ENHANCED PERFORMANCE OF BUILDINGS RETROFITTED WITH FRICTION DAMPERS WITH RESTRAINERS

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Abstract: The behaviour of slotted-bolted connections when the maximum slip travel exceeds the available slot length is presented. The drastic degradation behaviour of slotted-bolted connections under bolt impact is demonstrated. Remedy by means of the concept of restrainers is proposed, which is also advantageous in suppressing the build-up of resonance motion by virtue of the change in system stiffness at different stages. Nonlinear dynamic analyses are performed on two buildings equipped with the friction dampers with and without restrainers. The first structure, a 6-storey steel building with moment-resisting frame, is subjected to strong near-field earthquakes. The second one, a typical 6-storey reinforced concrete building in Bangkok with non-seismic design, is investigated under moderate ground motions induced by distant earthquakes. In the worst loading cases, the damper with restrainers is superior to that without restrainers, resulting in reduction of the peak inter-storey drift in the order of 16-20% and the peak slip travel 15-30%. The shorter slip travel obviously provides extra safety to the damper against bolt impact. Although the peak inter-storey drift and the peak slip travel of the device decrease with increasing in the restraining force limit, the extra gain has to be balanced with the potential of buckling of the bracing at the high restraining force in the case of retrofitting with bracings.

1. INTRODUCTION

The slotted-bolted connection (SBC) has been recognized as a low-cost passive friction-damping system for reduction of damages caused by strong motions (e.g. FitzGerald et al. 1989, Grigorian et al. 1993, Tremblay and Stiemer 1993). Most studies, except the one by Roik et al. (1988), do not address the situation when the slip travel of the SBC reaches the provided slip length. However, Roik et al. (1988) did not investigate the effect of stiffening due to the restraining action. Since earthquake characteristics are unpredictable, it cannot be guaranteed that in the most severe case the bolts would slide freely in the 'finite length' slot available. Furthermore, it is not practical, or even impossible in some cases, to provide long slots. It is thus significant to investigate the behavior of such a damping system in the event of bolt impact and the restraining effect thereafter. Advantages of the limited-slip friction damper over the conventional one are demonstrated through performance assessments of two buildings, one subjected to strong motion earthquakes, and the other to moderate ground motions induced by distant earthquakes.

2. CONVENTIONAL SBC UNDER BOLT IMPACT

Wanitkorkul (2003) performed a series of displacement controlled, cyclic tests on slotted-bolted connections, with and without impact on high-strength bolts. The damper specimen consisted of three A36 steel plates, i.e. one central plate and two cover plates each with a thickness of 11 mm. Brass plates were inserted between the central plate and the cover plates to create steel-on-brass contacting surfaces. The damper was designed to have two slots symmetrically placed on the central plate. All plates were clamped together using two 12-mm-diameter A325 high-strength bolts with “Belleville washers”. The total clamping force on each sliding surface was 108 kN (70% of bolt tensile strength). The test assembly was subjected to 30 displacement cycles without bolt impact, followed by 10 displacement cycles with impact on high-strength bolts, in general. Testing frequencies of 0.1, 0.5, and 1 Hz were used in the cyclic tests.

Figure 1 shows the hysteresis of a SBC specimen under the first thirty displacement cycles without bolt impact. Generally, the hysteresis under each testing frequency is similar, hence only the results from the testing frequency of 0.1 Hz are demonstrated. The resulting force-displacement relationships were nearly rectangular in shape, which was consistent with the results obtained from many past researches. The initial friction forces were around 45 kN, and they became stable around 95 kN after approximately 20 displacement cycles. Despite the fluctuation of the friction forces, steel-on-brass friction type specimens were acceptable because the variation of frictions was in the initial stage, therefore it would not affect the behaviour of damper when the specified slip travel was reached upon bolt impact. The stiffness increased sharply as can be seen in Fig. 2 which illustrates the portion of the hysteresis of a SBC specimen during bolt impact. From the figure, the first yield-plateau indicates the level of friction, while the second one shows the yield strength of the connection. During cycling of bolt impact, degradation of the friction force is evident. More than 50% of friction force was lost after only 4-mm of displacement was imposed beyond the provided slot length. Bearing caused permanent deformations in the clamping bolts, with the consequence of the friction loss.

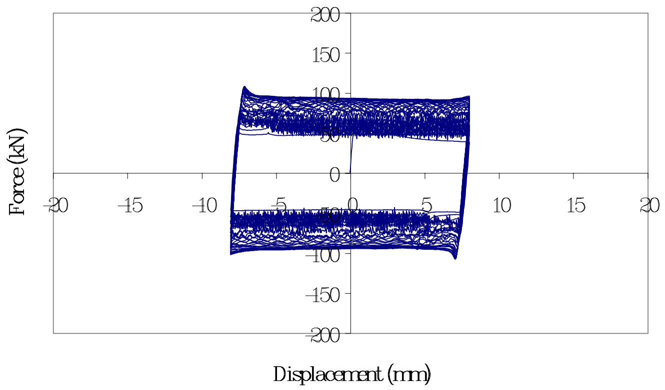


Figure 1 Force-Displacement Relationship of SBC: 30 Cycles, 0.1 Hz, without Bolt Impact

