AJBAS-Zahid-Ade Faisal-2012

by Ade Faisal

Australian Journal of Basic and Applied Sciences, 6(10): 129-138, 2012
ISSN 1991-8178

Effect Of Repeated Near Field Earthquake To The High-Rise Rc Building

[1]Mohd Zulham Affandi bin Mohd Zahid, [2]Taksiah A. Majid and [2]Ade Faisal

[1]School of Environmental Engineering, Universiti Malaysia Perlis, 02600 Arau, Perlis, Malaysia
[2]Disaster Research Nexus, School of Civil Engineering, Universiti Sains Malaysia, 14300, Nibong Tebal, Pulau Pinang, Malaysia

Abstract: Nowadays, most of the analyst and designer consider 'rare' single earthquake in seismic analysis and design. The repeated earthquakes are ignored even though the actual earthquake event occurs repetitively and the effect of the repeated earthquake is qualitatively acknowledged. Repeated earthquake is the repetition of medium-strong earthquakes at short time interval. The repeated earthquake was reported in many part of the world. This study investigates the effect of this event on the response of the reinforced concrete (RC) buildings. For that purpose, this study uses five generic RC models with different behaviour factor, q (i.e. 1, 1.5, 2, 4 and 6) in which the model with q = 1 is the strongest building and the strength of the building decreases with the increase of the q value. Furthermore, all models are 18-storey to represent high rise buildings in three dimensional in order to consider the effect of earthquake in orthogonal direction. There are 40 repeated near field ground motions which were combined randomly from 20 single near field ground motions employed in this study and 20 residual ground motion records. These repeated earthquakes are divided into two case, i.e. case 2 and case , in which case 2 has two consecutive earthquake(i.e. main shock and aftershock) and case 3 has three consecutive earthquakes (i.e. foreshock, main shock and aftershock). The duration between two consecutive earthquakes is 100s which is enough for the model to cease the motion. The near field earthquake contains special characteristic which is known as pulse effect that far field earthquake do not has, therefore, this study investigates also the effect of pulse period on the response of the RC buildings. The displacement ductility and storey ductility demand were considered to assess the response of the RC models and they are computed by using nonlinear time history analysis. It was found that repeated near field earthquake give significant effect to the respond of high-rise RC buildings compared to single near field earthquake.

Key words: high-rise building, RC building, repeated earthquake, near field earthquake

INTRODUCTION

The current practice is using 'rare' single earthquake in seismic design and analysis. The current seismic codes also ignore the influence of the repeated earthquake even though the effect of the repeated earthquake is qualitatively acknowledged (Hatzigeorgiou and Liolios, 2010). Repeated earthquake is characterized by the repetition of strong-medium earthquake ground motions after short interval of time. In these cases, the structures are already damaged in first earthquake and yet to be repaired, can be inadequate capacity to withstand subsequent earthquake. This accumulation of damage depends on the strength capacity of the building and on the characteristic of seismic events (Amadio *et al.*, 2003).

A few researchers have studied the effect of repeated earthquake on the buildings. Amadio *et al* (2003) showed that repeated earthquake can cause significant accumulation of damage and a consequent reduction in q-factor. Hartzigeorgiou has extensively study the effect of repeated earthquake to the buildings. Hatzigeorgiou and Liolios (2010) found that repeated earthquake lead to larger demand in comparison with corresponding single event. Furthermore, they also estimated the cumulative ductility demands of repeated earthquake using appropriate combinations of the corresponding demands of single earthquake and the proposed combination of ductility demands of single events is in good agreement with the results obtained from dynamic inelastic analysis.

In near field seismic region, the structures are excited by pulse like ground motion and non-pulse like ground motion or also known as residual ground motion in orthogonal direction (Baker, 2007). Since this study deals with near field ground motion, the both pulse like and non-pulse like motions are considered and therefore the RC building need to be modeled in three dimension (3D) instead of two dimension (2D) as used in previous research in investigating the influence of repeated near field earthquake, i.e: Hatzigeorgiou and Liolios (2010), Amadio *et al* (2003) and among others.

Corresponding Author: Mohd Zulham Affandi bin Mohd Zahid, School of Environmental Engineering, Universiti Malaysia Perlis, 02600 Arau, Perlis, Malaysia
E-mail: mohdzulham@unimap.edu.my.

The primary objective of this study to investigate the response of the high rise RC building under repeated near field earthquake. Therefore the response of the 18 storey building under single and repeated earthquake will be compared and the demand parameters used in evaluating the response of the buildings are displacement ductility and storey ductility demand. To be more detail, the influence of the building strength or the behaviour factor, q under repeated near field earthquake excitation will be investigated. Furthermore, since this study focuses on the near field earthquake, the effect of the pulse period, T_p under single and repeated earthquake excitation to the building response also will be evaluated.

MATERIALS AND METHODS

The 18-storey single bay model proposed by Ade Faisal (2011) was considered in this study as shown pictorially in Figure 1. This generic frame model has constant storey height of 3.6 m and 7.2 m of bay width. Moreover, this 3D model are extended from 2D models used by Medina and Krawinkler (2003) and Ruiz-Garcia and Miranda (2005). Note that the validation of 2D model was carried out by Medina and Krawinkler (2003).

