SELF-CENTERING DEMAND ASSESSMENT OF FRICTION SLIDING BRACED FRAME BUILDINGS

Laura Manolache¹, Ovidiu Serban¹, Lucia Tirca¹, Elide Nastri², Rosario Montouri², Vincenzo Piluso²

ABSTRACT

To reduce economic losses and disruptions caused by natural hazards, the development of innovative damage-resistant systems is required. These are envisioned to preserve structural members designed to remain in elastic range free of damage, as well as, to reduce damage of non-structural components which account for almost 80% of the building cost. To meet this need, in this study, an energy dissipation system and a partial self-centering system are assembled in parallel. Herein, a backup steel moment resisting frame able to provide elastic frame action is considered as a partial re-centering system for friction sliding braced frame (FSBF) because sliding occurs at lower interstorey drift when comparing with MRF beams yielding. Hence, a 8-storey prototype building located in Vancouver, BC are considered and designed with reference to provisions of National Building Code of Canada and the Steel Design standard. The investigated parameters are the peak interstorey drift, peak residual interstorey drift, as well as the peak floor acceleration. In this study, the variable parameters are: the participation of MRF to a combined system (e.g. 25%), the interstorey drift at yield of MRF versus that of FSBF, and the ductility related force modification factor ($R_d$) with values of $R_d = 4$.

The aim of assessing the self-centering demand is to diminish the residual interstorey drift of multi-storey FSBF buildings when subject to design earthquake loads corresponding to 2% probability of exceedance in 50 years.

¹ Department of Building, Civil and Environmental Engineering, Concordia University, Montreal, Canada
² Department of Civil Engineering, University of Salerno, Italy

SELF-CENTERING DEMAND ASSESSMENT OF FRICTION SLIDING BRACED FRAME BUILDINGS

Laura Manolache¹, Ovidiu Serban¹, Lucia Tirca¹, Elide Nastri², Rosario Montouri², Vincenzo Piluso²

ABSTRACT

To reduce economic losses and disruptions caused by natural hazards, the development of innovative damage-resistant systems is required. These are envisioned to preserve structural members designed to remain in elastic range free of damage, as well as, to reduce damage of non-structural components which account for almost 80% of the building cost. To meet this need, in this study, an energy dissipation system and a partial self-centering system are assembled in parallel. Herein, a backup steel moment resisting frame able to provide elastic frame action is considered as a partial re-centering system for friction sliding braced frame (FSBF) because sliding occurs at lower interstorey drift when comparing with MRF beams yielding. Hence, a 8-storey prototype building located in Vancouver, BC are considered and designed with reference to provisions of National Building Code of Canada and the Steel Design standard. The investigated parameters are the peak interstorey drift, peak residual interstorey drift, as well as the peak floor acceleration. In this study, the variable parameters are: the participation of MRF to a combined system (e.g. 25%$V_d$), the interstorey drift at yield of MRF versus that of FSBF, and the ductility related force modification factor ($R_d$) with values of $R_d = 4$. The aim of assessing the self-centering demand is to diminish the residual interstorey drift of multi-storey FSBF buildings when subject to design earthquake loads corresponding to 2% probability of exceedance in 50 years.

Introduction

The Concentrically Braced Frame (CBF) is a popular earthquake-resistant system with high stiffness and moderate ductility. While undergoing brace damage through inelastic cycles, the CBF system is prone to asymmetric response after braces reached buckling, damage is concentrated within a floor and large floor acceleration is exhibited. To overcome these drawbacks, friction sliding braces can be employed in association with a backup moment resisting frame (MRF) system. In light of this, the amount of energy dissipated by friction damper devices increases, while the system experienced lower residual interstorey drift due to re-centering capability of the MRF system.

After the Christchurch earthquake in New Zealand, the insurance policies have shaped the post-earthquake decisions for buildings structures. As a response to reduce the economic loss

¹ Department of Building, Civil and Environmental Engineering, Concordia University, Montreal, Canada
² Department of Civil Engineering, University of Salerno, Italy

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 in Japan it is generally less expensive to demolish and rebuild a new structure than to repair it if the damaged structure exhibited residual interstorey drifts greater than 0.5% $h_s$.

