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Determining the Optimum Slip Load of the Friction Damped Concentrically Braced Multi-Storey Timber Frame

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ABSTRACT: In this research, the application of sliding joints in concentrically braced frames (CBFs) will be assessed to determine the influence of friction damper on multi-storey timber buildings. Energy dissipation comes from the friction resistance between the sliding steel plates and as a result, it is feasible to have decreased floor accelerations and reduced forces in the structural members. Typically, dowel-type fasteners such as bolts, screws or nails are used for timber brace connections. Since it is difficult to ensure that timber connections fail by a combination of fastener yielding and timber crushing, timber buildings using traditional connection detailing can be considered as non-ductile structures. The investigated slip friction connection not only improves the earthquake resistance of the structure by providing the required ductility but also reduces the building damage through damping of energy. This paper presents the results of the dynamic analyses conducted on a conceptual ten storey CBF timber building, located in a high seismic zone. The response of the model with sliding joints is compared to the response of the conventional CBF model. The effect of sliding force on the response of the structure is also studied to determine a range of pre-stressing force. Different thresholds for sliding force are tested to determine the optimal level.

Keywords: Slip friction joint, timber connection, concentrically braced frame, multi-storey building.

1 INTRODUCTION

Sliding joints have been widely used in steel structures due to their efficiency in dissipating energy. The application of these connections in timber buildings is a new field. Since the material property of timber is different from steel and high strength to weight ratio of timber effects lighter building in comparison with steel structures, it is required to investigate the response of buildings having this type of energy dissipator to identify slip threshold for connections. The design of the lateral load resisting system is one of the most important parts of the design procedure and requires more investigation to understand the behaviour of structures notably when energy dissipator systems are utilised in the buildings.

Passive damping devices have been promisingly employed as one of the controlling systems for the structural response against earthquake excitations. Recently, friction dampers have been widely used in buildings due to their high-energy dissipation potential, cost efficiency and their undemanding application. Friction dampers require a force higher than their sliding force to commence the energy dissipation. A part of the input energy will be converted to thermal energy and appear in terms of temperature. Since the friction dampers have shown promising performance and high efficiency in dynamic loadings, many researchers have been both theoretically and experimentally studying slip friction connection.

Pall and Marsh (1982) proposed a type of friction damper at the crossing joint of steel X-braces. The advantage of their device is the absence of a compression member in the brace system. A comparison was made between the dynamic responses of friction damped bracing systems and ordinary braced steel frames by Colajanni and Papia (1993). Their results show that the assumption of ductility value greater than 3 for the cross-bracing system implies a high-risk condition, apart from fatigue and localized plasticity effects. Grigorian et al. (1993) investigated the energy dissipation of slotted bolted friction dampers. They studied the behaviour of a one story diagonally braced steel frame for four different earthquakes. Experimental results were presented for the fabricated slotted bolted connection for the mentioned frame subjected to the four displacement histories derived from selected earthquakes.

Filiatrault and Cherry (1988) used the relative performance index (RPI) which is based on the elastic strain energy to determine the optimum slip load for a structure. Aiken et al. (1988) studied the behaviour of a nine-storey steel building with friction dampers. They observed that the friction damped braced frame (FDBF) system could experience nonlinearity in the connections without demanding inelastic behaviour in the frame.

A quasi-static design procedure for a friction damped structure was introduced by Fu and Cherry (1999). In other research, Fu and Cherry (2000) simulated the seismic resistance of a single degree of freedom (SDOF) friction damped system to investigate the situation in which both dampers and frame members are inelastic. Butterworth (2000) proposed the use of slip-friction connections in concentrically braced steel frames. He concluded that an increase of elastic stiffness of the frame components could reduce the permanent offset of the building. Clifton et al. (2007) have developed a sliding hinge connection particularly for beam-column joints of steel frames. He developed a method to estimate the strength of the connection.

This study investigates the performance of CBF timber building using slip friction connectors. As it is demanded to determine a reasonable range for slip loads, the influence of initial slip force on the behaviour of the building has been discussed.

