Chapter 2: Literature Review

2.1 Deficiencies of sub-standard reinforced concrete column

In the past, buildings were designed and constructed without seismic consideration, the columns in such structure, sub-standard column, were designed to maintain the building load (gravity load) and static wind load only. In concern with dynamic load, especially earthquake, these columns reinforcement details violate the modern seismic design theory [Lao (1990)]. That deficiencies cause buildings perform the poor behavior, low ductility and brittle collapse when subjected to seismic excitation.

The investigation of buildings constructed following the sub-standard (pre-1970) seismic design approaches by Melek, Wallace et al. (2003) have led to the conclusions that lap splices in sub-standard columns were typically designed as compression splices with the lap length of about 20 to 24 times bar diameter and were poorly confined by small amount of transverse steels. Moreover, a lot of reinforced concrete bridge piers in low to moderate seismic zones may have inadequate lap splice between column longitudinal bars and starter bars projected from the footing at the base of column. The failure of the lap splice may lead to low lateral strength and poor ductility in cyclic responses of sub-standard RC columns subjected to seismic excitation.

In Thailand, the sub-standard Reinforced Concrete buildings were designed and built followed the codes published by the Engineering Institute of Thailand (EIT) under H.M. the King's Patronage. The first one, EIT standard 1007-34 [EIT (1991)], is a working stresses design method code. Second, EIT standard 1008-38 [EIT (1995)] is a strength design method code. Finally, the construction materials and practices is conformed the standard code EIT 1014-40 [EIT (1997)]. Thinh (2003) did a research to identify the deficiencies of design and construction practice in these EIT standard codes. He found that, these standard codes do not provide any seismic resistance, these lacks of the satisfactions of the seismic design philosophy:

Relative lap splice in EIT's Reinforced Concrete Columns is located just above the construction joint at floor level.

Widely spaced column ties that provide a little confinement, the requirement amount of transverse reinforcement is much lower than that in ACI 318M-99 (about 6 times).

The minimum lap splice length in column required in EIT 1007-34 [EIT (1991)]depends on the nominal yield strength of steel f_y that shown in Eq. (2.1a) and Eq. (2.1b) for compression and tension lap splice, respectively. That is also the requirement length in ACI 318-63, [Phatiwet (2002)]. In which, d_b is steel bar diameter.

$$l_d = \begin{cases} 20d_b \;\; if \;\; f_y \leq 300\, MPa \\ 24d_b \;\; if \;\; f_y = 400\, MPa \\ 30d_b \;\; if \;\; f_y = 500\, MPa \end{cases} \qquad \text{For compression slices} \qquad (2.1a)$$

$$l_d = \begin{cases} 24d_b \;\; if \;\; f_y \leq 300\ MPa \\ 30d_b \;\; if \;\; f_y = 400\ MPa \\ 36d_b \;\; if \;\; f_y = 500\ MPa \end{cases} \qquad \text{For tension slices} \qquad (2.1b)$$

Lap splice strength, is the maximum force that lap spliced can be maintained without bond failure, affect heavily to a column performance subjected to seismic load. In case of short lap splice length and poor transverse confinement, lap splice fails before bar can reach yield, that causes reinforced concrete column lateral strength suddenly drop and perform no ductility in hysteresis response. The experiments tested these sub-standard column with short lap splice can be found in many research, such as Kankam (1997), Xiao and Ma (1997), Bousias, Spathis et al. (2006), Harajli (2008).

Short lap splice leads to bond failure that causes degradation of strength of column prevents bars from reaching yielding. Two predominant modes of bond failure are 'pull-out' and 'splitting'[ACI408.2R92 (1992)]. Bond failure occurs in pull-out mode if the ratio of concrete cover to bar diameter is large or the concrete is very well confined [Harajli and Dagher (2008)]. The most important parameters that affect the pull-out bond strength under static and cyclic loading have long been discussed in detail by Eligehausen, Popov et al. (1983). On the contrary, when concrete cover is small or the steel bars are closely spaced or lap splice lengths are short, tensile splitting cracks tend to develop in the concrete under the radial component of the rib bearing forces parallel to the steel bars causing premature splitting bond failure [Harajli and Dagher (2008)]. In most of substandard RC column structures, bond failures occur in splitting mode rather than in pull-out mode due to short lap splice length.

In order to enhance the lateral strength and dynamic response of these sub-standard columns, there are some methods were used to retrofit these columns and strengthen lap splice strength. Steel jacketing is a method of using round steel plates to jacket around a column [Chai, Priestley et al. (1991), Aboutaha, Engelhardt et al. (1996)], and the gap between steel plates and column is grouted with a pure cement grout. It is found that, lap splice strength increase since lap spliced zone was steel jacketed [Pauley and Priestley (1992)]. Concrete jacketing is another method in which a relatively thick layer of reinforced concrete can be use to jacket around the columns, this method also provides an improvement in lap spliced strength [Pauley and Priestley (1992)]. Recently, Fiber Reinforced Polymer (FPR) is widely researched and applied on wrapping technology to strengthen sub-standard columns.

Next section (2.2) is going to represent a strengthening reinforced concrete column using Fiber Reinforced Polymer sheets to wrap around lap splice zone, as well as the effectiveness of FRP confinement on concrete compressive strength and bond stress between reinforcing bar and concrete.

2.2 Fiber reinforced polymer confinement impacts

Fiber Reinforced Polymer (FRP) is now considered the state-of-the-art technology in rehabilitating and strengthening of reinforced concrete (RC) structures. The sub-standard reinforced concrete column with short lap splice length can be strengthened by wrapping FRP around the lap spliced zone. There are three methods (Fig 2.1) to wrap FRP along the lap splice length [Teng, Chen et al. (2002)]. It can be fully wrapped by single or multiple layers. It can also be partially wrapped using FRP straps in a continuous spiral or discrete strips.

Some reinforced concrete columns confined with FRP sheets were tested by Xiao and Ma (1997), Ma, Xiao et al. (2000), Harries, Ricles et al. (2006), Bousias, Spathis et al. (2006), and Harajli and Dagher (2008). The tests showed that FRP confinement could prevent the splitting failure prior to yielding of main bars, thus developing the strength of short lap splice into post-yield range with higher ductility.

The strength of a lap spliced bar is based on bond stress between steel bar surface and surrounding concrete. There are many factors that affect bond stress and lap splice strength as consequence, the most factors were addressed by some research [Orangun, Jirsa et al. (1975), Harajli (2009)] are: concrete compressive strength, diameter of bars, ratio of concrete cover to bar diameter, ratio of lap splice length to diameter bar, transverse confinement condition. By wrapping FRP around lap splice zone, this zone is provided an additional transverse confinement, as a result, the compressive strength of concrete increase, and bond stress-slip relationship of lap spliced bar enhance. These help increase the lap splice strength.

2.2.1 Uniaxial stress-strain model of confined concrete

Richart, Brandtzæg et al. (1928; 1929) proposed stress-strain relationships of confinement concrete under hydrostatic pressure. The Fig 2.2 shows the hydraulic pressure chamber used for making pressure on concrete cylinder specimen. The proposed model was given as following expression [Eqs (2.2), (2.3)].

$$f'_{cc} = f'_c\left(1 + k_1 \frac{f'_l}{f'_c}\right) \tag{2.2}$$

$$\varepsilon'_{cc} = \varepsilon'_o\left(1 + k_2\left(\frac{f'_{cc}}{f'_c} - 1\right)\right) \tag{2.3}$$

Saatcioglu and Razvi (1992) proposed a model to predict the stress–strain response of reinforced concrete columns based on earlier findings by Richart, Brandtzæg et al. (1928) and Hognestad (1951). They noticed that the stress–strain response of confined concrete was comprised of two distinct regions as shown in Fig 2.4. Consequently, the parabolic ascending region was represented by a modified Hognestad (1951) equation for unconfined concrete under uniaxial load, while the linear descending region was constructed by joining two points corresponding to 85% and 20% stress level beyond the peak stress. A constant residual stress was assumed at 20% of the peak stress. The prediction was later compared to a large experimental database of various types of columns tested under fast and slow strain rates. The results indicated good agreement between the experimental data and the model.

