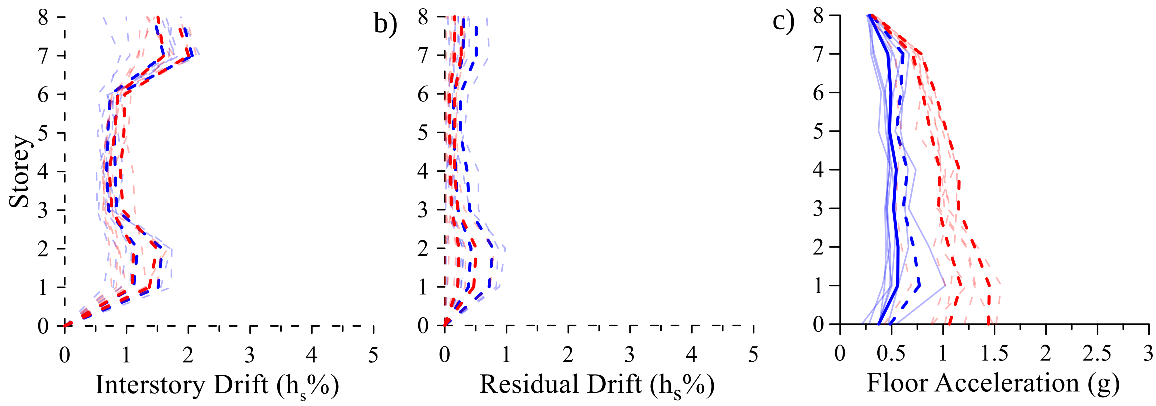


Fig. 5 - Seismic response of D-FSBF(5): a) interstorey drift, b) residual interstorey drift, c) floor acceleration

Conversely, under the subduction ground motion suite, the peak of the mean occurs at bottom floors and is about 1.1g. Thus, special details should be considered when acceleration sensitive components are installed in the buildings; these may be damaged when the floor acceleration is larger than 0.8 g.

To explain the behaviour of the friction dampers installed in an FSBF system, the response of 7th storey of Building Model #1 under the Loma Prieta record #787-360 is presented below. For buildings of normal importance category, the code limit for interstorey drift is 2.5%h_s which yields 95 mm for a storey height of 3800 mm. The projection of this drift with the angle formed by the brace with a horizontal line is 84 mm. Considering the safety factor of 1.3, the provided slip length is 1.3x84 = 110 mm which was used to model the fiction damper. Thus, after this slip length is consumed, the end bolts of friction damper hit the end of slotted hole and behaves in bearing. When this occurs, the damper does not fail and the axial force in the friction damper starts increasing above the slip force. At $S_a(T_1)$ = 0.27 g, which is the design level of scaled ground motion, large interstorey drift and residual interstorey drift are observed (Fig. 6a). From the associated hysteresis loops of the friction damper plotted in blue color in Fig. 6a, it is shown than the damper slips in one side until the stoke was almost consumed. When the demand increases to $S_a(T_1)$ =0.35g, the damper was pushed in the bearing phase, while responding sideways. Similarly, analyzing the seismic response of the 7th floor of D-FSBF(4) system, the behaviour is shown in Fig. 6b. As depicted, the residual interstorey drift is reduced, the response of the friction damper is more centered and the beams of the MRFs are still in the elastic range. After the beams of the MRFs experience hinging, the dual system leans in one direction and the system experiences a significant increase in residual interstorey drift. The yielding of the MRF's beams occurs around 1.5%h_s.