The seismic assessment of this building model was carried out with reference to five values of behavior factor or q value: the q values vary between q = 1 (strong building) and q = 6 (weak building). Note that the q values were estimated with reference to the ductility level i.e DCL, DCM and DCH for the seismic design of RC buildings as proposed by Eurocode 8.

It should be noted that, the fundamental period of the buildings are computed based on equation 3.6 as proposed by Eurocode 8. In order to achieve targeted building fundamental period, the weight at each floor and the moment inertia of the structural member need to be tuned. The result from the tune process, the weights at every floor for all models are assumed to be 1240kN and irregularity in mass along the height is not taken into account as there is no significant effect on the response of the structure (Wood, 1992: Al-Ali and Krawinkler, 1998: Miranda and Taghavi, 2005). This study adopts the beam to column ratio equal to 1.3 as proposed by Eurocode 8.

Besides that, this study also adopts overstrength factor of a_u/a_1 = 1.3 as suggested by Eurocode 8 for multi-storey multi-bay frame. Since the Eurocode 8 provisions do not explain on how to distribute the factor, in this study the factor was distributed uniformly along the height of the buildings.

The reduction of the stiffness along the height of the buildings *S(z/H)* is followed the method by Miranda and Reyes (2002) and the ratio of lateral stiffness at the top to the bottom storey, δ is equal to 0.25 as proposed by Ruiz-Garcia and Miranda (2006). For the purpose of having more realistic distribution of lateral stiffness, a decreasing stepwise distribution of lateral stiffness which followed parabolic stiffness distribution was used in this study. The lateral stiffness of the global structure changes for every three stories. The lateral stiffness was calculated using the equivalent cantilever method as explained by Taranath (2010).

This study employs a single component model which was developed with inelasticity along the member is lumped at both end of each member. In order to simulate the cyclic behaviour of reinforced concrete building, this study uses modified-Takeda hysteresis curve as proposed by Zarein and Krawinkler (2009). The moment at yield point, M_y is defined following Medina and Krawinkler (2003) method in which the maximum moment resulted from the linear elastic static lateral analysis is assigned as the M_y at the hinges location. FEMA-P695 recommends a constant value of 1.13 for M_c/M_y, hence, for this study this value is used for hardening stiffness. The yield rotation can be determined by the ratio of M_y to the elastic rotation stiffness (K_o = 6EI/L) as shown in Figure 2.

The yield rotation (Θy) is obtained by the ratio of M_y with elastic rotation stiffness (K_o) and the rotation capacity, i.e. plastic rotation capacity (Θp) and post-capping rotation capacity (Θpc) is equal to 0.04 and 0.06 as proposed by Zarein and Krawinkler (2009). Furthermore, Θu is the rotation ultimate, Θc rotation capacity and M_c is the moment capacity or capping moment. Besides that, *r* is the post yield stiffness ratio or bi-factor which is estimated based on the ratio of capping moment and yield moment (M_c/M_y) and ductility of plastic rotation capacity (μ,o).

The unloading and reloading parameters (a and (3) in hysteresis rule are assumed to be 0.3 and 0.6, for beam and column member respectively following the recommendation by LESSLOSS (2007) and Priestley *et al.* (2007). Moreover, this study considers member strength degradation based on the rotation ductility from Zarein and Krawinkler (2009) backbone curve, which is developed on hysteresis rule of Ibarra *et al* (2005).

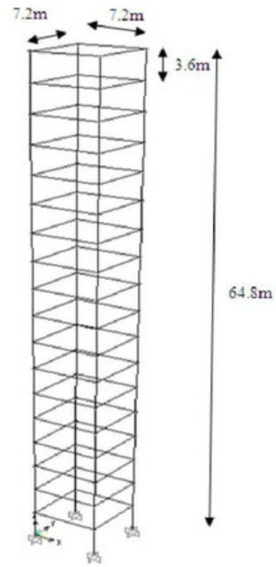


Fig. 1: 18-storey model

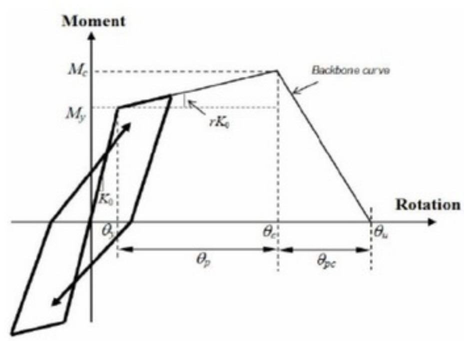


Fig. 2: Modified-Takeda hysteresis and backbone curve (Zarein and Krawinkler, 2009)

In this analytical study, two types of near field ground motion are used: i.e. natural near field earthquake (GM2) and synthetic near field earthquake or also known as residual ground motion (GM3). The original 20 natural near field earthquake records are downloaded from PEER NGA database with reference to the records published by Baker (2007) as shown in Table 1. The residual ground motions are extracted from natural near field earthquakes by removing the pulse component.

