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PRELIMINARY STUDY ON THE BEHAVIOUR OF A COLUMN-TOP ISOLATION SYSTEM

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ABSTRACT: Seismic isolation offers considerable improvement to earthquake resistance by means of a flexible layer between the ground and building. However, base isolation systems can add considerable expense to projects, especially for seismic retrofit applications. This is because significant funds must go towards constructing a seismic gap and additional rigid diaphragm at the base of the building. A column-top isolation system, in which isolators are placed directly at the column tops, below the floor diaphragm, has been suggested to mitigate some of the costs associated with isolation retrofit. This study considers the behaviour of a column-top isolation system using elastomeric bearings, accounting for rotational effects at the isolator-column interface and the P-Δ effects due to large isolation layer displacements. A steel moment frame was designed using historic code provisions and retrofitted with column-top isolation. Nonlinear time history analyses of the pre-retrofit and retrofit buildings were conducted under multiple current hazard levels to assess the performance and validity of the retrofit scheme.

1. Introduction

As seismic design provisions in the National Building Code of Canada have developed over the last 70 years, a significant portion of Canadian infrastructure is now known to be at risk during current expected seismic events (Caruso-Juliano, et al. 2014). Base isolation has been a popular system for significantly improving seismic performance, and has been effective in retrofit applications for improving the response of older structures (Matsagar and Jangid 2008). By providing a flexible layer between the ground and the structure as seen in Fig. 1 (middle), the building is decoupled from ground motions and experiences reduced story drifts and floor accelerations. Aside from structural performance, isolation also reduces the damage to non-structural components, which typically account for a major portion of the cost for most commercial buildings (Kelly 1993). For these reasons, seismic isolation has been recommended by the Federal Emergency Management Agency as an effective method for reducing the impacts caused by earthquakes (FEMA 2011).

Retrofit projects using base isolation, however, can be of considerable expense and are typically only considered in the case of historical structures. Typically, isolation layers are bounded by rigid diaphragms on the top and bottom which distribute forces, maintain equal displacement demands for all bearings, and keep the bearing end plates parallel. In retrofit installations, an additional diaphragm must be constructed for this purpose and contributes to the project expenses. Typical installations also require a seismic gap to be excavated with sacrificial moat covers to allow lateral motion during excitation, which also significantly contributes to project time and costs. To mitigate the cost associated with isolation retrofit, it has been suggested to place isolation bearings between the tops of the first story columns and the second story diaphragm (Matsagar and Jangid 2008), as shown in Fig. 1 (right). In this configuration, the additional diaphragm and seismic gap beneath the building are no longer necessary, however,

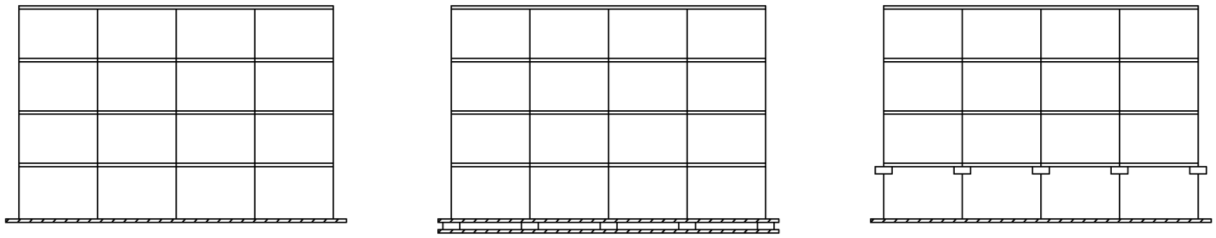

Fig. 1 **– Typical construction of a (left) moment resisting frame; (middle) base isolated frame; (right) column-top isolated frame**

bearings may not maintain parallel end plates due to flexible connecting elements. Previous studies have shown that as bearing end conditions change, the buckling behaviour (Imbimbo and Kelly 1997), and lateral stiffness (Ravari, et al. 2012) change, and thus, serious investigation is required.

