Response Sensitivity of Base-Isolated Steel Buildings to Near-Fault Ground Motions

P. Boroumand
PhD Student- Vienna University of Technology

M. Tehranizadeh
Professor- Amirkabir University of Technology (Tehran Polytechnic)

SUMMARY:
Since the late 1960s, a new classification of ground motions has been recognized, including those types of excitations usually taking place near enough to the ruptured zone, known as Near-Field earthquakes. By the improvement of the seismic data processing facilities, some basic characteristics of these kinds of motions such as high contents of energy and the limited high amplitude long duration pulses, specifically in the velocity and displacement records, accompanied by the significant vertical accelerations have been identified. Accordingly, some unexpected demands have been imposed on the structures in the region, not predicted by current seismic codes of practice. In this study, the performance of 8 Base-Isolated steel buildings with 5, 8, 10 and 15 stories under 6 Near-Fault ground motions have been discussed. Each type of buildings in height is designed once to satisfy the Immediate Occupancy (IO) level of performance with negligible superstructure nonlinearity and the other time, in order to achieve the Life Safety (LS) performance level, demonstrating significant superstructure nonlinear deformations. Having applied the as-recorded three components of the motions simultaneously to the buildings, the effects of Near-Field ground motion characteristics, namely the energy content, the acceleration, velocity and displacement amplitudes of the records on the nonlinear response of structures have been investigated. The results obtained from the study revealed that the energy content and the peak ground displacement (PGD) value of the Near-Fault motions make the most critical conditions for the isolated buildings under consideration and the peak ground acceleration (PGA) parameter usually has the least influence on them. The analyses have been conducted, using SAP2000v.11.0.8.

Keywords: Base-Isolation, Near-Field Ground Motions, Nonlinear Dynamic Analysis, Performance-Based Design

1. INTRODUCTION

Near-Field ground motions have been one of the subjects of enthusiasm in seismology and earthquake engineering since the late 1960s and the beginning of 1970s, when Housner's *et al* and Bertero's *et al* researches on Parkfield 1966 and San Fernando 1971 earthquakes revealed some characteristics of these strong motions[1]. Nowadays, some basic characteristics of these kinds of excitations, such as high amplitude long duration pulses, specifically in the velocity and displacement records and high contents of energy, as well as significant vertical component of the motion have been identified, nevertheless, there are not any specific definitions for Near-Field earthquakes, yet[2].

The extent of structural damage observed in some recent ground seismic activities such as Bam 2003, Chi Chi 1999, Kobe 1995, Northridge 1994, Tabas 1978 etc. in the vicinity of the ruptured zone, once again confirms the statement by D.Iwan that "the structural deformations taking place in the first few cycles of response under a pulse type excitation, cannot be approximated by any single mode of vibration and its shape of deformation will approximate the shape of the time history of the input displacement, independent of the period or other properties of the structure"[3]. That is to say, the current response spectrum analysis defined in the codes may predict the overall amplitude of response in some certain parts of the structure, however, it cannot anticipate the correct local deformations and rotations of the elements, being the causes of damages[3],[4]. Hence, in this study, the effects of different characteristics of the ground motions, recorded within a distance of 10 *km* from the surface of rupturing, on the nonlinear response of 8 Base-Isolated buildings are discussed. For this purpose, applying the performance-based design approach, two groups of 5, 8, 10 and 15-story isolated steel buildings are designed, once to satisfy the conventional expected level of performance of Base-Isolated structures, being Immediate Occupancy (IO) under MCE (Maximum Credible Earthquake) level of shaking according to FEMA 356 definitions and the other time, in order to meet the lower level of performance requirements with significant superstructure nonlinearities, compatible with Life Safety (LS) criteria[5], [6], [7]. In the next step, the nonlinear dynamic behavior of each building subjected to 6 Near-Field earthquakes will be discussed. As the principle motivation of such a sensitivity analysis has originated from the observed structural damage intensity similarities under some initially recognized as DBE earthquakes to those in the MCE class, this paper has aimed to suggest the most critical parameter among the selected ground motion characteristics, causing damage in Base-Isolated buildings[4]. It should be noticed that all the analyses have been conducted, using the three components of each excitation simultaneously, nevertheless, the performance discussion is mainly focused on the horizontal response of the structures[4].