Mander, Priestley et al. (1988) proposed a well-known model for evaluating the effect of steel confinement on the axial strength of concrete. The stress-strain relationship expression was given as below [Eqs (2.4), (2.5)].

$$f'_{cc} = f'_c\left(-1.254 + 2.254\sqrt{1 + \frac{7.94f_l}{f'_c}} - 2\frac{f_l}{f'_c}\right) \tag{2.4}$$

$$\varepsilon'_{cc} = \varepsilon'_o\left(1 + 5\left(\frac{f'_{cc}}{f'_c} - 1\right)\right) \tag{2.5}$$

Fardis and Khalili (1981) did experiment of circular FRP-encased concrete cylinders tested in concentric compression exhibit very high strength and ductility. Rectangular FRP-encased concrete beams were constructed also with varying amounts of unidirectional FRP reinforcement added at the bottom. Such beams have very good strength and ductility, and their deflections are almost completely reversible, even after loading to their peak capacity, provided that enough FRP reinforcement has been added at the bottom to prevent brittle failure by fracture of the FRP in tension. Fardis and Khalili (1981) proposed the model to evaluate the confine stress as [eq (2.6), (2.7)]. The subscript "$_{com}$" means properties of composite in hoop direction.

$$f'_{cc} = f'_c\left(1 + 2.05\frac{f_l}{f'_c}\right) = f'_c\left(1 + 4.1\frac{f_{com}t}{df'_c}\right) \tag{2.6}$$

$$\varepsilon'_{cc} = 0.002 + 0.001\frac{E_{com}t}{df'_c} \tag{2.7}$$

Base on regression analysis of existing data Cusson and Paultre (1995) determine the stress-strain relationship expression as Eqs (2.8)(2.9).

$$f'_{cc} = f'_c\left(1 + 2.1\left(\frac{f_l}{f'_c}\right)^{0.7}\right) \tag{2.8}$$

$$\varepsilon'_{cc} = \varepsilon'_o + 0.21\left(\frac{f_l}{f'_c}\right)^{1.7} \tag{2.9}$$

Based on a variety of reinforcing fiber types, orientations and jacket thickness, Karbhari and Gao (1997) developed and verified simple design equations, model (Fig 2.6) to estimate the response of composite confined concrete. Experimental results indicate that composite jackets significantly enhance the strength and pseudo ductility of concrete. The proposed expressions for ultimate strength and strain are seen to compare well with experimental data, and a phenomenological model for confinement based on the composite behavior of the confined system is presented. The results of the use of a number of predictive equations are compared with the experimental data. Eqs (2.10), (2.11) below show Karbhari and Gao (1997) model.

$$f'_{cc} = f'_c\left(1 + 3.1\nu_v\frac{2t}{D}\frac{E_f}{E_c} + 2\frac{f_l t}{Df'_c}\right) \tag{2.10}$$

$$\varepsilon'_{cc} = 1 - \left\{ \frac{(1+\varepsilon_{co})^2 \left[1 - \frac{f'_c}{E^{eff}} - 4.1 f'_c \nu_v \frac{2t}{D} \frac{E_{com}}{E_c E^{eff}}\right]}{(1+\varepsilon_{com})^2} \right\} \tag{2.11}$$

Samaan, Mirmiran et al. (1998) proposed a model (Fig 2.5) to predict the complete bilinear stress-strain response of FRP-confined concrete in both axial and lateral directions. The model [Eqs (2.12), (2.13)] is based on correlation between the dilation (expansion) rate of concrete and the hoop stiffness of the restraining member. The parameters of the model are directly related to the material properties of the FRP shell and the concrete core. The predicted stress-strain curves compare favorably with the results of the present study, as well as tests by others on both fiber-wrapped and FRP-encased concrete columns.

$$f'_{cc} = f'_c \left(1 + 6 \frac{f_l^{0.7}}{f'_c}\right) \tag{2.12}$$

$$\varepsilon'_{cu} = \frac{f'_{cu} - f_o}{E_2} \tag{2.13}$$

A uniaxial model for concrete confined with fiber-reinforced polymers (FRP), but also with steel jackets or conventional transverse reinforcement were proposed by Spoelstra and Monti (1999). The model, which is suitable to be inserted into fiber-type beam-column models, explicitly accounts for the continuous interaction with the confining device due to the lateral strain of concrete, through an incremental-iterative approach. The relation between the axial and lateral strains is implicitly derived through equilibrium between the (dilating) confined concrete and the confining device. This relation allows one to trace the state of strain in the jacket and to detect its failure. The model is compared with a set of experimental tests and shows very good agreement in both the stress-strain and the stress-lateral strain response.

$$f'_{cu} = f'_c\left(0.2 + 3\sqrt{f_{lu}}\right) \tag{2.14}$$

$$\varepsilon'_{cu} = \varepsilon_{co}\left(2 + 1.25 E_c \varepsilon_{ju} \sqrt{f_{lu}}\right) \tag{2.15}$$

Saafi, Toutanji et al. (1999) conducted an experiment for columns that are made of concrete-encased fiber reinforced polymer (FRP) tubes. The concrete-filled FRP tubes are cast in place. The tube acts as a formwork, protective jacket, confinement, and shear and flexural reinforcement. Results show that external confinement of concrete by FRP tubes can significantly enhance the strength, ductility, and energy absorption capacity of concrete. Equations to predict the compressive strength and failure strain, as well as the entire stress-strain curve of concrete-filled FRP tubes, were developed as Eqs (2.16).

$$f'_{cc} = f'_c \left(1 + 2.2 \left(\frac{f_l}{f'_c}\right)^{0.84}\right) \tag{2.16}$$

Another confinement model [Eqs (2.17)] of confined concrete was proposed by K. Miyauchi, Kuroda et al. (1999)

$$f'_{cc} = f'_c\left(1 + 2.98\frac{f_l}{f'_c}\right) \tag{2.17}$$

Based on a large test database assembled from an extensive survey of existing studies, Lam and Teng (2002; 2003; 2006; 2009) assessed available axial strength models for FRP-confined concrete. The test database is also deployed to examine the effect of various factors on the performance of FRP-confined concrete. Lam and Teng (2002) showed that the confinement effectiveness of FRP based on reported test results depends little on unconfined concrete strength, size, and length-to-diameter ratio of test specimens and FRP type, but depends significantly on the accuracy of the reported tensile strength of the FRP. The inherent variation of unconfined concrete strength also causes some scatter of data at low confinement ratios. Using those test data with accurate FRP tensile strengths, only two of the nine existing models are found to give close predictions. A new simple model (Fig 2.7) is finally proposed by Lam and Teng (2002), based on the observation that a linear relationship exists between the confined concrete strength and the lateral confining pressure from the FRP [Eqs (2.18)].

$$f'_{cc} = f'_c\left(1 + 2.15\frac{f_l}{f'_c}\right) \tag{2.18}$$

Existing analytical models for predicting the stress-strain or load-displacement response of fiber-reinforced polymer (FRP)-confined concrete are mostly derived for cylindrical plain concrete columns. In practice, however, typical concrete columns come in various shapes including circular, square, or rectangular and incorporate longitudinal and transverse steel reinforcements. Furthermore, strengthening or repairing is typically done while the column is under service loading. Maalej, Tanwongsval et al. (2003) proposed an analytical model to predict the load-displacement response of wall-like (i.e. high aspect ratio) reinforced concrete columns strengthened with FRP wraps with and without sustained loading. The model assumes that the general load-displacement response of the strengthened column consists of two distinct branches: a parabolic ascending branch and a linear descending branch. Shown in Fig 2.4, the ascending branch is influenced by the lateral confining pressure from the transverse reinforcement as well as the FRP wraps, while the descending branch is influenced by the buckling of the longitudinal reinforcement and the failure of the core concrete. The effective confined concrete area is shown in Fig 2.3. Comparisons between model results and experimental results indicate close agreement between the two.