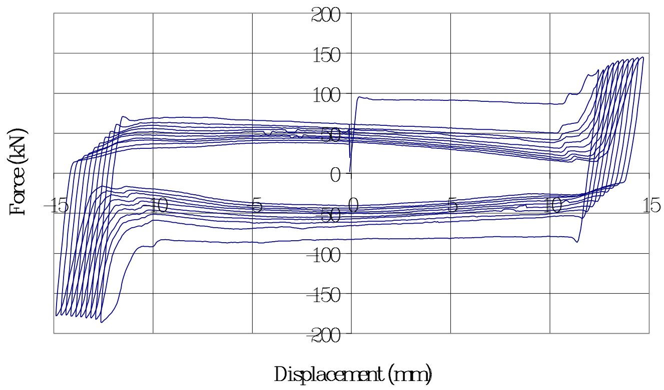


Figure 2 Force-Displacement Relationship of SBC: 10 Cycles, 0.1 Hz, with Bolt Impact

(a) (b)

Figure 3 SBC Specimen after Bolt Impact: (a) Central Plate and Bolts; (b) Cover Plates and Brass Plates

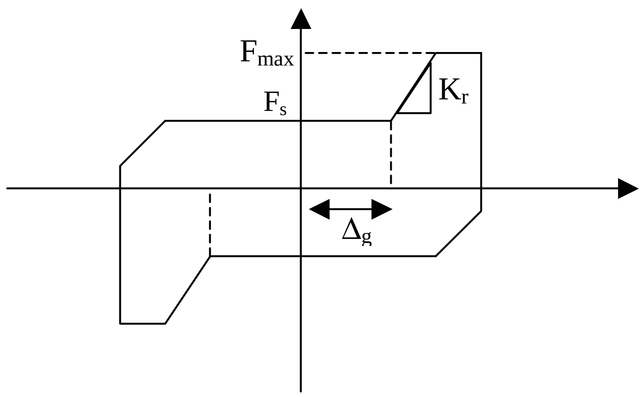


Figure 4 Damper Force-Slip Travel Relationship of Slotted-Bolted Connection with restrainers

Figure 3 shows each component of the specimen after test. Damages of the clamping bolts can be clearly seen in the figure. More experimental results can be found in the original document by Wanitkorkul (2003).

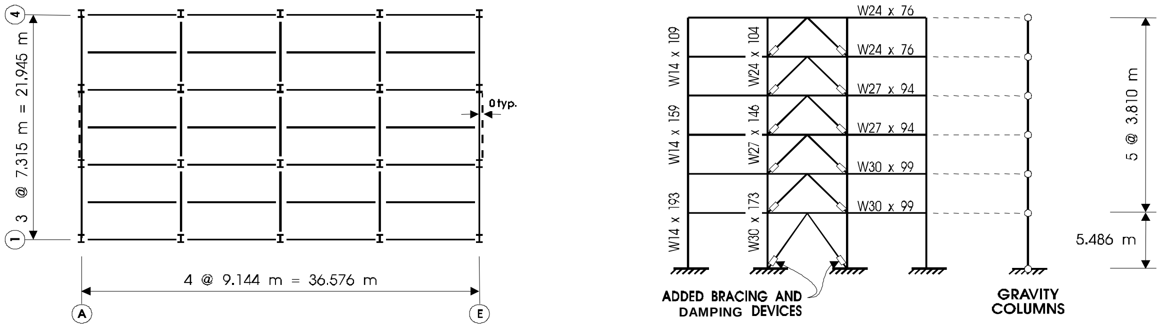
3. THE SLOTTED-BOLTED CONNECTION WITH RESTRAINERS

With the unacceptable behaviour of SBC under bolt impact witnessed, it is obvious that remedy is needed to prevent bolt impact. The restraining concept proposed by Roik et al. (1988) to prevent undesirable bolt impact is adopted in this study, with slight modification. The device with restrainers will slip at the predetermined slip load F_s. The restrainers will be activated when the slip travel is larger than the provided slip distance Δ_g, which results in increasing of the resisting force of the device at a stiffness K_r. This restraining force is further limited to a threshold value F_{max} which remains constant at the second yield plateau. Figure 4 shows the force-slip travel relationship of a slotted-bolted connection with restrainers. The restraining stiffness K_r of the device can be selected as suggested by Roik et al. (1988). It should be noted that the change in stiffness at different stages is advantageous in suppressing the build-up of resonance, should there be a tendency for such an event.

4. STEEL BUILDING

4.1 Building Model

We first take the case study of the steel structure considered by Filiatrault et al. (2001). The building is a 6-storey steel structure, rectangular in plan, and is braced by two exterior moment-resisting frames. Gravity loads acting on the frame during the earthquake are assumed equal to 3.8 kPa from roof dead load, 4.5 kPa from floor dead load, 0.7 kPa from floor live load, and 1.7 kPa from weight of the exterior cladding. Steel grade A36 is used for all members. The building, designed for seismic zone 4 in the United States, would not survive strong earthquakes when weld fractures occur in the welded beam-to-column connections. For retrofit, chevron-brace members are introduced in the central bay of the two exterior moment-resisting frames as shown in Fig. 5. The steel section HSS 300 mm x 300 mm x 15 mm is used for all chevron-brace elements. Slotted-bolted connections are incorporated at one end of all bracing members. Both conventional SBC and slotted-bolted connections with restrainers (SBC-R) are considered. The fundamental vibration period of the original building was 1.30 sec and was reduced to 0.67 sec after retrofitted with the proposed system.



