Study on the effect of rigid diaphragm beneath the seismic-isolation system on structural response

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SUMMARY
It appears to be standard practice, aided also by design code provisions, the use of rigid diaphragms just above and just below the isolation interface of seismically isolated buildings. Such practices result in increased costs of the seismic isolation designs putting them in competitive disadvantage over conventional design solutions. The present work aims at identifying and quantifying, if any, the effects and the role of a diaphragm just below the isolation interface of a 2 floor isolated building supported on top of ten cantilevered columns. It is shown through a nonlinear time-history parametric analyses that the use of a tie-beam grid on the column tops does not necessarily lead to improved behavior as compared to the behavior of the structural system without the tie-beam diaphragm.

Keywords: Seismic isolation, friction pendulum bearings, isolation diaphragm

1. INTRODUCTION

Modern seismic codes require from the designers of seismic-isolation systems to provide structural elements above and below the isolation interface sufficiently rigid in both horizontal and vertical directions. More specifically, Eurocode-8 states “A rigid diaphragm is provided above and under the isolation system, consisting of a reinforced concrete slab or a grid of tie-beams, designed taking into account all relevant local and global modes of buckling. This rigid diaphragm is not necessary if the structures consist of rigid boxed structures”. The reasons behind this particular requirement are neither stated nor explained in the commentaries of the code documents.

Such provisions as the one in Eurocode-8 (CEN [2004] ENV 1998), especially when they are not sufficiently justified, can adversely affect the proliferation of seismic-isolation technologies. This is because the use of seismic isolation technologies is penalized over conventional design solutions because of the additional costs of rigid diaphragms and/or grids of tie-beams.

The objective of the present work is to evaluate the role of a diaphragm or a grid of tie-beams below the seismic isolation interface. A three-storey seismically isolated structure is considered in the present work. The structure is supported by ten isolation bearings (friction-pendulum bearings) supported on top of ten columns, thus creating a “piloti” or soft story below the isolation interface. Two designs of the columns are considered in the study: (a) Case A, in which the column tops are connected with tie-beams, creating a diaphragm below the isolation interface, and (b) Case B, in which the column tops are free, thus not introducing a diaphragm below the isolation interface.

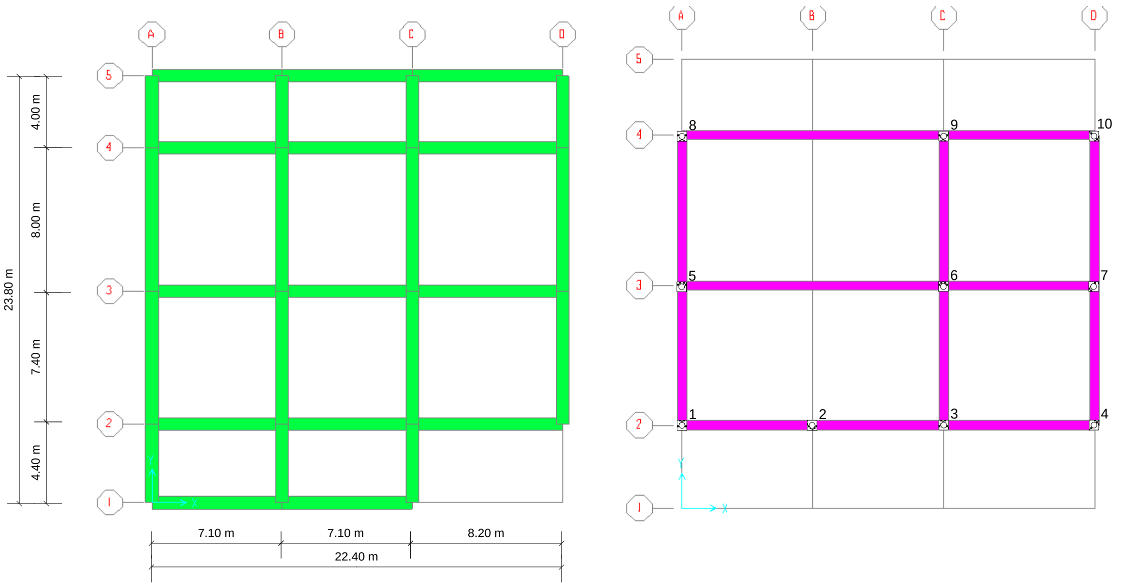
Above the isolation interface the 2nd story columns are connected with tie beams at their bottom, thus ensuring that a diaphragm is present above the isolation interface. The structural model and the model of the seismic-isolation hardware were constructed in SAP2000 (Computers and Structures Inc. 2007). A suite of seismic motions is used to excite the structural models.

