CHAPTER 3

ASSESSMENT OF SEISMIC DEFICIENCY OF EXISTING REINFORCED CONCRETE BUILDINGS IN BANGKOK

A study on seismic evaluation of RC buildings in Bangkok is presented. In this study, existing reinforced concrete buildings constructed as beam-column frames are investigated to identify the typical characteristics of building designed for gravity load only, especially the reinforcement detailing. This study focuses on reinforced concrete frames having 5-15 stories. According to previous studies, the buildings of this range will have a natural period around 1.0 second which corresponds to the peak acceleration response curve. For the buildings of this height, the fundamental vibration is supposed to play a dominant role. The first floor is supposed to be critical under earthquakes, thus the research conducted herein emphasizes the first floor sub-frame in both transverse and longitudinal directions of the buildings.

In this study, a practical method investigating seismic performance of the building is proposed. It aims to provide a guidance for practicing engineers to evaluate the seismic performance of the buildings. The method requires building geometry, cross sectional dimension, reinforcement details, material properties as input to construct the computer model of the buildings. The linear static analysis is employed to obtain shear force and bending moment at critical sections of members. The Demand Capacity Ratio (DCR) is then calculated. The capacity can be based on any structural design code. In this research, the ACI318 (1999) requirement for Intermediate Moment Resisting Frames is adopted as most Thai engineers are familiar with the American code. The proposed evaluation methodology consists of DCR determination, reinforcement detailing check and failure mode investigation.

3.1. Data collection of existing buildings

The architectural and structural drawings of 15 existing buildings constructed in Bangkok are collected. These buildings are listed in Table 3.1. As seen, the buildings have the number of stories ranging from 5-15 stories. They are constructed as beam-column rigid frame. The buildings are generally classified as essential and public facilities that cover apartments, schools, universities governmental offices and hospitals. In most buildings, the floor system consists of precast solid plank (PC plank) or precast hollowcore slab which is topped by cast-in-place concrete. Almost all buildings are provided with lift core for vertical transportation.

Table 3.1 General data of investigated buildings.

No.	Occupancy type of Building	No. of story	Total height (m)	Dimension W x L (m^2)	Type of Slab	Lift Core
1	Apartment, AP1	6	17.20	11.0x17.85	PC Plank	None
2	Apartment, AP2	9	22.90	13.2x29.2	PC Plank	2x2m.
3	Apartment, AP3	15	40.00	21.0x31.8	PC Plank	4.2x2.15m.
4	Apartment, AP4	8	22.40	13.1x30.3	PC Plank	2.7x3.2m.
5	Apartment, AP5	6	22.05	25.3x35.0	PC Plank	2.4x2.4m.

Table 3.1 General data of investigated buildings (continued)

No.	Occupancy type of Building	No. of story	Total height (m)	Dimension W x L (m^2)	Type of Slab	Lift Core
6	Academic, AC1	5	29.80	18.6x39.0	Cast in place	2.55x2.9ม.
7	Academic, AC2	6	26.00	18.8x58.5	PC Hollowcore	2.4x4.0m.
8	Academic, AC3	6	27.50	16.5x77.0	PC Hollowcore	2.5x4.3m.
9	Academic, AC4	5	20.25	13.4x36.0	PC Plank	None
10	City hall, CH1	5	25.34	24.0x77.0	PC Hollowcore	2.8x5.0m.
11	City hall, CH2	5	34.35	25.0x77.0	PC Hollowcore	2.8x5.0m.
12	City hall, CH3	5	30.95	25.0x77.0	PC Hollowcore	2.8x5m.
13	Hospital, HP1	10	43.70	37.6x43.6	Cast in place	3.8x3m. and 7.6x4.2m.
14	Hospital, HP2	9	42.50	26.95x58.5	Cast in place	2.7x3.2 m.
15	Hospital, HP3	5	20.00	21.0x21.0	PC Plank	3.1x2.3m.

3.2. Structural indices

Structural indices are defined as the parameters that characterize the behavior of beam, column and joint under the seismic action. Structural indices of buildings are calculated from design configurations such as sectional dimensions (Fig.3.1), quantity of longitudinal and transverse reinforcements, strength of concrete and reinforcement and others. The prominent structural indices used in predicting the failure modes are as follows.

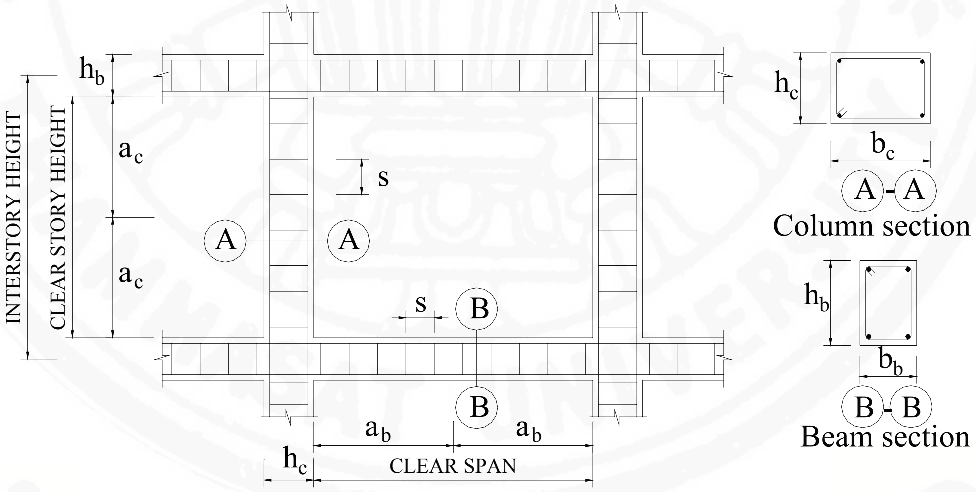

Fig.3.1 Definition of geometry parameters of structural indices

3.2.1 Beam and column indices

Both beam and column indices can indicate the comparative sectional properties, member capacity, and possible mode of failure of the member. Indices of both members have some common characteristics therefore the same indices are reported simultaneously as follows:

i) Shear span ratio (a_b / h_b, a_c / h_c): This index demonstrates the relative intensity of shear force and flexural moment acting on the member. If the shear span ratio is high, it is likely that the structure will fail in flexural mode.

a_b and a_c are shear span defined as the half of clear span of beam length and the half of clear story of column height, respectively. Both are measured from the joint face to a flexural inflection point of member along the member axis, and the inflection point was assumed to be set at the mid-height or mid-length of a member; h_b and h_c are beam height and column depth measured in the direction parallel to the applied force, respectively as shown in Fig 3.1.

ii) Nominal moment capacity to nominal shear capacity ratio ($M_{nc} / a_c V_n, M_{nb} / a_b V_n$) This index indicates a possibility of shear failure or flexural failure in the member. Based on the assumption that the inflection point is located at the mid-height of column or the mid-length of beam, larger value of this index indicates higher nominal flexural strength compared with shear strength, and a possibility of shear failure before flexural failure, and the value of the index equal to one indicates that shear force and moment reach the shear strength and flexural yield strength simultaneously.

a_c and a_b are the length measured along the column and beam axis from the joint, respectively. M_{nc}, M_{nb} are nominal moment of the reinforced concrete column and beam, respectively; V_n is shear capacity of reinforced concrete section; In column, V_n is determined as the formulas in ATC-40 while in beam, V_n is calculated as the formulas in ACI 318; V_c is shear strength provided by concrete; V_s is shear strength provided by transverse reinforcement; f_c' is specified concrete cylinder strength; f_{yt} is expected yield strength of transverse steel; λ is equal to 0.75 for light-weight aggregate concrete and to 1 for normal-weight aggregate concrete (λ is set to 1 in this study); k is 1 in regions of low ductility demand and is 0 in region of high ductility demand in this research; d is effective depth of flexural component; s is longitudinal spacing of transverse reinforcement; b_w is web width. For ATC-40 and ACI318 code, the units of stress, force, area, length are MPa, N, mm^2 and mm, respectively.