2 METHODOLOGY

2.1 Overview

A 10 storey braced timber structure is numerically analysed using SAP2000 program with seven different ground motions. The acceleration spectra for ground motions are shown in Figure 1. The records are particularly selected for the North Island of New Zealand (Oyarzo-Vera et al. 2012). The floor plan is 30 m by 30 m, and the structure has a floor height of 4 m. The seismic masses for each floor were calculated using the New Zealand Standard for Structural Design Actions (1170.5 2004). Figure 2(a) shows the structural plan of the model and Figure 2(b) demonstrates a 3D view of the analytical model. The structure has three bays in both X and Y directions. The location of the structure is assumed to be in Wellington, New Zealand, which is regarded as a high seismic zone.

The building is designed using target displacement and push over analysis. Due to the New Zealand standard, the maximum allowable drift is 2.5%, and consequently the ultimate displacement of the building is equal to 1 m. The ductility of four is assumed to determine the elastic displacement. This displacement is set as a target displacement for a push over analysis to obtain structural members size. It is desirable to concentrate all the nonlinearity of the building in slip friction connections. The base columns have pinned connections and braces are resisting lateral forces. All the members are designed based on the LVL 11 specifications. The ductility was chosen to make sure that by reaching the elastic deformation the structure has loads higher than serviceability limit state and lower than design basis earthquake loads. The influence of the slip load on the base shear and top floor displacement of the building is investigated using different sliding forces varied from serviceability limit state to design basis earthquake load.

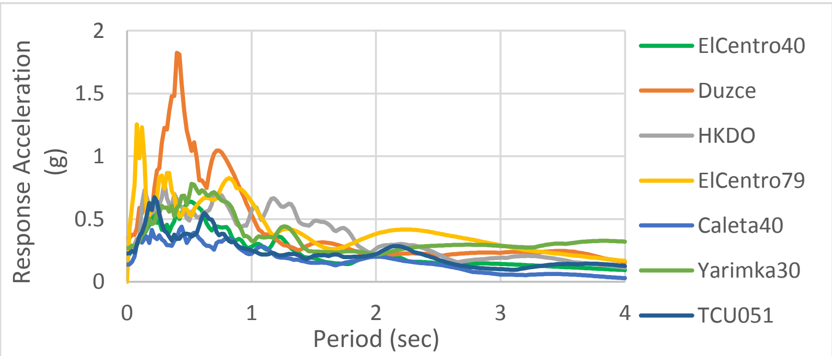
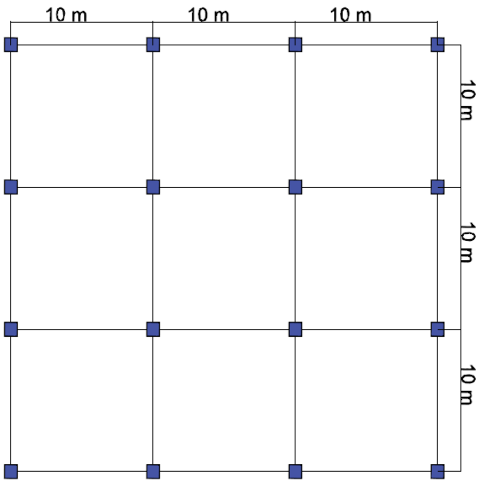
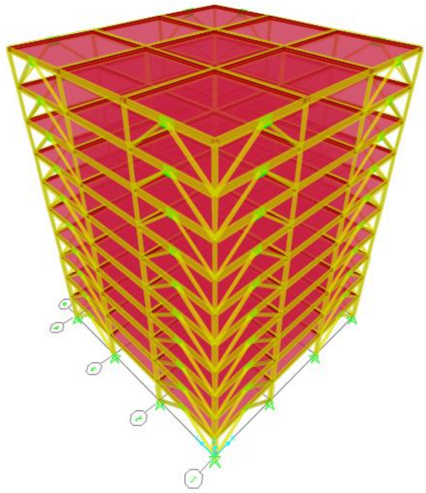