In this study, the challenge is to define the participation of the backup MRF to a combined dissipative system and to develop the design procedure for it. Nowadays, ASCE 7-10 standard [3] allows a variety of structural systems derived from the combination of any conventional systems. Yet, the specific design requirements for a built-up dual system are still limited. However, the standard envisions the strength proportioned relative to the elastic stiffness of each system, but no direct account of the relative distribution of energy dissipation between systems was proposed. Also, no account of interaction between systems in terms of inelastic displacement was considered. Previous studies on a similar dual system consisted of buckling restraint braces (BRB) and backup MRF explicitly tried to show that the amount of shear force assigned to the elastic MRF did not display differences in terms of drifts. However, shear reversal (MRF and BRB working against each other) was observed [4]. Furthermore, design requirements for the friction sliding braced frame (FSBF) are not provided in the NBCC and no ductility factor is suggested by researchers.

The objective of this research is three-fold: i) to emphasize the nonlinear seismic response of MD-CBF system versus the FSBF system with $R_d = 4.0$ ii) to assess the re-centering capability of low ductile MRF backup system added to multi-storey FSBFs, ii) to analyze and compare the response of ductile buildings with and without the consideration of backup MRF using OpenSees.

Prototype Building Models

An 8-story office building with a rectangular footprint of 35.5 m times 56.5 m was selected for investigation. The 8-storey building is located in Vancouver, British Colombia, Canada, on Site Class C (stiff soil). In this study, there are defined and analyzed three types of building models classified in function of their seismic force resisting system. Thus, Building Model #1 is braced in both orthogonal directions by four CBFs with a single diagonal brace per bay designed to act in tension-compression. Building Model #2 is similar with Building Model #1, whereas the difference consists of replacing the traditional brace by a friction-sliding diagonal brace composed of a Pall friction damper displaced in line with a brace support. The system was labelled friction-sliding braced frame (FSBF). Building Model #3 is similar with Building Model #2 to which was added a backup moment resisting frame (MRF) designed to re-center the seismic force resisting system while the friction-sliding braces behave in the nonlinear range. For Building Model #3, the FSBF system was designed to carry 100% of the computed base shear and the backup MRF was detailed as low-ductility system and designed to carry an additional base shear of 25%. The notation used for this system is FSBF-MRF$_{25\%}$. All three prototype buildings have a constant floor area. The plan view of Building Model #3 is shown in Fig. 1a. Considering the building symmetry in both orthogonal directions, the analysis is conducted for half of the building in the N-S direction only. The elevation is plotted in Fig. 1b and consists of two CBFs with friction-sliding diagonal braces linking a three-span MRF. The length of typical bay is 7.0 m, the height of ground floor is 4.0 m and that of typical floor is 3.8 m. The total building height, $h_n$ is 30.6 m. All CBF beams and braces are pinned connected at both ends and the CBFs 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 $F_y = 350$ MPa and tensile strength $F_u = 450$ MPa. All braces are made of hollow structural sections (HSS). The friction-sliding brace is illustrated in Fig. 1. As illustrated, the Pall friction damper is made of a middle plate that slides between two external channels bolted together by pretension bolts. The dead load at typical floor and roof level is $D_{L_{\text{type}}} = 4.0$ kPa and $D_{L_{\text{roof}}} = 3.8$ kPa, respectively. The snow load is 1.64 kPa and live load is 2.4 kPa. For cladding 1.0 kPa was considered. Thus, the seismic weight, $W$, of the building including 25% of snow load at the roof level is the same for all building models ($W = 62667$ kN).

