COMPARISON ON THE EFFECT OF EARTHQUAKE AND WIND LOADS ON THE PERFORMANCE OF REINFORCED CONCRETE BUILDINGS

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ABSTRACT:

In the year 2004 and 2005, tremors from the Sumatran earthquakes had brought safety concerns to the publics, government authorities, engineers and researchers especially when no earthquake design had been taken into practices in Malaysia. This study addresses the performance of reinforced concrete buildings with the comparison on the effect of earthquake and wind loads for existing buildings in Malaysia, so that the adequacy of the design capacity can be checked. This study investigated seven existing buildings from West and East Malaysia. The buildings were categorised as medium and high-rise reinforced concrete moment resisting frames. The UBC-97, CP3:1972 and the MS 1553:2002 are used as the design codes in determining the lateral loads from earthquake and wind. The design capacity calculation for the frames was based on BS 8110. There are four types of analyses adopted; (i) Free Vibration Analysis (FVA), (ii) Earthquake Static Equivalent Analysis (ESEA), (iii) Static Wind Analysis (SWA), and (iv) Earthquake Dynamic Response Spectrum Analysis (EDRSA). Results from FVA showed that five out of seven buildings produced high dynamic amplification factor in the range of 2.01 to 5.16. These values showed that buildings were responding dynamically due to the fequencies of earthquakes from neighbouring earthquakes which are similar to the building frequencies. From this study it can be noted that the ESEA normally produced larger lateral load design forces than that from the SWA and EDRSA. ESEA also result in larger base shear and deformation response include greater lateral displacement and inter-story drift in the buildings. However under the ESEA, the performances of buildings were generally deemed satisfactory even under the low intensity of earthquake level (0.20g). Based on story drift response from ESEA, there are potential failures for the medium rise buildings at lower storey levels. While for high-rise buildings, the failures can potentially occur at higher storey levels. The inter-story drifts indicator indicates that only the non-structural elements of the buildings would be possibly affected.

KEYWORDS: Earthquake, Wind loads, Concrete Buildings

1. INTRODUCTION

The objective of this study is: i) to identify the design base shear force for buildings in Malaysia by perform wind and earthquake static equivalent analyses, ii) to identify the design capacity level of buildings and the maximum allowable lateral load based on shear coefficients of wind and earthquake loadings by perform earthquake linear static and dynamic analyses, iii) to specify the damage level of buildings by perform earthquake non-linear dynamic analysis. All of existing buildings categorised as reinforced concrete moment resisting frame. Thus the results are checking by British Standard 8110 for the design capacity in each element. By adopting the displacement-based approach, various wind and seismic assessment methodologies including, static wind analysis, earthquake response spectrum analysis (ERSA), earthquake static equivalent analysis (ESEA) and earthquake time history analysis (ETHA) were implement. The performance of structures under wind and earthquake load use the state of the art analytical modeling tool of typical medium and high-rise building design. From the prediction of structural earthquake response by inelastic dynamic analysis, essential

information to structural designers on the safety level of the existing buildings due to seismic load could be provided.

Based on UBC-97 code, the dynamic analysis is required for regular structure over 240 feet (72.0 m) in height, while for irregular structure over 65 feet (19.5 m) in height. This code requires the irregularity divided into two wide categories: vertical structural and plan structural irregularity. Vertical irregularities include soft or weak stories, large changes in mass from floor to floor and large discontinuities in the dimensions or in plane locations of lateral load resisting elements. While planned irregular buildings include those which undergo substantial torsion when subjected to seismic loads, have re-entrant corners, discontinuity in floor diaphragms, discontinuity in lateral force path, or lateral load resisting elements which are not paralleled to each other or to the axes of the building. However for comparison purposes, all seven buildings were selected for dynamic analysis even though the buildings are mostly categorized as regular structures.

2. MATERIAL PROPERTIES

In Malaysia, the design of the reinforced concrete frames, is based on BS 8110 design code. A typical value of concrete compressive strength, f_c and reinforcing yield stress, f_y is equal to 27.6 N/mm^2 and 460 N/mm^2 respectively. The concrete modulus elasticity, E is 25000 N/mm^2 and shear modulus, G is 10000 N/mm^2 were taken in the design consideration. These values were applied for all modal frames which were assumed designed for similar strength of materials.