Plan

Elevation

Figure 5 Steel Building Model and Retrofit Scheme

Because of symmetry, only one exterior moment-resisting frame was modeled as a two-dimensional structure. Floor slabs and architectural elements were excluded. The panel zones of the beam-column joints were assumed to have no panel shear deformation and yielding during strong excitations. Large displacement effect was also considered in the analyses. P-Δ effect from interior columns was included by introducing a pin-ended gravity column into the building model, which represents all interior columns. Total gravity loads acting on the interior columns were applied to the gravity columns. Both the exterior frames and the gravity columns were constrained to undergo the same lateral displacement at each floor, representing a rigid floor diaphragm assumption. Bilinear moment-curvature relation with a curvature-hardening ratio of 2% was assigned to all columns.

The flexural strength degradation model suggested by Filiatrault et al. (2001) was introduced at both ends of all beams to account for the brittle behaviour of welded beam-to-column connections. It was assumed that the strength degradation was independent in positive and negative bending. Only fractures at the beam-to-column interfaces were considered. An elasto-plastic moment-curvature relation was specified for all beam elements. The connections were assumed to have no loss in shear capacity when weld fractures occurred. More details on the building model can be found in the original paper.

4.2 Parameters for Conventional Slotted-Bolted Connection (SBC) and Slotted-Bolted Connection with Restrainers (SBC-R) Model

For SBC, a slip load value (F_{s1}) of 1524-kN was assigned for the device on the first floor, which was equal to 40% of the buckling strength (P_{b1}) of the corresponding bracing element. In the case of SBC with restrainers, the maximum restraining force (F_{max}) was limited to 60% of bracing buckling load to avoid damages to the bracing members, Based on the design procedure proposed by Filiatrault and Cherry (1990), slip loads of the connections on the second to the sixth floor were assigned a value equal to 80% of slip load on the first floor. Similarly, limits on restraining forces on the other floors were also assigned to be 80% of the maximum restraining force of the device on the first floor. The restraining stiffness was set equal to the corresponding bracing stiffness on each floor. A limit on slip travel was selected based on the maximum slip required in the most severe case, which will be discussed later. Axial springs with an elasto-plastic axial force-displacement relationship were used for all bracing members.

4.3 Earthquake Ground Motions

Three near-field earthquakes were considered in the nonlinear dynamic analyses, i.e. the 1989 Loma Prieta and the 1966 Parkfield earthquakes, which have the same peak ground acceleration of 0.48g, and the 1940 El Centro record with PGA's of 0.34g. Figure 6 depicts the accelerograms and the pseudo-acceleration response spectra with 5% damping associated with these earthquakes.

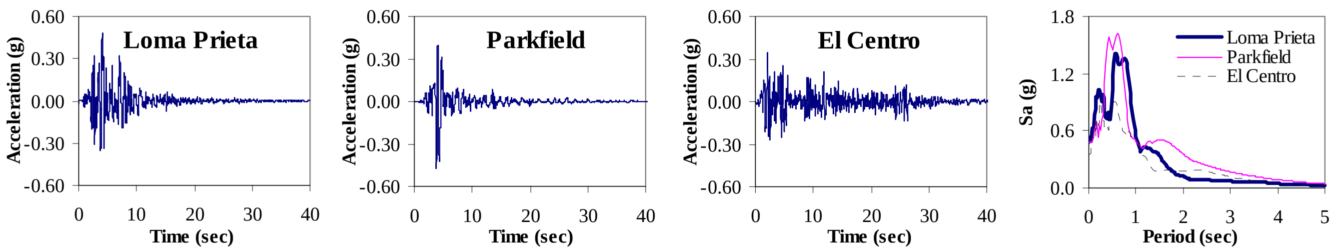


Figure 6 Accelerograms and Pseudo-Acceleration Response Spectra with 5% Damping

4.4 Performance

Nonlinear dynamic analyses were performed using the computer program RUAUMOKO (Carr

2000). The unretrofitted building collapses under all earthquakes considered with the peak inter-storey drift greater than 5% which indicates that retrofit is required for this structure.

4.4.1 Performance of Building with SBC

The most severe scenario results from the Parkfield earthquake, which results in the peak inter-storey drift of about 1.2% on the ground floor and the maximum curvature ductility of 3.8 in the ground floor columns. Damage is concentrated only on the first two floors with no yielding taking place on the others. The Loma Prieta earthquake, having the second highest spectral value, results in the maximum inter-storey drift of 0.8% and a peak curvature ductility of 2.7 in the ground floor columns with less damage compared with Parkfield. For the El Centro motion, it causes the least damage which is consistent with its lowest spectral value compared with other near-field records. Figure 7 illustrates the peak slip travels of the device obtained from analyses. The maximum slips of the device on the first and second floors are 35 and 20 mm, respectively, which result from the case of the Parkfield earthquake. These slip values are used for calculation of slip limits as discussed in the next section.

