INFLUENCE OF SEISMIC ISOLATION LEVEL ON A BUILDING RESPONSE

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ABSTRACT :

Base isolation systems are applied in building systems to improve their seismic performance by isolating the structure from full intensity of the seismic input at the ground level. In some instances, due to the economic or environmental constraints, it is not possible to apply the isolation devices at the ground level. Instead, isolators may need to be placed at the top of ground floor columns. This is common when isolation is used in existing structures as a mitigation measure. The columns may then be subjected to altered internal forces below the isolation level. To study the consequences of this application, a simple five-story existing residential building was employed. An isolation system consisting of friction pendulum devices has been designed as a retrofitting alternative to improve the projected poor seismic performance of the building to resist effects given by the current Turkish seismic design code. Three dimensional model of the building is analyzed under the effect of recorded ground motion at the building region and the code specified design spectrum. In addition to the isolator displacements, the internal force demands from time history analyses have compared with the member capacities both before and after isolation. Results reveal that the increased force demands in the columns below the isolation level make these members critical and the isolation becomes ineffective. In this type of applications, either top of the columns should be restrained to obtain diaphragm action or the columns need to be retrofitted.

KEYWORDS: base isolation, seismic performance, existing building, retrofit

1. INTRODUCTION

Seismic isolation is a widely accepted method to improve the seismic performance of the buildings by isolating the structure from the seismic effects of the earthquake ground motions. In general, isolation devices such as Lead Rubber Bearings (LRB) or Friction Pendulum Bearings (FPB) are employed at the foundation level of the buildings. In seismic isolation, the fundamental aim is to reduce the seismic demands by lengthening the natural period of the building, which is achieved by the additional flexibility of the isolation units. Increased fundamental period of the building is much greater than both its unisolated period and the predominant period of the ground motion. Thus, the earthquake forces exerted on the isolated building is much less than the ones for unisolated building. On the other hand, increased structural flexibility as well as the fundamental period has adverse effects on the building's horizontal displacement, which should be taken into consideration.

Other than the base level, seismic isolation may be employed at different building story levels due to some economic or environmental constraints. In such cases, seismic performance of the members above the isolation levels improve considerably, whereas those below the isolation units may experience additional seismic forces compared to its existing condition. Therefore, necessary measures should be taken for the structural components under the isolation level. In this study, a regular five-story existing residential building was employed to investigate the effect of seismic isolation level on its seismic performance. Especially, laterally unbraced columns that are under the isolation units can be subjected to additional internal forces and failure may occur if the column capacities are not sufficient. The main objective of this study is to investigate the effectiveness of the story level seismic isolation other than the base isolation and to examine the relevant measures if the columns are subjected to higher seismic forces. The sample building is analyzed using code specified response spectrum and linear time history analyses are performed using the ground motion recorded in 1992 Erzincan earthquake, where the sample building is located. Seismic demands are calculated from the analysis results of the building for the unisolated and ground floor level seismically isolated cases individually. Seismic demands of the members are compared with the section capacities of the members to investigate the efficiency of the isolation for both existing and seismically isolated cases.

2. DESCRIPTION OF THE BUILDING

A five-story existing residential building was investigated in the study. Due to the importance of the building, FPB isolation is applied to improve its seismic performance. Due to the economic and practical constraints, FPBs are placed over the ground floor columns. According to the Turkish Earthquake Code (TEC2006), this building is classified as a regular structure. As can be seen in Figure 1, building has plan dimensions of 6.80^{m}x9.35^{m} with reinforced concrete members having concrete strength of 16 MPa and reinforcement steel yield strength of 220 MPa. Column sections have a longitudinal reinforcement arrangement of 8∅18 satisfying the code minimum requirement of 1% and have a poor confinement ratio with ∅8 transverse steel at every 25cm. Only in the ground story, 15cm thick wall members are present at various column spans. Overhangs are present at the 2nd, 3rd and 4th stories. Some basic geometric properties of the building are given in Table 1.

