CHAPTER 2

LITERATURE REVIEW

In many cities in South East Asia, the conventional RC buildings that were designed and constructed for gravity load only show that their seismic performances are low when subjected to earthquake event. The strength and the stiffness of those buildings were decreased obviously. One of the important factors is believed to be non-ductile reinforcement details- especially with no or little horizontal transverse reinforcement within joint core (Leon,1987). Because of this, many researches have studied hysteresis behavior of the interior and exterior beam-column joint more than 35 years (Li, 2002) and still ongoing. Most of the researches consisted of experimental program, analytical model of beam-column joint and RC frames response. The summary of these researches are provided as follows.

2.1 Shear force on interior beam-column joint

When a building frame encounters a seismic event, a building frame is indirectly acted by lateral story forces. The moments and the shear forces were generated in the beams and columns of a building frame. Horizontal shear force is also transferred through the joint. At the face of beam-column joint, the longitudinal bar passing through the joint panel is subjected to tension force on one face of joint and subjected to compression force on the opposite face as shown in Fig. 2.1. Hence, the bar must develop bond force within limited column width to equilibrate these forces. Moreover, the corner of joint is acted by the compression stress resulting from bending of the beam member and from combination of bending and axial force of column. In general, the force mechanism within beam-column joint of the building frame during seismic events is known to be complex (Paulay et al,1978; Bonacci and Pantazoupoulou, 1993).

Some theories attempt to describe force transfer within the joint in the form of a horizontal shear force at the mid-level of the joint under lateral load as shown in Fig 2.1, which may be calculated in the following equation 2.1 and 2.2

$$V_{jh} = T + C'_c + C'_s - V_c \tag{2.1}$$

$$V_{jh} = T + T' - V_c \tag{2.2}$$

Where T is tension force, T' is tension force on opposite column face, C'_c and C'_s are compression force on concrete and steel, respectively and V_c is column shear force.

Other theories are available to explain force transfer within the joint in the form of stress (NZS,1995; Park and Paulay,1975). In their model, there are two mechanisms in the joints. In the first mechanism, the diagonal main strut force is derived from the compression forces transferred from the column and beam compression zones(Fig 2.1(a)). Another mechanism is the diagonal sub-strut mechanisms wherein the forces are transferred from the longitudinal column and beam bars via the bond between the concrete and the reinforcing steel of the column and the beam. The diagonal sub-strut stress field is generated along the length of bars over joint core (Fig 2.2(b)). The two mechanisms are superposed together to obtain the total behavior.