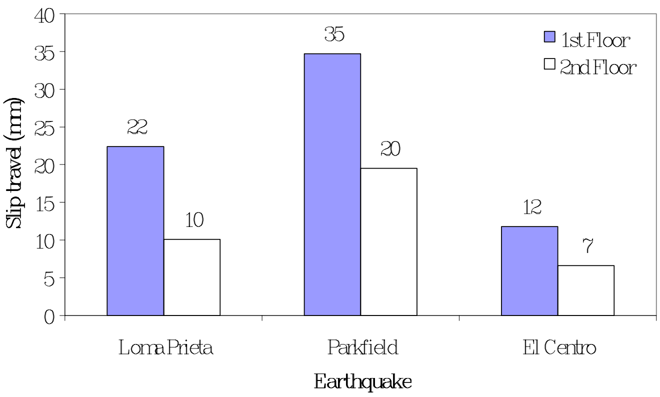


Figure 7 Maximum Slip Travel of SBC: $F_{s1}/P_{b1} = 0.40$

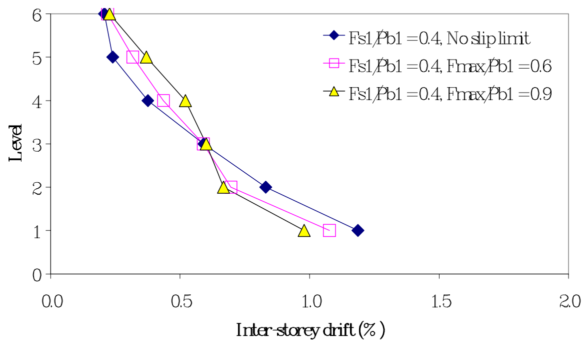


Figure 8 Peak Inter-Storey Drifts: Parkfield Earthquake

4.4.2 Performance of Building with SBC-R

Half of the maximum slip travel resulted from the case of Parkfield earthquake was applied to the SBC-R model as the slip threshold for activating the restrainers in the SBC-R model. Hence, the slip travel of the SBC-R on the first floor was limited to 17.5 mm when restraining effect takes place. However, rather than using different slip limits on each floor, the slip limit assigned to all devices on the other floors was 10 mm which was half of the maximum slip occurred on the second floor.

Figure 8 presents the peak inter-storey drifts of the structure subjected to the Parkfield earthquake. For the case of Loma Prieta, the maximum slip travel is close to the limit value; hence, responses from the system without and with limited slip are similar. For Parkfield excitation, with limited slip, the peak floor displacement is reduced by 11%. The inter-storey drifts on the first and the second floors are reduced by 9 and 16%, respectively, while those on the other floors increases by 6-33% because restraining actions stiffen the lower floors compared to other floors. Although the inter-storey drifts in those upper floors increase, the maximum value is still less than the immediate occupancy limit of 0.7% as per ATC (2000). The maximum curvature ductility in the ground floor columns is reduced to 3.5 which amounts to 8% reduction while there are no yielding of columns on the other floors.

It seems that the advantage of restrainers in terms of response reduction is not promising.

However, it is worth noting that, without restrainers, bolt impact may occur should the intended slip travel be accidentally reduced by, for instance, error in workmanship. Significant degradation of the friction force of the device would result as mentioned earlier, which can result in much worse performance or even collapse of the structure. The advantage of the restraining action is evident from the damper force-slip travel plot shown in Fig. 9. With the maximum normalized restraining force (F_{max}/P_{b1}) of 0.60, the maximum slip travel is reduced by 15% from the case without restrainers. Further reduction of 20% results when F_{max}/P_{b1} is increased to 0.90.The shorter travel obviously provides extra safety against the undesirable bolt impact. Hence, utilization of restrainers in slotted-bolted connections is desirable and beneficial.

Although the benefits of SBC-R are evident, the device also induces larger base shear to the system. Figure 10 shows the peak base shear with different slip loads and restraining force limits. With restrainers, the total base shear of the retrofitted structure increases, which directly affects the foundation system. The magnitude of the additional base shear induced by SBC-R depends on the level of slip load and restraining-force limit, and the earthquake considered. For the case of the Parkfield earthquake with a 40% normalized slip load, the total base shear increases by 15% and 39% with the normalized restraining force of 60% and 90%, respectively, while the corresponding increase for the case of Loma Prieta excitation is about 15% only.

It should be noted that the selected parameters, i.e. slip load, maximum restraining force, etc., are for demonstration purposes. To determine the optimum values, more parametric studies are required. This is out of scope of this study.