Figure 1. Building Model #3 geometry: a) plan view, b) elevation of FSBF backed up by MRF

**Design Procedure**

The seismic force resisting system selected for Building Model #1 is moderately ductile CBF designed for a ductility-related force modification factor $R_d = 3.0$ and the overstrength-related force modification factor $R_0 = 1.3$. To design the MD-CBF members the equivalent static force procedure is applied according to NBCC requirements [5]. Thus, the fundamental period of vibration is $T_a = 0.025h_n$ leading to 0.77 s. In case that a dynamic analysis is employed, the first mode period $T_1$ can be used but it cannot exceed $2T_a = 1.54$ s. To determine the lateral earthquake design force at the base of the building the following equation is used:

$$V = S(T_a)M_vI_EW/(R_dR_o)$$

where $S(T_a)$ is the 5% damped spectral response acceleration expressed as a ratio to gravitational acceleration that corresponds herein to $2T_a$, $M_v$ is a factor accounting for higher modes effect on base shear and $I_E$ is the importance factor for earthquake load. For normal importance category buildings, $I_E = 1.0$ and from calculation $M_v = 1.0$. However, the design base shear $V_d$ shall not be taken less than 0.8$V$ for regular buildings. The site-specific response spectral accelerations for Vancouver (Site Class C) at specified periods of 0.2, 0.5, 1.0 and 2.0 s are 0.848, 0.751, 0.425 and 0.257 g, respectively. These values correspond to 2% probability of exceedance in 50 years. Torsion caused by accidental eccentricity and notional loads were neglected. From Eq. (1) resulted $V = 5263$ kN. The first-mode and second-mode period of vibration evaluated from eigenvalue analysis in the N-S direction is given in Table 1. It is noted that for comparison purpose the product $R_d \times R_0$ was considered 4.0 instead of 3.9. To size the beams and columns of MD-CBF the capacity design method provided in CSA/S16-14 [6] was applied.
The design procedure used for the seismic force resisting system employed in Building Model #2 is not covered in the NBCC and CSA/S16 provisions. To carry out the design for the FSBF system a proposed force-based design method similar with that presented for CBFs is applied. For preliminary design the same $2T_a$ period was considered. However, there is no overstength in the FSBF system while the displacement ductility can be accommodated through fabrication. In light of this, the design was conducted for $R_d = 4.0$ and $R_0 = 1.0$ leading to the same $R_dR_0$ product as considered for the MD-CBF system. Therefore, for the preliminary design, slip forces triggered in friction-sliding braces are calculated the same as the axial forces transferred to traditional CBF braces. Furthermore, the same lateral earthquake design force at the base of the building, V that was computed for the MD-CBF system can be considered to size the slip forces, $F_{slip}$ of friction damper devices at all floors. It is noted that the friction-sliding brace is connected at one end to a traditional gusset plate and at the other end the friction damper is bolted to a gusset plate. Until the friction damper installed in a diagonal brace is activated, which means it reaches the slip force, the FSBF system behaves as a traditional CBF. To assure elastic response of braces supporting friction dampers, a safety coefficient of 1.3 was applied when proportioning the HSS brace members. Hence, according to [7], all HSS braces used as support for friction dampers were designed to carry a factored load of 130% $F_{slip}$ of attached device, while the brace compressive resistance $C_r$ is larger than the factored load. Thus, in the case of FSBF system, friction sliding devices were proportioned to dissipate the input energy through friction, while all adjacent members such as HSS braces, W-shape beams and columns, as well as their connections were designed to remain in the elastic range. The first-mode and second-mode vibration period resulted from eigenvalue analysis in the N-S direction is also given in Table 1.

Table 1. Characteristics of Building Models

<table>
<thead>
<tr>
<th>Building model</th>
<th>Static equivalent method</th>
<th>Dynamic analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$2T_a$</td>
<td>V</td>
</tr>
<tr>
<td>Building Model #1: MD-CBF</td>
<td>1.54</td>
<td>5263</td>
</tr>
<tr>
<td>Building Model #2: FSBF</td>
<td>1.77</td>
<td>0.61</td>
</tr>
<tr>
<td>Building Model #3:FSBF-MRF25%</td>
<td>1.73</td>
<td>0.59</td>
</tr>
</tbody>
</table>

Building Model #3 was designed similar with the Plate Wall system provided in NBCC. As aforementioned, the earthquake resistant system is composed of a FBSF and a backup MRF detailed as low-ductile and proportioned to respond to 25% base shear. The same design method as explained for Building Model #2 was used. Meanwhile, beams of MRF were designed to possess sufficient flexural resistance such that 25% of the applied factored storey shear force is resisted by the MRF members. Axial loads in beams and gravity load effects on beams were not considered in calculating this resistance. Although a slightly larger overstrength-related force modification factor can be used such that $1.0 \leq R_0 \leq 1.1$, herein, the same $R_dR_0$ product was considered.