2. MATERIALS AND METHODOLOGY

In this chapter, the properties and selection criteria of the applied earthquake records, geometrical and structural material properties, as well as the design and analysis procedures are briefly and separately reviewed, as follows.

2.1. Characteristics of the Ground Motions

The ground motions used in this research are classified under the Design Base Earthquakes (DBE) and Maximum Credible Earthquake (MCE). The selection criteria of the DBE earthquakes is the closeness of the Peak Ground Acceleration (PGA) parameter to the Iranian Seismic Code of Practice Hazard Map for the same level of shaking and the existence of the Peak Ground Velocities (PGV) in the approximate range of 100 *cm/s*< PGV<130 *cm/s*, should the earthquake be initially categorized as an MCE one[4], [5], [6], [8]. The characteristics of DBE and MCE earthquakes are shown in Table 2.1. All the motions are applied to the structure as they are recorded at the stations, thus the analysis outputs can represent the real loading impact of the selected excitations.

2.2. Design and Modeling Procedures

Isolators used in this study are Lead-Rubber Bearings (LRBs), designed as a bi-linear object, applying the static procedure, introduced in reference [10], assuming the lead core yield stress, the shear modulus of the elastomer and the design shear strain to be 10 *MPa*, 0.4 *MPa* and 100%, respectively.

The analytical characteristics of the LRBs and their dimensions are mentioned in Table 2.2. where W represents the dead load of the superstructure and T is the nominal period of the equivalent single degree of freedom isolated building. In addition, β represents the equivalent viscous damping ratio at the corresponding displacement, D is the displacement of the center of the mass of the isolation system, K_{eff} shows the equivalent linear stiffness at the design displacement while K_1 and K_2 are the initial elastic and the post yield stiffness of the isolators, in the order of appearance.

Table 2.1. Characteristics of the Near-Field Records[4],[9]

Earthquake	Station	Distance	PGA X	PGA Y	PGA UP	PGV X	PGV Y	PGV UP	PGD X	PGD Y	PGD UP	Hazard Level
		(km)	(g)			(cm/s)			(cm)			
Northridge 1994	Arleta Fire Station	9.2	0.34	0.31	0.55	40.44	23.13	17.71	15.08	10.66	8.62	DBE
Northridge 1994	LA Dam	2.6	0.51	0.35	0.42	63	50	19.46	21.25	15.09	8.69	DBE
Imperial Valley 1979	El Centro Array 5	1	0.52	0.38	0.54	46.87	90.53	38.43	35.4	63	19.8	DBE
Tabas 1978	Tabas	3	0.835	0.85	0.69	97.76	121.22	44.41	38.7	95	16.4	MCE
Kobe 1995	Takatory	0.3	0.61	0.61	0.27	127.2	120.7	16	35.7	33.7	4.47	MCE
Northridge 1994	Sylmar Converter Station	6.2	0.61	0.9	0.59	117.4	102.2	34.6	54.3	45.2	25.7	MCE

Eventually, F_y represents the yield force of the isolation system. In all the mentioned parameters, the indices D and M distinguish the quantities at the design and the maximum displacements, in the order of appearance[4],[10],[11].

Table 2.2. Analytical Characteristics of the LRB Isolation System[4]

Strucutre	W	T_D	T_M	β_D	β_M	D_D	D_M	K_{eff}	K_1	F_y	K_2/K_1
	(ton)	(s)		(%)		(cm)		(kN/m)		(kN)	(%)
5	71	2.5	2.8	25	17	21	39	450	2645	44	10
8	108	2.5	2.8	25	17	21	39	690	3999	66	10
10	132	2.8	3.2	28	19	23	44	664	3368	77	10
15	193	3	3.4	28	20	25	47	845	4285	114	10

In the next step, the structures are designed preliminarily, according to ASCE 7-05, AISC 360-05 and FEMA 356 Guidelines as 3D special steel moment frames- the lateral resisting system, capable of behaving satisfactorily enough in both the performance objectives- with horizontally rigid floors[4],[5],[12],[13].

