CHAPTER 3

SEISMIC INVESTIGATION OF EXISTING BUILDINGS IN LOW TO MODERATE SEISMIC ZONES

South East Asian countries such as Singapore, Malaysia and Thailand are located in regions of low to moderate seismic zones. In the past, the reinforced concrete buildings in Thailand are traditionally constructed according to the American Concrete building code (ACI) while the British code(BS) is adopted in Singapore and Malaysia. Both design concepts are based mainly on gravity load (Li, et al, 2002). By ignoring earthquake load and considering wind load in the design, the lateral strength of the buildings may not be enough to sustain seismic in long term (Kunnath, et al, 1995).

3.1 Characteristic assessment of non-seismically detailed buildings

According to the recent assessment program of existing reinforced concrete buildings in Thailand, the database of 17 mid-rise buildings in Bangkok was gathered by Chaimahawan (2006). All buildings were reinforced concrete beam-column frame without shear wall and had 5-21 stories. Types of buildings covered essential facilities including university, school, apartment, governmental office and hospital as shown in Table A-1 of Appendix A. These buildings were designed according to the ACI building code for gravity load but the seismic consideration is not taken into account in building design.

From the examination of 17 existing buildings, many buildings had column stronger than beam, complying with the strong column weak beam requirement despite no capacity design was applied in the design of these buildings. A possible reason is that these old buildings were designed based on working stress design concept. As a result, several buildings had sufficient capacity to resist the earthquake load specified in the code though the earthquake is entirely ignored in the design

Nevertheless, from the investigation, most of the assessed buildings lack seismic reinforcement details. It was found that none of them had stirrups in the joint. The column longitudinal bars were usually spliced just above the joint with lap length of 350 mm. The column ties and beam stirrups is widely-spaced in potential plastic hinge such as the column end or the beam end; the end hooks are non-seismic. In addition, it is possible that the buildings perform non-ductile behavior, the size of column traditionally designed was quite small. Since the column size contributes to the determination of joint dimensions, it also has influence on joint shear strength. If the column size is too small, it will result in the premature shear failure on the beam-column joint. The ductile behavior is necessary when the structure encounters the reversed cyclic loading.

The characteristic assessment is supposedly applicable not only to buildings in Thailand, but also to buildings designed and detailed according to the ACI code for gravity load only in general. Fig 3.1 shows some deficiencies of RC building in Thailand.

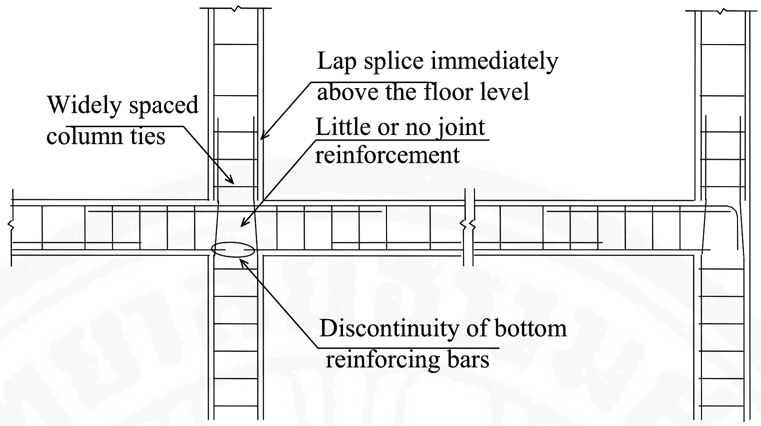

Fig.3.1 Typical non-seismic reinforcing detail of RC frame designed for gravity load only

3.2 Problem of shear force within interior beam-column joint

It is generally known that when earthquake occurs, the beam-column building frames are shaken in the same magnitude as that of earthquake as if the buildings are simulated by lateral load. The lateral load is transferred to column and causes bending moment and shear in both beam and column. Moreover, it either creates both horizontal and vertical shear forces in the joint region as shown in Fig. 2.1 or generates the results of shear force in the form of diagonal strut force as shown in Fig 2.2. From many previous investigations, the joint shear force is found to be several times greater than shear forces in beams and columns (Paulay and Priesley,1992). On this reason, the beam-column joint is one of the most critical components in the lateral load path of a frame (Hakuto,1999; Paulay et al.,1978).