In order to study the effect of repeated ground motion, based on Hartzigeorgiou's (2010) method, the above ground motions were combined to become ground motion Case 2 and Case 3. Generally, Cases 1 to 3 can be considered as triple earthquakes, where everyone of their discrete part is multiplied with appropriate factors as follows:

Case 1: (0.0000, 1.0000, 0.0000) as shown in Figure 3 (a)
Case 2: (0.0000, 1.0000, 1.0000) as shown in Figure 3 (b)
Case 3: (0.8526, 1.0000, 0.8526) as shown in Figure 3 (c)

It should be noted that, this study adopts the Ade Faisal's (2011) combination of repeated near field earthquake in which the fore-shock and after-shock are residual ground motion and only main shock is near field ground motion. This combination of ground motions are used in fault-normal component to observe the effect of pulse period on the response of the building. In fault-parallel component, the combination of repeated earthquake consists of residual ground motion only. On top of that, the fore-shock and after-shock are randomly selected using random function in MS-EXCEL to match them with main-shock.

Moreover, all the ground motions used in this study will be scaled to the spectral acceleration ordinate at fundamental period of the building, Sa(T_1), as recommended by Shome *et al.* (1998) to make them comparable. The response spectrum use in this study is the design spectrum of Eurocode 8 for condition of soil type B with peak ground acceleration a_g = 0.36g. The ag value is based on 475-years return period of earthquake that reflecting the condition of Seismic Zone III at Greece (Salomos *et al*., 2008).

RESULTS AND DISCUSSION

The 18-storey RC models were subjected to 40 single ground motions (i.e. 20 GM2 and 20 GM3) and 40 repeated near field ground motions RGM2 in orthogonal direction and the structural assessment was carried by means nonlinear time history analysis.

Table 1: List of near field earthquake

No	Year	Record Name	Station Name	PGA (g)	
				Major (Normal)	Minor (Parallel)
1	1984	Morgan Hill	Coyote Lake Dam (SW Abut)	1.080	0.814
2	1989	Loma Prieta	Gilroy-Gavilan Coll	0.294	0.414
3	1989	Loma Prieta	LGPC	0.944	0.537
4	1992	Landers	Lucerne	0.704	0.807
5	1994	Northridge-01	Jensen Filter Plant	0.518	1.068
6	1994	Northridge-01	Jensen Filter Plant Generator	0.518	1.067
7	1994	Northridge-01	Syimar-Converter Sta East	0.828	0.528
8	1994	Northridge-01	Syimar-Olive View Med FF	0.733	0.595
9	1999	Kocaeli, Turkey	Gebze	0.241	0.203
10	1999	Chi-Chi, Taiwan	CHY028	0.664	0.848
11	1999	Chi-Chi, Taiwan	TCU049	0.286	0.250
12	1999	Chi-Chi, Taiwan	TCU052	0.375	0.393
13	1999	Chi-Chi, Taiwan	TCU053	0.224	0.142
14	1999	Chi-Chi, Taiwan	TCU054	0.157	0.190
15	1999	Chi-Chi, Taiwan	TCU068	0.564	0.431
16	1999	Chi-Chi, Taiwan	TCU075	0.331	0.274
17	1999	Chi-Chi, Taiwan	TCU076	0.310	0.419
18	1999	Chi-Chi, Taiwan	TCU082	0.235	0.190
19	1999	Chi-Chi, Taiwan	TCU102	0.295	0.162
20	1999	Chi-Chi, Taiwan	TCU103	0.133	0.168

Preliminary eigenvalue analysis was carried out to determine the modal properties of the structural system. The 18-storey single-bay model has natural period equal to 1.71 s for first and second mode of vibration with an effective modal mass percentage equal to 99.957%. The computed period is similar to those relative to existing high-rise RC building (e.g. Taranath, 2010).

For the seismic performance assessments, the structural response quantities are expressed in term of global behaviour, i.e. displacement ductility, μ_d and storey ductility, μ_s. Displacement ductility demand μ_d, is the ratio of maximum displacement to the yield displacement while the storey ductility demand, μ_s is defined as the maximum interstorey drift normalized by the interstorey drifts at yield. The distributions of storey ductility demands over the height of the structure are studied to evaluate the storey response characteristics of MDOF system subjected to single and repeated near-field ground motions with forward directivity effects.

Figure 4 presents the relationship between the mean μ_d and behaviour factor, q or strength level of the structure. The designation C1, C2 and C3 represent case 1, case 2 and case 3, respectively. Note that the mean value of μ_d of RC building under GM2 _C 1 is higher than GM3_C1, especially when q > 2 and for high strength structure (q < 2), the variation of μ_d is very small. This is because, according to Baker (2004), the residual ground motion (GM3) has similar characteristic with far field ground motion in which there is no pulse observed in both earthquakes.

The mean of μ_s over the height of the buildings and the maximum standard deviation of the μ_s of all stories (σ_{max}) were computed and included in Figure 5. In general, the different of the building responds due to GM2_C1 and GM3_C1 increases as the q increases. To be more specific, the μ_d and μ_s increase as the q increases indicate that the weaker structure undergoes higher demands in comparison with stronger one.