Geometrically, the structures are modeled as bi-symmetrical buildings with five similar spans, each 5 *m* wide and the height of all the stories equals to 3 *m*. The dead and live loads applied to the internal floors are 700 kgf/m^2 and 400 kgf/m^2, respectively and the corresponding values on the roof are 600 kgf/m^2 and 150 kgf/m^2. The beams and columns are designed, using the seismically compact IPE and BOX sections made of structural mild steel with the yield stress 2400 kgf/cm^2 and the ultimate strength, 3600 kgf/cm^2. Finally, the nonlinear characteristics of the superstructure sections have been assigned to each end of the structural elements per FEMA 356 Guidelines. The following flow chart in Figure 2.1 quickly brushes up the major steps of the applied performance-based structural design. Eventually, the nonlinear isolator-superstructure interactions have been considered, applying the direct Wilson-ϴ method of integration with small enough time steps to satisfy the unconditional numerical stability by SAP2000v.11.0.8[4],[5],[14]. The final structural sections are summarized in Table 2.3.

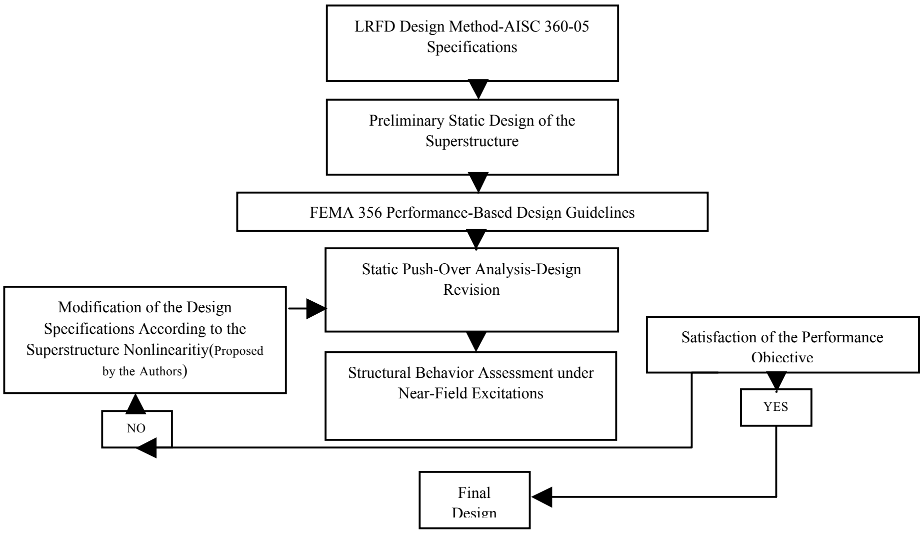

Figure 2.1. Performance-Based Design Steps of the Isolated Buildings[4]

Table 2.3. Final Structural Sections[4]