**Nonlinear Dynamic Analysis**

To investigate the nonlinear response of studied building models, all earthquake resistant systems were developed in OpenSees [8]. The selected direction of calculation is N-S and each model was built for half of building’s floor area. The MD-CBF system of Building Model #1 is geometrically
modelled as two-dimensional but the gusset plate connections of braces were modelled to allow the out-of-plane bending. Herein, braces were modelled as \textit{nonlinear forced based beam-column} elements with distributed plasticity, fiber cross-section formulations and three integration points per element. Each HSS brace was made of 16 elements, 240 fibers per cross-section and an initial out-of-straightness equal to 1/500 of brace length that was assigned in the out-of-plane direction. The \textit{Steel02} material was assigned to all MD-CBF members. To simulate the HSS brace fracture, fatigue material was assigned to parental material of HSS braces (\textit{Steel02}). In this study, the parameters used to define the fatigue material were calculated according to [9]. Brace to frame connections were modelled by springs assigned in the \textit{Zero-Length} elements. To simulate the brace’s gusset plate connection, two rotational springs were defined to allow rotation in-plane and out-of-plane, as well as a torsional spring was defined. The stiffness of rotational springs was based upon the geometry and properties of the gusset plate. All springs are made of \textit{Steel02} material. It is noted that the \textit{Zero-Length} element connects the brace member to a rigid link. Columns of MD-CBF were modelled as \textit{nonlinear forced based beam-column} elements with distributed plasticity and fiber cross-section discretization. Each column member was divided into 8 elements having an initial out-of-straightness equal to 1/1000 of its length. Each W-shape column was made of 120 fibers. Beams of MD-CBF were modelled as elastic members. For more information regarding the MD-CBF modelling the reader is referred to [10]. It is noted that 2% Rayleigh damping was assigned to members behaving in the elastic range.

The OpenSees model for the FSBF system is very similar with the model created for the MD-CBF. However, when the friction damper device starts slipping the supporting brace translates in-plane. The same model as above was considered for the HSS supporting braces and W-shape columns and beams. The particularity of this OpenSees model is the simulation of friction damper device which was defined as a translational spring inserted in the \textit{Zero-Length} element connecting the brace support to the rigid link. This translational spring is made of the uniaxial \textit{BoucWen} material. This material was selected because is able 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 \textit{BoucWen} material are: the initial elastic stiffness $k_0$, exponent $n$ that influences the sharpness of the model in the transition zones, $\alpha$ which is the ratio of the post-yield stiffness to the initial elastic stiffness, $\gamma$ and $\beta$ parameters controlling the shape of the hysteresis cycle. Other parameters such as $A_0$, $A$, $\nu$ and $\eta$ control the stiffness and strength degradation process. Herein, the above parameters were taken as $A_0 = 1$, $\alpha = A = \nu = \eta = 0$ while $\gamma$ and $\beta$ parameters were considered equal and calculated based on the equation: $\gamma + \beta = 1/(\Delta_y)^n$, where $\Delta_y$ represents the yielding displacement when the damper starts slipping. Thus, $\Delta_y = F_{\text{slip}}/k_0$ where $F_{\text{slip}}$ is the activation slip force and $k_0 = k_{\text{brace}} = A_{\text{brace}}E/L$. Regarding the sharpness parameter, $n = 10$ was selected for this study, as suggested in [11]. Again, a 2% Rayleigh damping was assigned to members behaving in the elastic range.