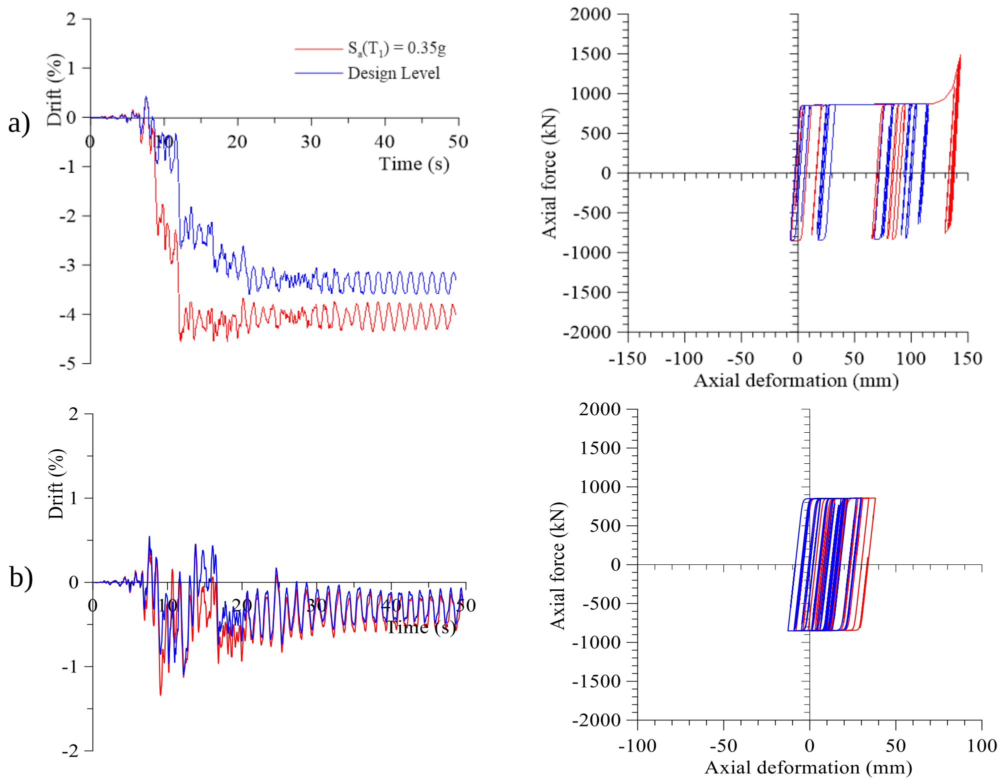


Fig. 6 - Response of the 7th floor under the #787-360 crustal record: a) FSBF system, b) D-FSBF(4) system

4.4 Incremental dynamic analysis to collapse

To assess the collapse safety of the studied structural systems, Incremental Dynamic Analysis (IDA) was employed [12]. The IDA curves are computed for each studied structural system subjected to crustal and subduction ground motions. To plot the IDA curves, the peak interstorey drift among all floors were selected to present the *Damage Measure* and the spectral acceleration at the structure's first-mode period to present the *Intensity Measure*. The seven IDA curves that show the response of Building Model #1 subjected to crustal ground motion suite are plotted in Fig. 7a and the response under subduction ground motion suite are plotted in Fig. 7b. The 50th percentile IDA curve is shown with a red line and the black dashed line shows the code demand at the first-mode period presented above. As depicted in Fig. 7a, the friction dampers of FSBF system start slipping at around 0.5%h_s associated to 0.1g. The response is stable until the interstorey drift is about 1.5%h_s and after that all IDA curves show softening behaviour where damage is accumulated at an increased rate for a small increase in intensity demand. Three out of seven crustal records subjected the building to collapse above the code demand and the 50th percentile IDA curve indicates collapse at $S_a(T_1)$ = 0.48g. In the case of the subduction records, the FSBF system cannot sustain the demand and failure occurs below the code level. When a 25% MRF was added to the FSBF, the response of dual system substantially improved. The IDA curves that show the response of D-FSBF(4) of Building Model #2 are depicted in Figs. 8a and 8b. Under both suites of ground motions, the response is stable until the MRFs beams experience hinging in flexure, which occurs at about 1.5%h_s interstorey drift. However, the softening behaviour of IDA curves shows a bit of hardening. After the 2%h_s drift is reached, the deterioration starts increasing. The 50th percentile IDA curve, obtained under crustal ground motions, indicates collapse at $S_a(T_1)$ = 0.79g. The IDA curves showing the building response to subduction ground motions are comparable but the hardening characteristic is lower. The 50th percentile IDA curve, obtained under subduction records, indicates collapse at $S_a(T_1)$= 0.66g. In the case of Building Model #3, the IDA curves computed for both crustal and subduction records demands are depicted in Fig. 9 and is similar with those shown for Building Model #2. The 50th percentile IDA curve, obtained under crustal and subduction records, indicates collapse of the D-FSBF(5) system at $S_a(T_1)$ =0.75g and $S_a(T_1)$ = 0.62g, respectively.