Harajli (2006) developed the stress-strain relationship consists of two stage, first state follows a second-degree parabola similar to the one suggested by Sheikh and Uzumeri (1980; 1982) [Eqs (2.19)]. The second stage, including the intersection point between the first and second state, has the stress-strain relationship follow [Eqs (2.20), (2.21)]

$$f_{cc} = f_{co}\left(2\frac{\varepsilon_{cc}}{\varepsilon_{co}} - \left(\frac{\varepsilon_{cc}}{\varepsilon_{co}}\right)^2\right) \quad for\ \varepsilon_{cc} \leq \varepsilon_{co} \tag{2.19}$$

$$f_{cc} = f_c' + k_1\left(f_{lf} + f_{ls}\frac{A_{cc}}{A_g}\right) \quad for\ \varepsilon_{cc} \geq \varepsilon_{co} \tag{2.20}$$

$$\varepsilon_{cc} = \varepsilon_o\left(1 + k_2\left(\frac{f_{cc}}{f_c'} - 1\right)\right) \quad for\ \varepsilon_{cc} \geq \varepsilon_{co} \tag{2.21}$$

2.2.2 Bond stress-slip model for confined concrete

Bond failure between steel and concrete is generally characterized by two modes, namely pullout and splitting ACI-408.1R-90 ; ACI-408.2R92). If the ratio of concrete cover to bar diameter is relatively large or the concrete is well confined, bond failure occurs in pullout mode due to the shearing off of the concrete keys between the bar ribs. On the other hand, if the concrete cover is small or the steel bars are closely spaced, tensile splitting cracks tend to develop under the radial component of the rib bearing forces parallel to the steel bars causing premature bond failure.

For most structural applications, bond failures are governed by splitting of the concrete rather than by pullout. The average bond strength at splitting bond failure is influenced by several parameters such as the ratio of concrete cover to bar diameter, the development or splice length, the concrete compressive strength and concrete confinement Orangun, Jirsa et al. (1975; 1977). Several analytical studies undertaken by incorporating local bond laws into numerical solutions schemes of the bond problem in both pull-out and splitting modes of bond failure, and allowed better understanding of the influence of various parameters on bond strength.

Eligehausen, Popov et al. (1983) proposed one of the most widely used bond stress-slip relationships, which is based on an experimental program at the University of California, Berkeley Fig 2.16.

Based on experimental results from bridge column tests, Lehman and Moehle (2000) proposed the bi-uniform bond stress-slip model shown in Fig 2.17. In this model, for slip values less than the slip corresponding to the yield strain in the bar, the uniform bond stress is approximated as $1\sqrt{f_c'}$ MPa ($12\sqrt{f_c'}$ psi). For slip values exceeding the slip at yield, the bond stress capacity is $0.5\sqrt{f_c'}$ MPa ($6\sqrt{f_c'}$ psi).

Giuriani, Plizzari et al. (1991) developed a refined analytical model involving the local mechanical properties of concrete and the interaction phenomena between the principal and transverse reinforcement. The theoretical results give a close fitting of several different experimental results.

Alsiwat and Saatcioglu (1992) proposed an analytical procedure for the force-deformation relationship of a reinforcing bar anchored in concrete. The procedure leads to computation of deformations in two parts, as extension and slippage (Fig 2.10). The extension of reinforcement is determined by establishing inelastic strain distribution along the embedment length of the bar. Constant average bond along the elastic length and frictional bond over the plastic length are assumed. The rigid body slippage of the bar is computed when the bar is stressed to the far end. An increase in the elastic bond stress is considered to develop the bar force with the available anchorage length. The elastic bond at the far end of the bar is used with a previously derived local bond-slip model to obtain the rigid body slippage of the bar. The procedure is applied to straight as well as hooked bars, subjected to pull, and simultaneous push and pull, simulating exterior and interior joints. The results are verified against a large volume of experimental data reported in the literature.

Saatcioglu, Alsiwat et al. (1992) conducted an experiment of large-scale reinforced concrete columns to investigate hysteretic behavior of anchorage slip in reinforced concrete structures. The columns were subjected to constant axial compression and unidirectional and bidirectional lateral-deformation reversals. The results indicate a significant increase in column rotation due to anchorage slip. Penetration of yielding into the column footing was observed in the tension reinforcement, while the compression yielding was localized at the column-footing interface. Axial compression resulted in early closure of the crack associated with anchorage slip, reducing related deformations. An analytical model (Fig 2.11) was developed by Saatcioglu, Alsiwat et al. (1992) for hysteretic moment-anchorage slip-rotation relationship. The model consists of a primary curve and a set of rules defining unloading and reloading branches. The primary curve is constructed by computing the extension and slippage of tension reinforcement in the adjoining member. This is accomplished by considering inelastic strain distribution along the embedment length of reinforcement, as well as the local bond-slip relationship. Hysteretic rules are obtained from experimental observations, and incorporate pinching of hysteresis loops as well as the effect of axial compression. Comparisons of experimental and analytical hysteretic relationships indicate excellent agreement.

Abrishami and Mitchell (1996) proposed models for bond stress-strain in cases of failure by pullout (Fig 2.8) and splitting (Fig 2.9).

Conducted experimental analysis, Kankam (1997) established the fundamental relationship between bond stress, steel stress, and slip in reinforced-concrete structures. Tests were conducted on double pullout specimens reinforced with 25-mm plain round mild steel, cold-worked and hot-rolled ribbed bars that had been fully instrumented internally with electrical resistance strain gauges. The results provided examples of the longitudinal variation of the steel strain (analogous to that between cracks). The method has proved to be capable of providing sufficient data for plotting the distribution of steel stress, bond stress, and slip between flexural cracks. The relationship between bond stress, steel stress, and slip was derived from the steel strain function, and has been represented by empirical formulas.

Xiao and Ma (1997) test three 1:2 scale model columns. One column was tested under the condition of "as built" and two others were tested after being retrofitted using prefabricated composite jacketing. The as-built column suffered brittle failure due to the deterioration of lap-spliced longitudinal reinforcement without developing its flexural capacity or any ductility. The retrofitted columns showed significant improvement in seismic performance (Fig 2.12). The failed as-built column was retested after being repaired. The repaired column also demonstrated significant improvement in ductility. An analytical model, which takes into consideration the bond-slip deterioration of lap-spliced longitudinal bars, has been developed for seismic assessment and retrofit design.

Sezen and Moehle (2003) based on experimental results and previous theoretical investigations; an analytical procedure [Eqs (2.22), (2.23)] is developed to characterize the bond-slip behavior. The procedure is used to compute column end rotations and corresponding lateral displacement due to longitudinal bar slip at beam-column interfaces under monotonic lateral load. A bi-uniform relationship is adopted to model the bond stress distribution along the development length of the anchored reinforcing bar as shown in Fig 2.13.

$$slip = \int_0^{l_d} \varepsilon dx = \frac{\varepsilon_s l_d}{2} \quad for\ \varepsilon_s \le \varepsilon_y \tag{2.22}$$

$$slip = \int_0^{l_{dy}} \varepsilon dx + \int_{l_d}^{l_{dy}+l'_d} \varepsilon dx = \frac{\varepsilon_y l_{dy}}{2} + \frac{(\varepsilon_s + \varepsilon_y) l'_d}{2} \quad for\ \varepsilon_s \ge \varepsilon_y \tag{2.23}$$

Ayoub (2006) proposed a new model for nonlinear analysis of reinforced concrete beam–columns with bond–slip (Fig 2.14). The model is derived from a two-field mixed formulation with independent approximation of forces and displacements. The state determination algorithm for the implementation of the model in a general purpose nonlinear finite-element analysis program is presented and its stability characteristics are discussed. Both, bond–slip and pull-out effects, are represented in the model. The accuracy of the model in representing global and particularly local parameters

Bamonte and Gambarova (2007) tested 48 cylindrical specimens reinforced with a single bar and subjected to a pull-out or push-in force. Four diameters are considered 5, 12, 18, and 26mm, with bonded length-to-bar diameter ratios equal to 3.5 HPC and 5 NSC. All specimens are highly confined by means of a steel jacket to prevent or control cover splitting and to investigate bond behavior in highly confined conditions. The proposed local bond stress-slip law (Fig 2.15): (1) is formulated as an extension of the law suggested in European Model Code MC90, also including some later proposals; (2) fits quite well with the available test data on short, well-confined anchored bars; (3) introduces the favorable effects that the confining reinforcement generally consisting of stirrups and longitudinal bars has on bond strength; and (4) may be easily introduced into a design code.