Table 1: Geometric properties of the building

Story #	Lx (m)	Ly (m)	Floor Area (m^2)	Story Height (m)	Number of Columns
1	6.8	9.35	63.58	3.15	12
2	6.8	10.2	69.36	2.75	12
3	6.8	10.2	69.36	2.75	12
4	6.8	10.2	69.36	2.75	12
5	6.8	6.35	43.18	2.50	6

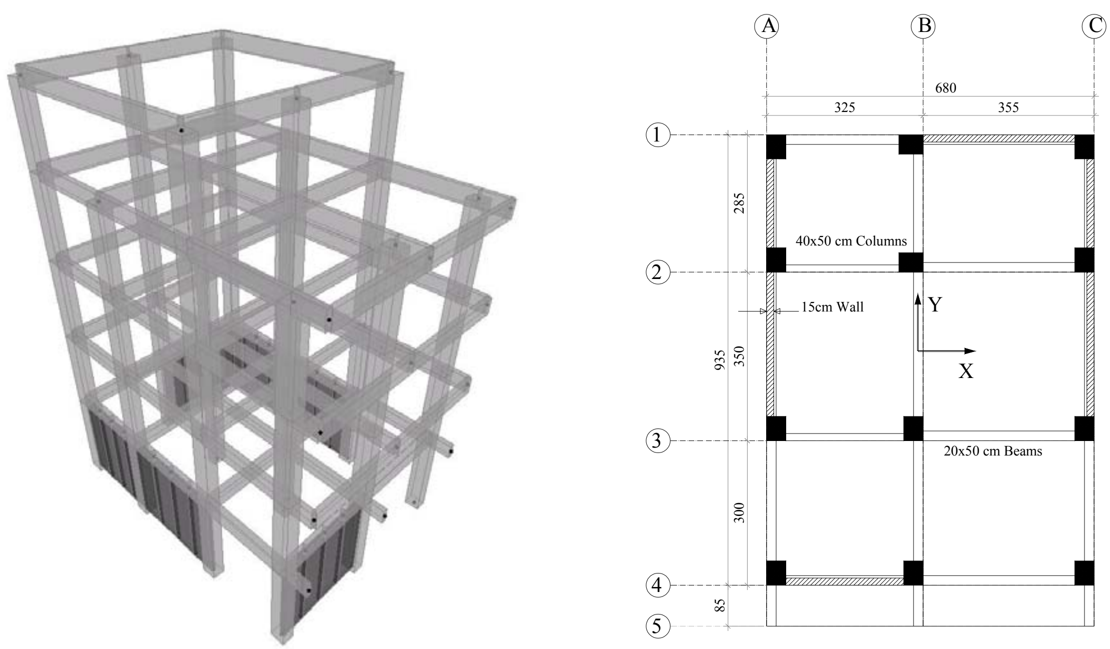

Figure 1: 3D and ground-story plan view of the building

3. SEISMICITY OF THE BUILDING SITE

The building is located in Erzincan at the eastern part of Turkey, which is a highly active seismic region. The city of Erzincan has been frequently affected by damaging earthquakes causing loses of lives and property. In the last century, three major earthquakes occurred in 1939, 1941 and 1992 with the magnitudes of 7.9, 5.9 and 6.8, respectively. When the seismic activity of the region is considered, it is believed that a damaging earthquake will likely occur in the future, affecting the city. Therefore, when the importance of the building is concerned, a design level earthquake may occur in the lifetime of the building and its seismic effects should be taken into consideration in the determination of building safety. It is assumed that a future event close to the building site (R=5km) with a magnitude Mw7.2 can be considered for the calculations. This earthquake is regarded as the "Maximum Capable Earthquake" as defined in the IBC2003. Also, shear wave velocity of the soil at the site is assumed as 500 m/s to be used in the calculations.

3.1. Estimation of Maximum Spectral Acceleration

According to the Turkish Seismic Zone Map, the city of Erzincan is in the 1st seismic zone (highest zone) with maximum ground accelerations given as 0.496g and 0.586g for the 475-year and 1000-year return period earthquakes, respectively. The seismic isolator calculations are carried out using spectral acceleration values at T = 1second (S_{M1}). In order to calculate S_{M1}, attenuation relationships developed by Boore et al. (1997) and Kalkan & Gülkan (2004) were used. For a M=7.2 seismic event and R=5km, S_{M1} is calculated as 0.54g and 0.40g with respect to Boore et al. (1997) and Kalkan & Gülkan (2004), respectively. Finally, an average acceleration of 0.5g for T=1s is decided for this scenario earthquake.

