Justification for adjustment of the lower bound F_p force for nonstructural components in base isolated buildings

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Requirement in ASCE 7-22

13.3.1.5 Nonlinear Response History Analysis In lieu of the forces determined in accordance with Equation (13.3-1), the nonlinear response history analysis procedures of Chapters 16, 17, and 18 may be used to determine the seismic design force for nonstructural components. Where the dynamic properties of the nonstructural component are not explicitly modeled in the nonlinear response history analysis, the seismic design force, Fp, shall be calculated as:

$$F_p = I_p W_p a_i \left[\frac{C_{AR}}{R_{po}}\right] \qquad (13.3\text{-}7)$$

where a_i is the maximum acceleration at level i obtained from the nonlinear response history analysis at the Design Earthquake ground motion. When a_i is determined using nonlinear response history analysis, a suite of not less than seven ground motions shall be used. If the supporting structure is designed using nonlinear response history analysis, the entire suite of ground motions used to design the structure shall be used to determine a_i. The value of the parameter a_i shall be taken as the mean of the maximum values of acceleration at the center of mass of the support level, obtained from each analysis. The upper and lower limits of F_p determined by Equations (13.3-2) and (13.3-3) shall apply.

F_p is not required to be taken as greater than

$$F_p = 1.6 S_{DS} I_p W_p \qquad (13.3\text{-}2)$$

and shall not be taken as less than

$$F_p = 0.3 S_{DS} I_p W_p \qquad (13.3\text{-}3)$$

The Issue:

The lower limit for seismic design force F_p used in the design of nonstructural components is determined from Equation (13.3-3). The coefficient 0.3 multiplying the S_{DS} is based on ground motion demands for non-base isolated buildings. It is well established that for base isolated buildings, the floor accelerations above the base isolation plane are reduced because of the presence of the base isolators. This reduction is not recognized when applied to the seismic design force F_p in ASCE 7, where the lower bound F_p is still maintained regardless if the building is base isolated or not.

Comparison of Recorded Floor Accelerations from Non-base Isolated and a Base Isolated Buildings from the Northridge Earthquake

To confirm the code-based forces used in the design of building diaphragms and nonstructural components attached to the diaphragms, a study was performed to document the floor accelerations from actual buildings where recorded motions were available and PGA > 0.28 g. Buildings selected for this study were chosen from the CESMD website where processed ground motions from the Northridge Earthquake were available. Nine buildings that met this criterion were chosen for this study. Of these buildings, one of the buildings is base isolated, the 7-story LAC USC Hospital building.

The instrumentation layout, peak floor accelerations at each of the instrumented floor and the response spectrum at the base, for three of the buildings are shown here in Figures 1 through Figure 9. They are the 13-Story Commercial building in Sherman Oaks, the 7-Story Hotel in Van Nuys, and the 7 Story University hospital in Los Angeles which is base isolated.