The crack widths of reinforced concrete flexural members are influenced by repetitive fatigue loadings. The bond stress-slip relation is necessary to estimate these crack widths realistically. Oh and Kim (2007) propose a realistic model for bond stress-slip relation under repeated loading. Several series of tests were conducted to explore the bond-slip behavior under repeated loadings. Three different bond stress levels with various number of load cycles were considered in the tests. The present tests indicate that the bond strength and the slip at peak bond stress are not influenced much by repeated loading if bond failure does not occur. However, the values of loaded slip and residual slip increase with the increase of load cycles. The bond stress after repeated loading approaches the ultimate bond stress under monotonic loading and the increase of bond stress after repeated loading becomes sharper as the number of repeated loads increases [Eqs (2.24), (2.25)]. The bond stress-slip relation after repeated loading was derived as a function of residual slip, bond stress level, and the number of load cycles. The models for slip and residual slip were also derived from the present test data. The number of cycles to bond slip failure was derived on the basis of safe fatigue criterion, i.e., maximum slip criterion at ultimate bond stress.

$$\tau_N = 0, \qquad s < s_{r(N-1)} \tag{2.24}$$

$$\tau_N = \tau_{max}\left(\frac{s - s_{r(N-1)}}{s_{p1}}\right)^{\alpha N}, \qquad s \geq s_{r(N-1)} \tag{2.25}$$

One of the most practical and effective means for improving the bond strength of steel bars is concrete confinement. Concrete confinement becomes particularly important with the more frequent use of HSC, for improving the ductility of bond failure Azizinamini, M et al. (1993). Also, concrete confinement is essential in areas of seismic hazard for reducing the bond deterioration and enhancing the energy absorption and dissipation capabilities of the structure or structural components under cyclic loading. The most common method for concrete confinement is the use of closely spaced ties or hoops within the development/splice region or anchorage zone. Other methods that have been gaining attention involves the use of high performance steel fiber reinforced concrete (FRC) in earthquake resistant structures such as concrete beam-column connections, hybrid steel beam-RC column connections, low rise structural walls, and coupling beams, as a means for reducing their structural damage or increasing their energy absorption and dissipation capacities. Furthermore, concrete confinement for improving ductility and structural strength can be achieved using fiber reinforced polymer (FRP) systems, which have evolved over the last few decades as a viable alternative to traditional materials and construction techniques. A summary of the materials properties and structural applications of FRP reinforcement, and guidelines for design and construction of externally bonded FRP systems for upgrading concrete structural elements are reported by ACI-440.2R-02).

While several experimental and analytical investigations have addressed the bond and slip characteristics of steel bars under monotonically increasing load, very few studies have focused on the bond stress-slip response under cyclic load, particularly when the mode of bond failure is by splitting. Also, with the increasing use of FRC systems and FRP composites for improving structural performance under seismic loading, evaluating the influence that FRC or FRP confining reinforcement has on the bond strength between steel bars and concrete under cyclic loading becomes of interest.

Base on the results of a series of experimental and analytical studies undertaken in the main part by Harajli (2008; Harajli and Dagher (2008; Harajli and Khalil (2008) and in part by other investigators on the use of external FRP jackets for bond strengthening of developed spliced steel bars in tension and its implications on the static and seismic response of concrete members. Harajli (2009) proposed a model [Eqs (2.26)] to calculate the ultimate bond stress in cases of concrete not only confined with internal transverse steel, but also confined with FRC (Fiber Reinforced Concrete) and FRP. The accuracy of the model was verified against experimental data.

$$u_{max} = 0.78\sqrt{f_c'}\left(\frac{c + K_c}{D_b}\right)^{2/3} \tag{2.26}$$

It is not found any research on bond-slip relationship of a lap splice confined by FRP in post-yield range. As a consequence, the post-yield strength of lap splice confined by FRP cannot be determined. In order to estimate the post-yield strength of lap splice confined by FRP, it is necessary to conduct a research on bond stress along lap splice confined by FRP in post-yield range of bar stress.

2.3 Experiments of strengthening lap splice by FRP wrapping

There are scarce experiments focused on strengthening lapsplice zone by FRP wrapping in literature. Xiao and Ma (1997) studied on seismic retrofit of reinforced concrete circular columns with poor lap-splice details using prefabricated composite jacketing. Three 1:2 scale model columns have been tested. One column was tested under the condition of "as built" and two others were tested after being retrofitted using prefabricated composite jacketing. The as-built column suffered brittle failure due to the deterioration of lap-spliced longitudinal reinforcement without developing its flexural capacity or any ductility.

Haroun and Elsanadedy (2005) conducted an experiment. In which thirteen half-scale circular and square column samples were tested in flexure under lateral cyclic loading. Three columns were tested in the as-built configuration whereas ten samples were tested after being retrofitted with different composite jacket systems such as: Carbon/epoxy, E-glass/vinyl ester, and E-glass/polyester. A brittle failure was observed in the as-built samples due to bond deterioration of the lap-spliced longitudinal reinforcement. The jacketed circular columns demonstrated a significant improvement in their cyclic performance.

Based on the flexural behavior of concrete columns retrofitted with carbon fiber-reinforced polymer jackets Sause, Harries et al. (2004). Harries, Ricles et al. (2006) investigated the use of carbon fiber-reinforced polymer (CFRP) jackets as a seismic retrofit measure for deficient lap splices. Three full-scale building column specimens with lap spliced longitudinal reinforcing bars were tested under combined axial and cyclic lateral load. The columns were intentionally designed such that the lap splices would fail prior to achieving the flexural capacity of the column. One column was tested without retrofit as a control specimen, while the others were retrofitted with CFRP jackets. It is stated that with a CFRP jacket retrofit, the nominal flexural capacity of the column may be achieved. The ductility of the repaired column, however, is limited by slip of the spliced bars resulting in a splitting failure in the lap splice region.

Six rectangular section RC column tests conducted by Bousias, Spathis et al. (2006). It was found that, the un-confined column strength with lap splice length less than 45 bar diameter drops fast under dynamic load. The confined column strength by FRP wrapping of the lap splice strength increasing as more and more number of FRP sheets wrapped. The strength increased of all confined columns with 5 layers of FRP in three groups 15db, 30db, and 45db were 26%, 16%, and 14% respectively. It was also found that, the positive effect of FRP wrapping on flexural resistance, ultimate deformation and energy dissipation declined with decreasing lap length.

Full scale unconfined and FRP-confined column specimens with lapspliced reinforcement at the base were tested by Harajli and Dagher (2008). Also, companion columns with continuous reinforcement and with internal steel confinement to satisfy the ACI building code requirements for regions of high seismic hazard (earthquake-resistant columns) were tested for comparison. Harajli and Dagher (2008) found that confining the spliced zone with FRP wraps increased the bond strength of the spliced bars, reduced the bond deterioration and pinching under cyclic loading, and increased the lateral load resistance and ductility of the columns.

Full-scale circular columns with spliced reinforcement were tested under lateral load reversals by Harajli and Khalil (2008). It was found that the mechanism by which the FRP confinement improves the bond strength of spliced bars in circular columns is similar to that in rectangular columns. All of the unconfined columns suffered premature splitting bond failure leading to almost total strength and stiffness degradation. Confining the splice zone with external FRP jackets improve column behavior in seismic performance.

2.4 Nonlinear modeling of reinforced concrete column

Lowes, Mitra et al. (2003) formulated a versatile two-dimensional macro-model that represents a reinforced concrete beam-column joint. In their model, the bond slip and shear deformation responses of joints were considered as separate springs. The zero-length bond-slip springs were representatives of the force transferring ability between flexural reinforcement and the surrounding concrete while the zero-length shear springs represented the ability of shear transferring at the interface between frame members and the joint.