This study considers the retrofit of a typical steel moment resisting frame (MRF), Fig. 1 (left), developed using loads prescribed in the 1965 National Building Code of Canada. The frame was outfitted with elastomeric bearings at the top of the first floor columns, herein referred to as column-top isolation. To account for the change in bearing end conditions, an analytical model for elastomeric bearings accounting for rotational flexibility, P-Δ demands, and stability was formulated. The performance was compared with the original structure under suites of ground motions matched to current seismic hazards at levels of 10% probability of exceedance in 50 years (10%/50yr) and 2% probability of exceedance in 50 years (2%/50yr). The demands on the supporting columns are assessed and compared to investigate minimum design requirements.

2. Building Design and Modelling

A four-story office building located in Abbotsford, BC, was selected for retrofit and comparison for this study. The frame was selected to have 4 bays with 6 meter spacing, and 4 stories with 4 meter heights. The building was designed as the MRF shown in Fig. 1 (left) using historical code provisions from the 1965 National Building Code of Canada. It was assumed that seismic loading governed and wind loading was neglected in the design. The 1965 building code prescribed the base shear as given below in Eq. 1, which includes the building weight, W, and a design load parameter, K (NRCC 1965).

$$V = K \cdot W \quad \text{where: } K = R \cdot C \cdot I \cdot F \cdot S \tag{1}$$

The build weight was defined as the dead load plus any loads from storage use, service equipment and machinery. The determination of the design load parameter, K, consisted of many factors to account for site specific hazards. The R factor, analogous to spectral acceleration in modern codes, was equal to the seismic zone number from the seismic zoning map shown in Fig. 2, except for zone 3 which was assigned an R factor of 4. The construction factor, C, analogous to ductility and overstrength, was equal to 0.75 for moment resisting frames following particular criteria, and 1.25 for all other buildings. The importance factor, I, similar to modern codes, was assigned as 1.3 for high importance buildings and 1.0 for all others. The foundation factor, F, analogous to current site class factors, was given as 1.5 for buildings on highly compressible soil and 1.0 for all other soils. Lastly, the factor S was given as shown in Eq. 2, where N is the number of stories in the building, to account for the effects of the structural period.

$$S = \frac{0.25}{9 + N} \tag{2}$$

Once the base shear was obtained, the code provisions distributed the base shear proportional to floor heights and weights in a similar fashion to modern practice using the equivalent lateral force procedure.

The site location of Abbotsford, BC is specified to be in zone 3 on the seismic zoning map, and was considered to be on normal soil conditions. Following the procedures outlined above, the design load parameter was determined to be K = 0.058, or a base shear of 5.8% of the seismic weight. The historical code prescribed no limits to lateral deflection but specified that adequate stability must be provided. The preliminary design had excessive lateral deflections using the loads prescribed, so modern interstory drift

Fig. 2 – Historical seismic zoning map (NRCC 1953)

limits of 2.5% were imposed to adhere to stability requirements. The resulting structure had a fundamental period of 1.2 seconds using a seismic weight defined by modern standards, which includes snow loading.

The structure was then retrofitted using a column-top isolation strategy as shown in Fig. 1 (right). The frame sections were unchanged, but first floor columns were shortened to accommodate the installation of isolation bearings so that floor heights were maintained. The isolation system was designed using a conventional design process by assuming the end plates remain parallel. As end plates are made flexible, the stiffness of the isolation layer is reduced, causing the isolation period to be slightly longer than initially calculated. If rotations are minor, this increase in period is negligible. Two different isolation bearings were designed for interior and perimeter columns to account for the different axial loads, but due to the light loads on all bearings, it was difficult to achieve an isolation period longer than 2.50 seconds. Similar to traditional base isolation, the isolation mode dominated the response of the retrofit structure, and the fundamental period of the isolated building was 2.48 seconds.