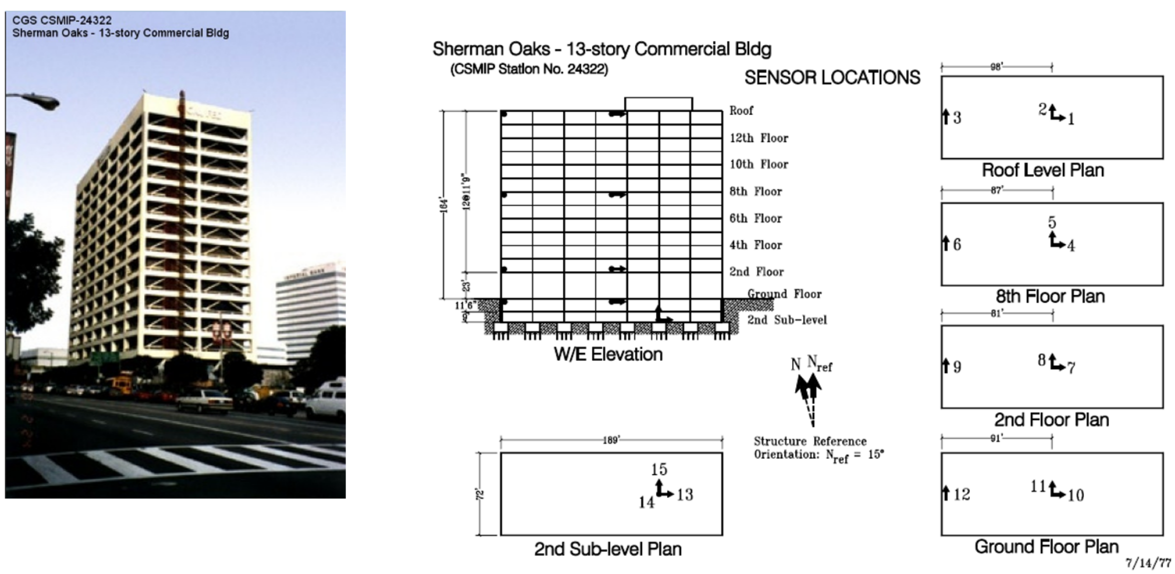

Figure 1: Sherman Oaks – 13-Story Commercial Building, instrumentation layout.

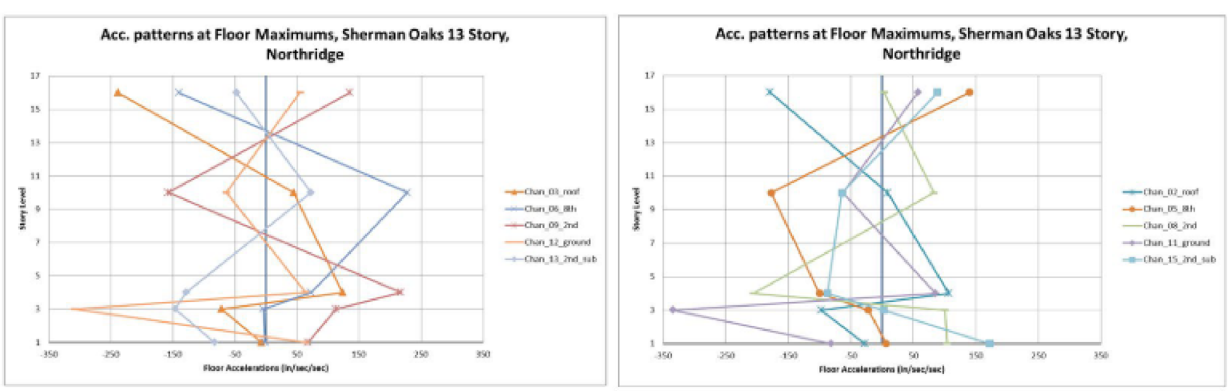

Figure 2: Acceleration values at instrumented floors at the maximum floor acceleration at the individual floor, Sherman Oaks – 13-Story Commercial Building.