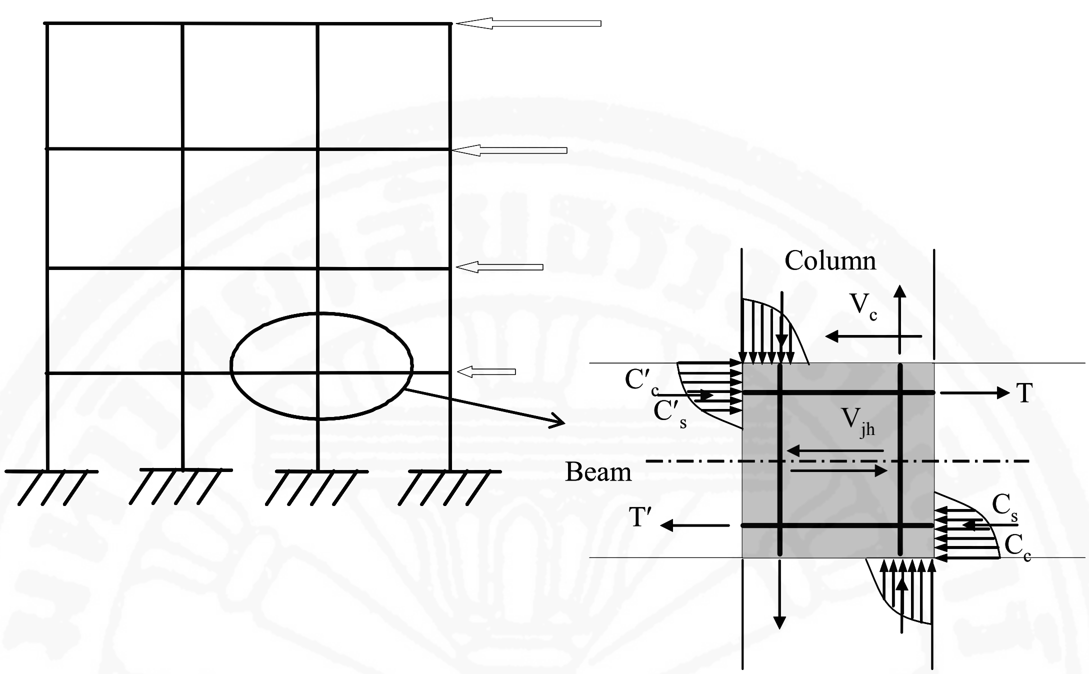
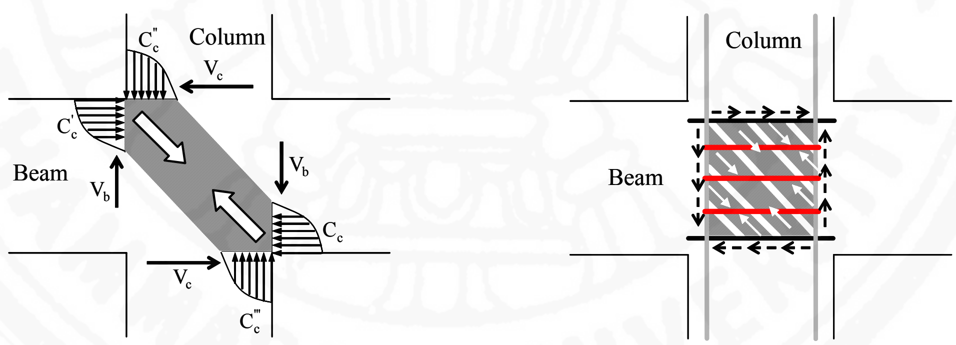


Fig. 2.1 Forces acting on the joint zone



a) Compressive strut mechanism b) Truss mechanism

Fig. 2.2 Mechanism of forces transmitted within a joint

Presently, Strut and Truss mechanism are generally adopted. Strut mechanism depends on the compressive strength of concrete mainly while truss mechanism depends on horizontal and vertical joint reinforcements. The joint reinforcement contributes the joint strength as long as concrete has sufficient strength to equilibrate tension in ties and bond anchorage of longitudinal bar is maintained in the joint. The concept of the strut and truss mechanisms is adopted in New Zealand code (1995). However, this resistant mechanism is not clearly proportional to strut and truss mechanism.

2.2 Experimental performance of beam-column joint

The previous researches (Aycardi,1994; Kunnath,1995; Hakuto et al,2000; Li et al,2002) studied hysteresis behavior of beam-column joint designed for gravity load only with nonseismic detail and tested to evaluate the performance of substandard beam-column joint. Some researchers tried to retrofit damaged buildings after being subjected to the simulated seismic loadings. Furthermore, some researches expected new joint reinforcement details for better seismic performance. However, the new joint reinforcement details have not been accepted yet due to the fact that only a few literatures were studied and that the construction method was too complicated. Many of the previous researches are summarized as follows:

Hanson and Connor (1967) studied the behavior of seven unconfined (isolated) beam-column joints. These tests demonstrated that the properly designed and detailed cast-in-place concrete frame can resist moderate earthquakes without damage and can survive severe earthquake without loss of strength. Hoop reinforcement in the joints connecting beams and columns was required to resist shear force. Moreover, the tests showed that properly detailed joints maintained their strength and the adequate energy absorption is provided by inelastic behavior (ductility) of the reinforcing steel and concrete in the beams.

Hanson and Connor (1972) tested 4 full-scale beam-column joint-a corner joints, an interior joint and two side edge joints. The experimental results confirmed that adequate energy absorption for seismic resistance can be provided at the joint core if anchorage, shear resistance and confinement were designed properly. The effectiveness of joint reinforcement designed to carry joint shear is dramatically shown in test result. One interior beam-column joint specimen with joint shear reinforcement designed to carry the entire shear in excess of that carried by the concrete, retained its strength and stiffness throughout the repeated reversals of load. Even after this severe loading, it was still demonstrated that this specimen had a substantial reserve capacity. Hysteresis behavior of another specimen, containing no joint shear reinforcement, showed identical results to the previous specimen except that it did not resist the high overload and resulted in a severe loss of stiffness and strength in the joint after the second repeated cyclic load. In every case, the required ductile behavior was provided in the beam at hinging section next to the column as long as the joint retained its shear strength and stiffness.