Matrin (2007) developed a reinforced concrete column two-dimension model Fig 2.18 to simulate behavior of reinforced concrete column without jacketing, subjected to seismic excitation. Basically, a single-curvature column model consists of two major parts; the first part is an elastic frame of $H - L_{\mathrm{p}}$ length, which represents the elastic behavior of column. The second part consists of a zero-length fiber section which is an assembly of nonlinear springs to represent the concrete and reinforcement nonlinear behavior. The second part is connected to the first part by a rigid link of L_{p} length. This model was found that give the close result to the experimental one in modeling sub-standard column with short lap splice [Matrin (2007)].

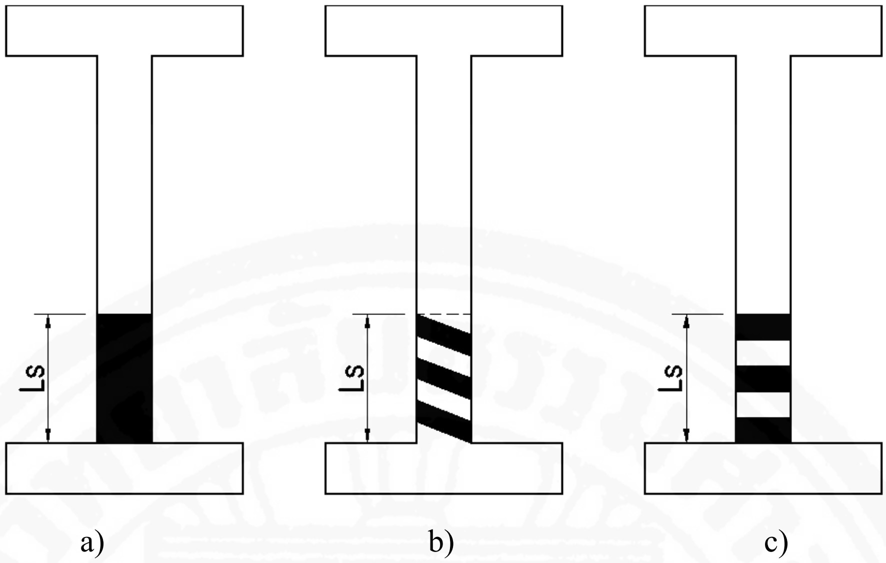

Fig 2.1: Typical FRP wrapping methods [Teng, Chen et al. (2002)]

a) Full wrapping using FRP sheets
b) Partial wrapping using FRP straps in continuous spiral
c) Partial wrapping using FRP straps in discrete strips

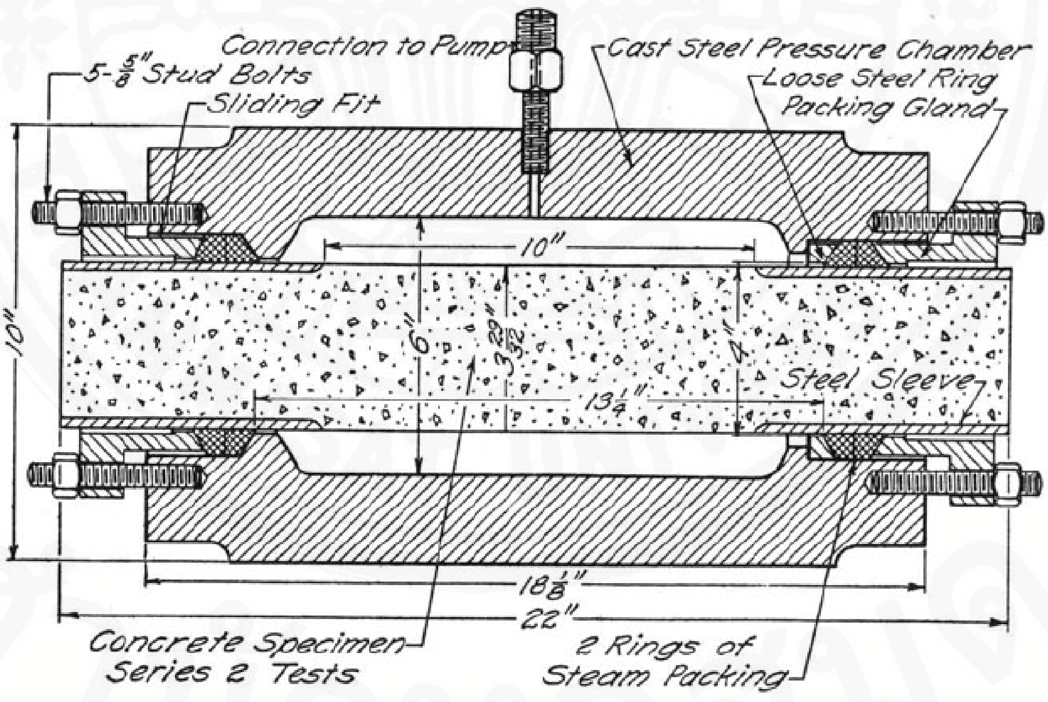

Fig 2.2: Hydraulic pressure chamber Richart, Brandtzæg et al. (1928; 1929)

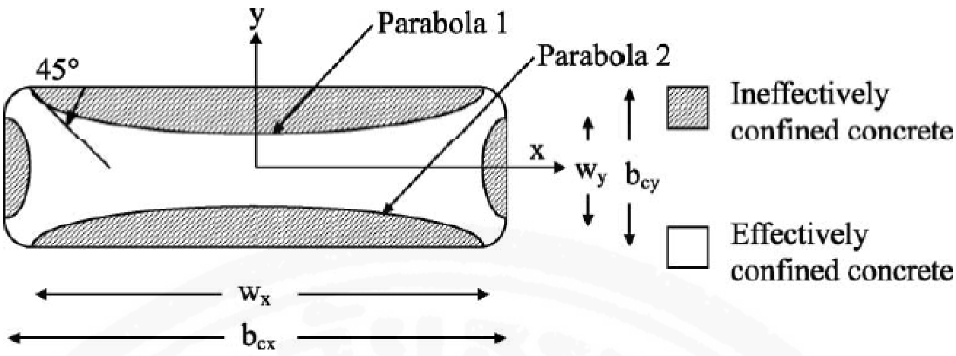

Fig 2.3: Effectively confined concrete area Mander, Priestley et al. (1988)

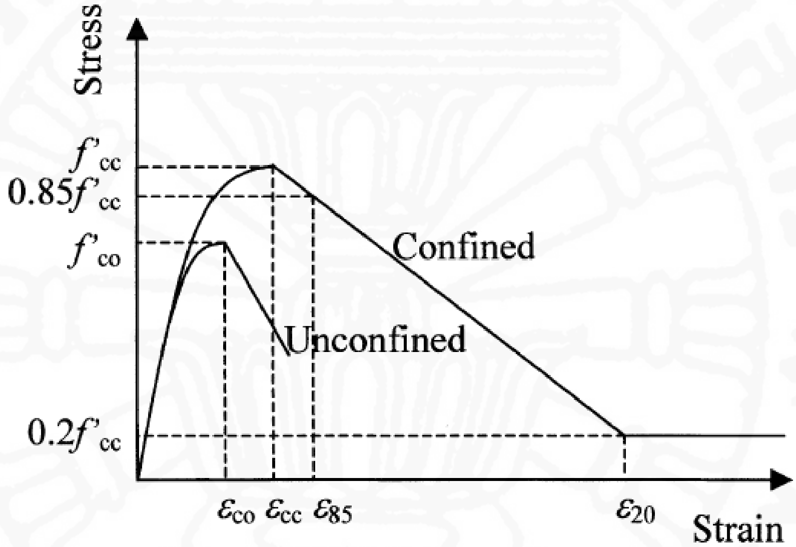

Fig 2.4: Stress-strain of confined concrete Saatcioglu and Razvi (1992)