3. LOADINGS

There are two types of loads that are considered in this study; i) gravity load ii) lateral load. Gravity loads carried the dead load and life load, whilst, lateral loads considered either wind, notional or earthquake loads.

3.1 Gravity load

This loading represents all tributary dead loads and live loads. The dead load included cladding and concrete self weight with concrete density equal to 24kN/m^3. Based on BS 6399: Part 1: 1984, the live load was taken as 3.0kN/m^2 for each building which are categorized under residential and offices occupancy class. The loads were distributed uniformly on all beams between of column lines.

3.2 Lateral load

In order to compare the performance of wind and earthquake loads on the reinforced concrete buildings, the static lateral loads of wind and earthquake forces were analysed. Besides, for the comparison purpose of existing lateral load design in BS8110, the notional load also included the load equal to 1.5% of the dead load.

3.3 Static Wind Load

Wind load design was considered in modal frame analyzed under static analysis. The design wind load using CP3: Chapter 5 with local condition of each building was taken into account.. Design wind speed in East and West Malaysia was taken into account for wind load design between 20m/s to 50m/s. The load was assumed to act parallel to the transverse frame direction to each floor.

Table 1 ists the design wind pressure for wind velocity of 20m/s and 50m/s.

Table 1 Design wind pressure for wind velocity of 20m/s and 50m/s

No.	Building	S_1	S_2	S_3	V=20 m/s		V=50m/s	
					V_s (m/s)	q (N/m^2)	V_s (m/s)	q (N/m^2)
1	Kuarters Kelas G Kudat	1.0	0.87	1.0	17.4	185.59	43.50	1159.95
2	Kuarters Kelas F Miri	1.0	0.88	1.0	17.6	189.55	44.00	1186.77
3	Hospital Besar KB	1.0	0.99	1.0	19.8	240.32	49.50	1502.00
4	Mahkamah Labuan	1.0	0.98	1.0	19.6	235.49	49.00	1471.81
5	JPN Putrajaya	1.0	1.02	1.0	20.4	255.11	51.00	1594.41
6	Mahkamah KT	1.0	1.04	1.0	20.8	265.21	52.00	1657.55
7	Pangsapuri Putrajaya	1.0	1.08	1.0	21.6	286.00	54.00	1787.51

Note:
$V_s = V.S_1.S_2.S_3$
$q = 0.613 V_s^2$

3.4 Static Equivalent Earthquake

Static equivalent lateral load (Eq.1) was analysed under static linear where the load designed was based on UBC-97. The Peak Ground Acceleration (PGA) of 0.05g to 0.2g was taken into account which was considered as low seismic zone in Malaysia. For the example of base shear design in each building, Table 2 lists the base shear for PGA 0.05g and 0.2g. This table also shows the seismic design parameters involved in the base shear design. Based on the static lateral force procedure, the base shear, V is distributed to each building storey by the formula:

$$F_x = \frac{(V - F_t)(w_x h_x)}{\sum_{i=1}^{n} w_i h_i}$$

(1)

where
F_t is force at the top where if T less or equal to 0.7 sec., F_t is equal to 0.0, while if T more than 0.7 sec., F_t is equal to $0.07 \leq TV \leq 0.25V$.
F_x is lateral force at level x
w_x, w_i is   portion of w located at level x and i respectively
h_x, h_i is   height in feet above the base at level x and i respectively
n is number of floors

3.5 Earthquake Response Spectra

Dynamic lateral force using site–specific spectrum was applied in each building. Design response spectra from UBC-97 as shown in Figure 1(a) was adopted. The seismic coefficients C_a and C_v, are correlated with soil profile type and seismic zone factor. In this study the site-specific soil was taken as under category S_D (stiff soil profile) with low intensity of ground accelerations (0.05g to 0.02g) was assumed. Figure 1(b) shows the design response spectra on S_D soil type for various intensities based on UBC-97 requirement. These spectra were applied in each building and analysed under dynamic linear analysis.