Durrani J. A. at el (1985) conducted a study to test three interior beam-to-column subassemblages designed according to the seismic provision of ACI code. The objective of the study was to evaluate the performance of interior joints which have different amount of transverse reinforcement less than that of recommendation of Committee 352. Other objectives were to investigate the effect of the level of joint shear stress on strength, stiffness and energy dissipation of beam-column subassembleages. The conclusions drawn from the test are as follows:

- The strength and stiffness loss have been observed also for structures with moderate volumes of transverse reinforcement and moderate shear and bond-stress demand.
- The joint hoop reinforcement helps to maintain the joint strength rather than it helps to reduce the loss of stiffness
- The joint shear stress affects the joint shear deformation, the slippage of beam and column bars, and thus the pinching of hysteresis loops. Keeping lower joints

shear seems to be more effective to reduce both the joint shear deformation and slippage of bars than the addition of more joint reinforcement.

- For higher energy dissipation, a combination of a lower joint shear and a moderate amount of joint reinforcement was found to be more effective than a combination of a higher joint shear and a large amount of joint reinforcement.
- A minimum column-to-beam nominal strength ratio of 1.5 was found to be suitable for design.

Kitiyama et al. (1987) tested two series of half-scale interior beam-column sub-assemblages (J- and C- series). The differences between J and C series were the amount of longitudinal beam bars. A degree of joint hoop parameter was used as a variable in this study. The experimental results presented that the joint shear demand should be restricted in proportion to concrete compressive strength. The joint shear failure is associated with the change of shear mechanism caused by the bond deterioration of beam bar within joint core and it is not feasible to prevent this deterioration so the ratio of the column width to the beam bar diameter must be limited as function of the strength of beam bar and concrete compressive strength. The role of the lateral reinforcement is considered to confine the connection rather than to resist shear.

Fattah et al (1887) tested 12 full-size interior beam-column assemblages under quasi-static loading. The objective of this research was to develop a different approach to move away beam plastic hinging zone from column face. From the study, the approach was adding supplemental intermediate longitudinal reinforcement placed in two layers at approximately the third points between the tension and compression reinforcement. These bars passed through the joints and extended into the beam a distance equal to 1.5 times the beam effective depth. The results are drawn as follows:

- Supplemental intermediate reinforcement can move the beam plastic hinging zone away from the column face, which led to a larger ductility demand in the beam. This increased demand was successfully provided by using small-diameter and intermediate-level reinforcement. The flexural-shear cracks were distributed widely along the beam.
- In addition, the beam section could remain elastic up to column drift of 3%, beam reinforcement yielding did not penetrate into the joint core.
- One result of the relocated plastic hinge was that the confinement of the joints can be achieved by using less transverse reinforcement volumes.

Leon (1990) clarified the influence of beam bar anchorage length on the hysteretic and shear performance of interior beam-column joints and indicate a definite interaction between shear stress and anchorage length. The results indicate that current design procedures would result in some damages to the joint if a weak girder-strong column design is used, and the frame is subjected to large seismic forces. Damage would be primarily in the form of joint shear cracking and joint shear deformation. Moreover, this research still indicated that;

- An anchorage length of 20 bar diameters is sufficient to anchor bars in interior beam-column joints. If anchorage lengths are in the order of 28 bar diameters then the beam will develop ultimate flexural strength
- Joint shear stress level less than $15\sqrt{f'_c}$ (kpi) does not produce substantial damage to the joint if the anchorage lengths are large. However, the joint shear stress is not a good indicator of joint damage or joint behavior.

- Most of force transfer is accomplished by strut mechanism. For a connection with large anchorage lengths (greater than 24 bar diameters), only a minimum amount of transverse reinforcement should be required in the joint core. The minimum joint transverse reinforcement shall be specified in accordance with the minimum requirement of column confinement.
- Deterioration behavior of joint due to joint shear strain cannot be ignored in the design though joint member is assumed to be rigid when the anchorage length equals or exceeds 28 times of beam bar diameter.