Deaggregation maps for Abbotsford, BC were used to determine expected hazards at both the 10%/50yr and 2%/50yr levels. Both hazard level deaggregations indicated a mean magnitude of 6.5 to 8.0 and a distance of less than 100 km for the majority of the hazards in the 0.5 to 3 second period range. A suite of ground motions for both hazard levels were selected based on the criteria from the deaggregation maps. The selected motions were scaled to a target response spectrum defined by modern code provisions in the 2010 National Building Code of Canada (NRCC 2010), and are summarized in Table 1. Scaling was

Table 1 – Selected ground motions

Earthquake		**Year**	**Station**	**Magnitude**	**Scale Factor**
10%/50yr	Tabas	1978	Dayhook	7.35	0.58
	Chuetsu-oki	2007	Kawaguchi	6.8	1.06
	Iwate	2008	Tamati Ono	6.9	0.97
	Darfield	2010	PEEC	7.0	2.31
2%/50yr	Loma Prieta	1989	Gilroy Array #6	6.93	1.93
	Northridge	1994	LA – UCLA Grounds	6.69	1.09
	Niigata	2004	NIGH11	6.63	0.73
	Iwate	2008	AKT023	6.9	1.17

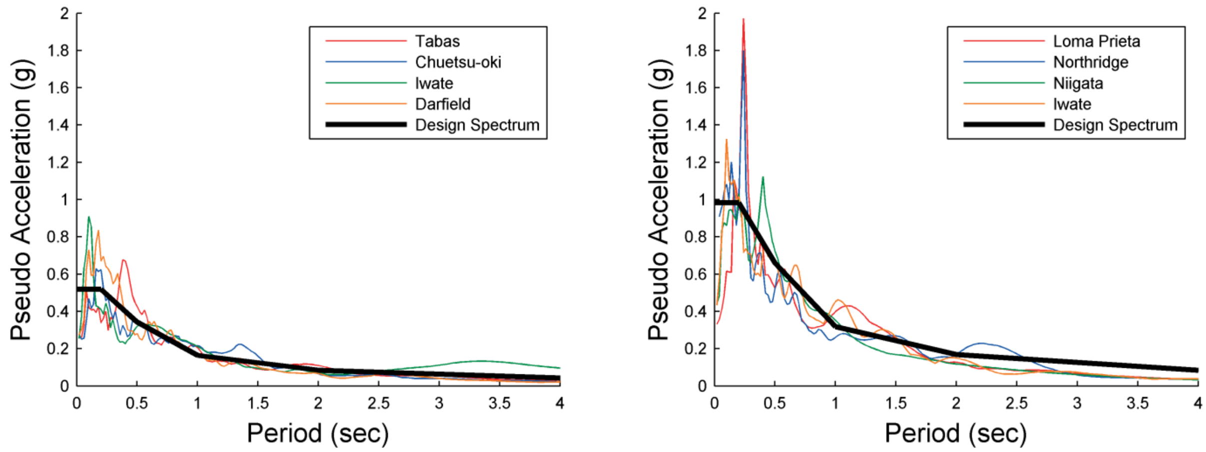

Fig. 3 – Response spectra of selected motions for (left) 10%/50yr; (right) 2%/50yr

performed by minimizing the mean square error of the motions and the target spectrum in the 0.5 to 2.5 second period range so that both the pre-retrofit and retrofit frame could be captured. A comparison of the response spectra of the selected motions to the target spectrum can be found in Fig. 3.

The scaled motions were used to conduct nonlinear time history analyses with the use of OpenSees software (McKenna and Fenves 2006). Models of both the MRF and retrofit frame were created primarily with nonlinear displacement based beam-columns with a Menegotto-Pinto model to capture typical steel behaviour. A damping of 5% was assumed for the structure, and a damping of 5% was also assumed for the isolation system as natural rubber bearings tend to have low energy dissipation. A new element was created and used to model the behaviour of elastomeric bearings under combined rotational-translational motion. The element is defined by two nodes with the positive sign convention shown in Fig. 4, and is capable of capturing the effects of end rotation on lateral stiffness and buckling.

3. Response Comparison

The global behaviour of both the pre-retrofit MRF and the retrofit frame was investigated to assess the performance of the column-top isolation system as a retrofit strategy. The interstory drifts and floor accelerations under the 10%/50yr and the 2%/50yr hazard levels are presented in Fig. 5 and Fig. 6, respectively. It is important to note that the interstory drift of the first floor in the retrofit frame was considered as the drift experienced by the supporting column underneath the isolator, rather than the drift between the first and second floors. The displacement of the isolation level is not included in the drifts shown. The results show lower demands in the pre-retrofit MRF than expected. The design of the MRF frame was governed by a 2.5% interstory drift limit using the code specified loads, while the performance reached a maximum interstory drift of 1.3% during the Loma Prieta earthquake. This is attributed to a longer fundamental period than predicted by the code, causing lower diaphragm forces and resulting deformations than anticipated.