To be used in the response spectrum analysis, design response spectrum for the building is calculated using the TEC2006 code considering the Z2 building local site class and the seismic zone of the building. This response spectrum is further scaled to obtain S_{M1}=0.5g at T=1s as given in figure 2. Moreover, Erzincan 1992 earthquake record was used for the time history analysis. Only one component of the earthquake having the greater intensity was selected and scaled with a factor of 0.69 to satisfy the acceleration of S_{M1}=0.5g at T=1s. Scaled ground motion time history and its 5% damped response spectrum are shown in figure 3.

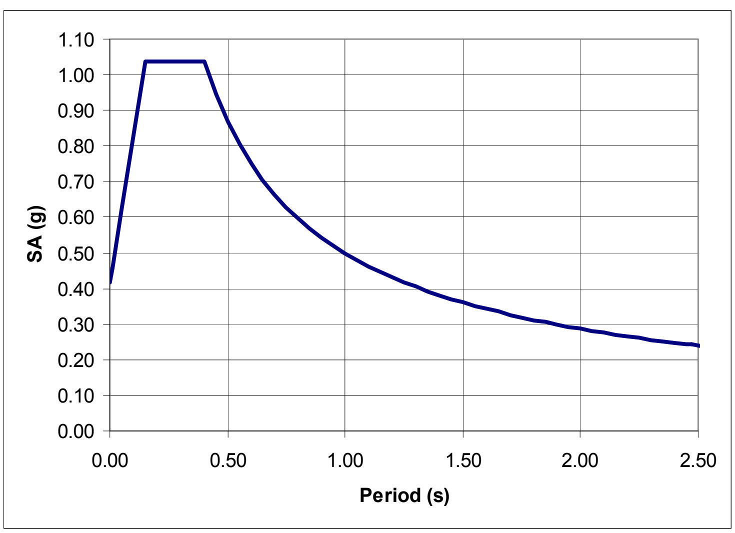

Figure 2: TEC2006 design response spectrum for the building

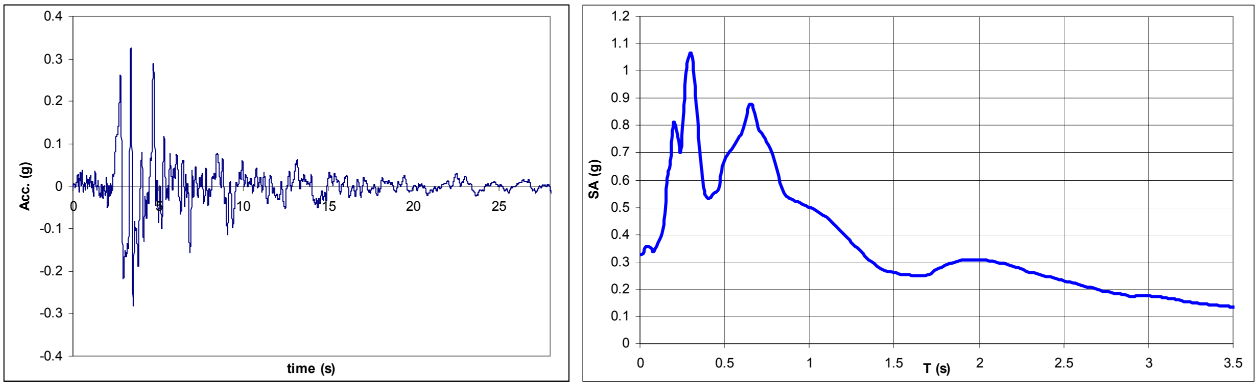

Figure 3: Erzincan 1992 EQ time history and 5% damped response spectrum with a scaling factor of 0.69

In addition to response spectrum, time history analysis using the Erzincan1992 ground motion was done for the existing and the seismic isolated building.