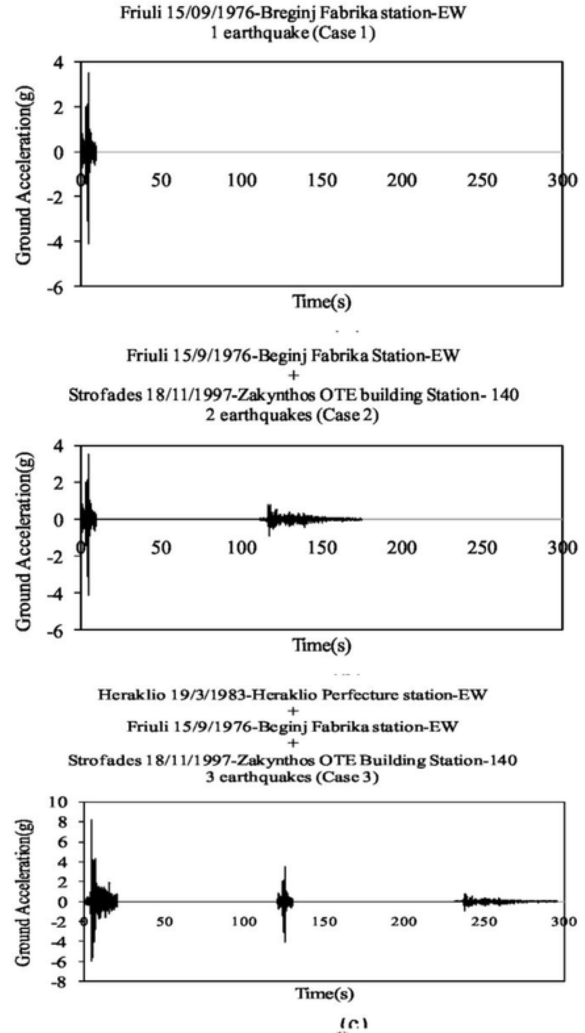


Fig. 3: Time history for single (case 1) and repeated earthquakes (case 2 and 3)

Note that the dispersion in structural response is larger for near field ground motions, as evidenced by the larger values of the maximum standard deviation of storey ductility demands when the structure is subjected to near field with forward directivity ground motions as opposed to residual ground motion and this pattern are consistent as the behaviour factor increases. Furthermore, the maximum standard deviation becomes larger as the buildings become weaker as shown in Figure 5. These findings agree with the observations made by Sehhati *et al* (2011), in which their work compared the μ_s due to near field and far field earthquakes.

Furthermore, repeated near field earthquakes i.e.: RGM2 (GM2_C2 and GM2_C3) impose higher value of μ_d compared to single earthquake as illustrated in Figure 4. Note that, the maximum μ_d of GM2_C2 and GM2_C3 are 28% and 18%, respectively, higher than GM2_C1.

Table 2 provides comparison of mean μ_s between single earthquake and repeated earthquake. The variations of mean μ_s are significant for building under repeated earthquakes (Case 2 and Case 3). Note that, the maximum μ_s tend to increase when the RGM2 is implemented in the time history analyses and the maximum μ_s increases as the q increases. For values of q corresponding to the strength of the building, the average increase of maximum mean of μ_s ranges between 36% (C2) and 90% (C3). The values computed for the frame subjected to the repeated near field earthquake can be higher than three times those computed for single near field earthquake. This finding demonstrates the importance of including the effects of RGM2 to estimate accurately the response of the building in near field area.

The number in the bracket shows the storey level where the maximum mean μ_s occurs. Note that, for the very strong building (q = 1), the maximum μ_s occurs at top storey and when the q increases the maximum μ_s migrates to bottom storey. Furthermore, the minimum μ_s always occur at the middle storey.

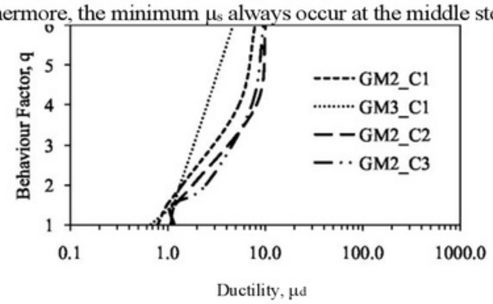


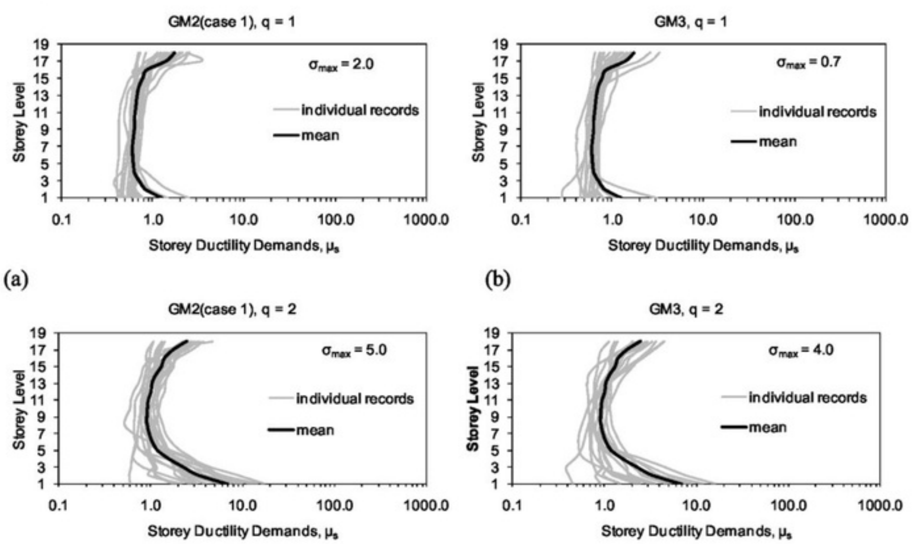
Fig. 4: Mean displacement ductility demand for 18-storey models