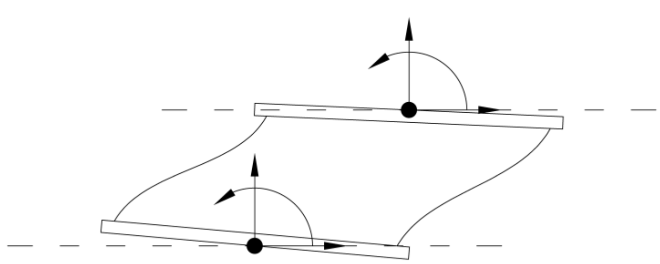

Figure 4 – Positive sign convention of formulated element

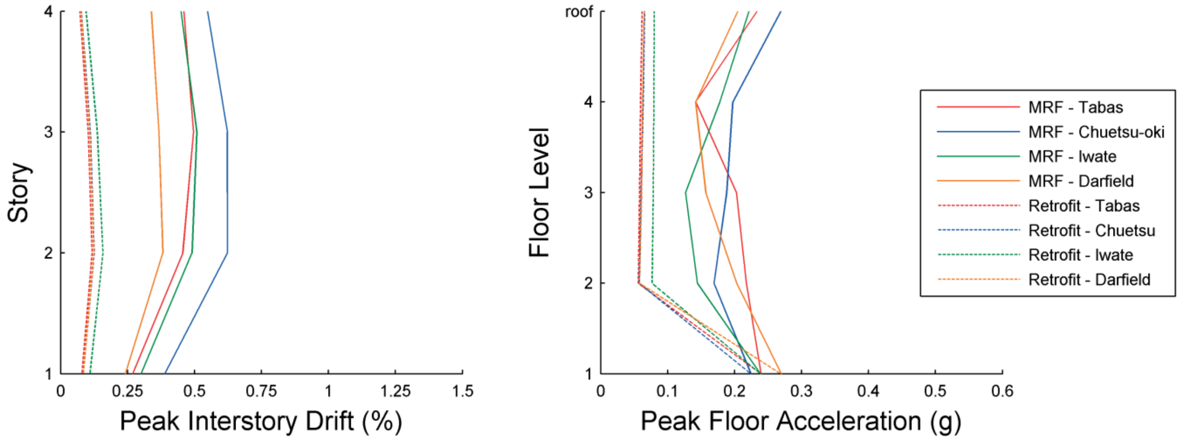

Fig. 5 – Responses under 10%/50yr motions: (left) peak interstory drifts; (right) peak floor accelerations

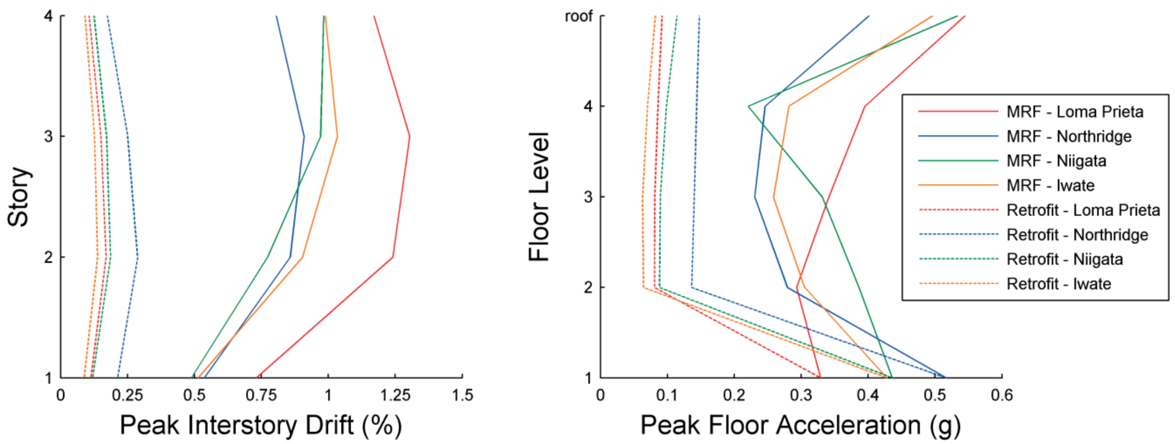

Fig. 6 – Responses under 2%/50yr motions: (left) peak interstory drifts; (right) peak floor accelerations