In Thailand, the design of reinforced concrete frame buildings normally considers the gravity load only according to the ACI building code. Hence, the building design and reinforcement details do not meet the modern seismic provision, i.e. New Zealand, U.S., Japan, and Europe. The detailing reinforcements in potential plastic hinge regions specified in modern seismic provisions are also aimed at making certain that the transverse reinforcement is sufficient to ensure adequate ductility and shear strength. Besides lacking of seismic ductile reinforcing details, the quite small size of column traditionally designed will result in shear stress concentration on the joint region. There are chances that the building possibly fails under shear mode. The premature shear failure of the beam-column joint could be brittle and sudden, thus joint shear failure mode is undesirable in the aspect of seismic behavior.

It can be stated that if the beam-column joint in conventional buildings lacks confining hoop then the column should be designed to satisfy weak beam-strong column requirement. When the column size is designed larger, the joint shear stress will be distributed less. Thus, it is possible that the buildings are safe when the earthquake excited.

3.3 Problem of bond deterioration on beam-column joint

Aside from the problem of shear force, the bar anchorage within beam-column joint is another significant problem. It is possible for some bar, to slip during seismic loadings. When buildings are subjected to lateral load, the beam-column joints carry the unbalanced moment derived from the column shear force. The unbalanced moment needs bond stress between longitudinal steel and concrete within joint core to develop the capacity of beam and column. Other than that, the bar anchorage affects the capacity of joint as well.

Bar anchorage within joint core has an significant influence such that if the bar slips, the bond stress of longitudinal bar will deteriorate. Under unbalanced moment, the compressive reinforcing bar on one side may actually change to tensile. As a result, the concrete compressive force is increased to equilibrate the change of tensile reinforcing bar (Shiohara,2001;Hakuto,1999;Leon,1990). Consequently, the joint shear force is transferred increasingly through a diagonal strut force. It can be stated that the bond deterioration accelerates the crushing of diagonal concrete strut which determines the joint shear failure. When the slippage of longitudinal bar anchorage happens, strength, stiffness and hysteresis behavior (ductility and energy dissipation) of beam-column frame derived from the contribution of the capacity of three components- beam, column and joint are reduced significantly to failure.

From the investigation of existing beam-column frame buildings, many of them agree with the weak beam-strong column requirement though their design criteria do not consider the seismic loadings. Nonetheless, the column size of typical existing buildings is quite small, resulting in limited bond anchorage within the column size.

On this reason, the modern seismic provision, i.e. New Zealand, U.S., Japan, and Europe recommend the adequate anchorage of longitudinal bars passing through or terminating in the joint(Hakuto,1999). One of the recommendations is the provision of minimum column depth to bar diameter ratio (h_c/d_b) to avoid common failure modes pertaining to the joint of interior beam-column connection. Joint shear and/or joint bond failure is considered undesirable since both lead to degradation of strength and stiffness of the frame (Paulay et al,1978). Furthermore, both failures are contributed to the inhabitation of weak beam-strong column mechanism.

3.4 Structural indices of RC beam-column frame on seismic performance

To characterize the seismic performances, an assessment of reinforced concrete frame was conducted. Chaimahawan and Pimanmas (2006) proposed a simple seismic evaluation methodology for RC beam-column frame which can assess the failure mode of 17 existing reinforced concrete buildings in Thailand well by using structural index as significant parameters. The structural indices of each component (beam, column and joint) indicate the behavior of each component under seismic action. Additionally, they can be used to model each component of actual building frame. Structural indices of buildings are calculated from their configurations such as sectional dimensions, total area of longitudinal and transverse reinforcement bars, strength of concrete and reinforcement steel, and etc. The structural and geometry indices for beam, column and beam-column joint were collected by Warnitchai at el(2004) as follows:

3.4.1 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 a direction parallel to applied force, respectively as shown in Fig 3.2.