4. DESIGN OF FRICTION PENDULUM BEARINGS

FPBs were designed at the top of the ground story columns to reduce the seismic effects of the earthquake ground motions. The target period of the isolated building is selected as 2.5s. Maximum lateral displacement of the isolation system, D_M was calculated considering a 10% effective damping. According to IBC2003, Eqn. 4.1. is used and B_M is taken as 1.2 for 10% effective damping.

$$D_M = \frac{\left(\frac{g}{4\pi^2}\right) S_{M1} T_M}{B_M} = \frac{9.8}{4\pi^2} \frac{0.5x2.5}{1.2} = 0.26\,m \tag{4.1}$$

Total lateral displacement was calculated considering the maximum displacement and the eccentricity between the mass center of the building and center of rigidity of the isolation system. 5% eccentricity is assumed for the building to calculate the total displacement according to the Eqn. 4.2 using the ground story plan dimensions.

$$D_T = D_M \left[1 + y_i \left(\frac{12e}{b^2 + d^2}\right)\right] = 0.26\left[1 + \frac{9.35}{2}\left(\frac{12x(0.05 * 9.35)}{6.8^2 + 9.35^2}\right)\right] = 0.31m \tag{4.2}$$

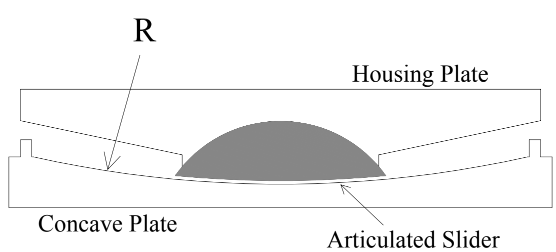

Figure 4: Details of a friction pendulum bearing

The radius of the spherical surface of the FPB as shown in figure 4, is calculated using the target period of the isolation system (Naeim and Kelly 1999). As the target period T=2.5s and g= 9.8 m/s^2 are inserted in Eqn. 4.3., R is calculated as 1.55m. It is decided to use R=1.6m for FPBs as the isolating devices.

$$T = 2\pi \sqrt{\frac{R}{g}} \tag{4.3}$$

Effective stiffness of each isolation unit is determined using Eqn. 4.4 based on the total axial load on the FPB, radius of the FPB, total lateral displacement and the friction coefficient between the articulated slider and concave surface of FPB, which is assumed as μ=0.05.

$$K_{eff} = \frac{W}{R} + \frac{\mu W}{D_M} \tag{4.4}$$

Since the total axial load varies for each isolation unit, different K_{eff} value is calculated and assigned in the analysis to model the effective stiffness of the FPB. Axial loads on each FPB are calculated from the gravity analysis and they are tabulated together with the corresponding K_{eff} values in table 2. Axes of the FPB's are shown in figure 1.

Table 2: K_{eff} for each FPB

Isolation Unit	Axes	W (kN)	Keff (kN/m)
FPB-1	A4	205.6	161.7
FPB-2	B4	341.8	268.8
FPB-3	C4	218.1	171.5
FPB-4	A3	288.5	226.8
FPB-5	B3	399.2	313.9
FPB-6	C3	302.8	238.1
FPB-7	A2	247.6	194.7
FPB-8	B2	379.0	298.0
FPB-9	C2	260.1	204.6
FPB-10	A1	205.6	161.7
FPB-11	B1	281.0	220.9
FPB-12	C1	216.0	169.9

Effective damping of the isolation system is calculated by Eqn. 4.5. as ξ_{eff}=0.13. This proves that the calculations for the spectral accelerations as well as the displacements were on the safe side. Because, initially assumed effective damping of 0.1 is less than the calculated ξ_{eff}=0.13, which results in smaller design displacements.

$$\xi_{eff} = \frac{2}{\pi} \frac{\mu R}{D_M + \mu R} \tag{4.5}$$

5. SEISMIC ANALYSIS OF THE BUILDING

Three dimensional elastic analytical model of the building for unisolated and isolated cases were generated using SAP2000 structural analysis program. FPB isolation units are modeled as elastic springs between the ground floor columns and the slab above using the effective stiffness values. Modal, gravity, response spectrum and linear time history analyses were carried out to obtain the dynamic characteristics of the building and to calculate the most unfavorable seismic demands. As can be seen in table 3, the existing system and the isolated system have the fundamental periods of 0.45s and 2.30s, respectively. First mode of the isolated system is a torsional mode with a period of 2.46s. Due to the higher mode effects, fundamental period of the isolated system is not exactly equal to the target isolation period of 2.5s, but very close to it. Because of the effective stiffness variation of the FPBs, which is due to the difference in the axial load levels, the 1st mode displays a torsional behavior. Mass participation ratios for the isolated system are very high and almost reach to 100% after the 3rd mode, which is not the case for the existing unisolated building.