The results of Fig. 5 and Fig. 6 indicate effective reductions of interstory drifts and peak floor accelerations at both hazard levels with the introduction of column-top isolation. In the 10%/50yr motions, the retrofit strategy provided a mean reduction of 75% for peak interstory drifts and a mean reduction of 63% for peak floor accelerations above the first floor. During the 2%/50yr motions, the retrofit provided an 81% mean reduction in interstory drifts and a 69% mean reduction in peak floor accelerations for the isolated levels. The introduction of the retrofit strategy results in floor accelerations that are nearly constant above the isolation layer. Interstory drifts of the first story columns of the retrofit frame experienced a 69% mean reduction for the lower hazard level, and a 76% mean reduction at the higher hazard level. This reduction is a result of reduced lateral loads in the upper portion of the building.

To study the behaviour of the isolation bearings, the displacements and end plate rotations were recorded for each motion, and peak responses can be found in Table 2. The Northridge motion provided the highest responses because of long period content in the ground motion close to the structural period of 2.5 seconds. A sample of the time history response of a corner isolation bearing is shown in Fig. 7, displaying the displacement and end plate rotations during the Loma Prieta motion at the 2%/50yr hazard level. The top plate of the isolation bearing is connected to the diaphragm above the isolation layer while the bottom plate is connected to the supporting column. As the top connection is more rigid than the bottom, the bearing tops experience less rotational demand than the bottom and is verified in the response. The rotational response history of both end plates are in phase with the lateral displacement of the bearing, but are small because of the stiff framing elements designed for the pre-retrofit MRF.

Table 2 – Peak bearing responses for the 2%/50yr earthquakes

Earthquake	Displacement (mm)	Top Plate Rotation (rad)	Bottom Plate Rotation (rad)
Loma Prieta	111.2	0.0014	0.0018
Northridge	209.7	0.0023	0.0033
Niigata	117.7	0.0015	0.0019
Iwate	86.8	0.0011	0.0014

It is noted that at the resting position at the start and end of the motion, the top and bottom plates have differential rotations indicating that end plates are not parallel (see Fig. 7). This is due to the unsymmetric loading on the corner bearing which has only one beam framing into the connection above it. This slight but permanent rotation results in the peak end plate rotations to occur at different times. This can be contrasted with a bearing in the center of the frame, shown in Fig. 8, which has no differential rotations at rest, and the peak rotations for both top and bottom end plates occur at the same time. Comparing the responses shows that both bearings experience the same displacement and have a similar bottom plate rotation response because the similar connection conditions to the supporting columns. However, the top plate rotations are smaller in the center bearing because of the added stiffness due to beams framing into the connection above from both sides.

In the pre-retrofit frame, the first story corner columns experienced lower shear and bending moment demands than the first story interior columns, leading to different amounts of deformation in each column. However, the installation of the isolation system redistributed these demands so that the supporting columns experienced approximately equal shear and bending moment demands regardless of location. Bending moments varied slightly due to different axial loads causing P-Δ effects, but these bending moments were small in comparison to the bending moments caused by shears at the column tops. The shearing forces were distributed equally to each column through the second floor rigid diaphragm, and resulted in approximately equal deformations in all of the columns. Isolation bearings, being located between the rigid diaphragm and the equally deforming columns, all experienced equal displacement demands because of the redistribution of forces.