For column (ATC-40) For beam (ACI 318)

$$V_n = V_c + V_s \qquad V_n = V_c + V_s \tag{3.1}$$

$$V_c = 0.29\lambda\left(k + \frac{N_u}{14A_g}\right)\left(\sqrt{f_c'}\right)b_c d \qquad V_c = \sqrt{f_c'}\,b_w d / 6 \tag{3.2}$$

$$V_s = \frac{A_v \cdot f_{yt} . d}{0.6s} \qquad V_s = \frac{A_v f_{yt} d}{s} \tag{3.3}$$

iii) Transverse steel index, $\rho_s \sqrt{b'' / s}$

This index derived from the experimental results is used as the indicator for the degree of concrete confinement and thus the ductility of the column (Park and Paulay, 1975). ρ_s is the volumetric ratio of transverse reinforcement and is calculated as the total volume of one layer of transverse reinforcement divided by volume of concrete having the cross-sectional area of a structural member measured center-to-center of transverse reinforcement and the height equal to transverse reinforcement spacing; b'' is a cross-sectional dimension of the column core measured center-to-center of confining reinforcement and perpendicular to the direction of applied force; s is longitudinal spacing of transverse reinforcement;

iv) Normalized associated shear force index, $\dfrac{V_a}{b_c d\sqrt{f_c'}}$

This index was specified in ATC-40 report "Seismic Evaluation and Retrofit of Concrete Buildings". It was used to indicate the level of curvature ductility of the member. The associated shear force is dependent on failure mode.

$$V_a = \frac{M_{nc}}{a_c},\ V_a = \frac{M_{nb}}{a_b} \qquad \text{For the expected flexural failure mode in column and beam, respectively} \tag{3.4}$$

$$V_a = V_c + V_s \qquad \text{For the expected shear fail in member} \tag{3.5}$$

v) Longitudinal reinforcing index, $\rho = \dfrac{A_s}{b_b d}$ and $\rho' = \dfrac{A_s'}{b_b d}$

This structural index belongs to beam only. Once the value is high, the joint shear force increases causing high possibility for joint shear failure to occur (Paulay and Priestley, 1992). A_s and A_s' are the total amount of bottom and top reinforcement in the section, respectively.

vi) Longitudinal reinforcing index, $\rho_t = \dfrac{A_t}{bd}$

This ρ_t is the column reinforcing index. A_t is the total amount of longitudinal reinforcement in the section. High value means possible high shear demand in joint core. In other words, it is cited that increasing index ρ_t results in high possibility that the joint will be failed in shear mode.

vii) Axial force ratio, $\dfrac{P}{f_c' A_g}$

This index is applied to column. Where A_g is gross concrete sectional area, mm^2; P is the gravity load (N) including dead load and likely live loads. Dead load can be taken as the calculated structure self-weight (without load factors) plus realistically estimated weight of flooring and wall of the building.

3.2.2 Beam-column joint indices

Beam-column joint indices are effectively used to predict the failure mode because beam-column joint is a critical member where many load transfers took place. The structural and geometry indices of beam-column joint consist of bond index (BI), column depth-to-bar diameter ratio (h_c / d_b), column width-to-beam width ratio (b_c / b_b), column depth-to-beam depth ratio (h_c / h_b), column flexural capacity-to-beam flexural capacity ratio (M_{nc} / M_{nb}), joint shear-to-joint shear strength ratio (V_{jh} / V_{jn}) and confinement joint reinforcement ratio ($\frac{\rho_{sv} f_{ys}}{f_c'}$). Each index is described as follows:

i) Bond Index (BI):

Kitayama et al.(1985) introduced bond index to assess the severity of bond stress in comparison with the bond strength. The higher the bond index is, the more severe the beam bar bond is. The beam bar bond index (BI) is defined by dividing the average bond stress by the square root of the concrete strength. The bond strength is assumed to be proportional to the square root of the concrete compressive strength. The index increases for higher beam bar strength, large diameter of beam bars, narrower column depth, and weaker concrete strength. The bond deterioration is more likely to occur for a higher index value.

$$BI = \frac{f_y d_b}{2h_c \sqrt{f_c'}} \tag{3.6}$$

Where d_b is longitudinal beam bar diameter; h_c is column depth; f_y is specified yield strength of reinforcement; f_c' is compressive strength of concrete.

ii) Column depth to bar diameter ratio, h_c / d_b:

This index was specified in international standard code for limiting the bond stress along the beam longitudinal bars within the joint. It should be noted that this index is usually used in the seismic design procedure rather than bond index (BI). These two parameters are similar to each other except the material properties are included in the definition of bond index.

iii) Beam to column width ratio (b_b / b_c):

This index was defined in the research of Pessiki et al.(1990). From the studied results of Kurose et al.(1988), it can be seen that the joint shear strength increases as the beam width relative to the column width increases.

iv) Beam to column depth ratio (h_b / h_c):

In general, a beam-column joint with deep beams relative to the column depth exhibit lower strengths than square joints. Kurose et al.,(1988) suggested that as the diagonal compression strut in the joint becomes steeper it also becomes less effective in resisting horizontal joint shear.

v) Joint confinement index ($\frac{\rho_{sv} f_{ys}}{f_c'}$):

This index was found in State of the Art Report of Comite Euro-International du Beton,(1996). It was used to study the role of hoops in the resisting mechanism of the joint, the relationship between the fractions of total shear resisted by the concrete core. ρ_{sv} is the volumetric ratio of joint reinforcement and f_{ys} is the yield strength of that reinforcement.

vi) Column to beam moment capacity (M_{nc} / M_{nb}):

This index indicates whether the plastic hinge will be formed in column or in beam. It is presented in the form of the ratio of nominal moment capacity of column to that of beam. M_{nc} is nominal moment capacity of column and M_{nb} is nominal moment capacity of beam.

vii) Joint shear force over joint shear capacity ratio (V_{jh} / V_{jn}):

The index indicates the possibility of joint shear failure. This index was used in ATC-40 to indicate the possibility of joint shear failure occurrence. V_{jh} is shear force transferring within joint region and is calculated by the following formulas (Paulay and Priestley, 1992).

$$V_{jh} = (1+\beta)\lambda_o f_y A_{s1} - V_{col} \tag{3.7}$$

$$V_{col} = 2\left(\frac{l_1}{l_{1n}} M_{0,1} + \frac{l_2}{l_{2n}} M_{0,2}\right) / \left(l_c + l_c^{'}\right) \tag{3.8}$$

$$\beta = A_{s1} / A_{s2} \tag{3.9}$$

Where A_{s1}, A_{s2} is area of top and bottom beam reinforcement, respectively, λ_o is over strength factor and is set to be 1.25, V_{col} is the column shear force, $M_{0,1}$ is negative moment capacity of the right beam, $M_{0,2}$ is positive moment capacity of the left beam, $l_1, l_{1n}, l_2, l_{2n}, l_c, l_c'$ are showed in Fig.3.2.

$$V_{jn} = 0.083\lambda\gamma\sqrt{f_c'}A_j \tag{3.10}$$

$$A_j = b_j h_c \tag{3.11}$$

Where V_{jn} is joint shear strength and is calculated by the formula in ATC-40; λ is equal to 0.75 or 1.0 for lightweight or normal weight aggregate concrete, respectively (λ is set to be 1.0), γ is shear strength factor reflecting confinement of joint by lateral members (γ is set to be 12), f_c' is specified compressive strength of concrete in the connection (MPa), A_j is effective horizontal joint area, b_j is effective width of joint transverse to the direction of shear (mm), h_c is full depth of column (mm), b_j is effective width (mm), the other notations are shown in Figs. 3.2 and 3.3.

3.2.3 Structural indices of existing building

Based on the collected 15 existing buildings constructed in Bangkok, the structural indices were collected. The connection in the first floor was selected to represent each building. One typical beam, column and beam-column joint in transverse and longitudinal direction were selected to construct structural indices data base. The summary of overall structural indices in each direction is shown in Tables 3.2, 3.3 and 3.4. The detail of structural indices is list in Appendix A.