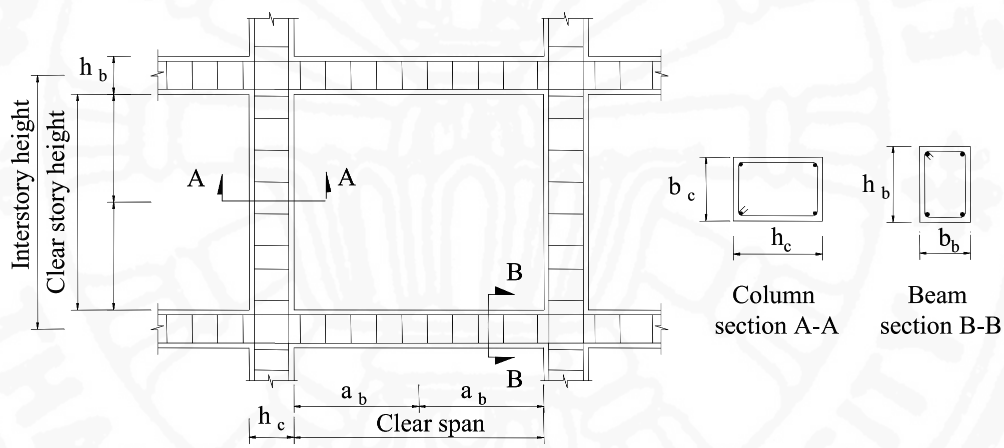

Fig 3.2 Geometries of beam-column frame

ii) Nominal moment capacity to nominal shear capacity ratio ($M_n/a.V_n$): 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 to 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.

Where *a* is the length measured along the column or beam axis from the joint. M_n and V_n are nominal moment and shear capacity of the reinforced concrete section, respectively. 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 code, the units of stress, force, area, length are MPa, N, mm^2 and mm, respectively. For ACI 318 code, the units of stress, force, area, length appertain to psi, kips, in^2 and in, respectively.

For column (ATC-40)	For beam (ACI 318)	
$V_n = V_c + V_s$	$V_n = V_c + V_s$	(3.1)
$V_c = 0.29\lambda\left(k + \frac{N_u}{14A_g}\right)\left(\sqrt{f'_c}\right)b_w d$	$V_c = \left(\frac{\sqrt{f'_c}}{6}\right)b_w d$	(3.2)
$V_s = \frac{A_v \cdot f_{yt} \cdot d}{0.6s}$	$V_s = \frac{A_v \cdot f_{yt} \cdot d}{s}$	(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, $\frac{V_a}{b_w \cdot d \cdot \sqrt{f'_c}}$

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.

$V_a = M_n / a$ For the expected flexural failure mode in member (3.4)

$V_a = V_c + V_s$ For the expected shear fail in member (3.5)

v) Longitudinal Reinforcing index, $\rho = \frac{A_s}{b_w \cdot d}$ and $\rho' = \frac{A'_s}{b_w \cdot d}$

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 top and bottom reinforcement in the section, respectively.

vi) Longitudinal Reinforcing index, $\rho_t = \dfrac{A_t}{b_w \cdot d}$

This ρ_t is the column reinforcing index. A_t is the total amount of longitudinal reinforcement in the section. High value means possibly 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 of flooring and wall of the building.

3.4.2 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_n) and confinement joint reinforcement ratio (ρ_s). 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 like to occur for a higher index value.

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

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 become steeper it also becomes less effective in resisting horizontal joint shear.

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

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 stress over joint shear strength ratio (V/V_n):

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 is shear force transferring within joint region and is calculated by the following formulas (Paulay and Priestley, 1992).

$$V = (1+\beta)\lambda_0 f_y A_{s1} - V_{col} \qquad 3.7$$

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

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

Where A_{s1},A_{s2} is area of top and bottom beam reinforcement, respectively, λ_0 is over strength factor and is set to be 1.25, V_{col} is the column shear force, V_{col} is calculated by the following formula, $M_{0,1}$ is negative moment capacity of the right beam, $M_{0,2}$ is positive moment capacity of the left beam.

$$V_n = 0.083\lambda\gamma\sqrt{f_c'}A_j \qquad 3.10$$

$$A_j = b_j h_c \qquad 3.11$$

Where V_n 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. 3.3 and 3.4.