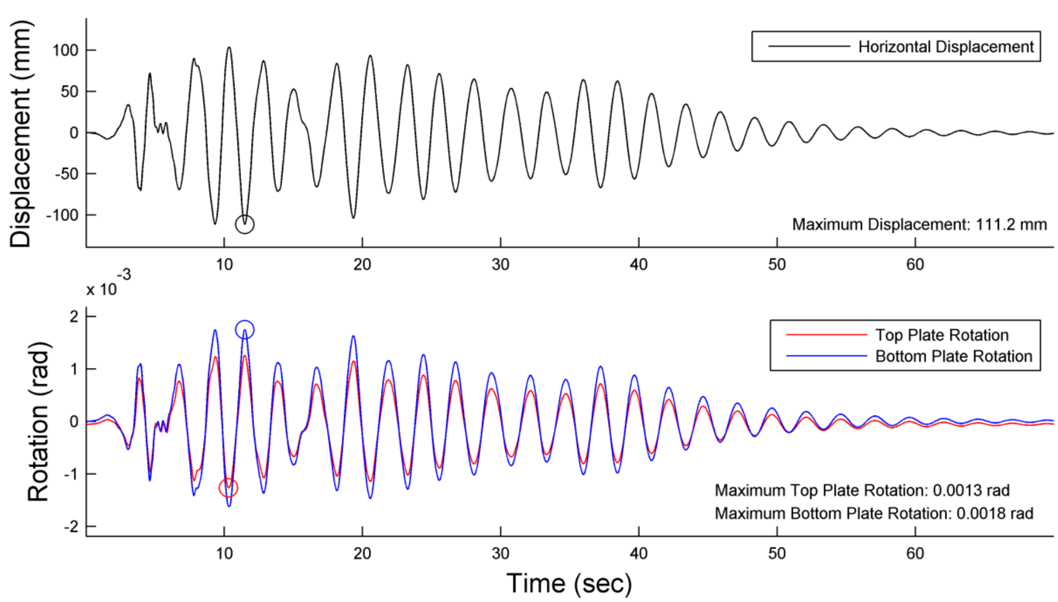

Fig. 7 – Corner bearing response during the 2%/50yr Loma Prieta earthquake

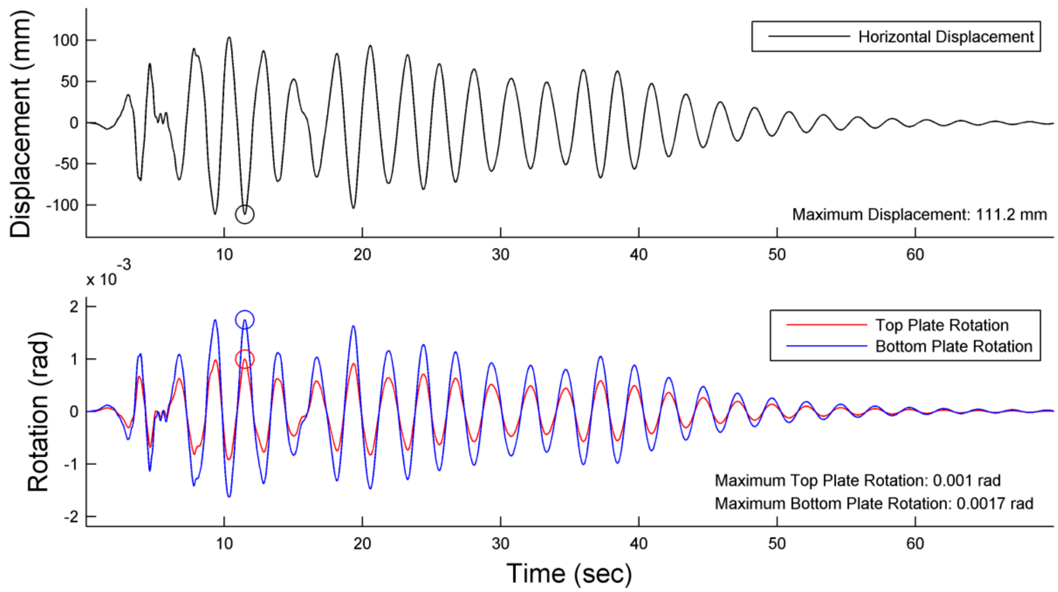

Fig. 8 – Center bearing response during the 2%/50yr Loma Prieta earthquake

The new demands imposed on the supporting columns become the key concern in this retrofit strategy in order to ensure safe and stable performance. Plastic hinge formation at the base of the supporting columns can form a soft story collapse mechanism, and prevention of this mechanism is paramount. While the introduction of an isolation layer will inherently decrease shears and moments in the columns, eliminating the bottom diaphragm means that the column top fixities are relaxed and decrease the column capacities. Peak bending moments at both ends of the supporting columns from each motion are given in Table 3. The results indicate significantly reduced demands on the first story columns with a mean reduction of 96% at the column tops, and a mean reduction of 77% at column bases. The large reduction in demands is attributed to the isolation layer, which relieved frame action that occurred in the pre-retrofit MRF. As the pre-retrofit MRF deformed, large moments occurred throughout the beams and columns in the structure, and were distributed into the first floor columns to be resisted by the foundation. In the retrofit frame, global demands were reduced by a lengthened structural period. The bending moments at the tops of the retrofit columns are further reduced due to the limited ability of the isolator bearings to transmit bending action from the frame above into the supporting columns.

Table 3 – Peak bending moments in first story columns, M_y = 1694 kN-m

Earthquake		MRF Column End Moments (kN-m)		Retrofit Column End Moments (kN-m)	
		Top	Bottom	Top	Bottom
10%/50yr	Tabas	316.2	669.0	10.2	162.6
	Chuetsu-oki	408.7	972.9	10.9	172.6
	Iwate	329.3	757.5	14.0	221.4
	Darfield	277.1	609.8	10.8	171.4
2%/50yr	Loma Prieta	706.4	1783.1	14.5	229.2
	Northridge	597.1	1355.2	27.0	432.8
	Niigata	638.4	1285.5	15.3	242.6
	Iwate	580.4	1287.1	11.3	178.8

Although the supporting columns experience a reduction in demands, a reduction in capacity is also present in the retrofit strategy. If bending moments at column tops are considered to be negligible in the retrofit frame, then the supporting columns act as cantilevers and the buckling load reduces. In this fixed-free case the critical load is significantly reduced. However, in the case of the retrofit frame studied, the pre-retrofit MRF columns were designed for large axial forces induced by the frame action described previously. Once the isolation system was installed, columns in the retrofit frame experience significantly lower axial demands and, thus, were far from the buckling loads.