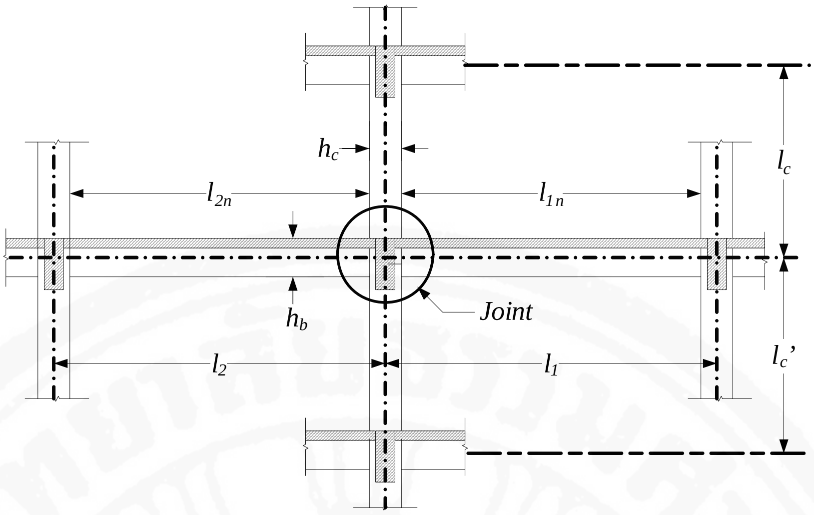

Fig. 3.2: Interior beam-column sub-assemblage

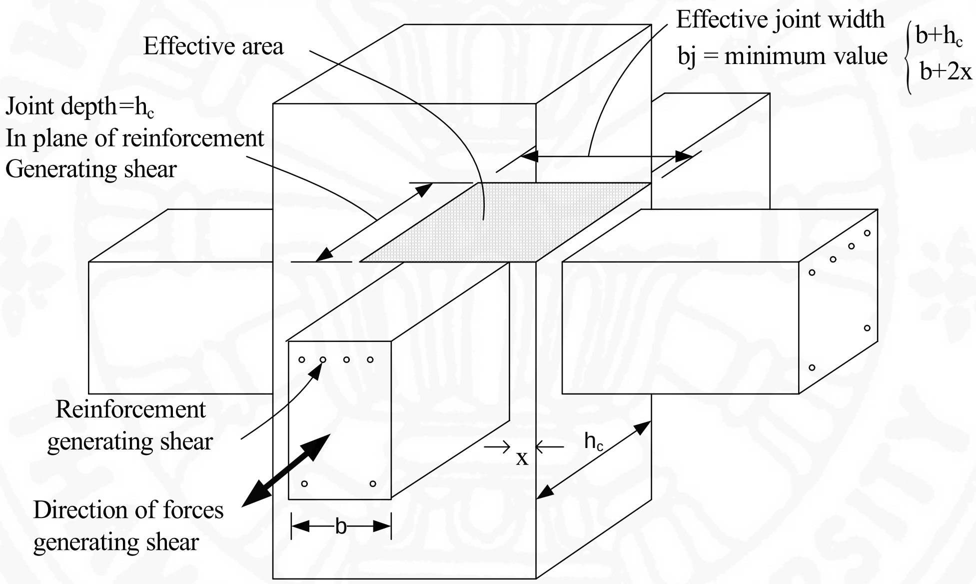

Fig. 3.3 Effective joint area

3.2.4 Analysis of structural indices data

The maximum, minimum, average and standard deviation of these indices are given in Table3.2-3.4. As shown in the table, there is a lot of scattering in collected data because of a great variety of actual buildings. However, the analysis based on the average of structural indices are as follow.

The average value of the shear span ratio (*a/h)* in column and beam was found to be around 4.08 to 5.76. This indicated that the type of failure should be somewhere between shear and flexural failure. In the study of Jaradat et al. (1998), columns with shear span ratio of 2.0 to 3.5 were tested. The result shows that columns with the shear span of 2.0 and longitudinal steel ratio of 0.020 failed by the sudden brittle failure mode.

The average value of Normalized yield flexural-to-shear strength ratio ($M_{nc}/(a_c V_n)$) is found to be around 0.22 – 0.56 in column and $M_{nb}/(aV_{nb})$ around 0.44-0.76 in beam. Both values are less than 1. Thus, from this index, flexural failure may occur in either column or beam.

Table 3.2 Structural indices for column

Direction/ Connection	Items	$\frac{a_c}{h_c}$	$\frac{M_{nc}}{a_c V_n}$	$\frac{P}{f_c' A_g}$	ρ_t	$\rho_s \sqrt{b''/s}$	$\frac{V_a}{b_c d \sqrt{f_c'}}$
Transverse Interior	Maximum	6.85	1.27	0.903	0.0982	0.0128	7.51
	Minimum	1.64	0.11	0.200	0.0135	0.0014	1.42
	Average	4.08	0.48	0.345	0.0477	0.0060	3.56
	Standard deviation	1.78	0.32	0.180	0.0253	0.0030	1.70
Transverse Exterior	Maximum	3.50	0.62	0.267	0.0287	0.0028	4.81
	Minimum	2.08	1.64	0.205	0.1909	0.0075	11.09
	Average	1.50	0.46	0.226	0.0129	0.0081	4.41
	Standard deviation	1.64	0.95	0.193	0.0310	0.0040	7.02
Longitudinal Interior	Maximum	9.89	1.51	0.482	0.0982	0.0143	3.78
	Minimum	2.06	0.22	0.191	0.0126	0.0021	1.41
	Average	5.73	0.38	0.278	0.0507	0.0077	2.30
	Standard deviation	2.01	0.33	0.091	0.0258	0.0034	0.72
Longitudinal Exterior	Maximum	9.89	1.18	0.447	0.0982	0.0133	7.95
	Minimum	2.53	0.16	0.041	0.0089	0.0019	0.96
	Average	5.76	0.37	0.210	0.0402	0.0065	2.29
	Standard deviation	2.02	0.27	0.121	0.0293	0.0034	1.72

Table 3.3 Structural indices for beam

Direction/ Connection	Items	$\frac{a_b}{h_b}$	$\frac{M_{nb}}{a_b V_n}$	ρ	ρ'	$\rho_s \sqrt{b''/s}$	$\frac{V_a}{b_b d \sqrt{f_c'}}$
Transverse Interior	Maximum	6.86	1.38	0.0297	0.0297	0.0099	3.76
	Minimum	1.63	0.29	0.0057	0.0046	0.0014	0.97
	Average	4.68	0.76	0.0134	0.0144	0.0044	2.13
	Standard deviation	1.87	0.40	0.0072	0.0079	0.0026	0.96
Transverse Exterior	Maximum	6.35	1.58	0.0214	0.0214	0.0099	3.67
	Minimum	1.55	0.43	0.0056	0.0039	0.0014	1.19
	Average	3.74	0.77	0.0107	0.0133	0.0042	2.21
	Standard deviation	1.74	0.37	0.0049	0.0056	0.0022	0.84
Longitudinal Interior	Maximum	7.13	0.86	0.0250	0.0250	0.0092	4.55
	Minimum	2.96	0.13	0.0032	0.0026	0.0013	0.68
	Average	4.25	0.44	0.0084	0.0095	0.0036	1.44
	Standard deviation	1.11	0.21	0.0056	0.0065	0.0028	1.00

Table 3.3 Structural indices for beam (continued)

Direction/ Connection	Items	$\frac{a_b}{h_b}$	$\frac{M_{nb}}{a_b V_n}$	ρ	ρ'	$\rho_s \sqrt{b''/s}$	$\frac{V_a}{b_b d \sqrt{f_c'}}$
Longitudinal Exterior	Maximum	7.25	1.25	0.0250	0.0250	0.0092	4.55
	Minimum	1.83	0.13	0.0043	0.0032	0.0013	0.86
	Average	4.00	0.59	0.0107	0.0104	0.0035	1.88
	Standard deviation	1.22	0.31	0.0060	0.0057	0.0021	1.06