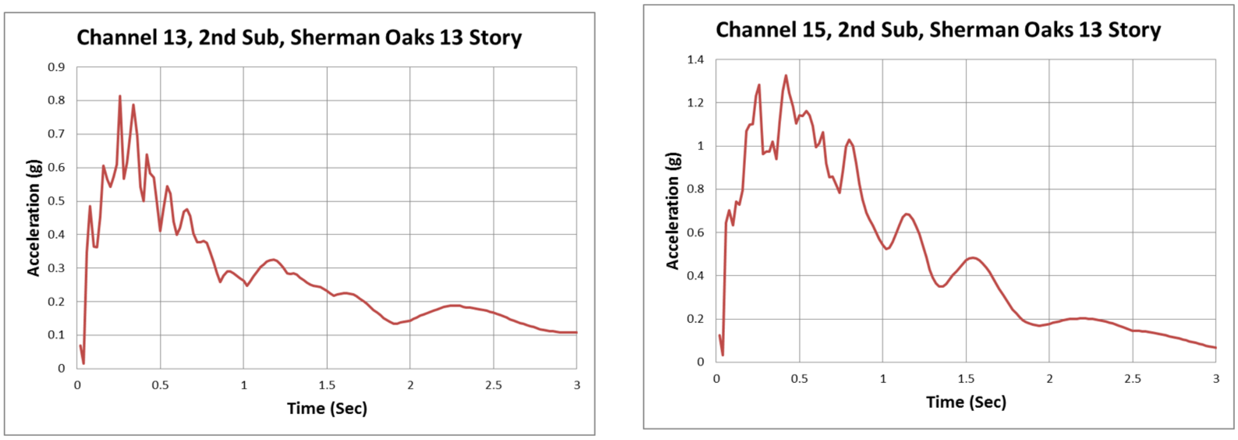

Figure 3: Response spectrum at structure soil interface, Sherman Oaks – 13-Story Commercial Building.

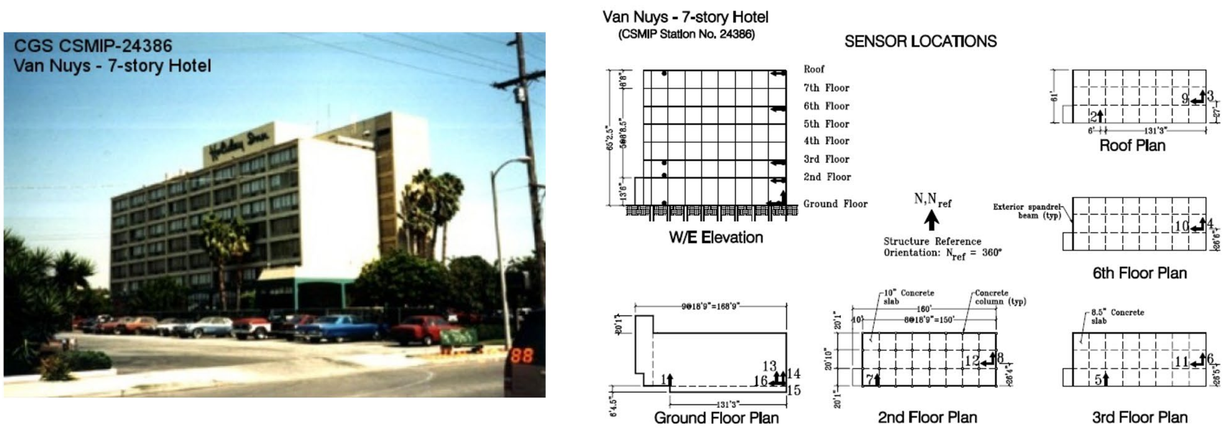

Figure 4: Van Nuys – 7-Story Hotel, instrumentation layout.

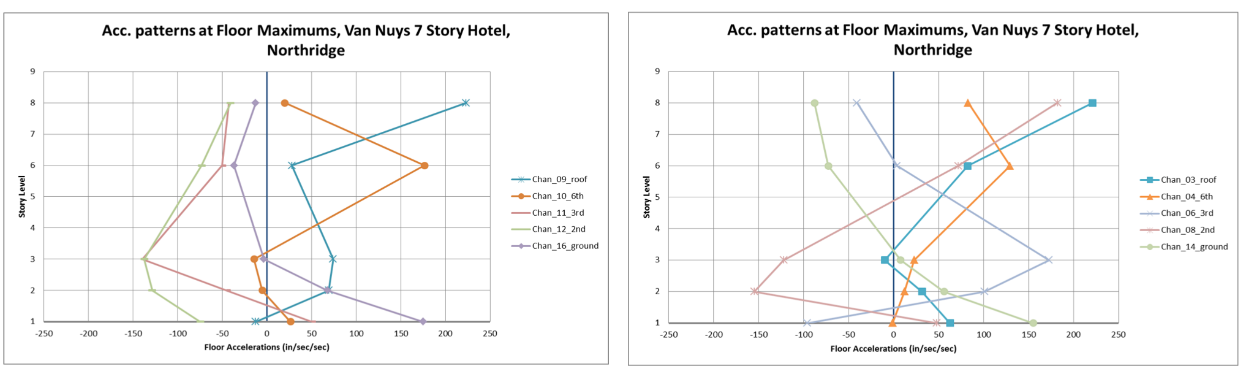

Figure 5: Acceleration values at instrumented floors at the maximum floor acceleration at the individual floor, Van Nuys – 7-Story Hotel.