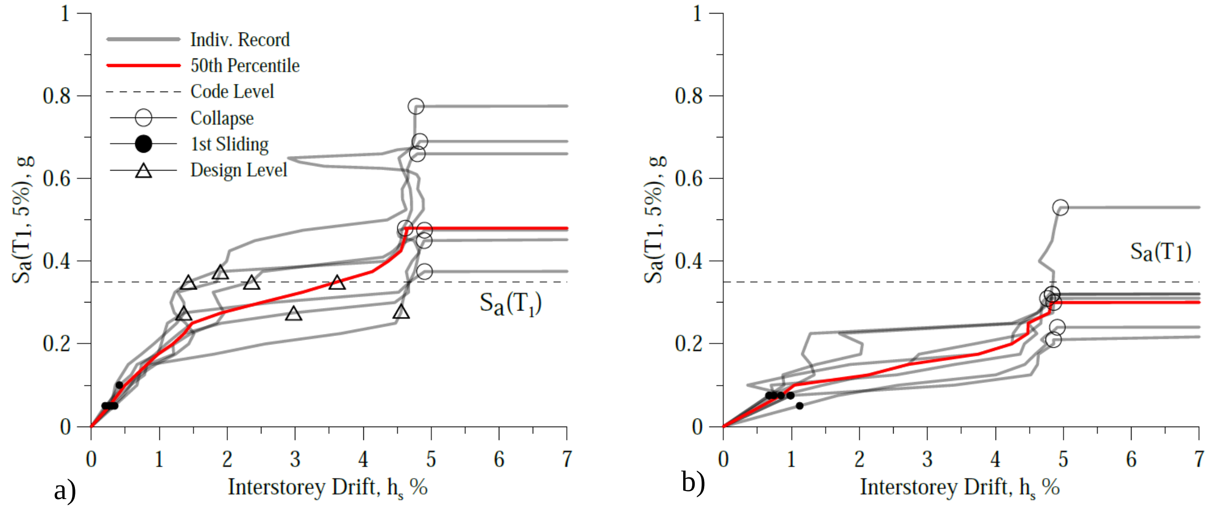


Fig. 7 - IDA for FSBF system of Building Model #1 under: a) crustal records, b) subduction records

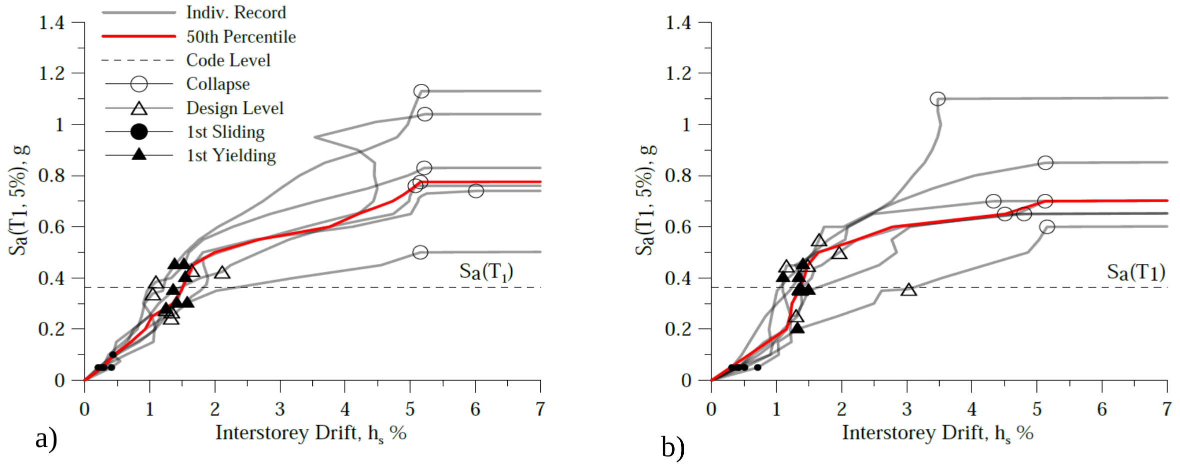


Fig. 8 - IDA for D-FSBF(4) system of Building Model #2 under: a) crustal records, b) subduction records

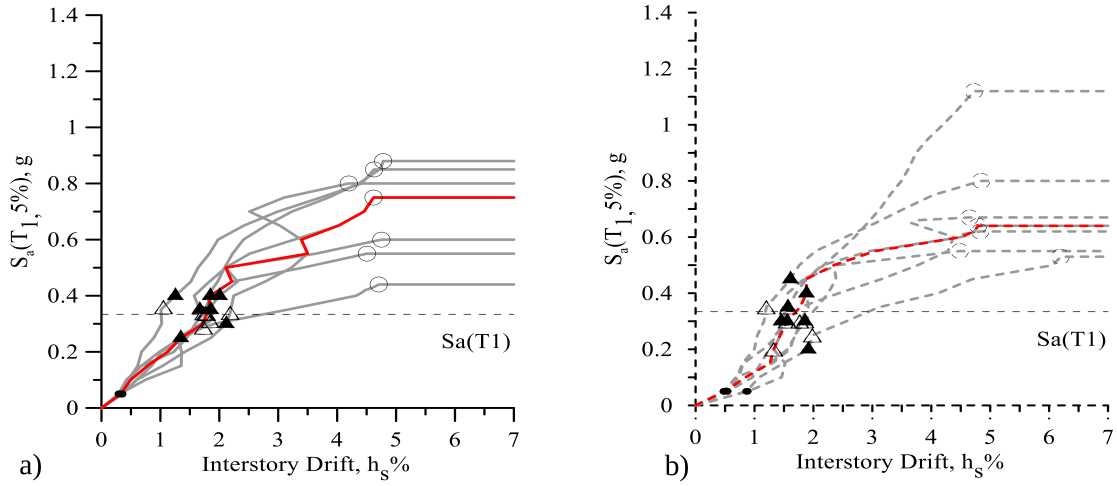


Fig. 9 - IDA for D-FSBF(5) system of Building Model #3 under: a) crustal records, b) subduction records