Table 3.4 Structural indices for beam-column joint

Direction/ Connection	Items	BI	$\frac{h_c}{d_b}$	$\frac{b_b}{b_c}$	$\frac{h_b}{h_c}$	$\frac{M_{nc}}{M_{nb}}$	$\frac{V_{jh}}{V_{jn}}$	$\frac{\rho_{sv} f_{ys}}{f_c'}$
Transverse Interior	Maximum	8.61	35.00	1.00	1.75	4.28	4.08	0.000
	Minimum	2.77	15.00	0.38	0.63	0.64	0.22	0.000
	Average	5.17	23.86	0.72	1.17	2.00	1.33	0.000
	Standard deviation	1.59	6.88	0.21	0.38	1.29	0.96	0.000
Transverse Exterior	Maximum	10.76	40.00	1.00	2.00	26.45	2.54	0.000
	Minimum	2.59	12.00	0.38	0.50	0.47	0.06	0.000
	Average	5.43	24.29	0.74	1.23	6.30	0.80	0.000
	Standard deviation	2.20	9.69	0.19	0.50	6.84	0.60	0.000
Longitudinal Interior	Maximum	7.23	33.33	0.60	2.00	10.11	3.91	0.000
	Minimum	2.90	16.67	0.15	0.75	0.34	0.20	0.000
	Average	5.30	22.84	0.44	1.40	3.92	1.00	0.000
	Standard deviation	1.40	5.87	0.13	0.36	2.95	0.91	0.000
Longitudinal Exterior	Maximum	8.28	33.33	0.67	2.00	20.60	1.95	0.000
	Minimum	2.90	12.50	0.15	0.63	0.73	0.10	0.000
	Average	5.67	21.47	0.43	1.37	6.83	0.55	0.000
	Standard deviation	1.57	5.75	0.15	0.44	6.27	0.57	0.000

By observing axial load ratio ($\frac{P}{f_c' A_g}$), the mean value is found to be around 0.21-0.345. It can be implied that the tributary area has a little effect on this index because sectional area and axial load are proportional to the tributary area.

Inspection of transverse reinforcement ratio ($\rho_s \sqrt{b''/s}$) indicated that columns and beams have a very low confinement level. The average value of transverse reinforcement is found to be around 0.0058-0.0065 in column and around 0.0035-0.0044 in beam. It is believed that the columns and beams in Thailand have low level of confinement, which results in low ductility level and high possibility to fail by shear failure.

The average value of longitudinal reinforcing index in column ρ_t is around 0.0402-0.0521. This index is significantly high so that the interior column has high possibility of shear failure to occur.

The average value of longitudinal reinforcing index ρ and ρ' are found to be around 0.0084-0.0134 and 0.0095-0.0144, respectively. This index is significantly high if the column posses a small tributary area. This could be explained from the fact that the sectional area of beam decreases while the reinforcement steel area is the same. Therefore, there is high possibility of joint shear failure occurrence in this group of building.

The average value of normalized associated shear force index ($\dfrac{V_a}{b_w \cdot d \cdot \sqrt{f_c^{'}}}$) is around 2.29-4.05 in column and around 1.44-2.21 in beam. From Table 9-7 in ATC-40, this index was classified into three levels: less than 3, between 3 to 6, and more than 6. The average value of this index for columns falls in the range of less than 3 and between 3 to 6 while in case of beam, the values are less than 3. This indicated that columns have moderate level of ductility and beams have high level of ductility.

The average value of Bond Index (BI) is around 5.17-5.67. It can be concluded that high possibility of occurring bond deterioration in beam-column joints under seismic loadings.

By observing the ratio (h_c / d_b) the mean value is around 21.47-24.29. This value satisfies ACI 318-05 (h_c / d_b>20) while it does not meet the requirement of New Zealand standard.

The average value of joint shear stress over joint shear strength ratio (V_{jh} / V_{jn}) is around 1.0-1.33 for interior connection and 0.55-0.80 for exterior connection. Based on this index, interior beam-column joints have high possibility to fail by joint shear failure while exterior beam-column joints have high possibility to fail by beam flexural failure.

The average value of column to beam moment capacity $\dfrac{M_{nc}}{M_{nb}}$ is around 2.0-6.83. This illustrates that columns are stronger than beam.

The value of joint confinement index ($\dfrac{\rho_{sv} f_{ys}}{f_c^{'}}$) is zero in all buildings. It can be seen that there are no transverse reinforcement within the joint area for all building. It is well known that the truss mechanism can work well only when transverse reinforcement exists in joint and good bond condition in beam bars passing through the joint, hence joint shear strength is mainly consisted of strut mechanism. Therefore, there is high possibility that brittle shear failure will occur in joint under seismic action.

It can be seen that some structural indices indicate compatible behavior but some indices are opposed. Therefore, the methodology of seismic evaluation by using these indices should be developed and tested in laboratory in order to confirm the method.

3.3 Evaluation methodology

The proposed evaluation methodology consists of linear static analysis of structures in order to obtain demand capacity ratio (DCR), reinforcement detailing check and flowcharts for failure mode investigation.

3.3.1 DCR determination

To obtain demand capacity ratio (DCR), the linear static analysis is conducted and the seismic demand and capacity are calculated and compared. Seismic demands include shear force and moment in beam, column and joint which are calculated from analysis of structures under the action of earthquake loading specified in governing code. In Thailand, the No.49 Ministerial Law based on UBC (1985) is adopted. Other more recent codes can be used as well. Capacities are calculated based on accepted design codes, such as ATC-40 and ACI318 (1999). For each building, the analysis should be conducted for both transverse and longitudinal directions. The procedure to evaluate DCR is described below,

i) Approximate weight (W) of building, including likely live load, for example 40% of specified live load.

ii) Calculate base shear force based on No.49 Ministerial law by the following formula,

$$V=ZIKCSW \tag{3.12}$$

Where V = base shear force

Z= Zone factor

I = Importance factor

K = Horizontal force factor

C = Coefficient of building natural frequency

S = Soil factor

iii) Distribute base shear V to each frame based on relative stiffness. For each frame, distribute lateral forces along building height.

iv) Model and analyze the structure by the computer to determine moment and shear forces in beam, column and joint. There are two load cases in the analysis.

Load combination 1:

$$U\ 1 = 0.75(1.4DL+1.7LL\pm1.87E) \tag{3.13}$$

Load combination 2:

$$U\ 2 = DL+0.4LL\pm1.87E \tag{3.14}$$

Load combination 1 is stated in the ACI318 (1999) building code. This load shall be used to check DCR compatibility with forces specified in the No.49 Ministerial law. Load combination 2 is considered to represent the more realistic situation under earthquake where actual live load is assumed to be 40%. Shear, moment and axial force obtained from this load combination will be used to examine the possible failure modes.

v) Calculate corresponding capacity of beam, column and beam-column joint based on ACI318 (1999) seismic requirement for Intermediate Moment Resisting Frame (IMRF) and ATC-40.

vi) Compare existing reinforcement detailing in beam, column and beam-column joint with ACI requirement for Intermediate Moment Resisting Frame (IMRF).

vii) Calculate Demand Capacity Ratio (DCR). Failure is considered to take place when DCR is greater than 1.0. The values of DCR used in this method consist of,

$\frac{M_{ub}^{-}}{M_{nb}^{-}}$ where M_{nb}^{-} is negative moment capacity of beam and M_{ub}^{-} is negative moment demand of beam.

$\frac{V_{ub}}{V_{nb}}$ where V_{nb} is beam shear capacity of section and V_{ub} is shear force demand of beam.

$\frac{V_{ju}}{V_{jn}}$ where V_{jn} is joint shear capacity of beam-column joint, V_{ju} is joint shear force demand from associated moment. V_{ju} can be calculated from equation (3.15)

$$V_{ju} = (M_{ub}^{+} / jd + M_{ub}^{-} / jd) - V_{col} \tag{3.15}$$

$\frac{V_{uc}}{V_{nc}}$ where V_{nc} is column shear capacity of section and V_{uc} is shear force demand of column.

$\frac{M_{uc}}{M_{nc}}$ where M_{nc} is moment capacity of column and M_{uc} is moment demand of column.

The meaning of subscription *u* is based on No.49 Ministerial Law demand, subscription *n* represents capacity of each member and subscriptions *c*, *b* and *j* indicate location of members, column, beam and joint, respectively.