Kitiyama et al. (1991) attempted to develop earthquake resistant design criteria of RC interior beam-column joints. The design criteria emphasized the protection of the joint during an intense earthquake. They studied experimentally the role of joint lateral reinforcement and the effect of transverse beam and slabs. The conclusion can be pointed out as: the beam bar bond deterioration should be tolerated in the interior beam-column joint; joint shear must be resisted by the diagonal compression strut; joint shear failure can be delayed by limiting joint shear stress level and providing joint lateral reinforcement. The joint shear resistance can be enhanced by transverse beams and slabs. Here, the design provisions were suggested in order to supplement the building performance to a ductility factor of 4.0 as follows:

- The ratio of column depth(h_c) to the beam bar diameter(d_b) must be limited as in the given formula: $h_c / d_b \geq fy / 9\sqrt{f'_c}$, where f'_c is concrete compressive strength (kgf/cm^2); f_y is yield strength of longitudinal bar (kgf/cm^2).
- The joint shear stress(ν_j, kgf/cm^2) must be limited as follows $\nu_j \leq 0.25f'_c$
- A minimum lateral reinforcement ratio of 0.4% is recommended. This value may be reduced if joint shear stress is sufficiently lower than $0.25f'_c$
- The nominal joint shear strength(ν_{jn}) may be enhanced up to 1.3 times, $\nu_{jn} \leq 0.33\,f'_c$, if beams frame into four vertical faces of the joint and if at least 2/3 of each joint face is covered by framing beams
- Column axial stress smaller than $0.3\,f'_c$ does not exhibit beneficial effect on the bond resistance along the beam bar within a joint, and the column axial stress smaller than $0.5f'_c$ does not influence the joint shear strength.

Bonacci and Pantazopoulou (1993) gathered the database results of 86 beam-column joint tests compiled from published literature and from the results of a simple mechanical model developed using equilibrium, kinematic, and material considerations. They studied the influence of variables such as axial load, amount of transverse reinforcement, concrete strength, presence of transverse beams, and bond demand on the strength and behavior of beam-column joints. The results of 86 building joint subassemblages tested in the laboratory found that anchorage failure contributed to joint failure for 19 specimens and found that 67 specimens were damaged by joint failure resulted from shear force. It is also concluded that these mentioned variables are largely responsible for the differences in the joint behavior.

Aycardi at el.,(1994) investigated experimentally the behaviors of one-third scale RC members under simulated seismic loading, designed for gravity load only. Four one-third column specimens and two specimens are interior and exterior beam-column specimen including slab and transverse beam. The research showed that the structural components, with nonseismic details such as lap splices in potential plastic hinge zone, lack of joint reinforcement, minimum transverse reinforcement in column for shear and confinement, and discontinuous positive bottom beam bar at the joint core, could reach their nominal strength capacities and sustained the gravity loads for large cyclic deformations. However, the exterior subassemblage showed a weak beam-strong column mechanism due to inappropriate anchorage of the bottom beam bar. Interior beam-column subassemblage presented a weak column-strong beam mechanism and did not protect interior subassembleage from a substantial amount of lateral load at 4% drift. Additionally, the experimental results of subassemblages showed that a hybrid type of failure mechanism will be more likely to occur for complete structural frame.

Hakuto at el.,(1999) presented the theoretical reduction in the flexural strength and ductility of beams resulted from bond deterioration of longitudinal beam bar within interior beam-column joints. The effect of a range of ratios of column depth (h_c) to diameter of longitudinal reinforcing bar(d_b) is one of parameters leading to that reduction. The small ratio of h_c/d_b causes some slippage of longitudinal beam bars that is bond deterioration, which results in a reduction of compressive stress in beam bar at the adjacent beams and may actually be in tension. When analysis takes into account the effect of actual stress in the compression beam bar then flexural strength and ductility of beam-column frame will be reduced as a result of increasing tensile stress. This outcome should be considered by specifying the maximum allowable ratios of h_c/d_b in the design standards and codes.