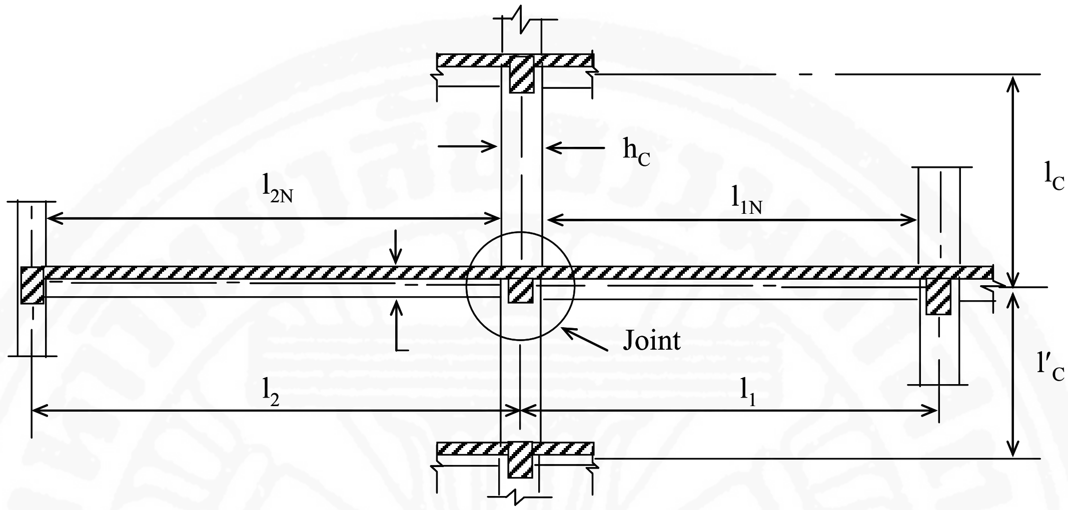

Fig. 3.3 Interior Beam-Column Subassemblages (Source: Paulay and Priestley, 1992)

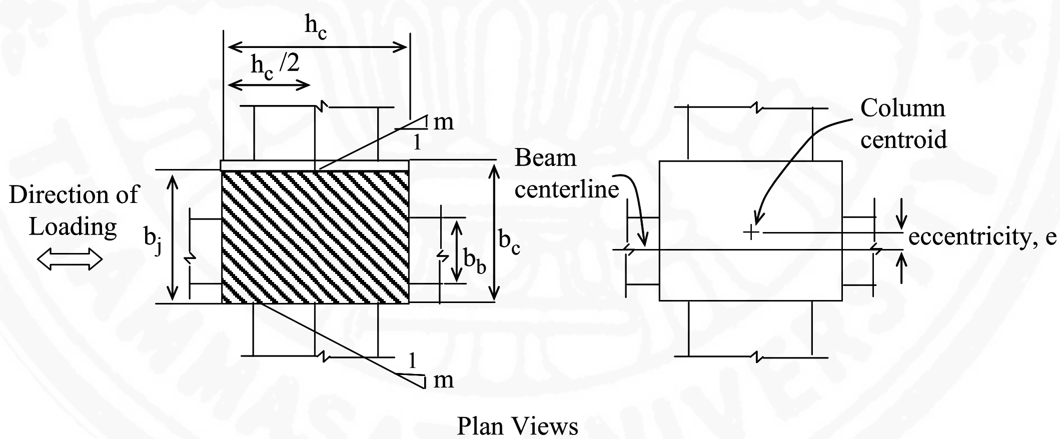

Fig. 3.4 Determination of effective joint width (bj) (ACI-318)

3.5 Relevant of column tributary area of buildings

These buildings were designed according to the non-seismic provisions of ACI building code which considers gravity load only. These buildings were grouped based on their column tributary area into three categories as buildings with large, medium and small column tributary area (Fig.3.5). The database of 17 reinforced concrete mid-rise buildings in Bangkok was gathered by Chaimahawan and Pimanmas (2006). All buildings were reinforced concrete beam-column frame and had 5-21 stories. Table A-1 of Appendix A provides types of building, number of story, column size and column tributary area of the conventional interior connection of the 17 observed buildings.