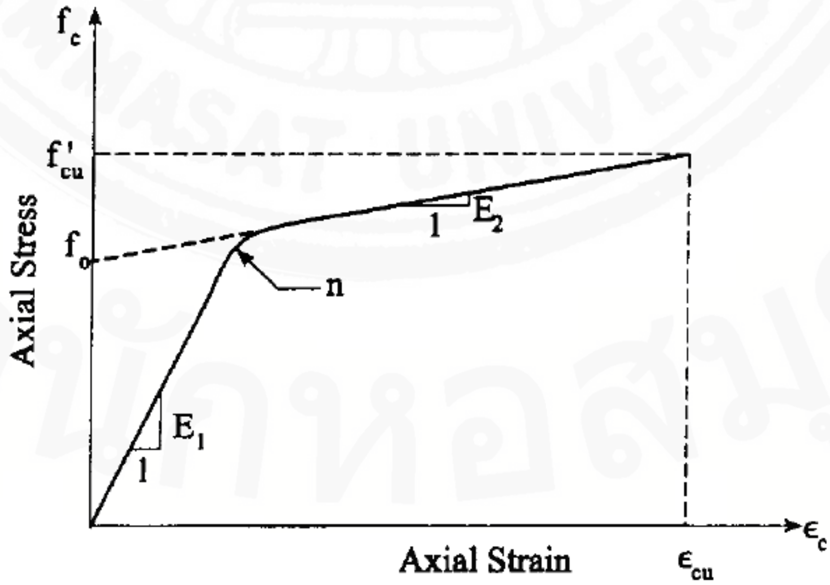

Fig 2.5: Bilinear confinement model Samaan, Mirmiran et al. (1998)

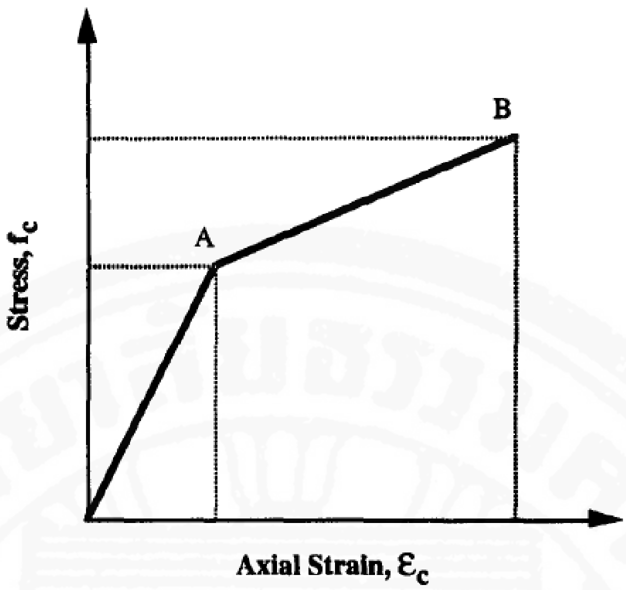

Fig 2.6: Stress-strain response, Karbhari and Gao (1997)

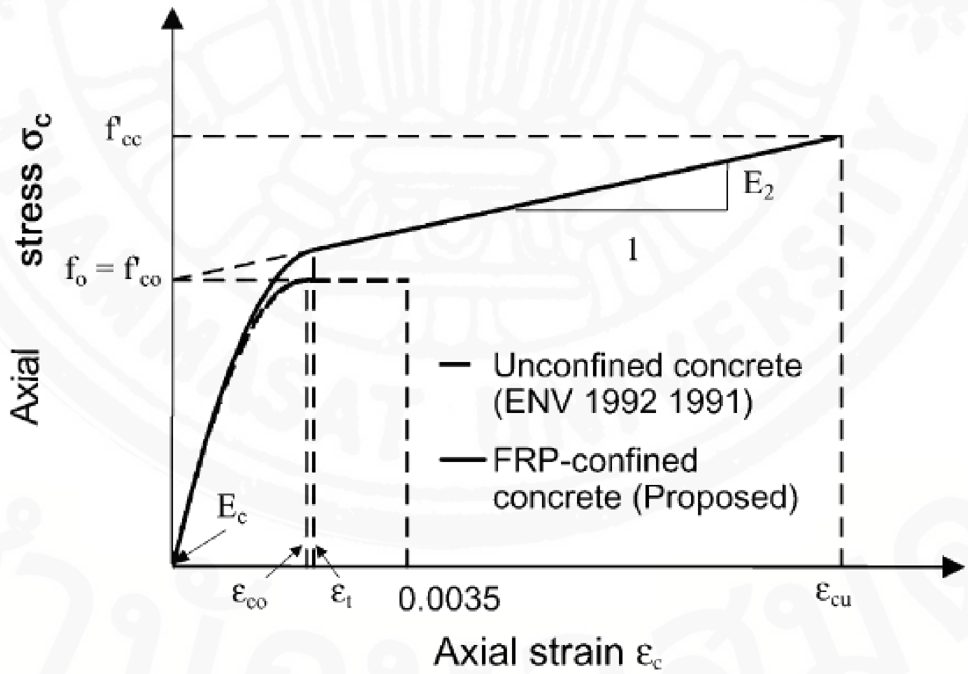

Fig 2.7: Proposed stress–strain model for FRP-confined concrete Lam and Teng (2002)

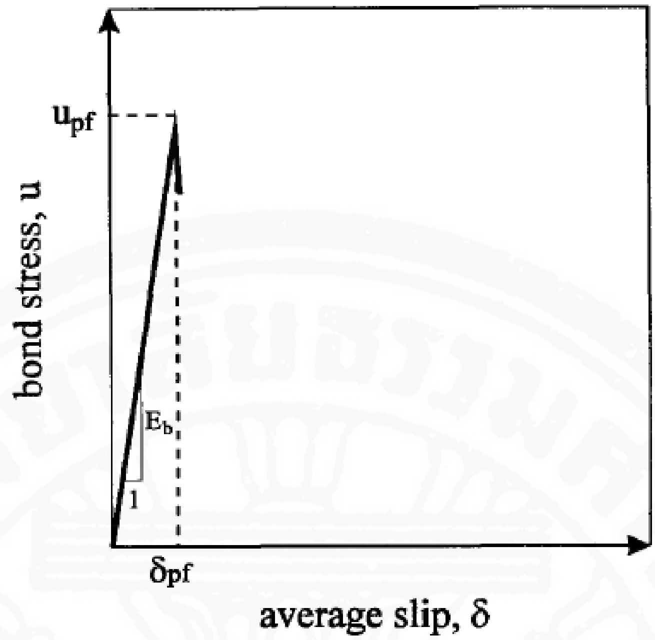

Fig 2.8: Analytical model for pull-out failure Abrishami and Mitchell (1996)

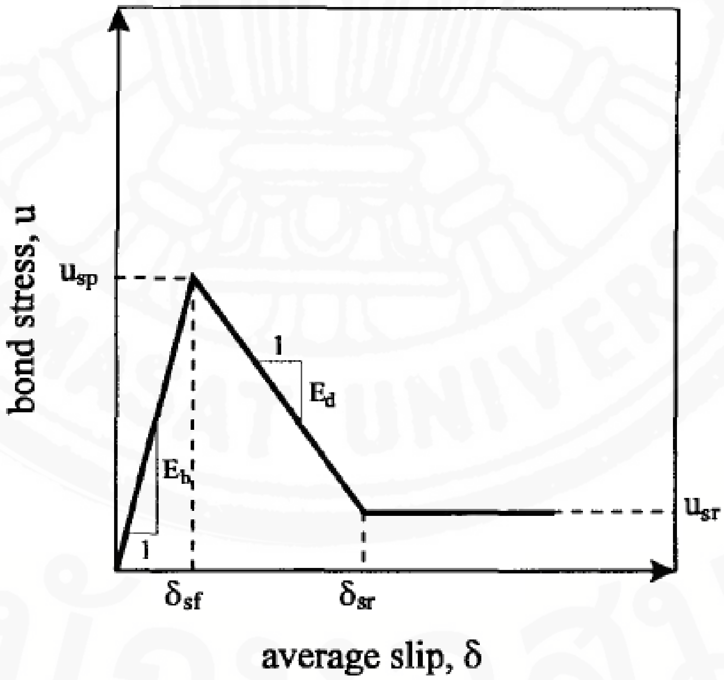

Fig 2.9: Analytical model for splitting failure Abrishami and Mitchell (1996)