Performance Level	Story														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
IO	IPE360	IPE360	IPE330	IPE300	IPE240										
	Box 28*28*2	Box 28*28*2	Box 28*28*2	Box 20*20*1.5	Box 20*20*1.5										
LS	IPE330	IPE270	IPE270	IPE240	IPE200										
	Box 28*28*2	Box 28*28*2	Box 28*28*2	Box 20*20*1.5	Box 20*20*1.5										
IO	IPE500	IPE500	IPE450	IPE400	IPE400	IPE330	IPE300	IPE240							
	Box 36*36*2	Box 34*34*2	Box 30*30*2	Box 30*30*2	Box 28*28*2	Box 28*28*2	Box22*22*1.5	Box 20*20*1.5							
LS	IPE360	IPE360	IPE360	IPE330	IPE300	IPE300	IPE270	IPE220							
	Box 34*34*2	Box 34*34*2	Box 30*30*2	Box 30*30*2	Box 25*25*1.5	Box 24*24*1.5	Box22*22*1.5	Box 20*20*1.5							
IO	IPE550	IPE550	IPE500	IPE450	IPE450	IPE400	IPE400	IPE360	IPE300	IPE240					
	Box 38*38*2.5	Box 36*36*2.5	Box 32*32*2	Box 32*32*2	Box 28*28*2	Box 28*28*2	Box 28*28*2	Box 25*25*1.5	Box22*22*1.5	Box 20*20*1.5					
LS	IPE360	IPE400	IPE400	IPE360	IPE360	IPE360	IPE300	IPE300	IPE270	IPE220					
	Box 36*36*2.5	Box 36*36*2.5	Box 32*32*2	Box 32*32*2	Box 28*28*2	Box 28*28*2	Box 26*26*1.5	Box 24*24*1.5	Box22*22*1.5	Box 20*20*1.5					
IO	IPE750	IPE750	IPE600	IPE600	IPE550	IPE550	IPE550	IPE500	IPE500	IPE500	IPE500	IPE450	IPE400	IPE330	IPE240
	Box 50*50*4	Box 44*44*3	Box 38*38*2.5	Box 38*38*2.5	Box 34*34*2.5	Box 34*34*2.5	Box 30*30*2.5	Box 30*30*2.5	Box 30*30*2.5	Box 28*28*2.5	Box 28*28*2.5	Box 28*28*2	Box 28*28*2	Box22*22*2	Box 20*20*1.5
LS	IPE400	IPE450	IPE450	IPE450	IPE450	IPE450	IPE400	IPE400	IPE360	IPE360	IPE330	IPE330	IPE300	IPE270	IPE220
	Box 42*42*3	Box 40*40*3	Box 38*38*2.5	Box 38*38*2.5	Box 34*34*2.5	Box 34*34*2.5	Box 30*30*2.5	Box 30*30*2.5	Box 28*28*2.5	Box 28*28*2	Box 28*28*2	Box 26*26*2	Box 26*26*1.5	Box22*22*1.5	Box 20*20*1.5

3. RESULTS

As follows, the statistical analysis of the nonlinear response of the structures to the governing parameters in the interaction between the Near-Field ground motions and the Base-Isolated structures is represented. The outcome of these analyses is expected to first detect the characteristic parameter(s) of the Near-Field ground motions, having the most destructive effect on the isolated structures and accordingly justify the observed similarities of the structural damage intensity under some DBE motions to those of the MCE class as explicitly documented in reference [4], but eliminated from this text.

3.1. Inelastic Rotation Considerations

As the ground motion pulses impact the structure and propagate by means of waves through its height, the demand for damping increases and in the absence of additional damping devices, the energy of the quake will be dissipated through the inelastic deformations of the structural and nonstructural elements. Hence and in this study, in the absence of any observed plasticity in the columns, the influence of the characteristics of the aforementioned excitations, namely the energy content, the acceleration, velocity and displacement maximum amplitudes on the inelastic rotation of the beams in various degrees of superstructure flexibility, is discussed. Accordingly, the inelastic response of every building is quantified by defining the Average Plastic Rotation Ratio (*APRR*) parameter, calculated as the average of the ratio of the maximum inelastic rotation of the beams to the yield rotation of the corresponding section in all the stories of the structure and in each direction. In the other word, the nonlinear response of every building is summarized by two *APRR* factors, corresponding to the "X" and "Y" directions. However, since this statistical damage index represents an averaged maximum quantity, the distribution of the inelastic hinges throughout the structure height with the LS level of performance is represented as well, using the Story Rotation Factor (*SRF*), following Eqn. 3.1. In the mentioned relationship, the MR_i represents the maximum rotation ratio of the beams in each story and n is the number of stories in each building. It is obvious that the higher the SRF value is, the lower the homogeneity of the damage distribution will be. As an example, $SRF=0$ represents a structure that the maximum inelastic rotation ratio of all its beams throughout the height is almost the same, demonstrating an optimum structural behavior under a specific ground motion[15].

$$SRF = \frac{\sqrt[2]{\sum_{i=1}^{n}(MR_i - APRR)^2 \sum_{i=1}^{n} {MR_i}^2}}{(n-1)APRR} = \frac{\sum_{i=1}^{n}(MR_i - APRR) \quad \sum_{i=1}^{n} MR_i}{(n-1)APRR} \tag{3.1}$$