Table 3: Modal properties of the existing and isolated system for 10 modes

Existing

Mode	Period	Mass Participation		Cumulative Mass Participation	
#	(s)	UX (%)	UY (%)	UX (%)	UY (%)
1	0.45	52.4	0.0	52.4	0.0
2	0.37	11.8	0.0	64.2	0.0
3	0.35	0.0	62.9	64.2	62.9
4	0.13	8.2	0.0	72.4	62.9
5	0.13	0.0	8.2	72.4	71.1
6	0.12	1.6	0.0	74.1	71.1
7	0.08	2.3	0.0	76.3	71.1
8	0.07	0.0	5.2	76.4	76.3
9	0.07	1.8	0.0	78.1	76.3
10	0.05	2.7	0.0	80.8	76.3

With Isolation

Mode	Period	Mass Participation		Cumulative Mass Participation	
#	(s)	UX (%)	UY (%)	UX (%)	UY (%)
1	2.46	21.0	0.3	21.0	0.3
2	2.30	4.9	91.6	25.8	92.0
3	2.28	71.0	4.8	96.8	96.8
4	0.28	0.0	0.0	96.8	96.8
5	0.22	0.0	0.0	96.8	96.8
6	0.22	0.0	0.0	96.8	96.8
7	0.12	0.0	0.0	96.8	96.8
8	0.11	0.0	0.0	96.8	96.8
9	0.10	0.0	0.0	96.8	96.8
10	0.08	1.0	0.0	97.8	96.8

Displacement demands of the Erzincan (1992) earthquake for isolated and existing building differs considerably as shown in figure 5. Although the design displacement for the isolation units are calculated as 0.31m in Eqn. 4.1., due to the superstructure flexibility as well as the higher mode effects, maximum top story displacement of the isolated system is calculated as 0.344 m and 0.342 m for the X and Y directions of the building, respectively.

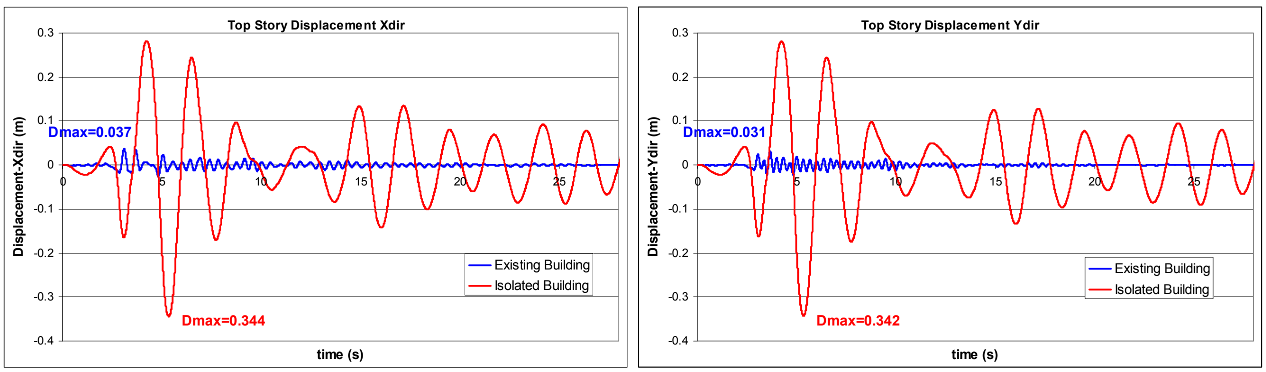

Figure 5: Elastic displacement of the existing and the isolated buildings in both directions under Erzincan1992 EQ

Response spectrum and time history analyses were performed after the application of gravity analysis. Member forces due to gravity analysis were calculated using the combination of 1.0·DL+0.3·LL as per TEC2006. Using response spectrum and time history analyses results, four different load combinations were formed in order to capture the most unfavorable seismic demands. Interaction analyses of the reinforced concrete beam and column sections were carried out using the material model suggested by Mander et al. (1988). For each column and beam member, PM2 (weak axis) and PM3 (strong axis) interaction curves were calculated. When the seismic demands on the sections and capacities were compared, it was found out that the beam members are mostly on the safe side. Therefore, comparisons for the existing and isolated building models were done with respect to the column members.