Comparisons between the two 1st-story column system designs are presented in terms of the isolation-system responses and the structural element responses in order to quantify the role of a diaphragm below the isolation interface.

2. SEISMICALLY ISOLATED STRUCTURE

2.1. Structure

The considered structure is a three-story reinforced concrete frame with a plan view shown in Fig. 2.1.(a). The upper two floors are carried by 10 columns of height 5.35 m (70cm X 70cm cross section) which are fixed at their base. The isolators are placed on top of each 1st floor column right under the floor of the 2nd level (see Fig. 2.2.) following the grid shown in Fig. 2.1.(b). Grid of beams together with the floor slabs of the 2nd level provide a diaphragm just above the isolation interface thus the structure of the two upper floors is a box-frame (see Fig. 2.3.).



(a) (b)

Figure 2.1. Plan view of (a) the building floors and (b) the Isolation Interface with the Grid of the tie-Beams of the Basement Columns.

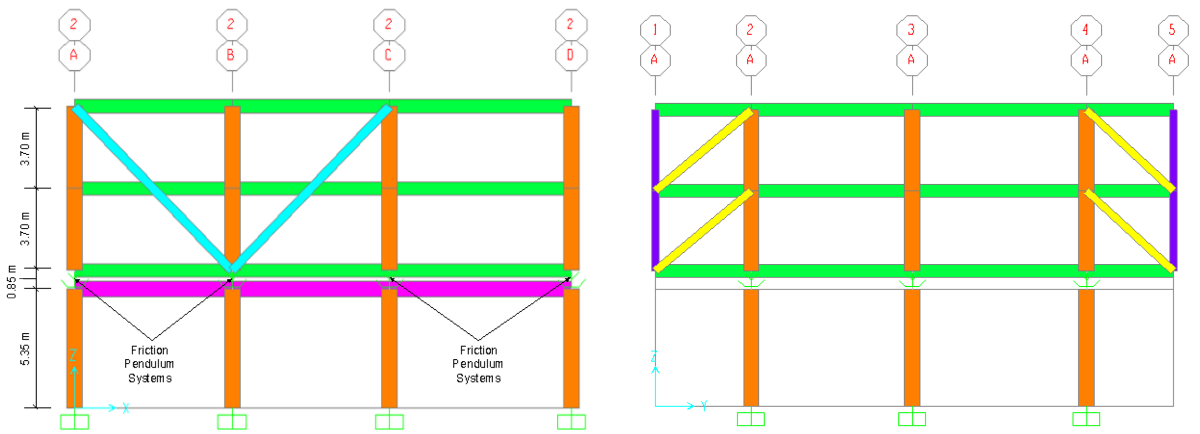


Figure 2.2. Side Views of (a) the Grid Line #2 of Case A with the Tie-Beams (Pink) on the Basement Column Tops, and (b) the Grid Line #A of Case B without Tie-Beams.

Fig. 2.2 presents the side views of the grid lines 2 and A of the plan. In the Fig. 2.2(a) the 1st floor columns of the grid line 2 are connected to their tops with tie-beams (purple colour) thus creating a diaphragm just below the isolation level, representing Case A of the analyses. Fig. 2.2(b) shows Case B of the analyses where the column tops are not connected with tie-beams, thus behaving independently as cantilevers. Case B does not conform with the Eurocode-8 requirement of providing a diaphragm both above and below the isolation level. The structure was modelled in SAP2000 considering Rayleigh damping with constants a=0.1839 and b=.863E-03.