Based on the collected data, the area range was 40-48 m^2, 20-30 m^2 and 9-18 m^2 for large, medium and small category, respectively. Four buildings (B1-B4), six buildings (B5-B10) and seven buildings (B11-B17) are classified as building with large, medium and small column tributary area, respectively.

The structural and geometry indices of the investigated buildings were calculated. Furthermore, it was found that the column tributary area is related to the column section, the ratio of column depth to diameter beam bar (h_c/d_b) and bond index (BI) as illustrated in Fig 3.6, Fig 3.7 and Fig 3.8, respectively. Fig. 3.6 shows the relationship between column size and column tributary area. It can be seen that the column size increases with the increase of column tributary area. This tendency is expected for structures designed primarily for gravity loads. Fig. 3.7 shows the relationship between column depth to bar diameter ratio (h_c/d_b) and square root of column tributary area. The square root of column tributary area indicates the equivalent span length. The figure shows the increasing of column depth to bar diameter ratio with the increase of equivalent span length. Fig. 3.8 shows the plot between bond index (BI) and square root of column tributary area. The tendency clearly shows the decreasing bond index with the increase of equivalent span length. These data reveals the fact that bond condition and the corresponding stress development will be more favorable in buildings with large column tributary area and column size.

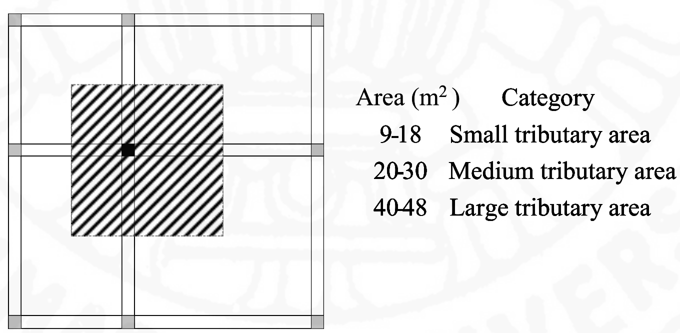

Fig. 3.5 Definition of column tributary area

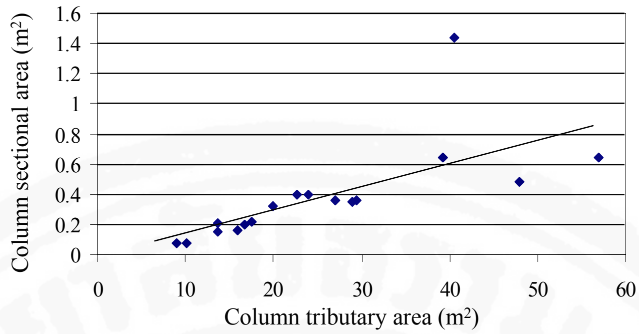

Fig. 3.6 Relation between column sectional area and column tributary area

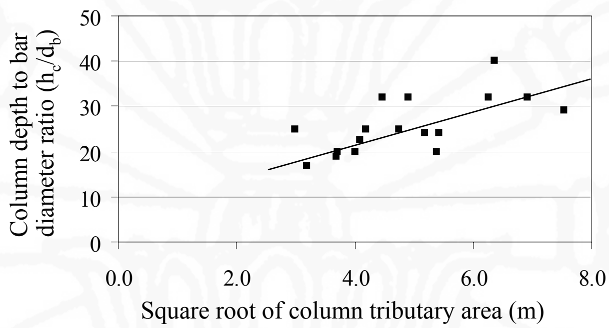

Fig. 3.7 Relation between column depth to bar diameter ratio (h_c/d_b) and square root of column tributary area

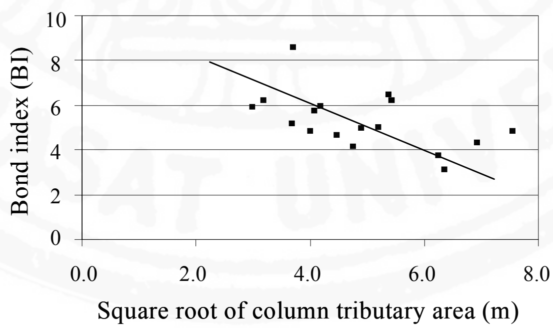

Fig. 3.8 Relation between bond index and square root of column tributary area