3.3.2 Reinforcement detailing check

The DCR determination is a check for load level specified in the code. Usually, the load level is lower than that actually occurs and the structure is assumed to demonstrate some reversed yielding behavior. As a result, all modern codes of earthquake design states the minimum requirement for reinforcement detailing to provide a certain level of ductility in structural members. In Thailand which is considered to be low to moderate seismic zone, the ACI (1999) requirement for Intermediate Moment Resisting Frame (IMRF) is adopted. The requirement for reinforcement detailing is shown in Figure 3.4 for beam, column and joint.

3.3.3 Acceptance criteria

A building is considered seismically acceptable if both of the following two conditions are satisfied.

i) Acceptance for force criteria

All critical elements of lateral force resisting elements have strengths greater than computed actions, that is, DCR is less than 1. This represents the strength check under code-specified load level.

ii) Acceptance for detailing criteria

All reinforcement detailing satisfies the code requirement. The detailing check is intended to check ductility and energy dissipation capacity of critical members of the building.

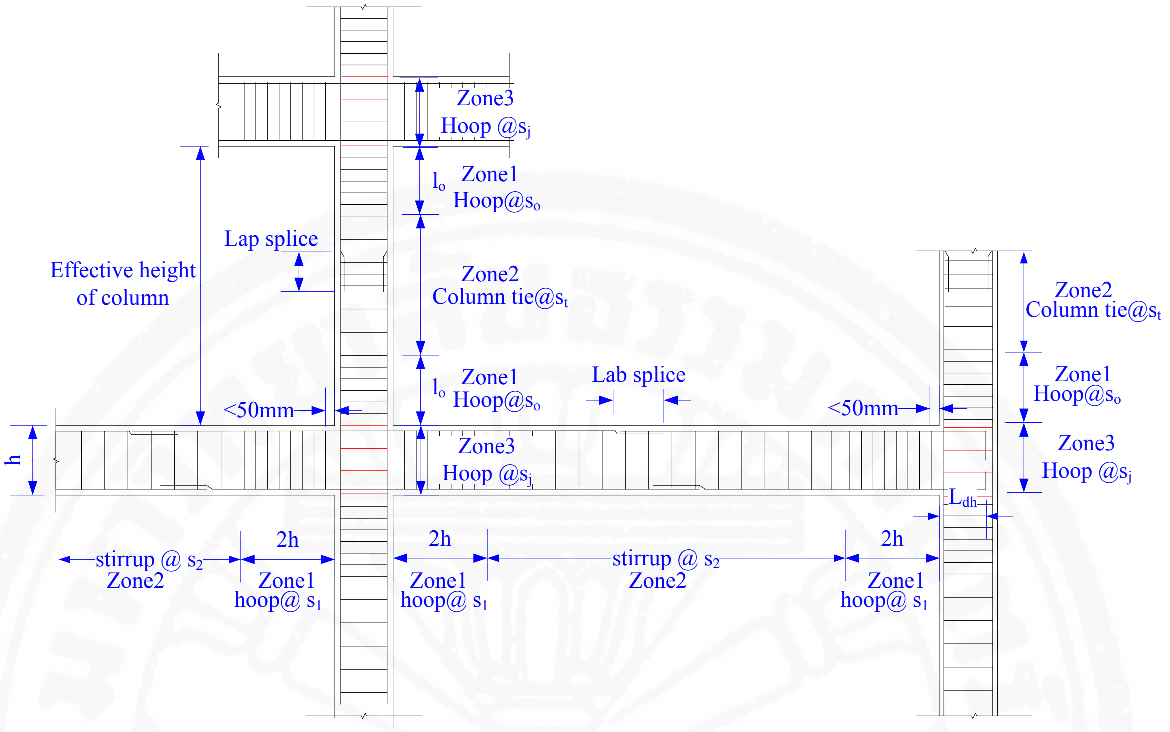

Fig. 3.4 Detailing criteria for beam, column and joint.

3.3.4 Results of DCR

The results of DCR comparison are shown in Fig.3.5 to 3.9. Summary of DCR check is shown in Table 3.5.

DCR for moment in beam (Fig.3.5) shows that 9 out of 15 buildings have DCR greater than 1. When investigating structural indices of 9 buildings, it is found that there are high value of shear span ratio (a/h) and low value of nominal moment capacity to nominal shear capacity ratio $(M_n/(aV_n))$ in transverse direction. Therefore, these 9 buildings have high possibility to fail by beam flexural failure. Another reason is because of the slab system using precast concrete slab spanning in the longitudinal direction of building, thus making the beam much stronger in transverse than in longitudinal direction.

DCR for shear in beam (Fig.3.6) shows that 2 buildings in transverse AC4 and CH3 do not pass the criteria. And 8 buildings in longitudinal direction AP3, AC3, AC4, CH2, CH3, HP1 and HP3 do not pass the criteria too because of the same reason as for moment.

As for DCR for moment in column (Fig.3.7), almost all building pass this criteria except building AC4, CH2 and CH3.

As for DCR for shear in column (Fig.3.8), all sections of the column have DCR less than 0.5 indicating that the sections have sufficient shear capacity to resist code-specified earthquake load. There are two reasons to support this, that is, the presence of lift core that reduces the lateral load transmitted to column and the traditional working stress design approach with low code-specified allowable material stresses.

The DCR for shear in beam-column joints is shown in Fig.3.9. As shown, almost all building pass this criterion because column has large size as explained previously. Except 4 buildings do not pass this criteria thus it is possible that brittle joint shear failure might occur in these buildings.

It is found that almost critical elements of lateral force resisting system have strengths greater than computed actions based on code specified earthquake load. However, it should not be concluded that the structure is safe against earthquake. As a matter of fact, the force that can develop in a structure depends on the structural capacity itself. Hence, the structure should be investigated under the condition that some members reach yielding. Moreover, the reinforcement detailing should be checked too.