Figure 1 Ground motions' acceleration spectra



(a) Structural plan

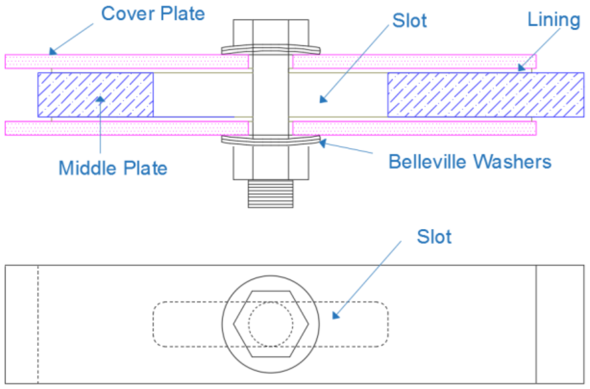


(b) 3D model

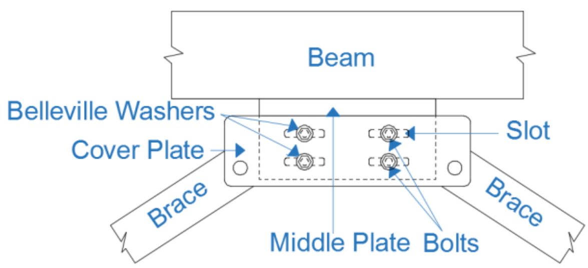
Figure 2 Structural configuration

2.2 Slip friction connection modelling

The slip friction connections consist of steel plates (brass or shims) which are bolted together through the slotted middle plate (Fig. 3). Since direct steel-to-steel contact generates unfavourably variable friction coefficient, various types of lining have been tested as facing materials. In this research, the overall behaviour of the connection is modelled regardless of the lining material. The variation of coefficient of friction value does not influence the overall behaviour since the slip force mostly depends on the bolt tension, which can be modified to provide the required force. The sliding holes included in the connection allow energy dissipation without any failure of the connection components. Horizontal forces from the ground motion are transferred to the brace members, and the horizontal component of the force will lead to sliding in the connector. The cover plates' role is to maintain the required static friction force to prevent the undesirable movement at low forces.



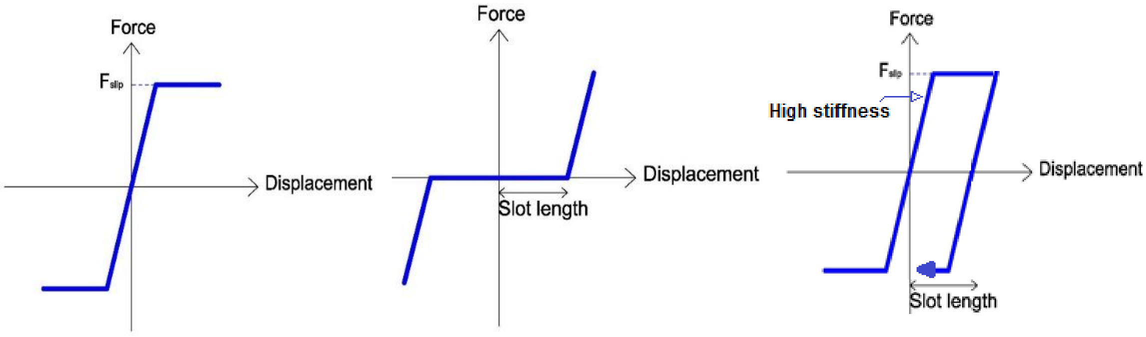
(a) Details of a connection



(b) Schematic application in the braced frame

Figure 3 Sliding bolted connection

For modelling the slip friction connection, three types of link elements, namely the multi-linear plastic link, the gap and the hook are used (Fig. 4). The multi-linear link element is employed to simulate the elasto plastic behaviour of the connection and as for the hysteresis type, the kinematic one is selected which provides no stiffness degradation during the analysis. The gap element works only in compression while the hook element works only in tension. These two elements are used to define the slot length of the connection. All the above mentioned elements can present the force displacement behaviour of the slip friction connection when they are properly positioned in the model. The slot length is calculated based on the inelastic displacement limit of the building, which is 75 cm as the assumed ductility is four. It is considered that the first mode is the dominant mode. Thus, the inelastic displacement should be accommodated by the sliding joints and be divided equally among the storeys. Since the minimum required travel length is 8 cm in each direction, the total amount of 16 cm for each story is required.



(a) Multi linear plastic link **(b) Nonlinear gap and hook element** **(c) Slip friction behaviour by combining multi linear plastic link, gap and hook element**

Figure 4 Link elements for modelling slip friction connection

2.3 Sliding force range

The structural response in terms of base shear and top floor displacement of the structure is highly dependent on the slip load and the travel length of each friction damper. When the slip initiation loads are high, the sliding does not occur. Therefore, the energy dissipation in the connection is negligible. As a result, the structure completely acts as an ordinary braced frame with elastic behaviour. An undesirable effect also exists when the slip initiation loads are small, which leads to insignificant energy dissipation and higher structural response. In this situation, the structure behaves more or less like an unbraced frame. There is a point in this range that can result in the minimum base shear or top floor displacement of the structure. It should be noted that choosing the slip load does not only depend on the minimum base shear or drift of the structure but also it relies on the overall cost of the connection and the feasibility of pre-stressing force.