The OpenSees model for the MRF included in the Building Model #3 system is defined as follows. Each MRF beam is made of \textit{beam with hinges element} with fiber-based cross-section discretization within the plastic hinge zone to which the \textit{modified Gauss-Radau integration scheme} was assigned. The strength and stiffness deterioration caused by flange local buckling which is simulated by assigning a calibrated low-cycle fatigue material model to flange fibers as referred in [12]. The MRF columns were modelled as \textit{nonlinear forced based beam-column} elements with distributed plasticity, fiber cross-section formulations and three integration points per element. \textit{Steel02} material was assigned to both beam and column members. Gravity columns of all studied building models were simulated using elastic elements to account for stiffness, as well as, for the
P-Δ effect. All gravity columns were connected to the earthquake resistant system at each floor by rigid links in order to simulate the effect of rigid diaphragm. For computations, the Krylov-Newton algorithm was used for all buildings.

**Ground Motion Selection and Scaling**

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. However, in this study only a set of seven crustal ground motions selected from the PEER-NGA ground motion database are considered and given in Table 2, where NGA is the record identification. All records comply with Site Class C. In the same table is provided the magnitude of the earthquake event, the earthquake duration (t), the Trifunac duration (t_D), the principal period of ground motion (T_p) and the main period of ground motion (T_m). According to the NBCC 2015 provisions, all ground motions are scaled such that the mean of seven to match or be above 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 value and the code design spectrum are given in Fig. 2, while all scaling factors, SF are given in Table 2.

![Figure 2: Response spectrum of scaled crustal ground motions](image)

**Table 2. Characteristics of crustal ground motions**

<table>
<thead>
<tr>
<th>NGA Component</th>
<th>M_w</th>
<th>Event/Station</th>
<th>t</th>
<th>t_D</th>
<th>T_p</th>
<th>T_m</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>802-90</td>
<td>6.93</td>
<td>Loma Prieta/Saratoga</td>
<td>39.99</td>
<td>8.02</td>
<td>0.22</td>
<td>0.567</td>
<td>1.20</td>
</tr>
<tr>
<td>1013-334</td>
<td>6.69</td>
<td>Northridge/ LA Dam</td>
<td>26.57</td>
<td>6.65</td>
<td>0.3</td>
<td>0.875</td>
<td>1.10</td>
</tr>
<tr>
<td>787-360</td>
<td>6.93</td>
<td>Loma Prieta/LGPC</td>
<td>39.645</td>
<td>11.61</td>
<td>0.3</td>
<td>0.685</td>
<td>1.30</td>
</tr>
<tr>
<td>787-270</td>
<td>6.93</td>
<td>Loma Prieta/ LGPC</td>
<td>39.61</td>
<td>12.66</td>
<td>1</td>
<td>0.883</td>
<td>1.45</td>
</tr>
<tr>
<td>779-000</td>
<td>6.93</td>
<td>Loma P./ Golden Gate Bridge</td>
<td>25</td>
<td>10.15</td>
<td>0.7</td>
<td>0.8</td>
<td>0.40</td>
</tr>
<tr>
<td>1013-064</td>
<td>6.69</td>
<td>Northridge/LA Dam</td>
<td>26.57</td>
<td>0.65</td>
<td>0.42</td>
<td>0.8</td>
<td>0.65</td>
</tr>
<tr>
<td>983-022</td>
<td>6.69</td>
<td>Northridge /Jensen Filter Plant</td>
<td>28.62</td>
<td>12.49</td>
<td>0.76</td>
<td>1.315</td>
<td>0.70</td>
</tr>
</tbody>
</table>
Seismic Response of Building structures

The nonlinear seismic response of Building Models #1 - #3 is expressed in terms of interstorey drift, residual interstorey drift and floor acceleration. The first-mode and second-mode period of studied buildings resulted from OpenSees are given in Table 1. As resulted, there is a slight difference between the vibration periods of studied buildings. The interstorey drift, residual interstorey drift and floor acceleration are plotted for Model Building #1 in Fig. 3, for Model Building #2 in Fig. 4 and for Model Building #3 in Fig. 5. The 50 percentile and 84 percentile of selected engineering demand parameters are also provided.