Using the results of the behaviour of column-top isolation for retrofitting an MRF structure, recommendations were formulated for the preliminary design requirements for supporting columns. As discussed previously, supporting columns may be conservatively interpreted as cantilevers. Estimation of the maximum bending moment at the base of these columns may be determined by:

$$M_{base} = S_a(T)\frac{Wh_1}{N} + S_d(T)P_i + M_{bt} \tag{3}$$

where $S_a(T)$ is the spectral acceleration at the fundamental period T, W is the seismic weight on the building, h_1 is the height of the first story, N is the total number of bearings, $S_d(T)$ is the displacement at the fundamental period T, P_i is the axial load on the column under consideration, and M_{bt} is the bending moment at the bearing top. The bearing top moment term is largely a function of the relative flexural stiffness of the bearing and connecting elements, along with the peak bearing displacement. A simplified method of determining this bending moment is presently under study and will be presented in future publications.

4. Conclusions

A steel moment resisting frame for a typical four-story office building was designed based on loads prescribed in the 1965 National Building Code of Canada. The frame was modelled and analysed through nonlinear time history analyses based on modern seismic hazards, and was used as the basis of a retrofit project to improve its seismic performance. A seismic retrofit strategy was proposed whereby isolation bearings are placed between the tops of the first story columns and the second floor slab, eliminating the need for an additional rigid diaphragm and installation of a seismic gap. An element was created for use in OpenSees to model the behaviour of elastomeric bearings under combined displacement-rotation to capture the behaviour changes due to P-Δ and end plate rotations. The retrofit frame was modelled and compared with the pre-retrofit frame to determine the benefits of column-top isolation. Lastly, recommendations for design requirements were given and a formula for estimating the maximum base moments in the supporting columns was developed.

The main findings of the study are as follows:

- Column-top isolation provides significant reductions in interstory drifts and floor accelerations.
- The rotations of the bottom plate of the rubber isolators are larger than top plate rotations due to connecting framing elements. However, the end plate rotations tend to be negligible.
- The first story columns have a significant demand reduction due to a new distribution of bending moments, and so the retrofit strategy may not require strengthening of the columns. However, as plastic hinge formation at the base of the columns must be avoided, this should be checked on a project-to-project basis.

5. References

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