Pulse Period:

Since this study deals with near field earthquake, the pulse period plays a vital role in the behaviour of the building subjected to near field event, therefore, the responses of the building model are re-analyzed in order to study the influence of pulse period on storey ductility demands, μ_s. Unlike the previous research (e.g. Kalkan and Kunnath, 2006, Alavi and Krawinkler, 2004 among others), this study investigate real pulse from ground motion records.

Table 3, 4 and 5 summarized the maximum μ_s over the height of 18-storey model for various value of the ratio T_1/T_p. In order to examine the dependency of structural response to the ratio of T_1/T_p and to make it comparable with previous study, this study considers T_1/T_p from 1.8 to 0.48 because within this range the forward directivity pulse renders similar structural response to that computed for an equivalent pulse model (Sehhati et. al., 2011) and most of previous studies used equivalent pulse model to study the effect of T_1/T_p (e.g. Kalkan and Kunnath, 2006, Alavi and Krawinkler, 2004 among others).

Based on the previous study by Kalkan and Kunnath (2006), the demands are higher for T_1/T_p near 1. In this study, however, for case 1, the maximum μ_s experienced by the high rise building with $T_1/T_p = 0.55$ is higher than $T_1/T_p = 0.95$ as shown in Table 3. Therefore, further investigation is carried out and it is found that the velocity spectra ordinate at T_1 for ground motion with $T_1/T_p = 0.55$ is about 80% higher than ground motion with $T_1/T_p = 0.95$ as shown in Figure 6. This finding indicates that the real pulse from ground motion records needs to be implemented in structural analyses in order to estimate accurately the response of the building in near field area.



(a) (b)

(b) (d)

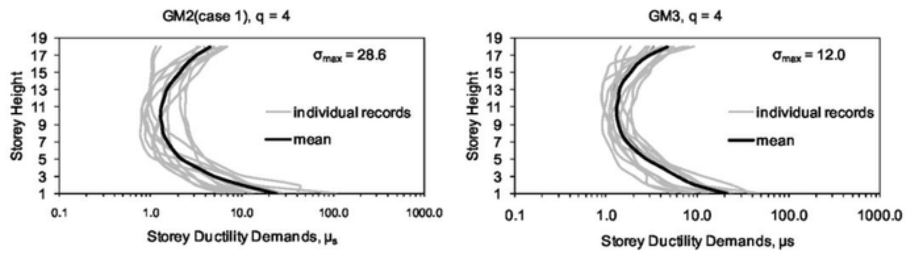


Fig. 5: Storey ductility demands, μ_s

Moreover, under repeated earthquake, the maximum μ_s endured by relative weak structure (q = 4 and 6) is higher for T_1/T_p = 0.95 compared to T_1/T_p = 0.55 as shown Table 4 and 5. The average increase of maximum μ_s experienced by relative weak structures is about 190% and 33% for case 2 and case 3, respectively.

Table 2: Comparison of maximum mean storey ductility demand between single earthquake and repeated earthquakes for various level of q-value

	Behaviour factor				
	q = 1	q = 1.5	q = 2	q = 4	q = 6
C1	1.8 (18)	3.3 (1)	6.9 (1)	23.8 (1)	38.4 (1)
C2	2.1 (18)	4.2 (1)	9.0 (1)	30.4 (1)	61.2 (1)
C3	2.1 (18)	4.4 (1)	8.9 (1)	45.6 (1)	116.7 (1)
C2/C1	1.17	1.27	1.30	1.28	1.59
C3/C1	1.17	1.33	1.28	1.92	3.04

Note: Value in bracket ( ) refers to storey level.

Table 3: Comparison of maximum mean storey ductility demand between single earthquake and repeated earthquakes for various T_1/T_p for case 1

Behaviour Factor	T_1/T_p					
	0.48	0.49	0.55	0.81	0.95	1.8
1.0	0.8 (18)	1.6 (18)	6.8 (17)	1.0 (18)	2.6 (18)	3.3 (18)
1.5	1.3 (18)	4.8 (1)	11.1 (1)	2.5 (18)	2.4 (1)	4.5 (18)
2.0	8.2 (18)	7.5 (1)	16.2 (1)	2.8 (18)	5.2 (1)	4.4 (18)
4.0	23.0 (1)	42.0 (2)	37.7 (1)	127 (1)	24.0 (1)	23.8 (2)
6.0	109.5 (1)	62.3 (1)	54.5 (1)	23.6 (1)	42.8 (1)	35.6 (1)

Table 4: Comparison of maximum mean storey ductility demand between single earthquake and repeated earthquakes for various T_1/T_p for case 2