5. Seismic Performance Evaluation

According to FEMA P695 procedure [6], the median collapse capacity, $\hat{S}_{CT}$, is defined as the intensity of the ground motion at which half of the records in the selected suite of ground motions cause collapse. The collapse margin ratio, CMR, is defined as the ratio between $\hat{S}_{CT}$ and $S_a(T_1)$. The value of these parameters computed for all buildings subjected to both ground motion suites are showed in Table 3. The collapse safety is evaluated based on the adjusted collapse margin ratio, ACMR, which is equal to CMR *x* SSF, where SSF is the spectral shape factor as per tables provided in [6]. To pass the collapse safety criteria, it should satisfy that ACMR $\geq$ $ACMR_{10\%}$, where $ACMR_{10\%}$ is computed as a function of β_{TOT} and represents the minimum permissible ACMR value corresponding to the 10% probability of collapse under a suite of records. Herein, β_{TOT} is the total system collapse uncertainty and is quantified based on the quality rating of the numerical model. To compute β_{TOT}, the following assumptions were made: (1) the design uncertainty was assigned "A-Superior" quality rating, β_{DR}=0.1; (2) the test data uncertainty was assigned "B-Good" quality rating, β_{TD}=0.2 and (3) the model uncertainty was assigned "B-Good" quality rating, β_{MDL}=0.2. The record-to-record collapse uncertainty is β_{RTR} = 0.4. These uncertainty values are used to calculate β_{TOT}=0.5. As presented in Table 3, both systems D-FSBF(4) and D-FSBF(5) pass the collapse safety criteria. It results that subduction ground motions subject the building to more severe demand than the crustal ground motions. Although the FSBF system shows borderline pass under crustal ground motions it fails under subduction ground motions and it is not recommended to be used in high seismic risk regions.

Table 3 - Evaluation of collapse safety according to FEMA P695 methodology

Item	**FSBF**		**D-FSBF(4)**		**D-FSBF(5)**	
Parameters	Crustal GMs	Sub. GMs	Crustal GMs	Sub. GMs	Crustal GMs	Sub. GMs
$\hat{S}_{CT}$	0.48	0.30	0.79	0.66	0.75	0.62
$S_a(T_1)$	0.346	0.346	0.365	0.365	0.334	0.334
CMR	1.39	0.87	2.17	1.81	2.24	1.85
ACMR	1.90	1.19	2.97	2.48	3.13	2.59
$ACMR_{10\%}$	1.90	1.90	1.90	1.90	1.90	1.90
ACMR/ $ACMR_{10\%}$	1.0	0.62	1.56	1.30	1.64	1.36
Pass/Fail	Borderline Pass	Fail	Pass	Pass	Pass	Pass

6. Conclusions

The effect of mega-thrust subduction records versus crustal records was assessed by analyzing the nonlinear response of an 8-storey prototype building braced by the FSBF and D-FSBF systems designed for R_dR_0=4 and R_dR_0=5. The innovative features of this study are as follow:

A detailed numerical model was developed in OpenSees to simulate the slipping phase of the friction dampers installed in-line with the braces, as well as, the bearing phase experienced when the demand is larger than the available damper's stroke. The MRF beams of D-FSBF systems were simulated to replicate the flexural hinging, as well as, the strength and stiffness deterioration caused by local buckling of I-shape flanges to which a calibrated low-cycle fatigue material model was assigned as recommended in [13].

Using the FEMA P-695 procedure, the performance of the studied structural systems was assessed. Thus, it was concluded that the FSBF system fails to meet the collapse safety criteria and is not recommended in high risk seismic regions. In the case of the FSBFs, after braces slip, the system deforms sideways and experiences large residual interstorey drift. To improve the seismic response of the FSBF system, a solution could be to provide continuous columns. When the D-FSBF system is considered, its behaviour under both suites of ground motions is significantly improve. Both D-FSBF systems, designed to resist seismic loads computed with R_dR_0=4 and R_dR_0 =5, pass the collapse safety criteria. The MRFs behave in the elastic range until the demand associated to the design level was reached. The peak of mean interstorey drift is within the code limit and the peak of the mean residual interstorey drift is about 0.5%h_s.

The effects of the subduction ground motions on the buildings are more severe than that of the crustal ground motions. This is explained by its long total duration and Trifunac duration of about 90 s. The subduction records induce about two times larger floor accelerations than crustal records.

7. Acknowledgements

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