The results of the *APRR* factor sensitivity to different Near-Fault motion parameters, for both the IO and LS levels of performance and the damage homogeneity index, *SRF*, have been analyzed by means of the diagrams of which one sample group for the 5-story IO and LS buildings is shown in Figure 3.1. The sensitivity of the structure to each characteristic of the Near-Fault motions is quantified by the R^2 factor, written at the corner of each diagram, as it reflects the convergence percentage of a linear interpolation, best fitting in to the obtained results. To comply with the defined page limitations, all the other structures R^2 factors for the different earthquake characteristics are summarized per Table 3.1. Following the outlines of the presented results, the major observations are listed and discussed below.

1. The *APRR* in short to medium isolated structures (5- and 8-story) with IO level of performance are mostly affected by the PGD parameter of the motion. Regardless of the intensity of all the other investigated parameters, it seems the higher the amplitude of the displacement pulse is, the more the structure is likely to respond nonlinearly.
2. Based on the observed analytical data, as the superstructure performance level shifts from the IO to the LS one and loses its relative rigidity, its sensitivity to the Near-Field ground motion

characteristics changes, accordingly. In the short to medium analyzed structures, the LS 5-story one is mostly damaged under the excitation with the highest PGV, though the 8-story building is extensively and plastically affected by the earthquake, having the highest energy content.

3. The *APRR* factor in high rise isolated structures such as 10 and 15 story buildings with IO level of performance increases as the energy content of the ground motion rises.

4. Moreover in the high rise LS cases, the 10 story building *APRR* factor increases the most with an increase in the energy content of the ground motion. However, the 15 story LS building shows the most *APRR* sensitivity to the PGD of the record. Regarding the negligible difference between the R^2 factor values of the energy content and the displacement amplitude per Table 3.1, one can claim that it is essentially the energy content, governing the nonlinear structural behavior of a Base-Isolated steel high rise building, disregarding its level of performance or specific period of vibration. Once the superstructure itself is flexible enough either through the formation of nonlinearities or due to its height, the energy content of the record, which itself is a function of the velocity of the impact and the record frequency content is expected to have the most critical effect on the nonlinear dynamic performance of the isolated structures.

5. Evaluation of the variation of the *SRF* parameter generally shows the same dependency as the *APRR* factor on the Near-Fault earthquake characteristics. The only case of exception in this study is the 15 story LS building, described in the preceding paragraph.

6. Per the observations, the *APRR* and *SRF* parameters of Base-Isolated steel buildings have the least sensitivity to the PGA of the ground motions in Near-Field regions. Implying that this parameter could not be considered as the most appropriate factor in the ground motion record selections for the analysis and design of at least seismically isolated buildings in Near-Field regions. Hence, scaling the time history of the records based on the comparison of their pseudo acceleration spectra with the Code pseudo acceleration spectra could be challenged in terms of its effectiveness to reasonably estimate the seismic demands on the structures.

Table 3.1. The R^2 Factor Variations of the APRR and SRF Parameters for the Earthquake Characteristics [4]

Structure	Performance Level	APRR R^2				SRF R^2			
		PGA	PGV	PGD	Energy	PGA	PGV	PGD	Energy
5	IO	0.006	0.065	0.61	0.33	-	-	-	-
	LS	0.49	0.88	0.58	0.76	0.32	0.37	0.09	0.14
8	IO	0.009	0.07	0.5	0.23	-	-	-	-
	LS	0.41	0.79	0.62	0.92	0.39	0.66	0.35	0.83
10	IO	0.03	0.33	0.56	0.86	-	-	-	-
	LS	0.33	0.41	0.77	0.79	0.47	0.66	0.41	0.73
15	IO	0.06	0.6	0.09	0.61	-	-	-	-
	LS	0.43	0.58	0.88	0.79	0.51	0.64	0.43	0.65

3.2. Base shear Investigations

In this part of the study, the effects of the Near-Fault motion characteristics on the maximum base shear of the superstructure, as one of the most important factors in the structural design procedures are investigated. Similar to the previous part, a linear interpolation function and a convergence percentage factor, R^2 form the base of judgment.