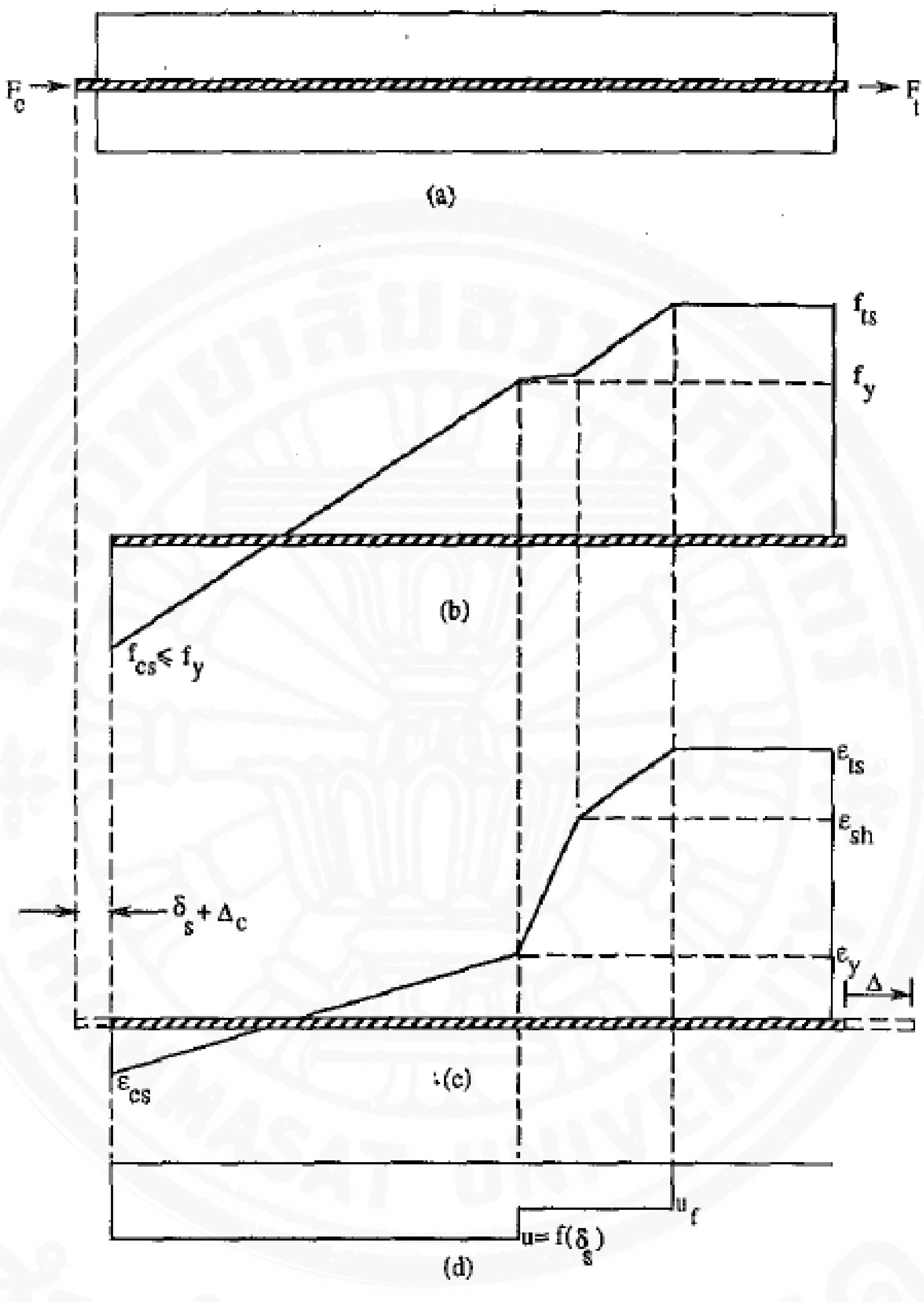

Fig 2.10: Steel stress-strain and bond-stress distribution Alsiwat and Saatcioglu (1992)

a. Reinforcing bar embedded in concrete
b. Stress distribution
c. Strain distribution
d. Bond stress between concrete and steel

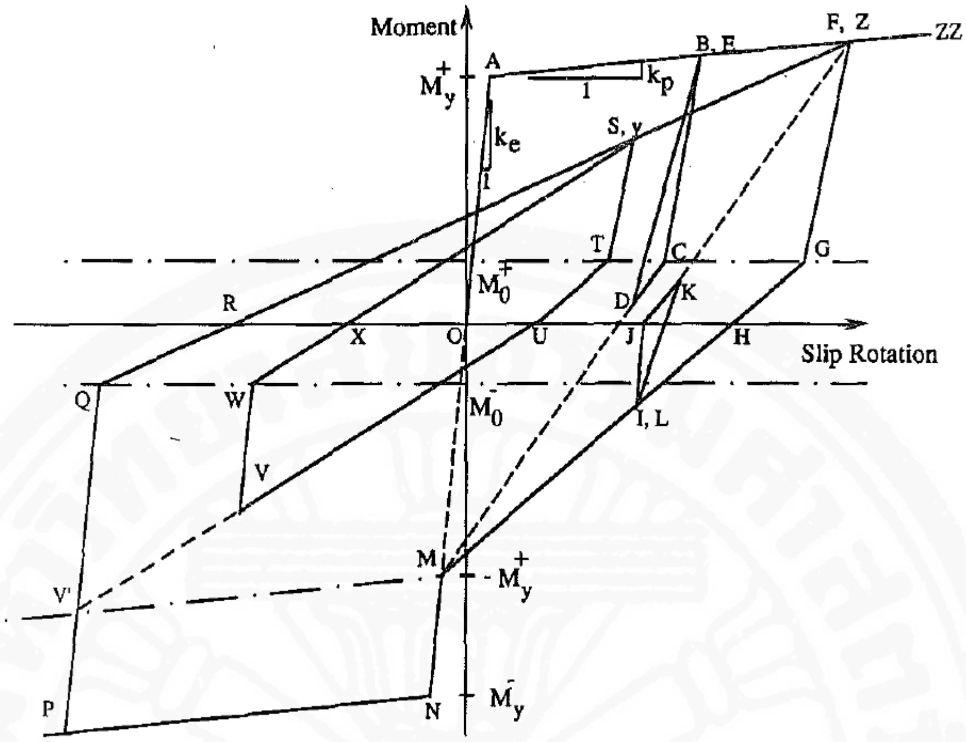

Fig 2.11: Proposed Hysteretic Model for Anchorage Slip by Saatcioglu, Alsiwat et al. (1992)

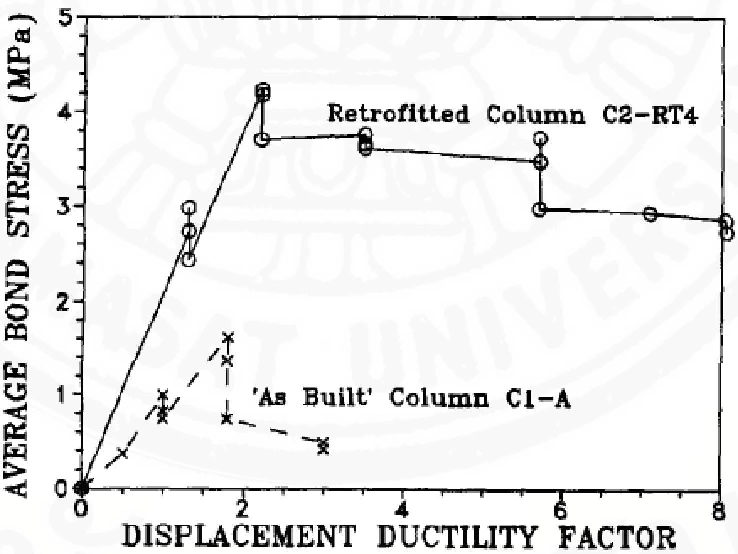

Fig 2.12: Degradation of bond stress in lapsplice bar Xiao and Ma (1997)