The minimum threshold for the sliding force is the serviceability load. Hence, the connections should be designed to avoid sliding under such circumstances. As an upper threshold for sliding force, friction connections are expected to slip before the structural members yield. In general, the lower limit should be higher than the serviceability load and could be considered as 30% beyond the SLS. In multi-storey timber buildings, as they are light, normally 1.3 times of wind governs the lower limit of sliding force. Furthermore, the upper limit should be lower than 25 % of members' capacity (Miri and Kahkeshan 2014).

This study shows that the slip initiation load of the friction damper has the most significant influence on tuning the response of the structure. The sliding forces in storey levels are distributed based on the proportion of the design shear force at each level. Thereafter, the sliding forces are changed between two mentioned limits to identify the structural response.

2.4 Loading protocol

For the loading protocol, seven earthquake records are applied to the model. The ground motions are chosen based on the New Zealand standard and the research conducted by Oyarzo-Vera et al. (2012). It is assumed that the structure is constructed on soil type C. As it is presented in Table 1; the records are scaled for two different levels of ground motions, serviceability limit state (SLS) and design basis earthquake (DBE).

Table 1. Scale factors of earthquake records

Record	Scale Factor for SLS	Scale Factor for DBE
ElCentro40	0.37	1.47
Duzce	0.22	0.87
HKD085	0.28	1.12
ElCentro79	0.33	1.34
Caleta	0.71	2.84
Yarimka	0.36	1.44
TCU051	0.58	2.33

The guideline of the National Earthquake Hazards Reduction Program (NEHRP) (1997) specifies that a structure with an energy dissipating system should be assessed for both DBE and maximum credible earthquake (MCE) records. The DBE refers to the earthquake having 10% probability of exceedance in 50 years (i.e. 475 years return period), and the MCE refers to the earthquake having 2% probability of exceedance in 50 years (i.e. 2450 years return period). Under the DBE, the structure is evaluated to ensure that the strength demands on structural elements do not exceed their capacities and that the drift in the structure is within the tolerable limits.

3 RESULTS

3.1 Structural response

As expected, increasing the slip initiation load reduces the structural response in terms of base shear and top floor displacement in comparison to the conventional braced structure without the damping device. Figure 5 represents the influence of the slip initiation load on the response of the structure.

Higher slip friction and travel length in the connection lead to more energy dissipation in the system and decrease the base shear. While the slip friction is low, the connection starts to slide at the low level of force and the amount of energy dissipation becomes insignificant. As a result, there is a slight reduction in top floor displacement. It should be noted that the sliding force has been increased in nine steps or eight ranges from serviceability limit state to DBE loads to evaluate the effects of different sliding levels. The results are normalized to the response of the undamped structure to demonstrate the effects of the connectors characteristics.

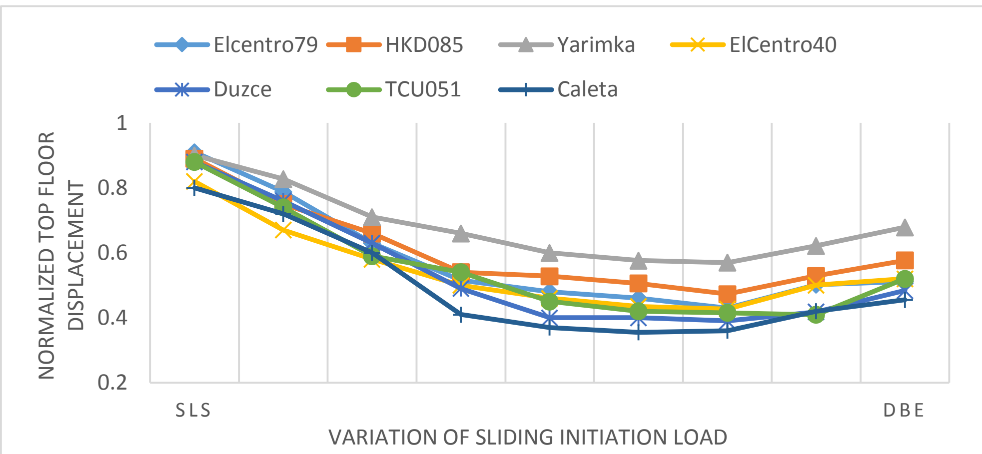


Figure 5 Top floor displacement normalized to the undamped structural response versus different sliding forces

Table 2 shows the maximum top floor displacement of the building for the selected records.