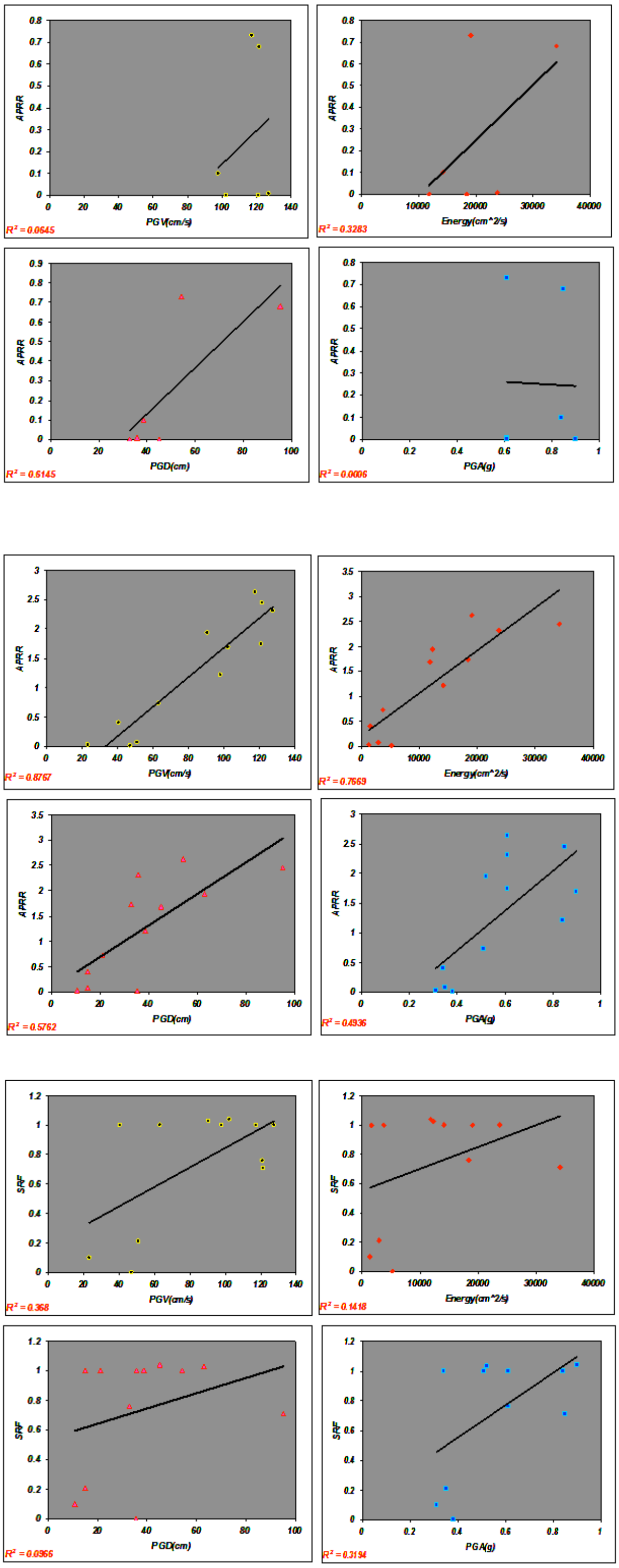

Figure 3.1. Variation of the APRR Factor in the 5-Story Buildings "IO" (Top), "LS" (Middle) and the SRF (Bottom) [4]

The results represented in Table 3.2 in addition to Figure 3.2 show that regardless of the superstructure rigidity or flexibility (different levels of performance) and the structure period of vibration (short to high rise buildings), the maximum value of base shear increases the most as the displacement amplitude of the record, PGD, increases. Its sensitivity to the energy content of the motion is the next prevailing factor and the PGA parameter affects it the least.