Hakuto at el.,(2000) reported the results of simulated seismic load tests on RC interior and exterior beam-column joint with substandard reinforcement details typical of buildings constructed before the 1970s. The details of the interior beam-column joints are considered as substandard details. The substandard details are described as no transverse joint reinforcements and poorly anchored longitudinal beam bar passing through the joint. The experimental results showed that the seismic performance of typical interior beam-column joint of frames without transverse reinforcement in joint would be poor in a severe earthquake if the nominal horizontal joint shear stress(ν_j) less than $0.17f'_c$ (MPa). However if the column of interior beam-column frame is enlarged, then no joint hoops are required because joint shear stress is decreased, thus it can be considered as ductile frame. At an imposed displacement ductility factor of 6, the failure mode at joint core and nominal horizontal joint shear stress is greater than $0.07f'_c$(MPa). Moreover, the research investigated the effect of beam bar diameter to column depth ratio ranging from 18.8 – 25.0. It is found that the seismic behavior was not significantly affected by the bond conditions along the column depth.

While the substandard reinforcement details of exterior beam-column joint contained very little transverse reinforcement in the member and joint core and the beam bar hooks were not bent into the joint core but the hooks of end of the top bars were bent up and the hooks of the end of the bottom bars were bent down. From the test, the performance under seismic loading was significantly improved when the ends of the hooks of the top and bottom longitudinal beam bars were bent into the joint core. By this bending type, exterior frame can tolerate a nominal joint horizontal shear stress of ν_j given as follows $\nu_j = 0.053 f'_c$ MPa or $\nu_j = 0.31\sqrt{f'_c}$ MPa when plastic hinging occurs in the beam.

Stehle et al (2001) studied two half-scale RC interior wide-band beam-column connection under lateral earthquake loading. The first connection had details according to Australian standards without any special provision for seismicity, and the second connection had details similar to the previous connection but with minor modifications. First, longitudinal beam bars passing more than half of effective depth (*d/2*) outside the column face were debonded for a *d*-distance from the column face. This was to avoid torsion cracks of the portion of the wide beam transferring out of balance moment to the side faces of the joint core. Second, the bottom longitudinal beam bars extend past the column to avoid loss of anchorage. Third, some stirrups were added to the beam in the plastic hinging region at a spacing of d/2 to provide some confinement. The study can be concluded as follow: The first connection exhibits main deficiencies in performance are as follows; 1) Torsion crack of the portion of wide beam transferring out-of-balance moment through the side faces of the joint core and 2) The poor anchorage of bottom beam bar. The second connection is shown to perform very well up to the maximum 4% column drift ratio, and strength is not reduced by the debonding mechanism due to alternative load path availability.

Li, et al (2002). This paper reports the experimental results of four full-scale interior beam-column joints of six-story reinforced concrete frame, two oblong joints in transverse direction frame and two beam-wide column joints in longitudinal direction frame. Joint dimensions have been found to have a significant influence on the seismic behavior of frame. Two of them were typical beam-column joint in Singapore, designed to sustain the gravity loading and a notional horizontal load required by BS 8110 but no seismic loading was considered. The typical reinforcement details of such frame were: 1) column main bars were lap spliced just above the floor level. 2) bottom beam bars were terminated and lap spliced within the joint core, and 3) there was no joint transverse reinforcement within the joint core. While the reinforcement details of the other two specimens were modified, one contains 15% and 24% of joint transverse reinforcement required by NZS 3101 and were tested for comparisons with the control ones.

The experimental results demonstrate that providing a small quantity of joint transverse reinforcement and relocating the column bar lap splices to other positions, the performance of such joints can be improved, achieving higher ductility factor. Additionally, more energy was dissipated in the modified joint. However, joint shear failure was the predominant failure mode in all specimens. The extensive joint cracking and joint deformation were considered to be the main causes for reduction in stiffness. For any oblong joints having a joint shear stress ratio(ν_{jh}/f'_c) of less than 0.2, the improvement of the behavior of this kind of joints cannot depend on the restriction of beam bar diameters rather than the increase of the joint transverse reinforcement. However it is also suggested that the requirement on the ratio of beam bar diameter to column depth, which is recommended by joint ACI-ASCE Committee 352, should be more strict.