Behaviour Factor	T_1/T_p					
	0.48	0.49	0.55	0.81	0.95	1.8
1.0	1.5 (18)	1.6 (18)	6.8 (16)	2.5 (18)	2.6 (18)	4.6 (17)
1.5	5.0 (1)	4.7 (1)	9.0 (17)	3.9 (18)	6.5 (1)	4.5 (18)
2.0	11.5 (1)	7.6 (1)	16.2 (1)	12.2 (1)	15.1 (1)	4.4 (18)
4.0	120.5 (1)	66.2 (1)	44.0 (2)	124.0 (1)	127.4 (1)	23.8 (2)
6.0	190.7 (1)	62.3 (1)	65.5 (2)	47.5 (1)	190.0 (1)	35.6 (1)

Table 5: Comparison of maximum mean storey ductility demand between single earthquake and repeated earthquakes for various T_1/T_p for case 3

Behaviour Factor	T_1/T_p					
	0.48	0.49	0.55	0.81	0.95	1.8
1.0	1.3 (18)	1.7 (17)	6.8 (17)	7.8 (18)	2.7 (18)	6.0 (17)
1.5	2.5 (18)	4.8 (1)	10.0 (1)	3.5 (18)	2.3 (1)	4.4 (18)
2.0	5.9 (1)	5.8 (1)	14.7 (1)	12.5 (17)	9.7 (1)	9.4 (1)
4.0	117.9 (1)	126.3 (1)	33.8 (1)	125.0 (1)	51.5 (1)	23.5 (1)
6.0	190.8 (1)	105 (1)	170.5 (1)	176.0 (1)	190.7 (1)	14.3 (18)

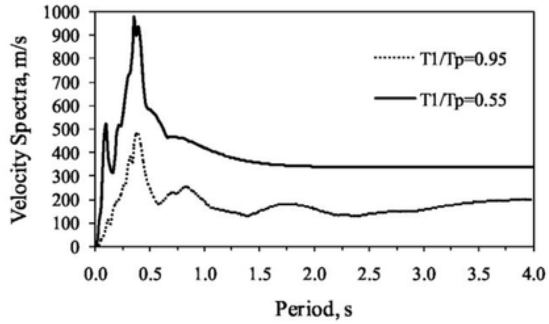


Fig. 6: Velocity spectra for ground motion with $T_1/Tp = 0.95$ and 0.55

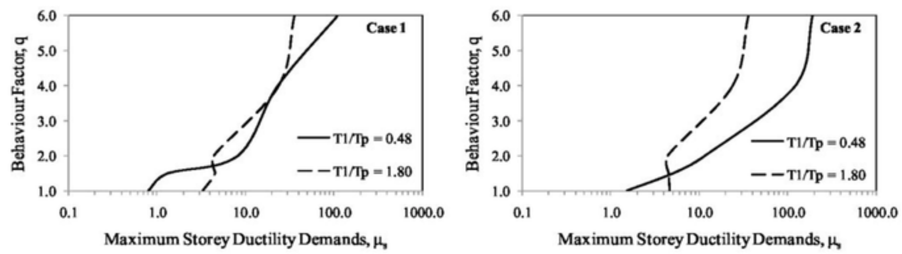


Fig. 7: Comparison of maximum storey ductility demands, μ_s for $T_1/T_p = 0.48$ and 1.80 for case 1(left) and case 2 (right)

Figure 7 illustrates the comparison between maximum μ_s due to near field ground motion with high value of pulse period with respect to fundamental period of building ($T_1/T_p = 0.48$) and high fundamental period of building with respect to the pulse period ($T_1/T_p = 1.8$). Note that, for case 1, the maximum μ_s experienced by relative strong structure (q = 1 and 1.5) under ground motion with $T_1/T_p = 1.8$ is higher than its counterpart. Then, when q increases the maximum μ_s of $T_1/T_p = 0.48$ becomes higher than $T_1/T_p = 1.8$.

This results show that the effect pulse period is significant for relative weak structure (q = 2, 4 and 6).

Furthermore, under repeated earthquake of case 2, only building with q = 1 experienced higher demands under $T_1/T_p = 1.8$ which is two times higher than $T_1/T_p = 0.48$. Then, for the rest of the structures the maximum μ_s due to $T_1/T_p = 0.48$ are higher than $T_1/T_p = 1.8$ and its gap becomes wider as the q increases. This finding indicates that the effect of pulse period may be significant for relative weak structure under repeated earthquakes.

Conclusion:

In this paper, the inelastic dynamic response of five different behavior factor, q of high rise RC buildings has been investigated to study the effect of repeated near field earthquake. A detailed study of the problem leads to the following problem:

a) The average increase of displacement ductility demands, μ_d experienced by high rise buildings under repeated near field earthquake of case 2and case 3 is 28% and 18%, respectively, higher than single near field earthquake.

b) The distribution of storey ductility demand, μ_s is significant under repeated near field earthquake. The average increase of maximum μ_s is 36% and 90% under repeated near field earthquake case 2 and case 3. The values computed for the frame subjected to the repeated near field earthquake can be higher than three times those computed for single near field earthquake. This finding demonstrates the importance of including the effects of RGM2 to estimate accurately the response of the building in near field area.

c) The increase of displacement and storey ductility demand increase as the behaviour factor, q increases.