Table 2. Top floor displacement of the building

Slip Load	Top Floor Displacement for Different Records (cm)						
	ElCentro79	HKD085	Yarimka	ElCentro40	Duzce	TCU051	Caleta
SLS	88.3	84.3	90.0	74.6	82.7	81.4	71.6
Step 1	76.4	71.1	82.7	61.0	71.5	68.5	64.5
Step2	61.1	62.5	71.0	52.8	59.2	54.6	53.7
Step 3	50.0	51.2	66.0	45.5	46.1	50.0	36.7
Step 4	46.6	50.0	60.0	41.9	37.6	41.6	33.1
Step 5	44.6	47.9	57.6	39.6	37.6	38.9	31.8
Step 6	41.7	44.8	57.0	38.9	36.7	38.4	32.2
Step 7	48.7	50.1	62.1	45.5	39.3	37.9	37.6
DBE	49.6	54.6	67.8	47.3	45.5	48.1	40.7

The relation between the sliding force and the maximum base shear of the model is shown in Figure 6. It can be observed that the increasing of the sliding force decreases the base shear. Sliding force in the range of 50 to 75 percent of DBE force results in a lower range of responses. However, increasing the sliding force leads to a higher base shear. It is worth pointing out that the response of the structure depends on the level of sliding force and the amount of travel length. The base shear goes up when slip initiation loads are close to the DBE level. This condition occurs since the friction damper could not fully travel the slot length. Therefore, the amount of energy dissipation would be diminished and as a result, the higher base shear could be observed.

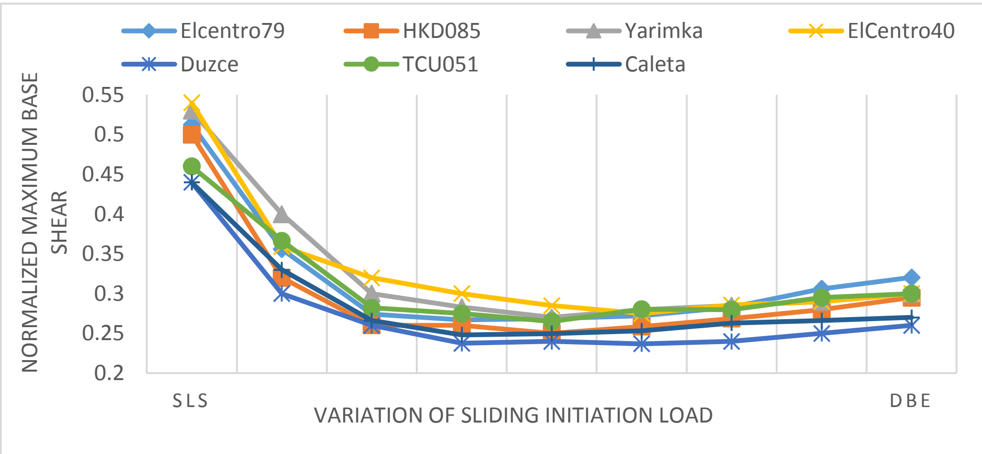


Figure 6 Normalized base shear versus different sliding forces

The sliding force of the building for each storey is shown in Table 3. The SLS force of the sliding connector is determined and the maximum value in each storey is considered as for the connectors. These values are also specified for the DBE.

The base shear of the structure during the ElCentro79 ground motion in three different conditions, undamped structure, and damped structure with sliding force obtained from the DBE and damped structure with sliding force obtained from the 75 percent of the DBE is shown in Figure 7. In this case, the maximum base shear is reduced to less than half.