In the first model, the building is modeled with its existing properties. As illusrated in figure 6, almost all the column PM2 and PM3 demands are outside the interaction curve for the existing building. This proves the necessity for the seismic mitigation of the existing building to protect it from the earthquake induced seismic hazard. In the second model, in order to improve the seismic performance of the building, FPB isolation units are employed at the top of the ground floor columns. In this case, superstructure column demands were all inside the interaction diagram as shown in figure 7 for both principal axes of the column section, which was already expected for an isolation system. However, some of the ground floor columns below the isolation level which are not adjacent to the wall members experienced excessive seismic demands even greater than the seismic demands when the building was not seismically isolated. These excessive moment demands are due to the excessive displacement (0.34m) of the FPBs at the top of the column. The stiffness of the FPB times the relative displacement of the FPB results in shear force demand on the columns that behave as cantilevers due to absence of diaphragm action. This shear force times the column height gave the resulting moment demand on the column section at the base. The columns that are at the ends of the wall members did not experience such big moment demands since the resulting shear force due to the FPB displacement is resisted by the wall members and column demands reduced considerably.

Excessive seismic demands on the laterally unsupported columns may cause the failure of the building and necessary precautions need to be taken. First alternative may be to increase the capacity of the unsupported columns by jacketing. For this alternative, the column sections should be increased such that the column capacity will be greater than the seismic demand. In the second alternative, top of the ground floor columns should be connected to each other in order to provide a diaphragm effect under the isolation units. This can be achieved by either connecting the column ends with horizontal brace members or by inserting partial concrete slabs in some of the bays. The diaphragm action makes all the ground floor columns displace all together and most of the shear force resulting from the displacement of the isolation units is carried by the stiffer wall members. Since wall members have sufficient capacity to resist such amount of shear force, the building can be able to resist all seismic actions safely. In the analytical model a full diaphragm action was considered for the top joints of the ground floor columns. As can be seen in figure 8, all the superstructure columns in the isolated building are within the interaction no matter whether the diaphragm action is implemented in the ground floor. However, the fundamental difference occurs in the ground floor column demands. Ground floor column demands for isolation system with diaphragm elements are much less than the ones for isolation system without diaphragm elements when the figures 7 and 8 are compared.