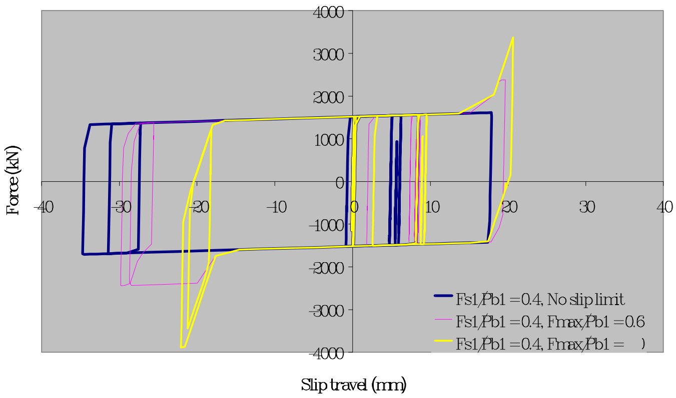


Figure 9 Damper Force- Slip Travel of SBC and SBC-R: Parkfield Earthquake

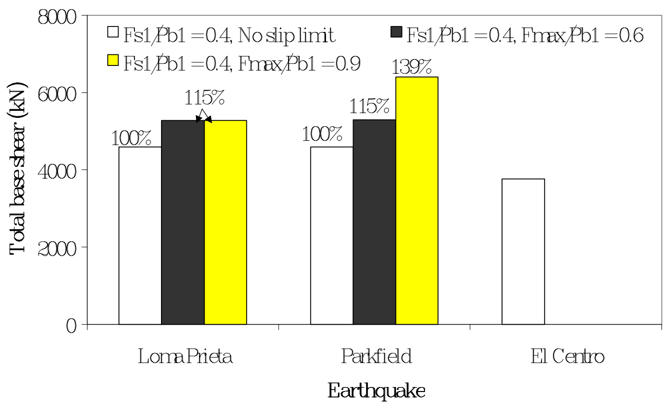


Figure 10 Total Base Shear for Retrofitting with Different Restraining Force Limits

5. RC BUILDING

5.1 Building Model

An existing 6-storey reinforced concrete building with brick masonry, which has been in use as an apartment building in suburban Bangkok (Bang Na area), Thailand since 1989, is next considered for seismic performance evaluation. It has typical details which represent many buildings in Bangkok. The exterior frames have masonry walls in all spans and all storeys except the ground floor level while the interior frames have masonry walls only in the exterior spans from the second to the fifth floors. Figure 11 shows the plan and elevation views of the building. Because of symmetry, only half of the building was modeled. The reinforcement details of the beams and columns are typical of non-ductile design in Thailand (Wanitkorkul 2003).

The Takeda hysteretic model was assigned for all beams and columns. A flexural strength degradation model was also introduced to the inelastic springs of all beam and column elements when the ultimate section capacity was reached to simulate the post peak behaviour of the member. The model simulating the brick masonry wall proposed by Crisafulli (1997) was also included. Figure 12 depicts the masonry wall model. The strength of each spring representing masonry can be calculated as follows:

For shear spring, $$P_{u,shear} = \frac{\tau_m L_m t}{\cos\alpha} \tag{1}$$

For axial spring, $$P_{u,axial} = \frac{f'_m A_{ms}}{2} \tag{2}$$

where $P_{u,shear}$ *and* $P_{u,axial}$ are the strengths of the equivalent shear and axial springs, respectively; τ_m and L_m are the shear strength and the horizontal length of the masonry wall, respectively; t is the thickness of the wall; α is the inclination angle of the equivalent shear spring; f'_m is the compressive strength of the masonry; and A_{ms} is the area of the equivalent masonry strut. More details can be found in the original document.