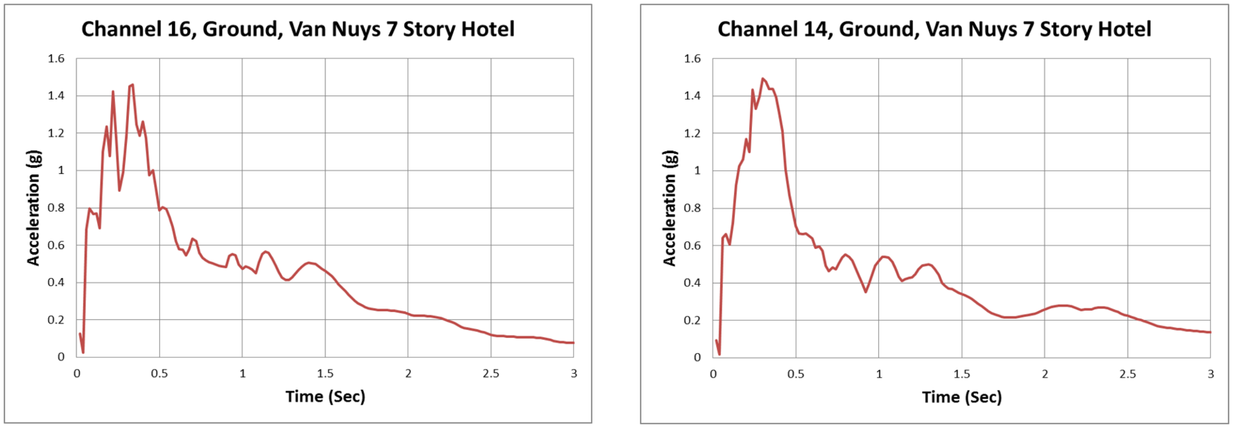

Figure 6: Response spectrum at structure soil interface, Van Nuys – 7-Story Hotel.