This works also investigate the effect of pulse period on the ductility demand of the building under single and repeated near field earthquakes. It is found that the effect pulse period is significant for relative weak structure (q = 2, 4 and 6). The average increase of maximum μ_s experienced by relative weak structures is about 190% and 33% for case 2 and case 3, respectively. Furthermore, the gap of the μ_s between single and repeated near field earthquake becomes wider as the q increases. This finding indicates that the effect of pulse period may be significant for relative weak structure under repeated earthquakes.

ACKNOLEDGEMENT

Support for this work was provided by Ministry of Higher Education of Malaysia through research grant (No: 100/PAWAM/814115), UniMAP and USM Fellowship Scheme. Several people contributed to this research, including but not limited to Prof. Dr. George D. Hartzigeogiou from Democritus University of Thrace, Greece.

REFERENCES

Ade Faisal, 2011. Influence of repeated earthquake on the seismic demand of inelastic structures, Final Presentation of Research Progress (Ph.D Thesis), School of Civil Engineering, Universiti Sains Malaysia, Pulau Pinang, Malaysia

Al-Ali, A.K.A. and H. Krawinkler, 1998. Effects of vertical irregularities on seismic behavior of buildings structures, *Report No. 130*, John A. Blume Earthquake Engineering Center, Department of Civil and Environmental Engineering, Stanford University, Stanford

Alavi, B. and H. Krawinkler, 2004. Behavior of moment resisting frame structures subjected to near-fault ground motions. Earthquake Engineering and Structural Dynamics, 33(6): 687-706.

Amadio, C., M. Fragiacomo and S. Rajgelj, 2003. The effects of repeated earthquake ground motions on the non-linear response of SDOF systems. Earthquake Engineering and Structural Dynamics, 32: 291-308.

ASCE, 2006. Minimum design loads for buildings and other structures, ASCE Standard No. 007-05, American Society of Civil Engineers, Reston, VA

ASCE, 2006. Seismic rehabilitation of existing buildings, ASCE Standard No. 045-06, American Society of Civil Engineers, Reston, VA

Baker, J.W., 2007. Quantitative classification of near-fault ground motions using wavelet analysis. Bulletin of Seismological Society of America, 97(5): 1486-1501.

Blume, J.A., 1968. Dynamic Characteristics of multi-storey buildings. Journal of Structural Engineering ASCE, 120(4): 1240-1254

Carr, A.J., 2008. RUAUMOKO – inelastic dynamic analysis program. Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand

CEN, 2003. Eurocode 8: Design of Structures for earthquake resistance. Part 1: General rules seismic actions and rules for buildings. Final Draft prEN 1998. European Committee for Standardization. Brussels

Chopra, A.K. and C. Chitanapakdee, 2004. Inelastic deformation ratios for design and evaluation of structures: single degree of freedom bilinear systems. *Journal of Structural Engineering*, 130(9): 1309-1319.

Chopra, A.K. and R.K. Goel, 2002. A modal pushover analysis procedure for estimating seismic demands for buildings. *Engineering and Structural Dynamics,* 31(3): 561-582.

Elnashai, A.S., L.D. Sarno, 2008. Fundamental of Earthquake Engineering. West Sussex: John Willey & Sons Ltd.

FEMA, 2000. NEHRP recommended provisions for seismic regulations for new buildings and other structures, 2000 edition, Part1: Provisions, FEMA 368, Buildings Seismic Safety Council for Federal Emergency Management Agency, Washington D.C.

FEMA, 2009. Quantification of building seismic performance factors, 2009 edition, FEMA P-695, Applied Technology Council (ATC), Redwood City

Hall, J.F., 2005. Problems encountered from the use (or misuse) of Rayleigh Damping. Earthquake Engineering Structural Dynamic, 35(5): 525-545.

Haselton, C.B., A.B. Liel, S.T. Lange, & G.G. Deierlein, 2007. Beam-column element model calibrated for predicting flexural response leading to global collapse of RC frame buildings, Report No. 2007/03, Pacific Earthquake Engineering Research Center, University of California at Berkeley, Berkeley

Haselton, C.B., A.B. Liel, S.T. Lange, G.G. Deierlein, 2009. Simulating structural collapse due to earthquakes: model idealization, model calibration and numerical solution algorithms: In: ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, 22-24 June 2009, Rhodes, Greece.

Hatzigeorgiou, G.D. and A.A. Liolios, 2010. Nonlinear behavior of RC frames under repeated strong ground motions, in press.

Hatzigeorgiou, G.D. and D.E. Beskos, 2009. Inelastic displacements ratios for SDOF structures subjected to repeated earthquakes. Engineering Structures, 31: 2744-55.

Hatzigeorgiou, G.D., 2010. Ductility demand spectra for multiple near- and far-field earthquakes. Soil Dynamics Earthquake Engineering, 30: 170-83.

Ibarra, L., R.A. Medina and H. Krawinkler, 2006. Hysteresis model that incorporate strength and stiffness deterioration. Earthquake Engineering Structural Dynamic, 34(12): 1489-511.