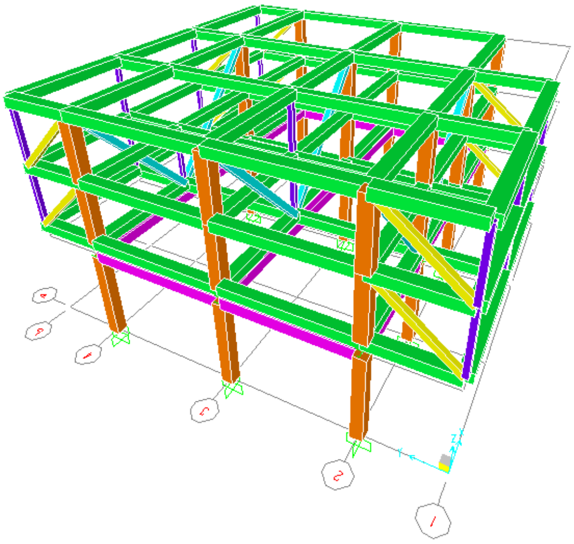


Figure 2.3. A 3D View of the Structural Model with tie-Beams (purple colour elements) in SAP2000.

2.2 Seismic Isolation System

The Isolation system consists of 10 Friction Pendulum Isolators (FPS) placed on a grid as shown in Fig. 2.1(b). The axial load (Dead and Live Loads) each isolator carries is shown in Table 2.1. The total weight carried by the isolation system is 16,394.2 kN. The isolation-system design called for isolation period of 2.5 secs with a radius of curvature of the isolators to be 1.55 m. The maximum and minimum coefficients of friction were taken to be 0.057 and 0.04 respectively.

Table 2.1. Axial Load Carried by Isolators.

Isolator #	Axial Load (kN)
1	1091.3
2	1857.4
3	1603.9
4	562.59
5	1637.6
6	2321.4
7	1030.7
8	2123.7
9	2943.5
10	1222.3
Wtot	16394.2

3. EARTHQUAKE MOTIONS

One set of ground motion time histories is considered in this study. The set is identical to the set used in Whittaker et al. (1998), and consists of 10 pairs of scaled acceleration time histories from 6 actual earthquakes with magnitude larger than 6.5, and epicentral distance between 10-20 km. Table 3.1 presents details and scale factors for the set of ground motions. The ground motions where selected from sites classified as soft rock to stiff soil. The scaling procedure, which is presented in Tsopelas et al. (1997), was such that while preserving their frequency content their average spectrum matched a target design spectrum (AASHTO soil Type II, A=0.4). Fig. 2.3 presents the spectra of these motions as well as their average spectrum.

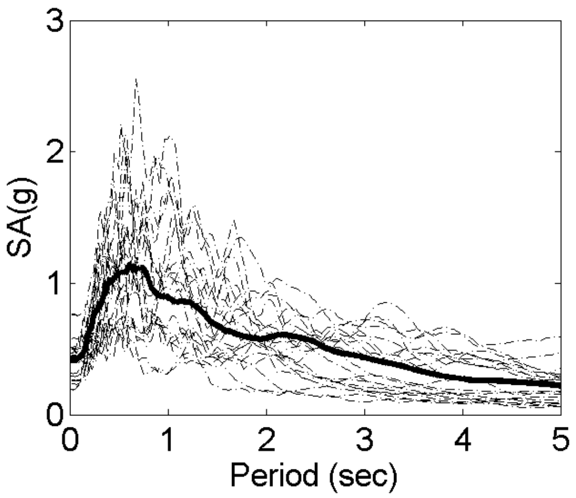


Figure 3.1. Response spectra for the set of Seismic Motions (bold lines indicate the average).

Table 3.1. List of Ground Motions Considered.

Record ID	Seismic Event	Station	Component	Scale Factor
1, 2	1992 Landers	Joshua (CDMG)	90, 0	1.48
3,4		Yermo (CDMG)	270, 360	1.28
5,6	1989 Loma Prieta	Gilroy 2 (CDMG)	0, 90	1.46
7,8		Hollister (CDMG)	0, 90	1.07
9,10	1994 Northridge	Century (CDMG)	90, 360	2.27
11,12		Moorpark (DCMG)	180, 90	2.61
13,14	1949 W. Washington	325 (USGS)	N86E, N04W	2.74
15,16	1954 Eureka	022 (USGS)	N79E, N11W	1.74
17,18	1971 San Fernando	241 (USGS)	N00W, S90W	1.96
19,20		458 (USGS)	S00W, S90W	2.22

4. ANALYSIS RESULTS AND DISCUSSION

Nonlinear time-history analyses were performed with SAP2000 with the structural model excited independently along the X and along the Y directions with every component of each seismic recording. The maximum responses in every analysis appeared along the direction of the excitation due to the fact that the superstructure is rather symmetric and experience limited torsional response. The responses were averaged over the 20 motions utilized in this study for each analysis. Therefore, results are obtained for excitation in every direction, which are averaged over the 20 motions. Table 4.1, 4.2 and 4.3 present the average responses of the isolation-system displacements at specific isolator locations, the drift and the shear forces at specific column locations above the isolation interface and the moments at the top and bottom of specific columns below the isolation interface for Case A and Case B analyses and for earthquake motion along the X and Y directions.