Park R.(2002) reported the results of some simulated seismic load tests on RC one-way interior and exterior bema-column joints with substandard reinforcement details typical of buildings constructed in New Zealand before the 1970s. The tests were conducted using both deformed and plain round longitudinal reinforcement. The results from the simulated seismic load tests indicated that

- The seismic performance of typical joint of pre-1970 without transverse reinforcement would be poor in a severe earthquake due to diagonal tension cracking in the joint core and bond slip.
- The bottom longitudinal reinforcement in the beams lap spliced in the potential plastic hinge regions near the column face showed that when plain round bar was used, the bond in the lap splice was totally lost in the first loading cycle and the beam did not reach its flexural strength while in the case of deformed bar used, deterioration of the flexural strength of beam occurred after a displacement ductility factor of 2 was reached.

Li at el (2004) conducted test to examine the feasibility of new joint detailing for low to moderate seismic risk regions. Four half-scale beam-column joint specimens cast of high strength concrete(HSC), containing respectively E detail, H detail, AD detail and CD detail were fabricated and tested under reversed cyclic displacement excursions. Joints without transverse reinforcement (E detail) have been proven unsatisfactory for strength and ductility, while conventional joints containing transverse reinforcement (H detail) commonly adopted in seismic regions cannot avoid reinforcement congestion. Two kinds of new joint details, i.e. AD (adding diagonal steel bars in the joint) and CD (bending some of the beam longitudinal reinforcement bars diagonally up and down in the joint). Unit CD could hardly reach the limited ductility range and the joint suffered severe inelastic damage and Unit AD behaved excellently with the relatively well intact joint. The test results showed that the various reinforcement detailing within the joint core exhibited different failure modes. It is concluded that the detail of Unit AD is suitable for joints in regions of low to moderate seismicity due to ease of fabrication compared to Unit H.

Cheejaroen (2003) tested three half-scale R/C interior beam-column subassemblages, constructed in RC mid-rise buildings in Bangkok under quasi-static cyclic loading. The first specimen was considered as a representative of small tributary area column while the second specimen represented medium tributary area column. The last specimen was simulated from beam-column joint whose column occupied large tributary area. All specimens were designed for gravity load only so they did not contain horizontal joint shear reinforcement and column lap splice were bundled just above joint core. The experimental result indicated the significant behaviors as follows: 1)the specimen which represented a large column tributary area show a good seismic performance even they had no transverse reinforcement within the joint region. While specimen occupied a small tributary area (less than 5x4 m^2) failed by joint shear within joint core and showed poor seismic performance.

2.3 Analytical Investigation for beam-column joint model

Internal force equilibrium mechanism on a beam-column joint was proposed by Park and Presley. They attempted to describe internal force mechanism in terms of Strut and Truss mechanism which has generally been accepted. However, these mechanisms can not clearly seperate a proportion of resistance force between concrete strut and joint reinforcement (Baglin, Leon, Hong and Lee). On this reason, shear resistance of interior and exterior beam column joint has been studied although there are not many researches in this aspect. Aside from Park and Presley's model, new model of joint shear failure is proposed by Shiohara. Mechanism of Shiohara's model based on moment-resisting system within beam-column joint and was proven by many experimental results. Furthermore, investigation of shear mechanism within joint panel was analysis by finite element method. However, internal force equilibrium within joint panel depend on not only joint capacity but also beam and column capacity. The previous researches are reported as follows:

Hwang and Lee(1999) proposed softened strut-and-tie models for exterior beam-column joint. This model could evaluate only the shear strengths of exterior beam-column joint for seismic resistance without considering the effect of bond deterioration which significantly affected the reduction of truss mechanism. The joint strength was based on diagonal strut mechanism and truss mechanism according to the geometries of the joint, and considered that the softening of diagonal strut and truss mechanism depended on the horizontal joint reinforcements and longitudinal column bars. This model was derived to satisfy equilibrium, compatibility, and the constitutive laws of crack reinforced concrete. The accuracy of the proposed model is satisfactorily correlated with other research's experimental results.

Baglin and Scott(2000) used SBETA program, a nonlinear finite element analysis software for analysis of reinforced concrete structures under plane stress condition. The finite element mesh adopts first-order quadrilateral elements only while reinforcing bars were specified by discrete element constrained at the boundary of the concrete element. The study focuses on the isolate effects of a single variable in the parametric study because there are a large number of variables that controls the capacity of joint. The general conclusions can be drawn as follows: The numerical models show very good correlation with load deflection of the test specimens. However, the deformation due to crack growth and dislocation of the models was inhibited by the smeared crack approach. The results of analysis can provide the failure mechanism to be clearly observed, with compression failure zones and dominant tensile crack clearly indicated.