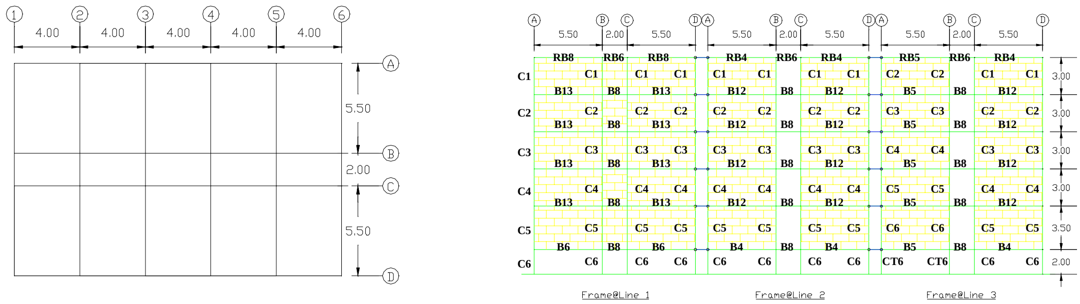


Figure 11 Plan and Elevation of the RC Building

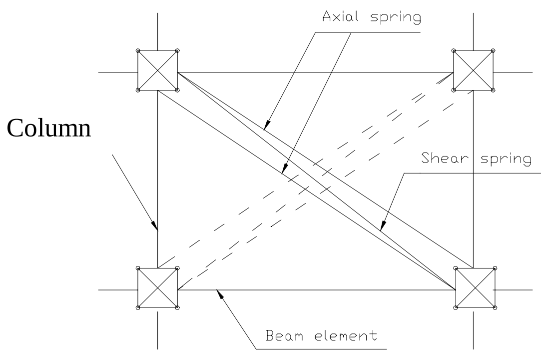


Figure 12 Masonry Wall Modeling

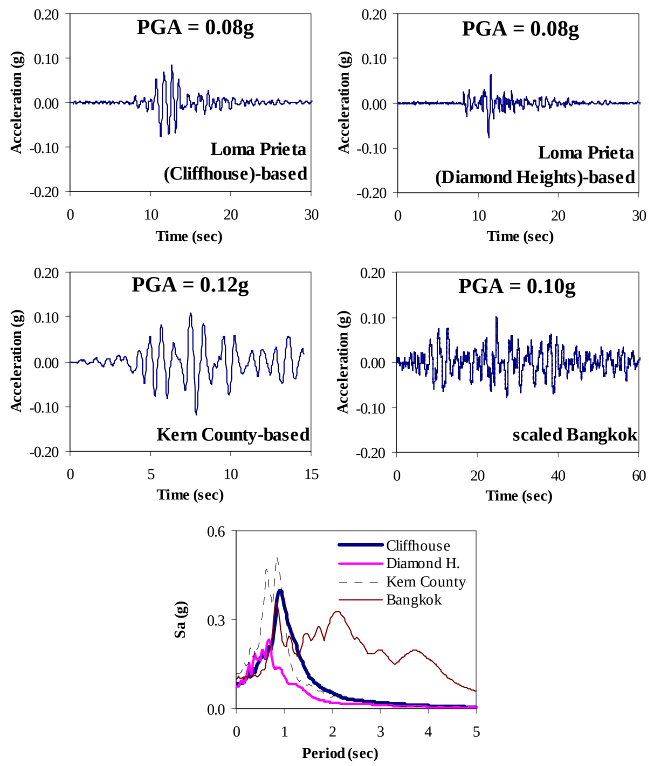


Figure 13 Accelerograms and Pseudo-Acceleration Response Spectra with 5% Damping

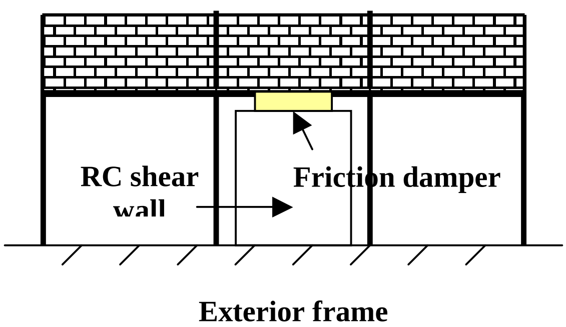


Figure 14 Retrofit Scheme

5.2 Earthquake Ground Motions

The ground motions induced by distant earthquakes were simulated using the available soil properties from field tests in Bangkok area (Wanitkorkul 2003). Three strong rock outcrop motions were selected as input motions for the analyses, namely, the 17 October 1989 Loma Prieta earthquake records at the Cliffhouse and the Diamond Heights stations and the Pasadena signal from the 1952 Kern County, California earthquake. Figure 13 depicts the resulting ground motions and their response spectra. The 1995 Bangkok excitation with scaled peak acceleration of 0.10g was also included in the investigation.

5.3 Performance

5.3.1 Response of Unretrofitted Building

For all the earthquakes considered, the damages are concentrated on the first few floors due to the poor structural system. Failure of two thirds of the ground floor columns occur at the storey drift level of less than 0.8%. This indicates non-ductile behaviour of the system. The Loma Prieta (Cliffhouse)-based excitation put the most severe demand on the building. So results will be presented based mainly on those from the Loma Prieta-based earthquake.

The maximum curvature ductility of 9.7 occurs in a few ground floor columns of the interior frames which is about 139% of the curvature capacity at ultimate moment resistance. These columns can be regarded as totally collapsed. However, no failures occur in ground floor columns of the exterior frames because of the lower level of axial load. High axial load level increases the strength of the column; however, it also decreases the ductility capacity. As for the brick walls, some infilled panels on the second and the third floor fail in the sliding-shear failure mode because of the low-strength of the mortar used in their bed joints.

5.3.2 Retrofit Scheme

The proposed retrofit scheme consists of incorporating a reinforced concrete shear wall on the ground floor with a frictional damping device connecting the wall and the beam in the central bay of the exterior frame (Fig. 14). This system has the advantage over the bracings with friction dampers system in that it does not require strengthening of the existing foundations, as will be explained later. Of course, the wall itself has to be constructed on a (new) separate foundation. This would be easier to build and also more economical. This scheme also does not induce high forces into the vulnerable beam-column joints. The 225-mm thick x 1200-mm wide shear wall is designed to respond without any yielding to avoid the pinching behaviour that would result in poor performance. This can be achieved with the use of the friction dampers which can limit the force transferred to the wall and provide an additional source of energy dissipation.

5.3.3 Performance of Building Retrofitted with SBC

After a couple of trials, a value of 3% of the total seismic weight (W), or 320 kN, was selected as the slip load of the device. The peak inter-storey drifts of the unretrofitted and retrofitted buildings are compared in Fig. 15. Only the results from the most severe earthquake, i.e. the Loma Prieta (Cliffhouse)-based excitation, are shown here. In general, retrofit reduces responses on the first floor. The retrofit system reduces the peak inter-storey drifts by 20%, compared with the unretrofitted building. However, stiffening the first floor induces more damage on the upper floors. Damage is distributed to the other storeys instead of concentrating on the first floor as indicated by the increase in the inter-storey drifts on those upper floors. However, even with an increase in the peak inter-storey drifts ranging between 3-17%, the resulting curvatures on the upper storeys are still small that neither the beams nor columns reach flexural failures.

Considering only the inter-storey drifts, it seems that the retrofit system is not so effective. However, in terms of collapse prevention, the retrofit system proves beneficial in preventing failures of the ground floor columns. The maximum curvature ductility of the ground floor columns is reduced from 9.7 in the unretrofitted system to 6.5 with the presence of the retrofit system in the worst scenario earthquake. This amounts to a reduction of 33% in the maximum curvature ductility demand of the ground floor columns compared with the unretrofitted building whose columns fail under the same excitation.