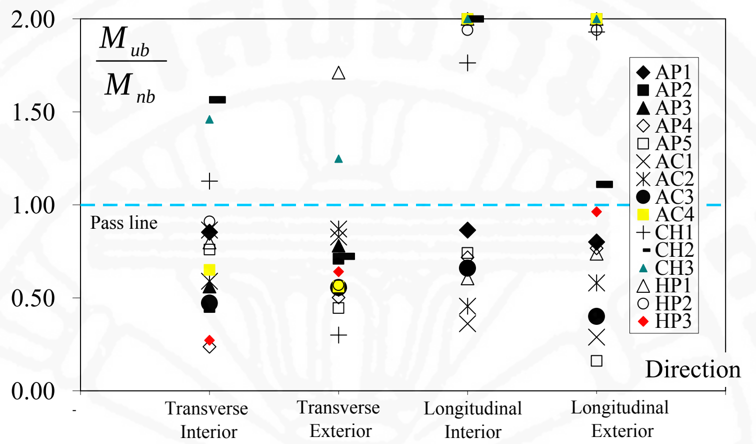

Fig. 3.5 DCR for moment in beam

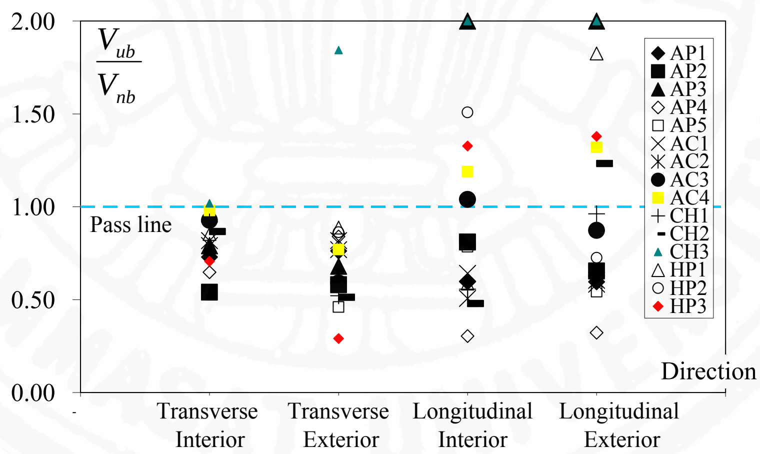

Fig. 3.6 DCR for shear in beam

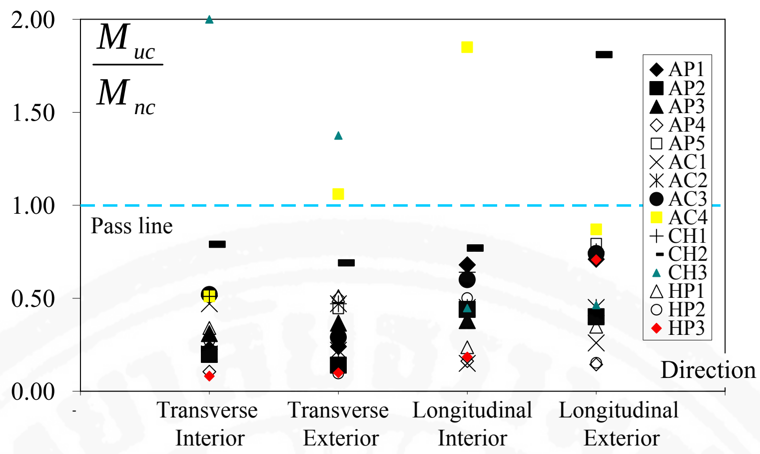

Fig. 3.7 DCR for moment in column

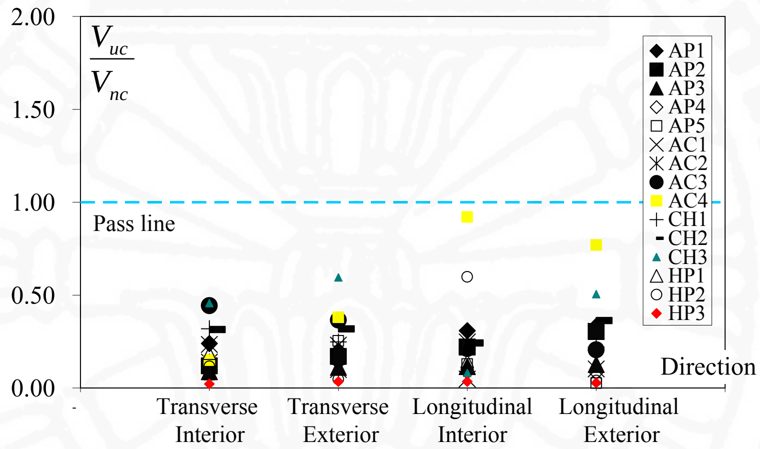

Fig. 3.8 DCR for shear in column

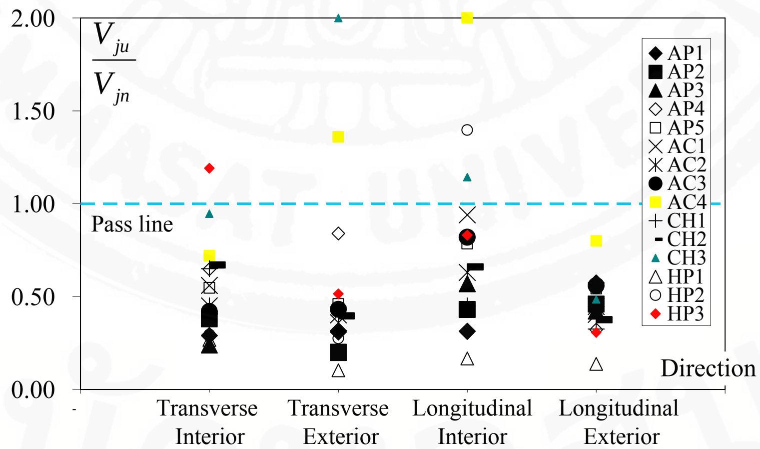

Fig. 3.9 DCR for shear in beam-column joint

Table 3.5 Summary of DCR check

Building	Transverse direction		Longitudinal direction	
	Interior span	Exterior span	Interior span	Exterior span
AP1	C	C	C	C
AP2	C	C	NC	NC
AP3	C	C	NC	NC
AP4	C	C	C	C
AP5	C	C	C	C
AC1	C	C	C	C
AC2	C	C	C	C
AC3	C	C	NC	C
AC4	C	NC	NC	NC
CH1	NC	C	NC	NC
CH2	NC	C	NC	NC
CH3	NC	NC	NC	NC
HP1	C	NC	C	NC
HP2	C	C	NC	NC
HP3	NC	C	NC	NC

Note: C=Compliance, NC=Not compliance

3.3.5 Result of reinforcement detailing check

The results of reinforcement detailing check show that all buildings do not satisfy the reinforcement detailing. All present transverse steels are simply tied without end details as seismic hook. There are not enough transverse steels in beam and column, especially in plastic hinge zone (zone1). Moreover, there are no transverse steels in beam-column joint in all buildings. This means that all investigated buildings lack ductility. An example of detailing check list is shown in Table 3.6 and another detailing check list is shown in Appendix A.

Table 3.6 Example of Detailing criteria for transverse direction-interior span.

Building	Location	Transverse steel	Existing	Minimum requirement	Results
AP1	Beam	Zone 1 ($2h_o$)	RB9 @ 0.20	RB9 @ 0.15	NC
		Zone 2	RB9 @ 0.20	RB9 @ 0.16	NC
	Column	Zone 1 (s_o)	3-RB9@0.20	3-RB9 @ 0.20	C
		Zone 2 (s_t)	3-RB9@0.20	3-RB9 @ 0.40	C
	Joint	Zone 3 (s_j)	None	RB9 @ 0.40	NC
AP2	Beam	Zone 1 ($2h_o$)	RB6 @ 0.10	RB6 @ 0.04	NC
		Zone 2	RB6 @ 0.10	RB6 @ 0.05	NC
	Column	Zone 1 (s_o)	3-RB6 @ 0.20	3-RB6 @ 0.20	NC
		Zone 2 (s_t)	3-RB6 @ 0.20	3-RB6 @ 0.20	C
	Joint	Zone 3 (s_j)	None	3-RB6 @ 0.20	NC

Note: C=Compliance, NC=Not compliance

Table 3.6 Example of Detailing criteria for transverse direction-interior span (continue).

Building	Location	Transverse steel	Existing	Minimum requirement	Results
AP3	Beam	Zone 1 ($2h_o$)	1-RB 9 @ 0.20	1-RB 9 @ 0.09	NC
		Zone 2	1-RB 9 @ 0.20	1-RB 9 @ 0.09	NC
	Column	Zone 1 (s_o)	3-RB 9 @ 0.20	3-RB 9 @ 0.16	NC
		Zone 2 (s_t)	3-RB 9 @ 0.20	3-RB 9 @ 0.32	C
	Joint	Zone 3 (s_j)	none	3-RB 9 @ 0.32	NC
AP4	Beam	Zone 1 ($2h_o$)	1-RB 6 @ 0.20	1-RB 6 @ 0.04	NC
		Zone 2	1-RB 6 @ 0.20	1-RB 6 @ 0.05	NC
	Column	Zone 1 (s_o)	3-RB 6 @ 0.20	3-RB 6 @ 0.14	NC
		Zone 2 (s_t)	3-RB 6 @ 0.20	3-RB 6 @ 0.28	C
	Joint	Zone 3 (s_j)	none	3-RB 6 @ 0.28	NC
AP5	Beam	Zone 1 ($2h_o$)	1-RB 9 @ 0.15	1-RB 9 @ 0.11	NC
		Zone 2	1-RB 9 @ 0.15	1-RB 9 @ 0.17	C
	Column	Zone 1 (s_o)	2-RB 9 @ 0.20	2-RB 9 @ 0.15	NC
		Zone 2 (s_t)	2-RB 9 @ 0.20	2-RB 9 @ 0.30	C
	Joint	Zone 3 (s_j)	none	2-RB 9 @ 0.30	NC
AC1	Beam	Zone 1 ($2h_o$)	RB 9 @ 0.20	1-RB 9 @ 0.15	NC
		Zone 2	RB 9 @ 0.20	1-RB 9 @ 0.16	NC
	Column	Zone 1 (s_o)	3-RB 9 @ 0.20	3-RB 9 @ 0.21	C
		Zone 2 (s_t)	3-RB 9 @ 0.20	3-RB 9 @ 0.43	C
	Joint	Zone 3 (s_j)	none	3-RB 9 @ 0.43	NC
AC2	Beam	Zone 1 ($2h_o$)	1-RB 9 @ 0.125	1-RB 9 @ 0.09	NC
		Zone 2	1-RB 9 @ 0.125	1-RB 9 @ 0.09	NC
	Column	Zone 1 (s_o)	4-RB 9 @ 0.20	4-RB 9 @ 0.20	C
		Zone 2 (s_t)	4-RB 9 @ 0.20	4-RB 9 @ 0.40	C
	Joint	Zone 3 (s_j)	none	4-RB 9 @ 0.40	NC
AC3	Beam	Zone 1 ($2h_o$)	1-RB 9 @ 0.125	2-RB 9 @ 0.09	NC
		Zone 2	1-RB 9 @ 0.125	2-RB 9 @ 0.09	NC
	Column	Zone 1 (s_o)	3-RB 9 @ 0.30	3-RB 9 @ 0.06	NC
		Zone 2 (s_t)	3-RB 9 @ 0.30	3-RB 9 @ 0.06	NC
	Joint	Zone 3 (s_j)	none	3-RB 9 @ 0.06	NC
AC4	Beam	Zone 1 ($2h_o$)	1-RB6@0.10	2-RB6@0.06	NC
		Zone 2	1-RB6@0.10	2-RB6@0.06	NC
	Column	Zone 1 (s_o)	2-RB6@0.15	2-RB6@0.14	NC
		Zone 2 (s_t)	2-RB6@0.15	2-RB6@0.25	C
	Joint	Zone 3 (s_j)	None	2-RB6@0.25	NC