Table 3.2. The R^2 Factor Variations of the Maximum Dynamic Base Shears[4]

Performance Level	Structure	Maximum Dynamic Base Shear R^2			
		PGA	PGV	PGD	Energy
IO	5	0.003	0.12	0.64	0.49
	8	0.003	0.13	0.72	0.53
	10	0.04	0.14	0.94	0.77
	15	0.1	0.073	0.98	0.69
LS	5	0.34	0.63	0.83	0.66
	8	0.38	0.6	0.89	0.7
	10	0.42	0.52	0.96	0.72
	15	0.32	0.46	0.93	0.66

4. CONCLUSIONS

Due to some controversial nonlinear structural responses observed in the analysis of 8 Base-Isolated Steel buildings under Near-Field ground motions, a parametric study to investigate the sensitivity of seismically isolated structures with various superstructure degrees of rigidity (LS and IO levels of performance) and periods of vibrations (short to high rise structures) to the dominant characteristics of the Near-Field earthquakes (PGA, PGV, PGD, Energy Content) seemed to be inevitable [4],[11]. The data, summarized in the previous chapters implies that dealing with Base-Isolated steel buildings subjected to Near-Fault motions, it is more rational to consider the PGD or Energy content of the quake rather than its PGA as the characteristic parameter to select or even scale the motions, used in dynamic design analyses or performance evaluation procedures.

Furthermore, the represented numerical results, which for the sake of brevity were summarized by two sets of diagrams and tables, explain and justify the nonlinear structural response similarities between some of the ground motions initially classified as the DBE ones and those in the MCE category. According to the nonlinear time history data analysis documented per reference [4], the performance of the structure under the “Y” component of Imperial Valley 1979 ground motion could be a neat example of such effects where the PGA characteristic of the motion is quite low enough to fall within the conventional DBE class of excitations, however, its notable PGD value induces damage states comparable to the MCE Northridge S 1992 and Kobe 1995 earthquakes. Though the detailed time history results are avoided in this text for the sake of brevity, keen readers may consult reference [4] for more information.