The maximum slip travel of the device resulting from each excitation, when no restrainer is provided to limit the slip travel, is shown in Fig. 16. The maximum slip travel of 8.4 mm results from the case of earthquake simulated by using the Loma Prieta (Cliffhouse station) record.

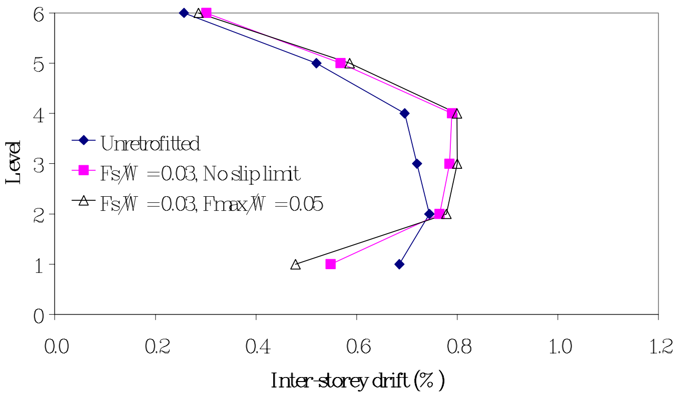


Figure 15 Peak Inter-Storey Drifts: Loma Prieta (Cliffhouse)-Based Earthquake

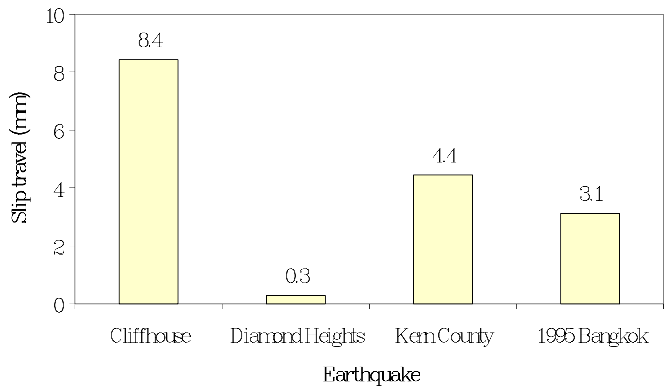


Figure 16 Peak Slip Travel of Friction Damper

5.3.4 Performance of Building Retrofitted with SBC-R

Half of the maximum slip required by the case of the earthquake generated from the scaled Loma Prieta earthquake (Cliffhouse station), or 4.2 mm, is assigned as the threshold slip for activating the restraining action, and hence additional restraining force. This will be referred to as the limited-slip case. With this value of slip limit, the other generated signals cause (practically) no impact in the device. Thus, only the excitation simulated by using the scaled Loma Prieta (Cliffhouse station) record is considered in the analyses. Since a high maximum restraining force induces high demand on the foundations, only 5% of total seismic weight of the building, or 510 kN, is assigned as the limit on the maximum restraining force.

Figure 15 compares the computed peak inter-storey drifts of the building for different cases. Similar to the previous case, retrofit with limited-slip friction damper reduces the peak inter-storey drift response on the first floor by about 13% but slightly increases those on the upper floors by 1-3% compared with the case of conventional damper. No flexural failures occur in any structural element. With slip limit, the maximum deformation of the ground floor column is reduced; hence, less damage occurs. The maximum curvature ductility of the ground floor column is reduced to 5.1 which is decreased by 22% compared with the case of no slip limit.

Comparison of the base shears for different building systems is shown in Fig. 17. The base shear resisted by the original frames is reduced by 7% compared with the retrofitted structure with no slip-limit friction damper, while the total base shear increases by 3%. The decrease in base shear in the original frames when retrofitted with SBC-R (or even without restrainers) makes it unnecessary to strengthen the existing foundations, which is an advantage. Figure 18 shows the force-slip travel of the device. With slip limit, maximum slip travel decreases to 5.3 mm which is a 37% reduction compared with the case without limit on slip travel (8.4 mm). Again, the restrainers provide extra safety to the device against bolt impact, which is an advantage over the conventional one.