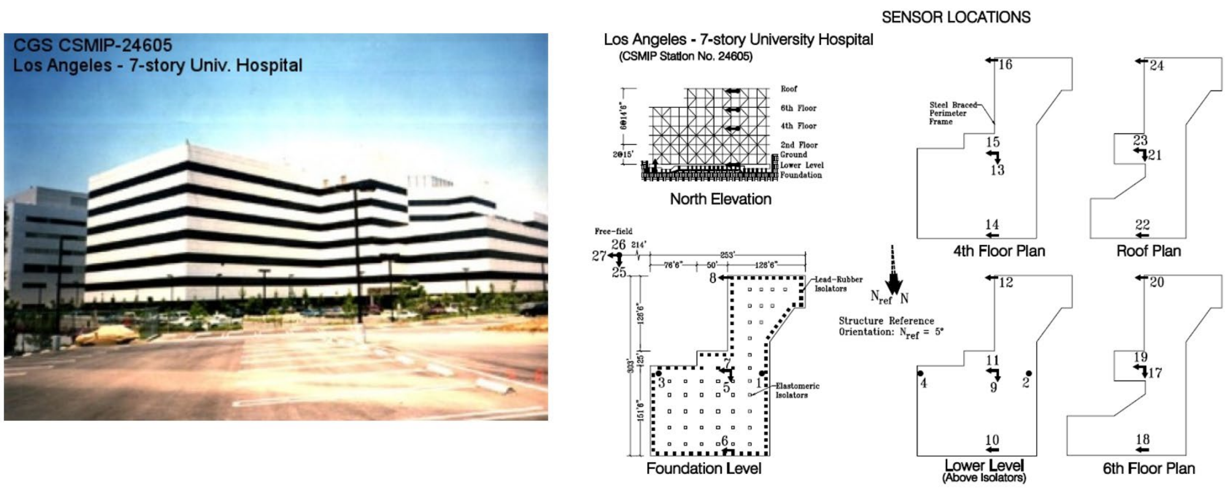

Figure 7: Los Angeles – 7-Story University Hospital, instrumentation layout. Base Isolated building.

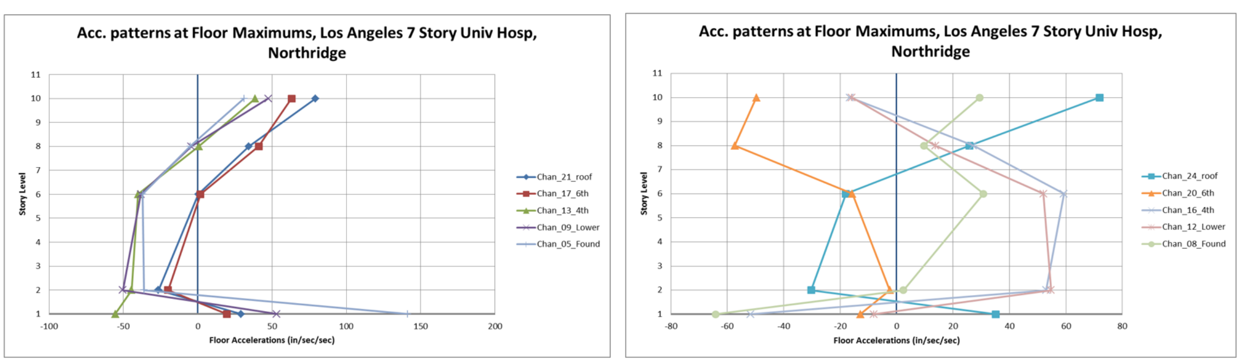

Figure 8: Acceleration values at instrumented floors at the maximum floor acceleration at the individual floor, Los Angeles – 7-Story University Hospital (Base Isolated).

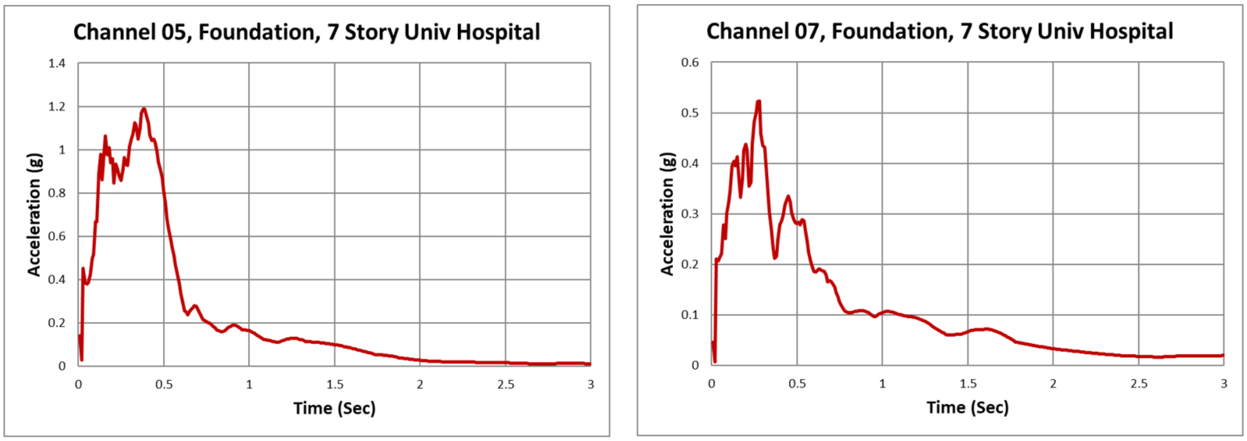

Figure 9: Response spectrum at structure soil interface, Los Angeles – 7-Story University Hospital (Base Isolated).