Table 2 Design base shear of buildings for ground acceleration 0.05g and 0.20g

No.	Building	W (kN)	h_n (ft)	C_t	T (sec.)	R	I	N_v	$A_g = 0.05g$				$A_g = 0.20g$			
									C_a	C_v	Z	V (kN)	C_a	C_v	Z	V (kN)
1	Kuarters Kelas G Kudat	450.00	47.56	0.03	0.543	3.5	1.0	1.0	0.08	0.13	0.05	27.00	0.28	0.4	0.2	90.00
2	Kuarters Kelas F Miri	1063.8	49.69	0.03	0.561	3.5	1.0	1.0	0.08	0.13	0.05	63.80	0.28	0.4	0.2	212.80
3	Hospital Besar KB	1161.22	87.90	0.03	0.861	3.5	1.0	1.0	0.08	0.13	0.05	46.45	0.28	0.4	0.2	150.96
4	Mahkamah Labuan	1850.01	82.66	0.03	0.822	3.5	1.0	1.0	0.08	0.13	0.05	190.70	0.28	0.4	0.2	534.10
5	JPN Putrajaya	26873.28	142.35	0.03	1.236	3.5	1.0	1.0	0.08	0.13	0.05	806.20	0.28	0.4	0.2	2418.60
6	Mahkamah KT	5646.00	119.39	0.03	1.084	3.5	1.0	1.0	0.08	0.13	0.05	198.10	0.28	0.4	0.2	726.30
7	Pangsapuri Putrajaya	3028.25	168.59	0.03	1.404	3.5	1.0	1.0	0.08	0.13	0.05	90.85	0.28	0.4	0.2	242.26

Note:

W   = total seismic dead load
h_n = building height
C_t = building system coefficient
T   = fundamental period of vibration
R   = structural system coefficient
I   = importance factor
N_v = near sources factor

C_a and C_v = seismic coefficient dependent on the soil condition at the site and regional seismicity

Z   = seismic zone factor
V   = base shear
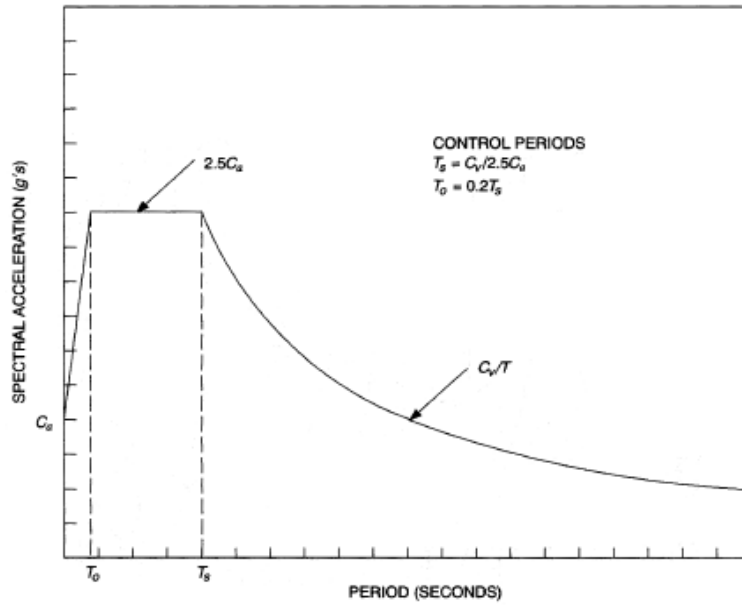
(a) Fundamental RS based on UBC-97
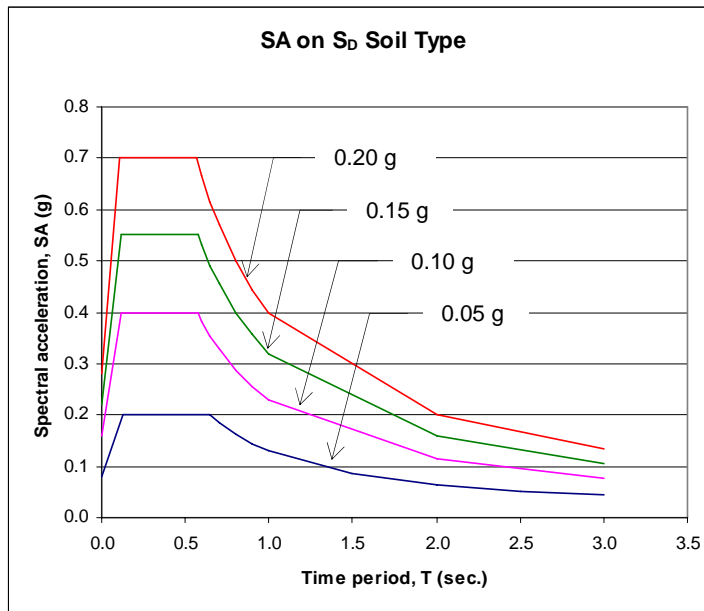
(b) RS for various ground acceleration on S_D soil type