The displacements of the isolation system are larger for Case A where the diaphragm is provided below the isolation interface with the tie-beams at the top of the columns. It is worth noting, that the isolation displacements are larger in Case A, not only in terms of average values, but for every single seismic motion. This behavior is rather expected since the structural system below the isolation interface in Case B is more flexible than in Case A, resulting in some of the displacements to be partially distributed to the cantilever columns. In other words, the isolators together with the cantilevered beams form an effective "isolation system" rather than the isolators themselves to be the isolation system.

The 1st-floor column drifts in Case A are consistently smaller than these in Case B, as explained above, with the maximum values for the columns presented in Table 4.2 to be 4 ‰ for Case A and 8‰ for Case B. The picture of the shear forces is the opposite of the drifts; the shear forces in Case B are much smaller than these in Case A.

Table 4.3 presents the moments at the top and bottom of the 1st-floor columns. As expected, the moments at the bottom of Case B cantilever columns are approximately 40% higher than the ones in Case A. The presence of tie-beams at the top of the cantilevers results in a more uniform distribution of the moment along the height of the columns, instead of concentrating it at the bottom cross section which is the case in Case B.

Table 4.1. Average Isolation System Displacements.

		Isolation Displacement (m)	
		Case A	**Case B**
Isolator #1	**EQ-X**	0.147	0.153
	EQ-Y	0.154	0.164
Isolator #6	**EQ-X**	0.149	0.135
	EQ-Y	0.145	0.131
Isolator #10	**EQ-X**	0.149	0.160
	EQ-Y	0.148	0.149

Table 4.2. Average Drift and Shear Force of the Basement Columns.

		Relative Displacement (m)		**Shear Force (kN)**	
		Case A	**Case B**	**Case A**	**Case B**
Column 2-A (Isol. #1)	**EQ-X**	0.014	0.028	267.7	228.1
	EQ-Y	0.017	0.029	327.7	238.7
Column 3-C (Isol. #6)	**EQ-X**	0.019	0.045	477.6	360.8
	EQ-Y	0.021	0.044	567.2	353.6
Column 4-D (Isol. #10)	**EQ-X**	0.021	0.030	422.2	242.7
	EQ-Y	0.013	0.030	244.0	242.7

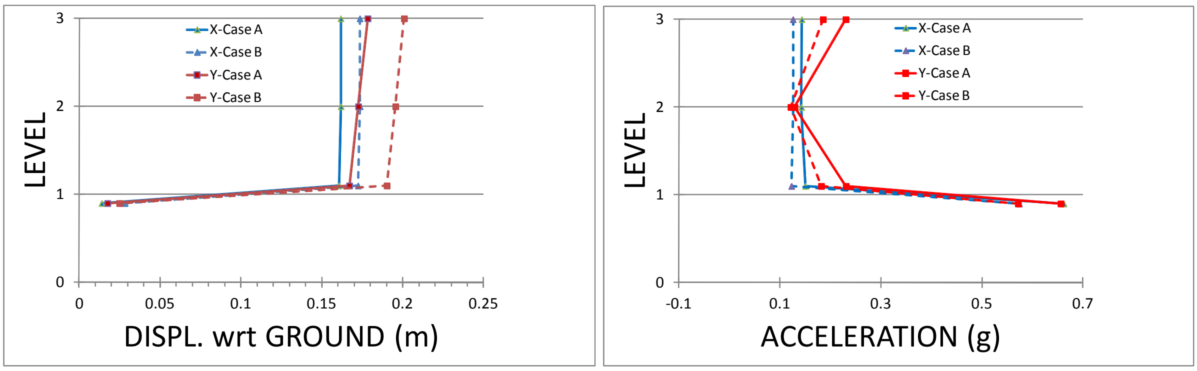
Table 4.3. Average Moments at the Two Ends of Basement Columns.