Note: C=Compliance, NC=Not compliance

Table 3.6 Example of Detailing criteria for transverse direction-interior span (continue).

Building	Location	Transverse steel	Existing	Minimum requirement	Results
CH1	Beam	Zone 1 ($2h_o$)	RB 9 @ 0.20	1-RB 9 @ 0.10	NC
		Zone 2	RB 9 @ 0.20	1-RB 9 @ 0.10	NC
	Column	Zone 1 (s_o)	2-RB 9 @ 0.30	2-RB 9 @ 0.20	NC
		Zone 2 (s_t)	2-RB 9 @ 0.30	2-RB 9 @ 0.27	NC
	Joint	Zone 3 (s_j)	None	2-RB 9 @ 0.27	NC
CH2	Beam	Zone 1 ($2h_o$)	1-RB 9 @ 0.15	1-RB 9 @ 0.10	NC
		Zone 2	1-RB 9 @ 0.15	1-RB 9 @ 0.10	NC
	Column	Zone 1 (s_o)	2-RB 9 @ 0.30	2-RB 9 @ 0.20	NC
		Zone 2 (s_t)	2-RB 9 @ 0.30	2-RB 9 @ 0.40	NC
	Joint	Zone 3 (s_j)	None	2-RB 9 @ 0.40	NC
CH3	Beam	Zone 1 ($2h_o$)	1-RB 9 @ 0.20	1-RB 9 @ 0.11	NC
		Zone 2	1-RB 9 @ 0.20	1-RB 9 @ 0.11	NC
	Column	Zone 1 (s_o)	3-RB 9 @ 0.30	3-RB 9 @ 0.20	NC
		Zone 2 (s_t)	3-RB 9 @ 0.30	3-RB 9 @ 0.40	C
	Joint	Zone 3 (s_j)	None	3-RB 9 @ 0.40	NC
HP1	Beam	Zone 1 (2ho)	1-RB 9 @ 0.15	1-RB 9 @ 0.14	NC
		Zone 2	1-RB 9 @ 0.15	1-RB 9 @ 0.15	C
	Column	Zone 1 (so)	3-RB 9 @ 0.30	3-RB 9 @ 0.21	NC
		Zone 2 (st)	3-RB 9 @ 0.30	3-RB 9 @ 0.32	C
	Joint	Zone 3 (sj)	None	3-RB 9 @ 0.32	NC
HP2	Beam	Zone 1 (2ho)	1-RB 6 @ 0.15	1-RB 6 @ 0.08	NC
		Zone 2	1-RB 6 @ 0.15	1-RB 6 @ 0.09	C
	Column	Zone 1 (so)	3-RB 9 @ 0.40	3-RB 9 @ 0.21	NC
		Zone 2 (st)	3-RB 9 @ 0.40	3-RB 9 @ 0.43	C
	Joint	Zone 3 (sj)	None	3-RB 9 @ 0.43	NC
HP3	Beam	Zone 1 (2ho)	2-RB 9 @ 0.15	2-RB 9 @ 0.09	NC
		Zone 2	2-RB 9 @ 0.15	2-RB 9 @ 0.09	NC
	Column	Zone 1 (so)	4-RB 6 @ 0.15	4-RB 6 @ 0.13	NC
		Zone 2 (st)	4-RB 6 @ 0.15	4-RB 6 @ 0.13	NC
	Joint	Zone 3 (sj)	None	4-RB 6 @ 0.13	NC

Note: C=Compliance, NC=Not compliance

3.4 Investigation of failure mode

The DCR analysis presented above relies on the force specified in No.49 Ministerial law. This force may or may not occur in a real earthquake since the actual forces developed in a structure depend on its capacity. Hence, using the force level specified in the code for the evaluation of existing structures may not be fully rational. A more meaningful approach is to determine the possible failure modes when the structure is displaced until yielding takes place in some members of the structure. The staged failure mode is very important to the building retrofit. For example, when flexural DCR exceeds 1.0, it may simply mean that the member yields without failure. The retrofit for flexural

DCR exceeding 1.0 may not be important as long as the member can yield with some ductility. The secondary failure mode such as beam or column shear failure after yielding and post-yield joint shear failure is more significant. In this respect, this research presents two flowcharts for identifying the failure modes of the structure.

3.4.1 Load flowchart.

The load flowchart is for checking the possible failure modes under code-specified lateral load. It is applied with load combination 2 (U= DL+0.4LL$\pm$1.87E) with likely live load acting on the structure. The load flowchart is shown in Fig. 3.10.

3.4.2 Yielding flowchart

This yielding flowchart is intended for a situation when earthquake motion moves the structure until yielding develops in some members of the structure. This allows an opportunity to investigate staged failure modes. The yielding flowchart is shown in Fig. 3.11.