Figure 1 Response Spectra (RS) applied on structural buildings based on UBC 97

4.   DYNAMIC AMPLIFICATION

The dynamic characteristics of building from the free vibration analysis which includes number of mode, time period, frequency and angular frequency. It indicates the first four mode shapes, time period, frequency, and angular frequency for seven modal frames which were produced using SAP 2000 computer program.

To study the response of buildings due to local earthquakes, dynamic amplification factors determine the dynamic characteristics of the building and local earthquakes. Most of major local earthquake loading that happened in Sumatra and East Malaysia were preferred for studies on our building structures. The three local earthquakes from Sumatra are named as Acheh, Nias and Semangko earthquake with magnitudes 9.0, 9.0 and 8.0 on the Richter scale respectively. Besides that two of local earthquakes from East Malaysia have been chosen which namely Tawau and Bintulu where each earthquake has magnitude 6.0 and 7.5 on the Richter scale respectively.

The results from the seven modal frames indicate that four buildings have dynamic respond due to local earthquakes as shown in Tables 3 and 4 for Peninsular and East Malaysia buildings respectively. For Peninsular Malaysia, Block 3B Pangsapuri Putrajaya has the largest value of amplification factor of 3.70 and 2.01 due to Nias and Acheh earthquake respectively. Where else the JPN building in Putrajaya and Mahkamah Kuala Terengganu had a factor of 2.95 and 2.04 respectively due to Semangko earthquake. For East Malaysia, Quarters Kudat has the largest value of amplification factor of 5.16 due to Bintulu earthquake. Kuarters Miri also indicated high dynamic response due to Bintulu earthquake which is 2.44 of amplification factor. Other buildings seem to behave minimum dynamic response or statically under earthquake loads due to the factors of about 1.0.

Table 3 Dynamic magnification for buildings in Peninsular Malaysia

Site Buildings		Epicentre	Distance (km)	$\overline{T}$	$\varpi = \dfrac{2\pi}{\overline{T}}$	ω	$\beta = \dfrac{\varpi}{\omega}$	$D = \dfrac{1}{\sqrt{\left(1-\beta^2\right)^2 + \left(2\xi\beta\right)^2}}$
Mahkamah Kuala Terengganu	to	Acheh	825	3.02	2.08	4.65	0.45	1.25
		Nias	750	2.77	2.27	4.65	0.49	1.31
		Semangko	610	1.88	3.34	4.65	0.72	2.04
Blok 3B Kuarters Putrajaya	to	Acheh	625	2.36	2.66	3.74	0.71	2.01
		Nias	500	1.95	3.22	3.74	0.86	3.70
		Semangko	325	1.14	5.50	3.74	1.47	0.85
Jabatan Pendaftaran Negara Putrajaya	to	Acheh	625	2.36	2.66	4.79	0.56	1.44
		Nias	500	1.95	3.22	4.79	0.67	1.82
		Semangko	325	1.14	5.50	4.79	1.15	2.95
Hospital Besar Kota Bharu	to	Acheh	790	2.91	2.16	5.60	0.39	1.17
		Nias	700	2.61	2.41	5.60	0.43	1.23
		Semangko	575	1.79	3.51	5.60	0.63	1.64

Note:

$\overline{T}$ = Natural period of earthquake
$\overline{\omega}$ = Angular frequency of earthquake
ω = Angular frequency of building
β = Frequency ratio
D = Dynamic magnification factor

Table 4 Dynamic magnification for buildings in East Malaysia

Site Buildings		Epicentre	Distance (km)	$\overline{T}$	$\varpi = \dfrac{2\pi}{\overline{T}}$	ω	$\beta = \dfrac{\varpi}{\omega}$	$D = \dfrac{1}{\sqrt{\left(1 - \beta^2\right)^2 + (2\xi\beta)^2}}$
Mahkamah Labuan	to	Tawau	275	0.59	10.66	6.07	1.76	0.48
		Bintulu	430	0.81	7.79	6.07	1.28	1.51
Kuarters Kudat	to	Tawau	280	0.60	10.54	6.55	1.61	0.63
		Bintulu	490	0.89	7.06	6.55	1.08	5.16
Kuarters Miri	to	Tawau	475	0.87	7.23	5.89	1.23	1.92
		Bintulu	500	0.90	6.95	5.89	1.18	2.44

5.   CONCLUSIONS

5.1 Static and Dynamic Analyses

1) The earthquake static equivalent analysis is mostly governed by lateral load design, followed by the static wind load and the dynamic response spectrum analysis.