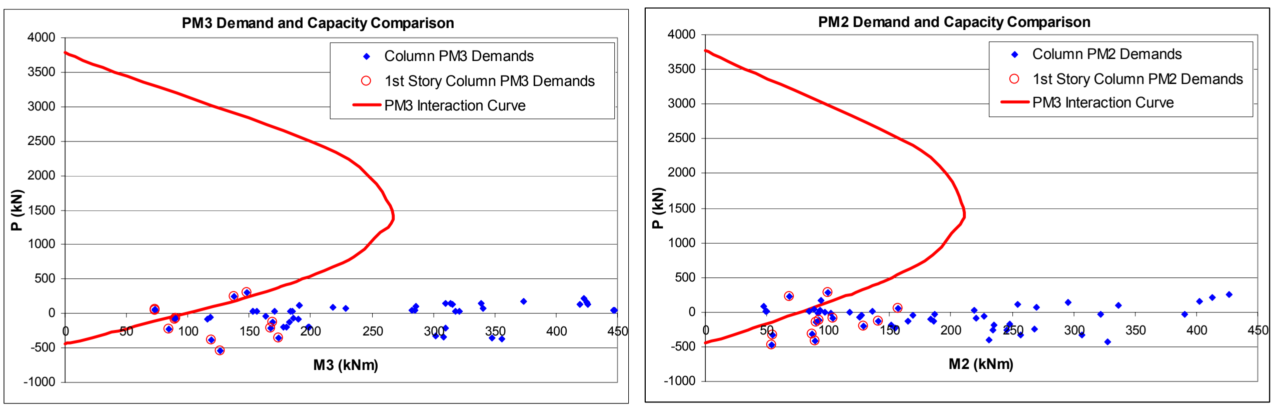

Figure6. Column PM2 and PM3 seismic demands and capacities of Existing Building

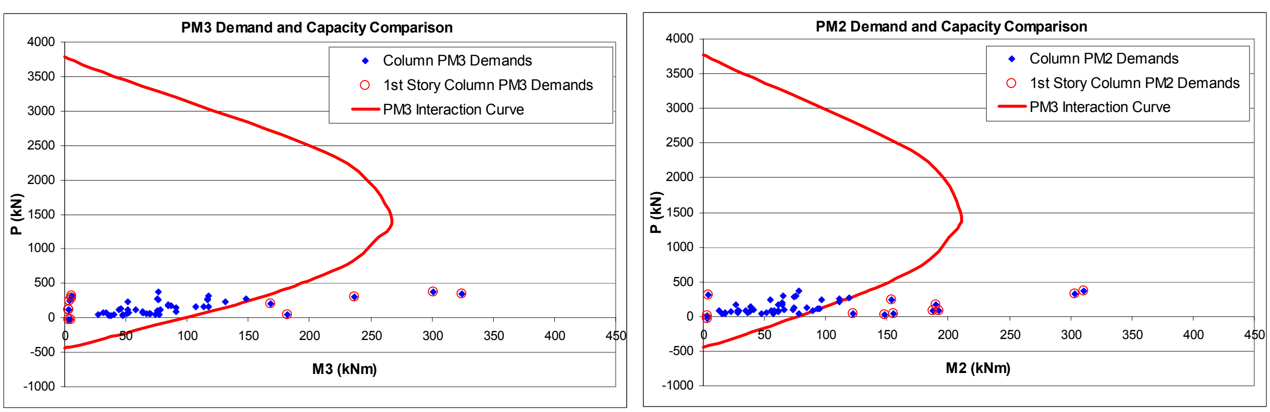

Figure7: Column PM2 and PM3 seismic demands and capacities of Isolated Building

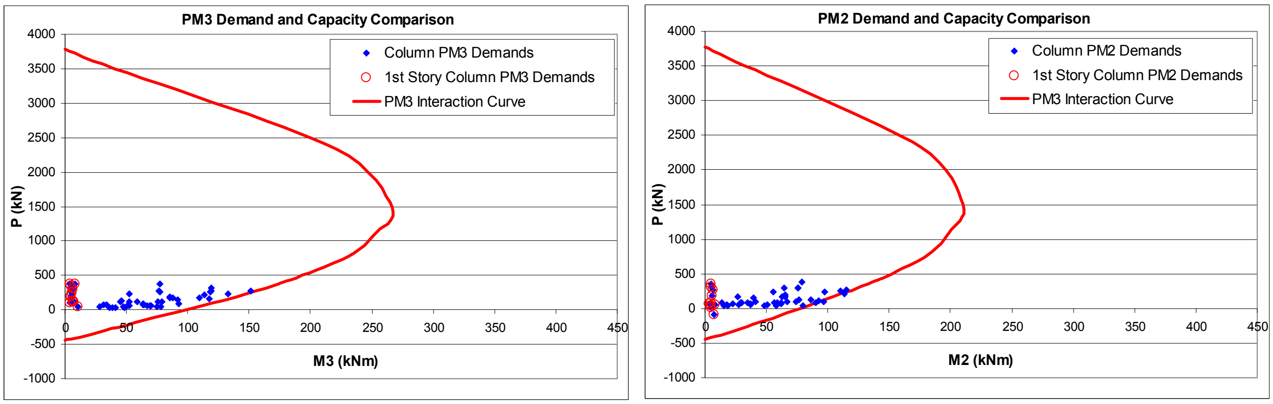

Figure8: Column PM2 and PM3 seismic demands and capacities of Isolated Building with Diaphragm Members

6. CONCLUSIONS

Performance of a typical five-story existing residential building was investigated under the effect of seismic actions. The existing capacity of the building was found to be inadequate and as a seismic mitigation strategy seismic isolation using FPBs was considered and designed to reduce the seismic demands on the building. However, due to the economic and environmental constraints, friction pendulum isolation units were employed at the top of the ground floor columns instead of the base level. Although superstructure column demands decreased considerably, laterally unsupported ground floor column demands experienced additional seismic demands compared to unisolated existing case. It has been found that to overcome such an undesirable situation, the capacity of these columns could be improved by column jacketing or ground floor columns could be constraint to displace all together instead of experiencing individual displacement causing excessive seismic demands for the unsupported columns. In order to provide additional constraints at the top of ground floor columns, axial brace elements or concrete slab can be considered as alternative solutions. In this study, it is emphasized that seismic isolation does not always guarantee safe lower member seismic demands, especially if the isolation units are employed at the story levels other than base level. All members under and over the isolation units should be checked. If they displace independently from the story members, occurrence of additional seismic demands will be unavoidable unless necessary precautions are taken.

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