Hwang and Lee(2000) proposed the analytical model for predicting shear strengths of interior RC beam-column joints, called the softened strut-and-tie model. The model is based on the concept of struts and ties and derived to satisfy equilibrium, compatibility and the constitutive laws of cracked reinforced concrete. This proposed model consists of the diagonal, horizontal and vertical mechanism. Diagonal mechanism assumed that the direction of the diagonal concrete strut coincides with the direction of principal compressive stress of the concrete. The horizontal and vertical tie mechanisms assumed that the joint hoops within the center half of the joint core are considered fully effective and the other joint core hoops are included at a rate of 50%. The joint shear strength is calculated as the concrete compressive stress on the nodal zone as it reaches its capacity and the concrete bearing force to be examined is the summation of compressions from the diagonal, flat, and steep struts, The proposed method was found to reproduce test results of 56 interior joint from the literature with reasonable accuracy

Shiohara (2001) proposed a new mathematical joint model for predicting the moment strength and pointed out an irrationality in the existing joint models for shear failure, adopted by current design codes for RC interior beam-column connections. The joint panel of the new model is divided into four triangular concrete segments. The rotational movement of the segments caused uneven opening of the cracks like flexural cracks. The model can account for the effects of parameters including the concrete compressive strength and amount of transverse reinforcement in the joint core. Twenty interior beam-column connections were investigated and the new joint model can be correlated with experimental shear force of the investigated specimen. And from the investigation, the following is concluded:

- In the test of beam-column connections, joint shear stress is not proportional to story shear. Joint shear is increased until the end of the test in most specimens, even if the joint shear deformation apparently increased and the story shear decreased.
- The cause of the degradation in story shear was due to the finite upper limit of anchorage capacity of beam bar. When the anchorage resistance attained its capacity, stress in compressive reinforcement at the column face shifted from compression to tension. On this reason, it caused the decrease of moment resistance that finally led to the degradation of story shear.

Calvi G.M., et.al (2002) considered the substandard joint of both interior and exterior beam-column joint response to evaluate the global performance of RC frame. The joint was designed in the absence of specific seismic code provisions. Smooth bars are anchored with terminal hooks. The column designed for gravity only is considerably weaker than beam. An equivalent moment rotation spring, governing the relative rotation of beam and column elements, is adopted to represent the joint behavior either in the linear or non-linear range. Beam and column elements converging in the joint are modeled as one-dimensional frame elements with concentrated inelasticity at the critical section interface, defined through appropriate moment curvature based on section analysis. The moment-rotation characteristics of the joint spring might be directly derived, based on equilibrium considerations, from the corresponding principal tensile stress and shear deformation curve. After first crack, the beam and column elements possessed hardening behavior until the tensile stress (p_t) reached $0.42\sqrt{f'_c}$ which was assumed for interior joint, where alternative shear transfer mechanism can be activated, while elasto-perfect plastic behavior or an approximate strength degradation model can be adopted for exterior Tee-joints. The main results can be summarized as follows:

- The numerical analysis carried out using a model for joint behavior which accounts for the limit strength of the panel zone without considering strength degradation at higher deformation demand seems to show that the "favorable" aspects of joint damage may be relevant and therefore joint damage may be not as detrimental as usually thought.
- Exterior joint can be extremely vulnerable and reach collapse well before any significant damage takes place in columns and beams
- Due to bond degradation and slip in column bars, columns in interior joints can general sustain lower flexural moment than those expected when assuming prefect bond conditions

- The joint damage activates a shear hinge mechanism which can protect or delay soft-storey mechanism by spreading the inter-storey drift demand between the story above and below the joint and thus reducing the rotation demand in the adjacent column. This favorable effect is trade off with higher local deformation demand in the damaged joint panel.