2) The result of seven modal frames buildings indicate that the maximum axial forces under the wind and earthquake loads are mostly exceed the design capacity of column element except for the column at KM building frame.

3) Maximum shears (V_{max}) at beam elements for all modal frames were surpassing the design capacity of all load cases except for HKB and PPJ buildings.   While the V_{max} at the column elements are sufficient to resist all load cases for all modal frames except for JPNP and MKT.

4) The performances of beam element shows that maximum moment bending (M_{max}) are still below the beam design capacity for all load cases and analysis type except for MKT building.   However the performances of M_{max} at column elements for all buildings can be sustained under different load cases with various wind velocities and peak ground accelerations.

5.2   Comparison between Wind and Earthquake Analyses

1) From this study, it has been noted that the Earthquake Static Equivalent Analysis (ESEA) is most governed in lateral load design, rather than Static Wind Analysis (SWA) and Earthquake Dynamic Response Spectrum Analysis (EDRSA).   These results could be observed from the base shear and the deformation respond includes the lateral displacement and inter-storey drift of the buildings.

2) If the base shear obtained from wind forces with velocity ranges of 30m/s to 40m/s in SWA were used as design limit, the buildings may be found to be unable to withstand under lateral load even at the low earthquake intensity (0.05g) in ESEA.

3) The maximum internal result forces under the wind and earthquake linear analysis; (e.g.; SWA, ESEA and EDRSA) have similar manners of failure at beams and columns elements.   The maximum axial force at most building frames exceeded the design capacity of the column. While maximum shear force at most building frames exceeded the design capacity of beam.   The MKT building frame is the most

crucial frame where the maximum internal force at beam and column elements exceeded the design capacity except for bending moment at column.

5.3   Comparison between the Linear and Non-Linear Analysis

1) Based on the results of the earthquake non-linear dynamic analysis the performance of the medium and high-rise reinforced concrete buildings were generally deemed satisfactory under the low intensity earthquake level (0.20g).   The Inter-storey Drift Index (IDI) level shows that HKB had reached the fourth level, while KK, KM and PPJ buildings had reached the third level of the damaging potential. Based on lateral stability index by Scholl (1984), the fourth level indicates that non-structural damage is certain and structural damage is likely, while the third level indicates that non-structural damage is relatively certain and structural damage is likely.   Since this earthquake level would likely to fail at structural elements only in HKB building, it could be concluded that the resulting inter-storey drifts will possibly only affect the non-structural elements at the most buildings (partition etc.).   However based on Park and Ang Overall Damage Index, the maximum indices may reach is 0.107 (PPJ building), which categorised under the moderate damaging level where extensive large crack and spalling of concrete in weaker elements may occur.   Plastic hinges were mainly formed in beams of the buildings except for MKT building frame where the plastic hinge was initially formed at column element.   These results were subjected to the El-Centro earthquake records scaled up to 0.20g of ground acceleration.

2) Under the earthquake linear static analysis, the performances of the medium and high-rise reinforced concrete buildings were generally deemed satisfactory under 0.20g intensity earthquake level.   It can be shown from the Inter-storey Drift Index (IDI) level where KK and PPJ buildings may reach the second level of damaging potential which indicates that the non-structural damage is likely.   Based on the performance of frame elements, the maximum internal forces are failure at beam elements for KK, KM, ML, MKT and JPNP.   While for column elements all buildings are affected under axial force except for KM building.

3) From the vulnerable study of modal frames indicates that KM and PPJ modal frames were expected to collapse under 0.55g of earthquake intensity. While for JPNP, ML, and KK modal frames were expected to collapse at 0.9g, 1.17g and 1.2g of earthquake intensity respectively.   Nevertheless the highest resisting building frames due to earthquake ground motion belonged to the HKB building frames with 1.9 g of earthquake intensity level.

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