Thus, in the case of MD-CBF system, the interstorey drift at ultimate limit state (design level) is below the 2.5% $h_s$ code limit where $h_s$ is the storey height. Further, the 84 percentile residual interstorey drift and floor acceleration is lower than 0.5% $h_s$ and 0.8 g, respectively. In a study reported in [2] it was concluded that is less expensive to demolish and rebuild a new building than
to repair it when the damaged structure shows residual interstorey drifts greater than 0.5\% \( h_s \). Regarding the recorded floor acceleration magnitude, there are nonstructural components sensitive to acceleration that may be damaged at 0.8 g. Thus a structural system able to experience lower floor acceleration is recommended. From data provided in Fig. 4 resulted that the FSBF system experienced large interstorey drift, as well as, large residual interstorey drift and lower floor acceleration which is about 0.6 g. Herein, the peak residual interstorey drift shows a mean of 1.5\% \( h_s \) at the 7\(^{th}\) floor. However, the 50 percentile interstorey drift is lower than 2.5\% \( h_s \). Thus, the FSBF system should be re-centered in order to avoid large residual displacements. The results for Model Building #3 show maximum interstorey drift less than 1.0\% \( h_s \), maximum residual interstorey drift less than 0.3\% \( h_s \) and floor acceleration of 0.4 g. Moreover, there is shown a more uniformly distributed demand across the building height. Thus, among the three structural systems, the FDBF-MRF\(_{25}\%\) shows the better seismic performance.

![Figure 5. Seismic response of Building Model #3 (FSBF-MRF\(_{25}\%\))](image)

The response of selected structural systems subjected to Loma Prieta record #779 is presented in Fig. 6 in terms of time-history series of interstorey drift recorded at the 7\(^{th}\) floor and hysteresis loops experienced by brace of MD-CBF, friction damper of FSBF and re-centered friction device. It can be seen from Fig. 6 that in the case of FSBF system, at the end of the seismic event, the building does not return to its initial equilibrium position and it remains to oscillate in one side. This behaviour leads to large residual interstorey drifts. Beneath, it is shown that adding a back-up MRF it behaves as an elastic frame action able to re-center the lateral force resisting system. As resulted from Fig. 6, the friction damper device experienced a slipping length twice as that of the MD-CBF brace. It is worth to note that the FSBF-MRF\(_{25}\%\) system experienced lower frame action effect when the MRF beams start hinging in flexure. This phenomenon occurs when the system experienced larger interstorey drift than 1.7\% \( h_s \). For lower interstorey drifts the MRF behaves in the elastic domain.

**Conclusions**

This study was conducted to assess the seismic response of three 8-storey Building Models with constant floor area that were braced by different earthquake resistant systems without and with self-centering capability. The following findings are reported:
Figure 6. Response of buildings studied recorded at the 7th floor under the #779 record: MD-CBF time-history interstorey drift and brace hysteresis (1st row), FSBF time-history interstorey drift and friction damper hysteresis (2nd row), and FSBF-MRF\textsubscript{25\%} time-history interstorey drift and re-centered friction device hysteresis (3rd row)

1. The 8-storey MD-CBF buildings designed for Vancouver, B.C. shows large floor acceleration of about 0.8 g, the peak interstorey drift was within the code limit of 2.5\% \( h_s \) and the peak of mean residual interstorey drift was slightly lower than 0.5\% \( h_s \) which is the acceptable limit suggested by researchers.

2. The 8-storey FSBF building show high ductility capacity, lower floor acceleration than the previous system but large residual interstorey drift of about 1.0\% \( h_s \). This system is prone to concentrate damage within a floor. A force-based design method is presented to size the system members by considering a ductility factor of \( R_d = 4.0 \). It is suggested to study the system response when designed for slightly greater \( R_d \) factors.
3. The proposed FSBF-MRF system is able to uniformly distribute damage among floors, to exhibit lower interstorey drift and half floor acceleration comparing to the MD-CBF building. This system experienced the lower residual interstorey drift recorded as 0.3% $h_s$. It is also reported that the 25% MRF is able to perform in the elastic range until the FSBF-MRF system experienced a peak interstorey drift less than 1.7% $h_s$.

4. An additional study should be carried out in order to quantify the MRF contribution to a combined system with different height and ductility factor.

Acknowledgments

The financial support provided by the IC-IMPACTS Centers of Excellence Network of Canada and “Fonds de recherché du Quebec – Nature et Technologies” is gratefully acknowledged.

References