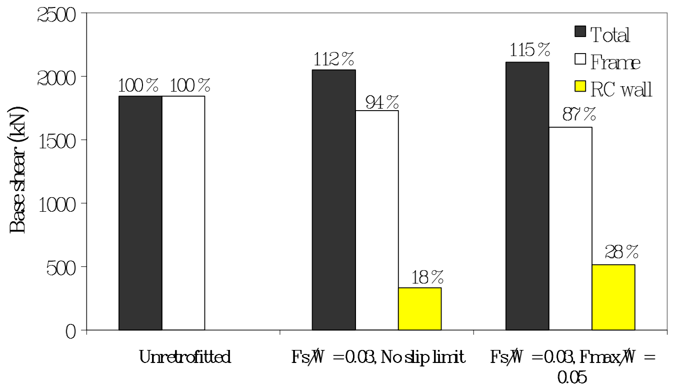


Figure 17 Base Shear Distributions: Loma Prieta (Cliffhouse)-Based Earthquake

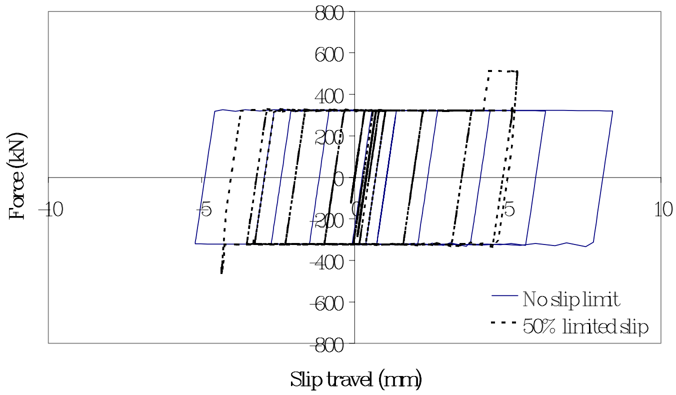


Figure 18 Force-Slip Travel of Friction Damper: Loma Prieta (Cliffhouse)-Based Earthquake, $F_S/W = 0.03$, $F_{max}/W = 0.05$

5.3.5 Storey-Level Damage Index

To quantify damage on each floor the Park and Ang (1985) damage index (DI) is adopted with slight modification to determine the storey-level damage index of each storey as follows:

$$DI = \left(\frac{U_{max}}{U_u}\right)_{storey} + \left[\frac{\beta_s(\int dE_{h,s})_{storey} + \beta_m(\int dE_{h,m})_{storey}}{(F_u \times U_u)_{storey}}\right] \quad (3)$$

where U_{max} and U_u are the maximum floor displacement and the ultimate floor displacement capacity, respectively; F_u is the ultimate shear strength of the storey associated with U_u, obtained from the pushover analysis; $dE_{h,s}$ and $dE_{h,m}$ are the incremental hysteretic energies absorbed by structural components and masonry panels, respectively; and β_s and β_m are the model constant parameters for structural components and masonry, respectively. A value of 0.1 for β_s was suggested by Valles et al. (1996). Ang and Kwok (1987) suggested a value of 0.075 for unreinforced brick masonry. Hence, these values were adopted for damage index calculation.

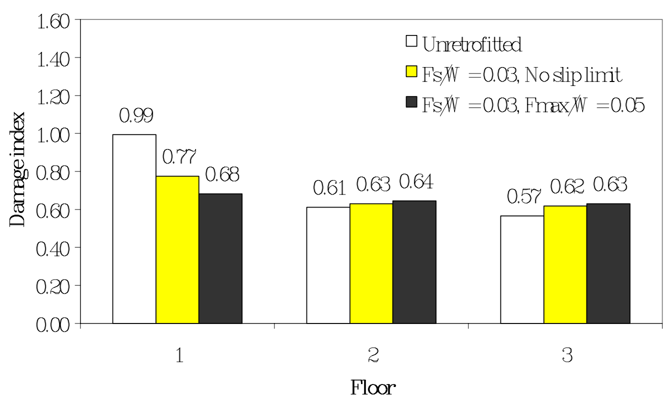


Figure 19 Storey Damage Index: Loma Prieta (Cliffhouse)-Based Earthquake

Figure 19 depicts the calculated damage indices for buildings subjected to Loma Prieta (Cliffhouse)-based earthquakes. Since damages generally concentrate on the first three floors, only damage indices of these storeys are presented. It should be noted that for the unretrofitted building under the worst scenario earthquake (Loma Prieta (Cliffhouse)-based ground motion with PGA of

0.08g), the ground-floor damage index is 0.99 which indicates that the storey is on verge of collapse. The damage indices of the other storeys are much less than 1, indicating no failure is imminent. Upon retrofitting with no slip limit friction damper, the damage index of the ground level is reduced to 0.77, a reduction of 23% compared with the unretrofitted building, while the damage indices of the other storeys are increased insignificantly. Further reduction in ground-floor damage index is achieved by utilizing slip limit with the maximum normalized force of 0.05. In this case, the ground-floor damage index is reduced to 0.68, which is 12% lower than the case without slip limit. These results are consistent with the findings described earlier.

6. CONCLUSIONS

This study focuses on the behaviour of the slotted-bolted connections (SBC) when the slip travel exceeds the provided slot length. The effectiveness of the dampers with restrainers (SBC-R) is demonstrated. Nonlinear dynamic analyses on two prototype buildings equipped with SBC and SBC-R were performed to evaluate the performance of the SBC-R as a seismic protection device. The following conclusions can be drawn from this study:

Bolt impact causes severe damage to the clamping bolts with significant degradation of the friction force of the device. Loss of friction can be more than 50%. Therefore, accidental impact of bolts in an unforeseen event can lead to disastrous result.

For the steel building under strong ground motions, the SBC-R is superior to the conventional SBC in the worst loading case, resulting in reduction of the peak inter-storey drift by 9-16% and the peak slip travel by 15-30% depending on the maximum normalized restraining force.

Although the peak inter-storey drift and the peak slip travel of the device decrease with increasing in the restraining force limit, the extra gain has to be balanced with the potential of buckling of the bracing at the high restraining force.

For the RC building under moderate ground motions, the limited-slip friction damper can reduce the peak curvature ductility demand by 22% compared with the conventional damper, which results in less damage in the ground floor columns.

The peak slip travel of the device is reduced by 37% with the use of restrainers, similar to the case of the steel building. This provides extra safety against bolt impact which can damage the damper.

Higher restraining force induces higher base shear; hence the adequacy of the foundations has to be properly addressed as well in retrofitting of buildings.

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