		EQ-X		**EQ-Y**	
		Case A	**Case B**	**Case A**	**Case B**
Column 2-A (Isol. #1)	**TOP**	452.7	192.2	560.3	198.8
	BOT	917.8	1410.0	1197.3	1473.1
Column 3-C (Isol. #6)	**TOP**	975.4	309.7	1303.6	302.8
	BOT	1442.1	2238.3	1731.4	2181.4
Column 4-D (Isol. #10)	**TOP**	731.1	206.0	407.1	206.3
	BOT	1417.1	1502.1	902.7	1501.5

In the following figures, level 0, 1, 2, and 3 represent the ground level, the isolation interface, the floor of the 3rd story, and the roof or 3rd floor, respectively. The 1st-story column tops are represented by the points just below 1, and the floor of the 2nd story is represented by the points just above level 1.

Fig. 4.1 presents the displacement with respect to ground of every level of the structure along the height of column above isolator #1. The displacements of the top of the basement columns for the Case B for excitation in both directions are larger than the displacements of Case A. This is rather expected since the structural system with the tie-beams, Case A, is stiffer than the system of Case B which consists of unconnected cantilever beams. This flexibility of the basement columns results in larger absolute displacements of the upper floor of Case B when compared with Case A. However, as shown in Fig. 4.2, the column shears are smaller for Case B.

Fig. 4.1(b) presents the acceleration at the levels of the structure. It is observed that there is a de-amplification of the accelerations above the isolation interface and as appears in the distribution of the shear forces the accelerations are higher for Case A analyses.



(a) (b)

Figure 4.1. Average (a) Floor/Level Displacements, and (b) Level Accelerations.

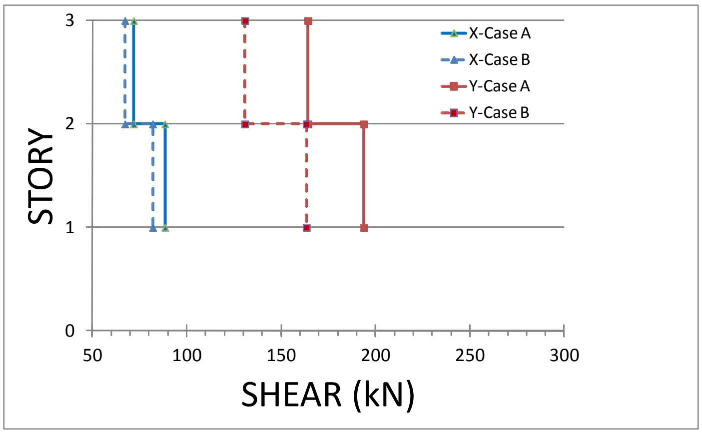


Figure 4.2. Average (over 20 motions) shear for Column 2-A directly above Isolator #1.

The total base shear at the 1st-story level below the isolation interface in both X and Y directions are approximately 20% and 16% for Case A and Case B respectively of the total normal weight carried by the isolation system.

5. CONCLUSIONS

The effects of the diaphragm below the isolation interface in the 3-story structure considered in these analyses are:

a) a small increase of the isolation system displacements compared with the displacements of Case B,
b) a reduction of the drifts experienced by the columns of the 1st story (which are connected with tie beams),
c) an increase of the shear forces in the 1st-story columns (base shear of the substructure) by approximately 25%, and
d) an increase of the floor accelerations and shear forces in the columns of the 2nd and 3rd stories compared to the responses of Case B analyses.

Based on the above results, it could be argued that the use of a diaphragm below the isolation system in the structure considered in this study does not appear to result in lower overall responses compared to the responses of the structure with the 1st-story columns behaving as cantilevers.

In any case, the presented results do not justify the strict requirements stated in Eurocode-8 for the use of diaphragms above and below the isolation interface of seismically isolated structures. The results of this limited study support the argument that the code should give the freedom to the designer to decide on the use of diaphragms given of course the structural components affected by the presence or absence of diaphragm are properly designed.

Further work is required considering different structural systems, different isolation-system configurations and diaphragm locations to generalize or even disprove the presented conclusions of this limited study.

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