Ibarra, L.F. and H. Krawinkler, 2005. Global collapse of frame structures under seismic excitations, *Report No. TR152*, John A. Blume Earthquake Engineering Center, Department of Civil and Environmental Engineering, Stanford University, Stanford

Kalkan, E. and S.K. Kunnath, 2006. Effects of fling step and forward directivity on the seismic response of buildings. Earthquake Spectra, 22(2): 367-390

LESSLOSS, 2007. Guidelines for displacement-based design of buildings and bridges. Editor: Fardis, M. *LESSLOSS report No. 2007/05*. IUSS, Pavia.

Medina, R.A. and H. Krawinkler, 2003. Seismic demands for non-deteriorating frame structures and their dependence on ground motions. *Report No. TR144*, John A. Blume Earthquake Engineering Center, Department of Civil and Environmental Engineering, Stanford University, Stanford

Miranda, E. and C.J. Reyes, 2002. Approximate lateral drift demands in multistory buildings with nonuniform stiffness. Journal of Earthquake Engineering ASCE, 128(7): 840-849.

Miranda, E. and S. Taghavi, 2005. Approximate floor acceleration demands in multistory buildings with nonuniform stiffness. Journal of Earthquake Engineering ASCE, 131(2): 203-211.

Miranda, E., 1999. Approximate seismic lateral deformation demands in multistory buildings. Journal of Earthquake Engineering ASCE, 125(4): 417-425.

Miranda, E., 2000. Inelastic displacement ratios for structures on firm sites. Journal of Structural Engineering, 126(10): 1150-1159.

Nakashima, M., K. Ogawa and K. Inoue, 2002. Generic frame model for simulation of earthquake responses of steel frames. Earthquake Engineering and Structural Dynamics, 31: 671-692.

Panagiotakos, T.B. and M.N. Fardis, 2001. Deformations of reinforced concrete members at yielding and ultimate. ACI Structural Journal, 98(2): 135-148.

PEER, 2008. PEER NGA Database, available at http://peer.berkeley.edu/nga/.

Priestly, M.J.N., M. Calvi and M.J. Kowalsky, 2007. Displacement-based seismic design of structures, IUSS Press, Pavia.

Rodriguez-Marek, A. and J.D. Bray., 2006. Seismic site effects for near-fault forward directivity ground motions. *Journal of Geotechnical and Geoenviromental Engineering ASCE,* 132(12): 1611-1620.

Ruiz-Garcia, J. and E. Miranda, 2006. Evaluation of residual drift demands in regular multi-story frames for performance-based seismic assessment. Earthquake Engineering and Structural Dynamics, 35: 1609-1629.

Salomos, G., A. Pinto, S. Dimova, 2008. A review of the seismic zonation in national building codes in the context of Eurocode 8. EUR23563 EN-2008. Joint Research Centre, Ispra.

SAP2000, 2005. Analysis Reference Manual. Computers and Structures, Ins., Berkeley.

Sehhati, R., A. Rodriguez-Marek, M. ElGawady and W.F. Cofer, 2011. Effects of near-field ground motions and equivalent pulses on multi-story structures. Engineering Structures, 33(2011): 767-779.

Seismosoft, 2007. SeismoSignal, available at: http://www.seismosoft.com/.

Shome, N., C.A. Cornell, P. Bazzuro, J.E. Carballo, 1998. Earthquake records and nonlinear MDOF responses. Earthquake Spectra, 14(3): 469-500.

Taranath, B.S., 2010. Reinforced concrete design of tall buildings, CRC Press, Boca Raton

Wood, S.L., 1992. Seismic response of R/C frame with irregular profiles. Journal of Structure and Earthquake Engineering ASCE, 118(2): 545-566.

Yi, W.J., H.Y. Zhang and S.K. Kunnath, 2007. Probabilistic constant-strength ductility demand spectra. Journal of Earthquake Engineering ASCE, 133(4): 740-75.

Zarein, F. and R.A. Medina, 2010. A practical method for proper modelling of structural damping in inelastic plane structural system. *Computers and Structures*, 88: 45-53.

Zarein, F., and H. Krawinkler, 2009. Simplified performance-based earthquake engineering. *Report No. 169*, John A. Blume Earthquake Engineering Center, Department of Civil and Environmental Engineering, Stanford University, Stanford.

AJBAS-Zahid-Ade Faisal-2012

ORIGINALITY REPORT

9%	%	%	9%
SIMILARITY INDEX	INTERNET SOURCES	PUBLICATIONS	STUDENT PAPERS

PRIMARY SOURCES

1	Submitted to Universiti Sains Malaysia Student Paper	2%
2	Submitted to Universiti Teknologi MARA Student Paper	2%
3	Submitted to Universiti Malaysia Perlis Student Paper	1%
4	Submitted to City University Student Paper	1%
5	Submitted to Universiti Kebangsaan Malaysia Student Paper	1%
6	Submitted to Durban University of Technology Student Paper	1%
7	Submitted to University of New South Wales Student Paper	1%
8	Submitted to Institute of Graduate Studies, UiTM Student Paper	1%
9	Submitted to Payson High School	

Student Paper

<1%

10	Submitted to Universiti Putra Malaysia Student Paper	<1%
11	Submitted to SASTRA University Student Paper	<1%

Exclude quotes	Off	Exclude matches	Off
Exclude bibliography	On		