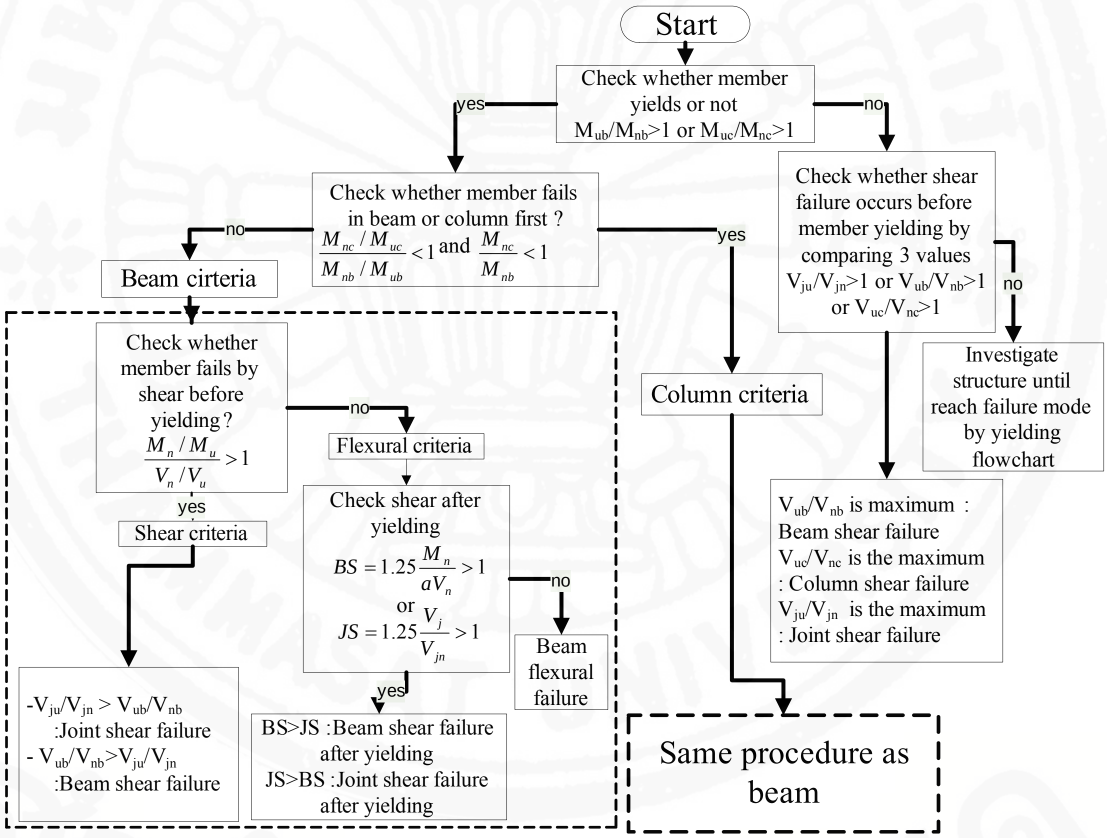

Fig 3.10 Load flowchart

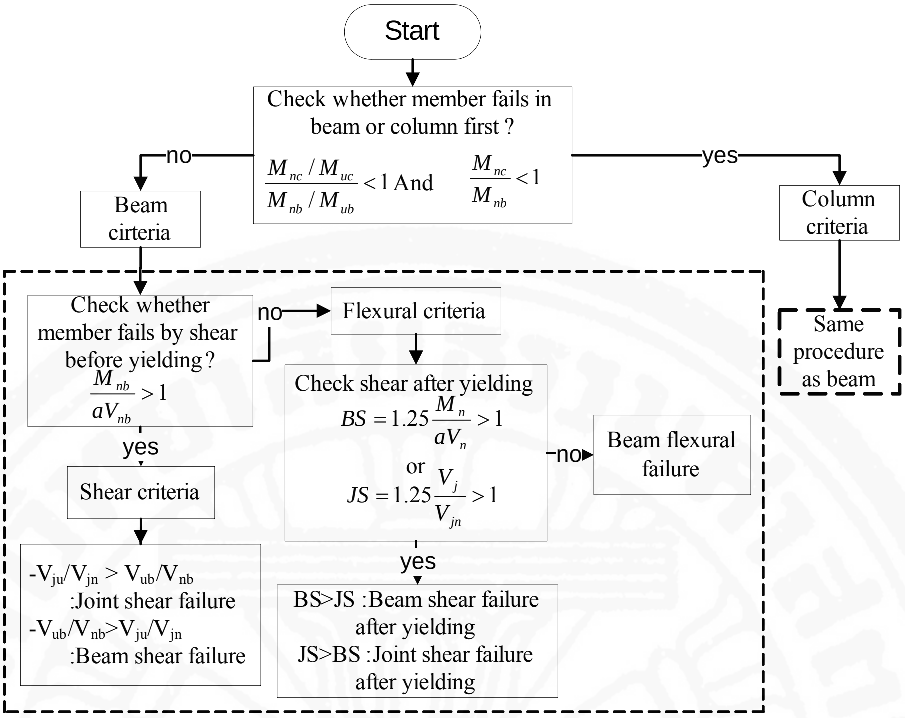

Fig.3.11 Yielding flowchart

3.4.3 Failure mode result

All 15 buildings are examined with the proposed flowcharts. The result shows 41.7% of beam flexural failure, 38.3% of joint shear failure and 20% of beam shear failure. It is seen that shear failure covers 58.3% of the total failure (Fig 3.12). The total failures can be classified as follows,

In transverse direction and interior span, 53.4% is joint shear failure, 33.4% is beam shear failure and 13.3% is beam flexural failure (Fig.3.12).

In transverse direction and exterior span, 46.7% is beam flexural failure and 33.3% is beam shear failure and 20% is joint shear failure (Fig 3.12).

In longitudinal direction and interior span, 60% is joint shear failure and 40% is beam flexural failure (Fig.3.12).

In longitudinal direction and exterior span, 66.7% is beam flexural failure and 20% is joint shear failure and 13.3% is beam shear failure (Fig.3.12).

According to the investigation, the majority of failure is found in beams. However, as mentioned, the beam flexural failure is rarely a problem as long as ductility is available. It is also noted that the evaluation with load flowchart shows less failures compared with yielding flowchart. This is due to traditional RC working stress design approach adopted in Thailand with lower allowable compressive strength of concrete and steel compared with ACI (1999) codes. Another reason is the presence of lift core that reduces the forces transmitted to beam-column frame.

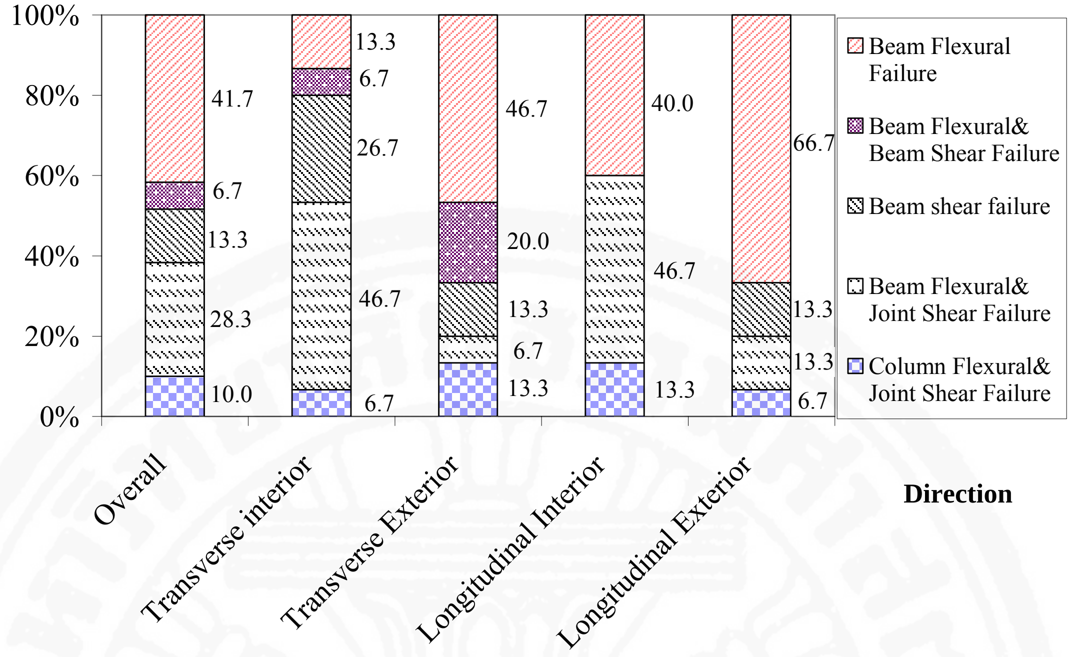

Fig. 3.12 Failure mode result

3.5 Conclusion

This chapter presents result of seismic assessment of 15 existing reinforced concrete buildings in Bangkok. All buildings were constructed as beam-column rigid frame. The building category covers school, apartments and governmental offices. The evaluation method consists of DCR determination, reinforcement detailing check and failure mode investigation. It is found that no buildings satisfy the reinforcement detailing requirement, primarily because of the lack of sufficient transverse reinforcement in beam, column and beam-column joint and non-seismic detail of hook anchorage. Based on DCR, 10 out of 15 buildings show some failures in the members under code-specified earthquake load. The failure mode investigation shows 41.7% of beam flexural failure, 38.3% of joint shear failure and 20% of beam shear failure. The load flowchart failure check is less critical than yielding flowchart because of traditional working stress design approach for RC structures in Thailand with lower allowable compressive strength of concrete and steel compared to codes. Another reason is the presence of lift core that reduces the forces transmitted to beam-column frame. According to the investigation, the joint shear failure is identified to be one of the most critical failures of RC buildings and need to be retrofitted.