For each of the buildings selected for this study, a response spectrum was created for the recorded input ground motion at the base of the building. From the response spectrum an S_{DS} value was estimated as an average value close to the peak of the response spectrum in the short period range. This estimated S_{DS} value was divided by the peak ground motion or PGA of the soil structure interface of the building, except for the base isolated building, where the input S_{DS} value is dived by the acceleration at the level above the isolation system as this forms the input motion for the structure above the isolation plane. These results for the three buildings mentioned above are shown in Table 1.

Table 1: Floor accelerations at recorded floors from the 1994 Northridge Earthquake for three instrumented buildings.

Building	Number of Stories	LFRS	Relative Location - Z	Recorded Acc. (g)	Floor Level	Derived S_{DS} from recorded motion	S_{DS} for Site Class D (ASCE 7-10)	Ratio of Accel at Base to Derived S_{DS} Value
Sherman Oaks - 13 Story Commercial Buildings	13	RC Moment Frame - X dir	1.00	0.46	14	1.1	1.459	
			0.63	0.46	8	1.1	1.459	
			0.25	0.53	2	1.1	1.459	
			0.13	0.87	0	1.1	1.459	
			0.00	0.45	-2	1.1	1.459	0.405
	13	RC Moment Frame - Y dir	1.00	0.62	14	0.65	1.459	
			0.63	0.59	8	0.65	1.459	
			0.25	0.56	2	0.65	1.459	
			0.13	0.80	0	0.65	1.459	
			0.00	0.21	-2	0.65	1.459	0.330
Van Nuys 7 Story	7	RC Moment Frame - X Dir	1.00	0.58	7	1.2	1.452	
			0.71	0.46	5	1.2	1.452	
			0.29	0.36	2	1.2	1.452	
			0.14	0.33	1	1.2	1.452	
			0.00	0.45	0	1.2	1.452	0.378
	7	RC Moment Frame - Y dir	1.00	0.57	7	1.4	1.452	
			0.71	0.33	5	1.4	1.452	
			0.29	0.45	2	1.4	1.452	
			0.14	0.40	1	1.4	1.452	
			0.00	0.40	0	1.4	1.452	0.287
Los Angeles 7 Story University Hospital	7	Base Isolated - Y Dir	1.00	0.21	8	1.1	1.715	
			0.75	0.11	6	1.1	1.715	
			0.50	0.10	4	1.1	1.715	
			0.03	0.13	0	1.1	1.715	
			0.00	0.37	0	1.1	1.715	0.119
	7	Base Isolated - X Dir	1.00	0.16	8	0.45	1.715	
			0.75	0.14	6	0.45	1.715	
			0.50	0.08	4	0.45	1.715	
			0.03	0.07	0	0.45	1.715	
			0.00	0.16	0	0.45	1.715	0.162

Summary of Findings

The results show that the peak floor acceleration (PGA) at the base of the building compared to the derived S_{DS}, ranges from 0.3 to 0.45 for non-base isolated buildings and is less than 0.2 for the base isolated building. A closer look at the response spectra for the ground motion in the two orthogonal directions of the base isolated building, shows that the ratio of the peak floor acceleration just above the isolation plane to the derived S_{DS} from the input ground motion at the base of the building has a 35% difference. For the direction where the input ground motion is higher (PGA = 0.37 from Table 1) and an estimated S_{DS} = 1.1g (Table 2), the ratio of S_{DS}/PGA is lower compared to the orthogonal direction where the PGA is lower (PGA =0.16 from Table 1) and estimated S_{DS} = 0.45g (Table 2). This shows that when the base isolators are activated, there is a larger reduction in acceleration demands above the isolation plane. This is even more pronounced when compared to non-base isolated building. These comparisons are shown in Table 2 and represented graphically in Figure 10.