Li et al. (2003) used program WCOMD-2D to verify the experimental results and to study the effect of some critical parameter such as joint transverse reinforcements, column axial load and bond condition on the joint's behavior. The behavior of interior beam-column joints were investigated by using nonlinear two dimensional 8-node isoparametric RC element. The anchorage of the beam longitudinal bars and column main bars was defined by using RC joint plates, located at the boundaries of the joint panel because the influence of bond slip on the behavior of structure is considerably high. The analytical result can be summarized as follows: 1) the behaviors of exterior and interior beam-column joint are different. The parameters influencing the shear capacity are different, 2) the most important factors affecting the shear capacity of exterior connections are the concrete compressive strength(f'_c), the joint aspect ratio, beam reinforcement (ratio, detailing, anchorage, and amount of stirrups and 3)for interior joint, the main parameters influencing the shear capacity is the concrete compressive strength

Lowes and Altoontash (2003) represented the response of reinforced-concrete beam-column joints under reversed-cyclic loading. The proposed model provides a simple representation of the primary inelastic mechanisms that determine joint behavior, failure of the joint core under shear loading and anchorage failure of beam and column longitudinal reinforcement embedded in the joint. The model is implemented as a four-node 12-degree-of-freedom element that is appropriate for use with typical hysteretic beam-column line elements in two-dimensional nonlinear analysis of reinforced concrete structures. Constitutive relationships are developed to define the load deformation response of the joint model on the basis of material, geometric, and design parameters. Comparison of simulated and observed response for a series of joint subassemblages with different design details indicates that the proposed model is appropriate for use in simulating response under earthquake loading

Hong and Lee (2004) presented a strut-and-tie model to determine the shear strength of interior beam-column joint without considering a capacity of connecting components such as beam member or column member. In this model, the joint shear strength is related to the deformation of plastic hinges at the adjacent beams and the system ductility is limited by joint shear failure. The effect of plastic hinge deformation of beams on shear strength of joint region is defined in terms of bond deterioration and softening of concrete compression. Modeling with only the diagonal strut mechanism and truss mechanism induces a conservative solution for the shear carrying capacity of beam-column joint because bond deterioration affects joint shear strength significantly. Bond deteriorated mechanism does not exist in strut-and-tie model of Hwang and Lee. The proposed model is compared with available test results in form of the column shear strength, showing good agreement. The beam-column joint model is useful for practitioner to evaluate the joint strength of existing joints and for the design not only in high seismic zone but low- or mid- seismicity zones where fully ductile behavior is not required.

Hegger et al. (2004) used program ATENA, assuming full bond between the reinforcement and the concrete, to investigate the behavior of exterior and interior beam-column joints. The concrete was modeled with nine-node isoparametrics shell element, while discrete bars were used to model the reinforcement. The fine element size in joint region is used while the elements of beam and columns were coarser. The failure criterion of the concrete is based on the biaxial stress according to Kupfer, Hilsdorf and Rusch. The analysis results indicated that the behaviors of exterior and interior beam-column joint are different. The parameters influencing the shear capacity are different. The most important factors affecting the shear capacity of exterior connections are compressive strength, the joint aspect ratio, beam reinforcement (ratio, detailing, anchorage, and amount of stirrups), while interior joint, the main parameter influencing the shear capacity is the compressive strength

Supaviriyakit at el (2007) presented a test of non-seismically detailed RC beam-column connections under reversed cyclic load. The tested specimens represented those of the actual mid-rise RC frame buildings designed according to the non-seismic provisions of the ACI building code. The evaluation of 10 existing reinforced concrete frames was conducted to identify key structural and geometrical indices. A column tributary area was chosen as a parameter for classifying specimens because of a correlation among the structural and geometrical characteristics and the column tributary area. Hence, The test results showed that specimens representing small and medium column tributary area failed by brittle joint shear while specimen representing large column tributary area failed by ductile flexure despite no ductile seismic details were provided. The detailed failure mechanism was investigated through a nonlinear finite element analysis. The two-dimensional reinforced concrete plate element was used to model beam and column whereas the one-dimensional discrete joint element was used to model the interface between beam and joint face. The finite element analysis revealed that the joint shear failure of specimen representing small to medium column tributary area was resulted from the collapse of principal diagonal concrete strut. It is shown the nonlinear FEM can satisfactorily predict the hysteretic loops, correct failure modes and cracking process. The collapse of diagonal concrete strut has been analytically identified as the principal of failure mechanism.