Table 3. Sliding force of the connections

	Sliding Connector Force (kN)	
Storey No.	**SLS**	**DBE**
1	835	2000
2	750	1850
3	665	1655
4	580	1365
5	495	1170
6	410	970
7	320	780
8	250	585
9	185	390
10	110	195

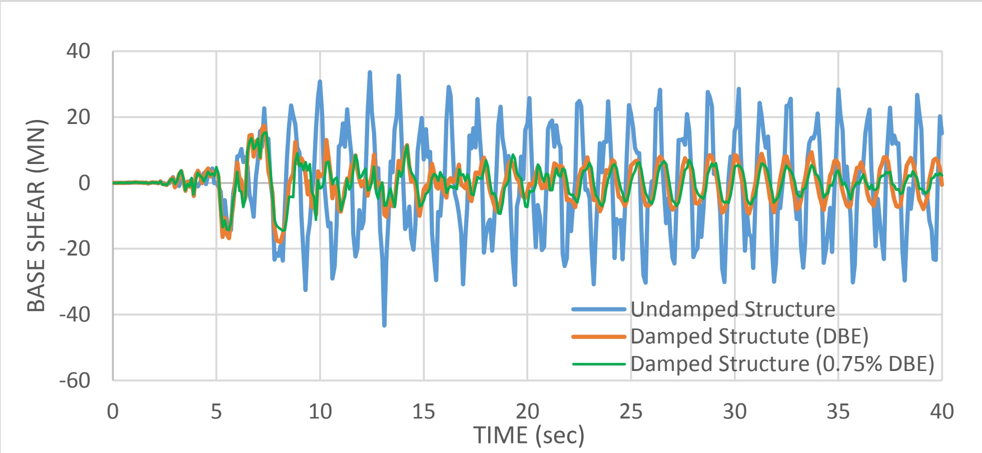


Figure 7 Base shear time history for ElCentro ground motion

The maximum normalized acceleration of each storey for the ElCentro79 excitation is shown in Figure 8. Generally, the rise in the sliding force led to a decline in each floor acceleration. For the first six storeies the same size of brace is used and for seventh to tenth storey the section size is reduced. Owing to the change of the brace stiffness at seventh storey, a sharp increase in the floor acceleration can be seen in Figure 8.

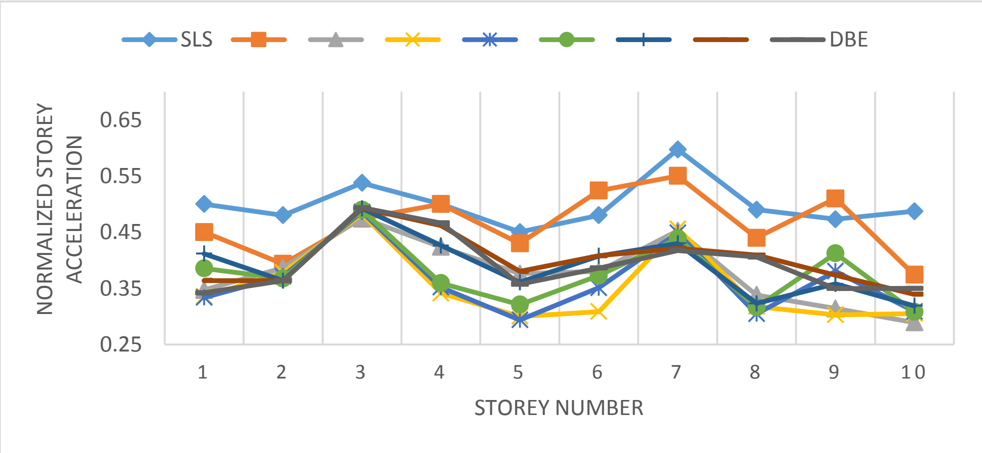


Figure 8 Maximum storey acceleration due to the ElCentro ground motion

4 CONCLUSIONS

The efficiency of the friction dampers in reducing the structural responses for a range of loading conditions has been studied for a ten storey concentrically braced frame building. The friction damper considerably reduced both the peak floor displacement and the base shear of the structure introduced to different seismic excitations. It was observed that both acceleration and displacement responses were decreased. Furthermore, with a gradual increase of sliding forces the maximum base shear substantially decreased, but after a specific stage, which depends on the record characteristics, this trend was altered and the base shear increased slightly. In the first three ranges, changing of the sliding force resulted in a significant reduction of the structural response while this decline was not substantial in the next steps. Nonetheless, with increasing of sliding forces, the base shear and the top floor displacement increased in the last two ranges due to lower energy dissipation. It implies that the variation of a sliding force can affect the base shear and the top floor displacement of the building. It can be deduced from the results that the reasonable sliding force range is within the fourth and sixth steps, which, in this case, is around 50 to 75 percent of the design basis earthquake loads.

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