Table 2: Summary of the ratio of PGA to the derived S_{DS} for nine instrumented buildings

Number of Stories	LFRS	Relative Location - Z	Recorded Acc. (g)	Floor Level	Derived S_{DS} from recorded motion	S_{DS} for Site Class D (ASCE 7-10)	Ratio of Accel at Base to Derived S_{DS} Value
19	X Braced Frame	0.00	0.20	-3	0.5	1.489	0.407
19	SMRF	0.00	0.32	-3	0.7	1.489	0.450
14	RC Shear Wall	0.00	0.28	0	0.75	1.462	0.371
14	RC Shear Wall	0.00	0.21	0	0.5	1.462	0.414
13	RC Moment Frame	0.00	0.45	-2	1.1	1.459	0.405
13	RC Moment Frame	0.00	0.21	-2	0.65	1.459	0.330
10	Precast Conc. Shear walls	0.00	0.34	0	1.15	1.56	0.297
10	Precast Conc. Shear walls	0.00	0.26	0	0.7	1.56	0.377
6	Steel MF	0.00	0.36	0	1.18	1.544	0.303
6	Steel MF	0.00	0.21	0	0.55	1.544	0.386
3	Brace Frame 3Story 2RC SW	0.00	0.32	0	0.9	1.484	0.352
3	Brace Frame 3Story 2RC SW	0.00	0.33	0	0.95	1.484	0.344
7	RC Moment Frame	0.00	0.45	0	1.2	1.452	0.378
7	RC Moment Frame	0.00	0.40	0	1.4	1.452	0.287
7	Base Isolated	0.00	0.13	0	1.1	1.715	0.119
7	Base Isolated	0.00	0.07	0	0.45	1.715	0.162
6	Steel Plate Shear Wall	0.00	0.80	0	1.75	1.695	0.455
6	Steel Plate Shear Wall	0.00	0.38	0	1.15	1.695	0.331

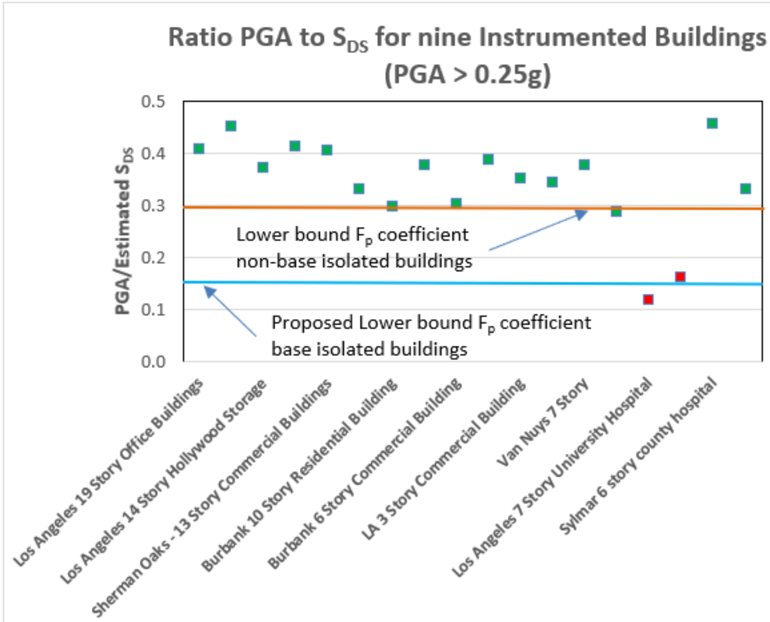

Figure 10: Comparison of PGA to the S_{DS} from nine instrumented buildings

Recommendation

This study supports the proposal that the lower bound limit F_p coefficient used in ASCE 7-22 Equation 13.3-3 should be lower for base isolated buildings, compared to non-base isolated buildings. A reasonable lower bound limit F_p coefficient appears to be in the range of 0.15 to 0.2.