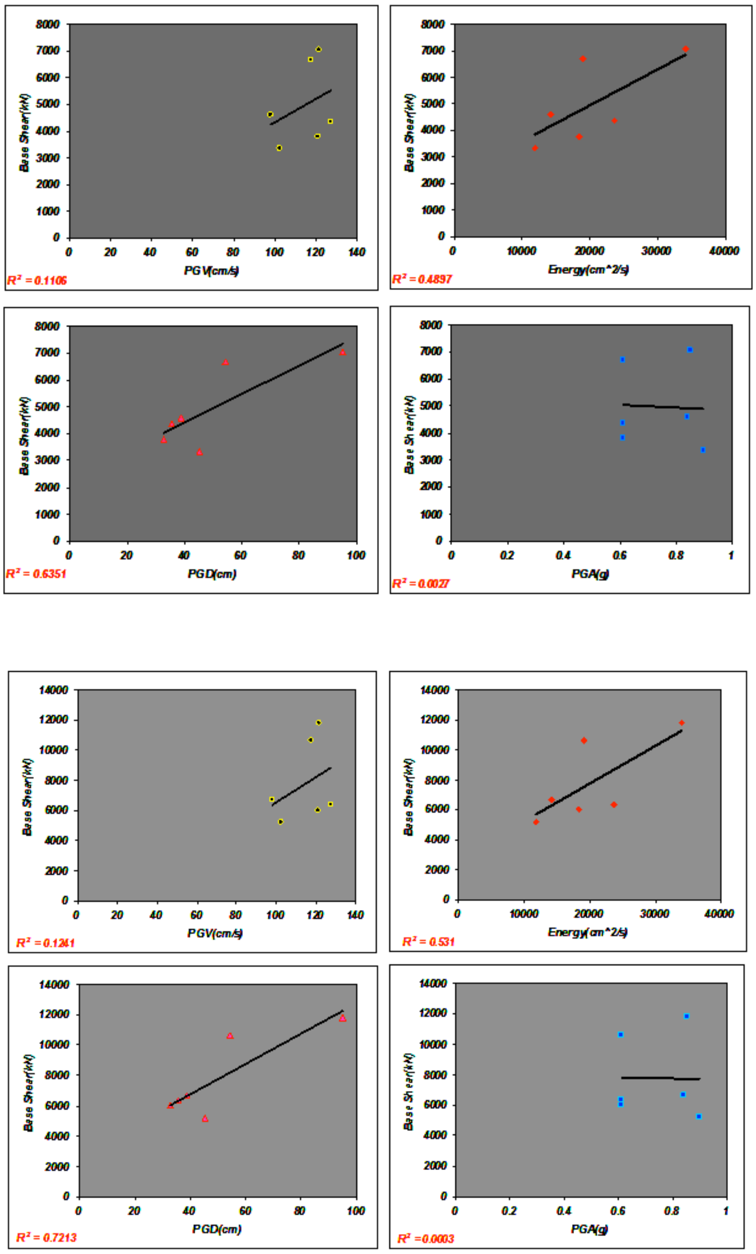

Figure 3.2. Variation of the Maximum Dynamic Base Shear verses the Near-Field Ground Motion Parameters in the "IO" Building, 5-Story(Top), 8-Story(Bottom) [4]

AKCNOWLEDGEMENT

The authors greatly acknowledge Amirkabir University of Technology to support this presentation at the "15th World Conference on Earthquake Engineering".

REFERENCES

1- Bertero, V.V, and Bozorgnia, Y. (2004). The Early Years of Earthquake Engineering and Its Modern Goal. *EARTHQUAKE ENGINEERING From Engineering Seismology to Performance-Based Design*. 2000 N.W. Corporate Blvd., Boca Raton, Florida 33431, CRC PRESS LLC

2- Jangid, R.S. and Kelly, J.M. (2001). Base isolation for near-fault motions. *J. Earthquake Engng Struct. Dyn*. **Vol 30**, 691-707

3- Iwan, W.D. (1995). Near-Field Consideration in Specification of Seismic Design Motions for Structures.*10th European Conference on Earthquake.* 257-267

4- Tehranizadeh, M., Boroumand, P. (2009). *Application of Performance-Based Design in Base-Isolated Steel Structures Under Near-Field Ground Motions*. MSc Thesis, Amirkabir University of Technology, Tehran, Iran

5- American Society of Civil Engineering for Federal Emergency Management Agency (2000). *FEMA 356: Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. Washington DC.: FEMA

6- American Society of Civil Engineering for Federal Emergency Management Agency (1998). *FEMA 273: Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. Washington DC.: FEMA

7- American Society of Civil Engineering for Federal Emergency Management Agency (1998). *FEMA 274: Seismic Rehabilitation Commentary*. Washington DC.: FEMA

8- Building and Housing Research Center (2005). *Standard No.2800-05:IRANIAN CODE OF PRACTICE FOR SEISMIC RESISTANT DESIGN OF BUILDINGS*. Tehran: BHRC

9- PEER Strong Motion Database. *Pacific Earthquake Engineering Research Center (PEER)*. Available at : <http://peer.berkeley.edu/smcat> [Accessed 15 July 2008]

10- Naeim, F. and Kelly, I.M. (1999). *Design of Seismic Isolated Structures, From Theory to Practice*. Chichester: JOHN WILEY & SONS, INC.

11- Tehranizadeh, M., Boroumand, P. (2011). Performance-Based Design of Seismically Isolated Steel Structures with Ordinary Occupancy Importance in Near-Fault Regions(Persian). *Sharif Journal of Science & Technology*. **Vol 27**

12- American Society of Civil Engineering. (2006). *ASCE/SEI 7-05: Minimum Design Loads for Buildings and Other Structures*. Virginia: ASCE

13- AMERICAN INSTITUTE OF STEEL CONSTRUCTION. (2005). *ANSI/AISC 360-05: Specification for Structural Steel Buildings*. Illinois: AISC

14- Computers and Structures, Inc. (2007). *CSI Analysis Reference Manual for SAP2000, ETABS and SAFE*. Berkeley: Computers and Structures, Inc.

15- Ghobarah, A. (2004). ON DRIFT LIMITS ASSOCIATED WITH DIFFERENT DAMAGE LEVELS. *PROCEEDING OF THE INTERNATIONAL WORKSHOP*