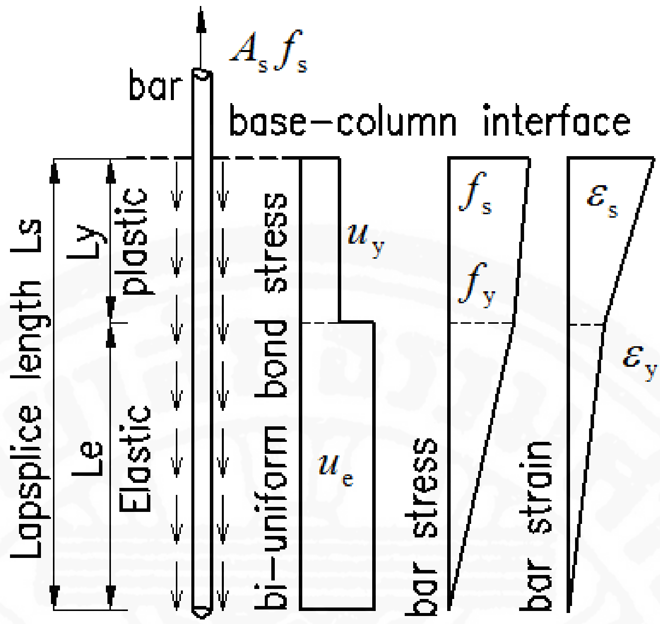

Fig 2.13: Bi-uniform bond stress distribution by Sezen and Moehle (2003)

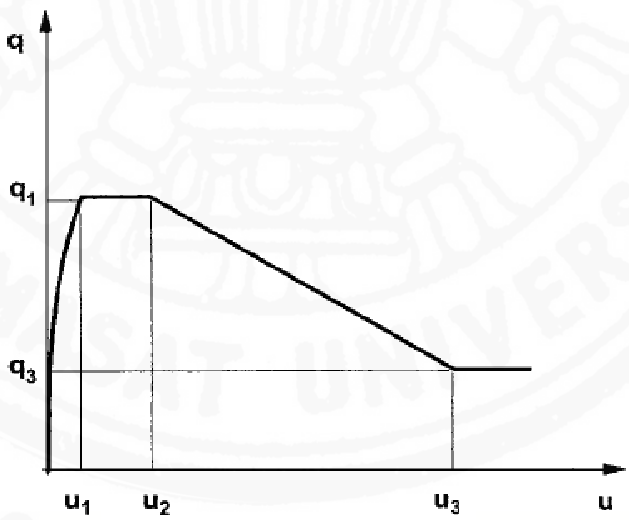

Fig 2.14: Bond stress–slip by Ayoub (2006)

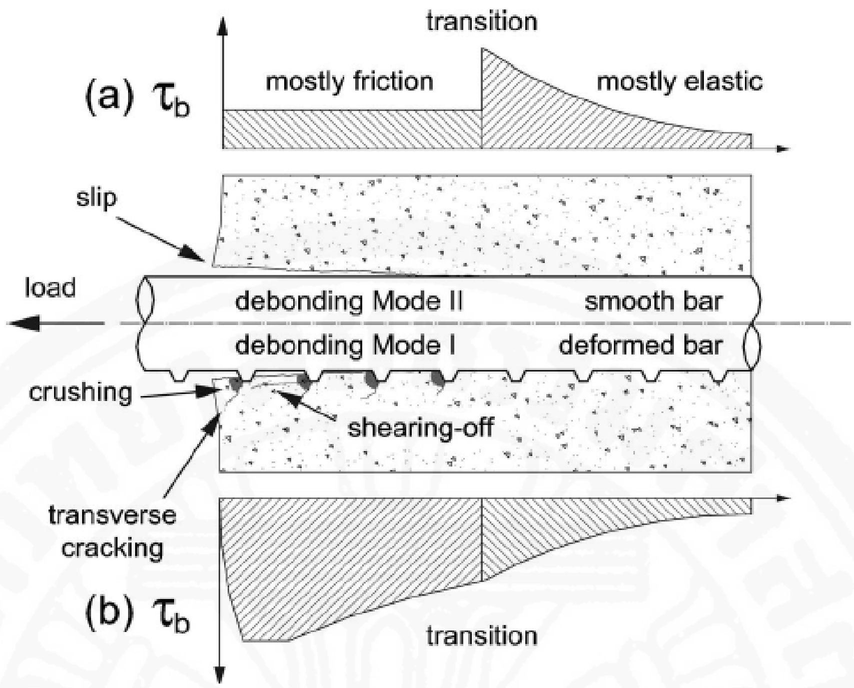

Fig 2.15: Bond mechanisms in Bamonte and Gambarova (2007)

a) smooth bars
b) deformed bars

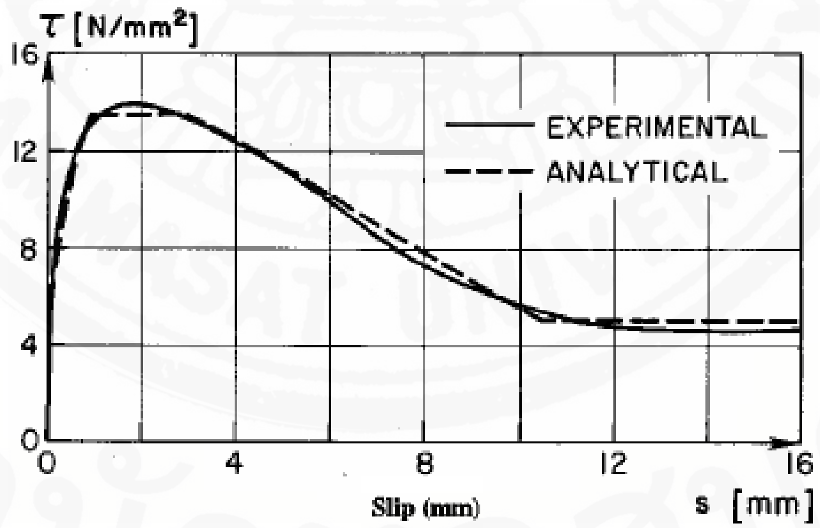

Fig 2.16: Bond stress-slip model proposed by Eligehausen, Popov et al. (1983)

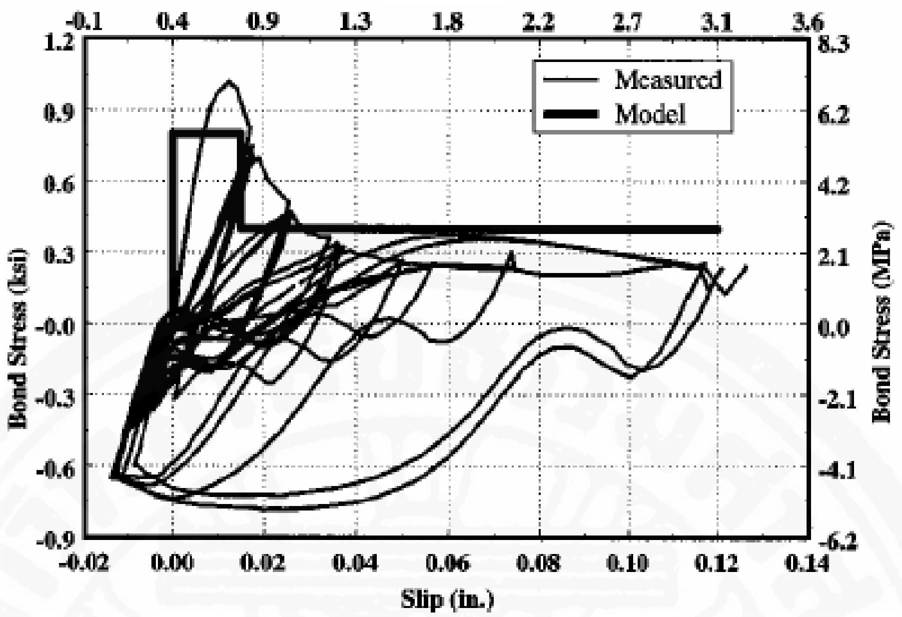

Fig 2.17: Bond stress-slip model proposed by Lehman and Moehle (2000)

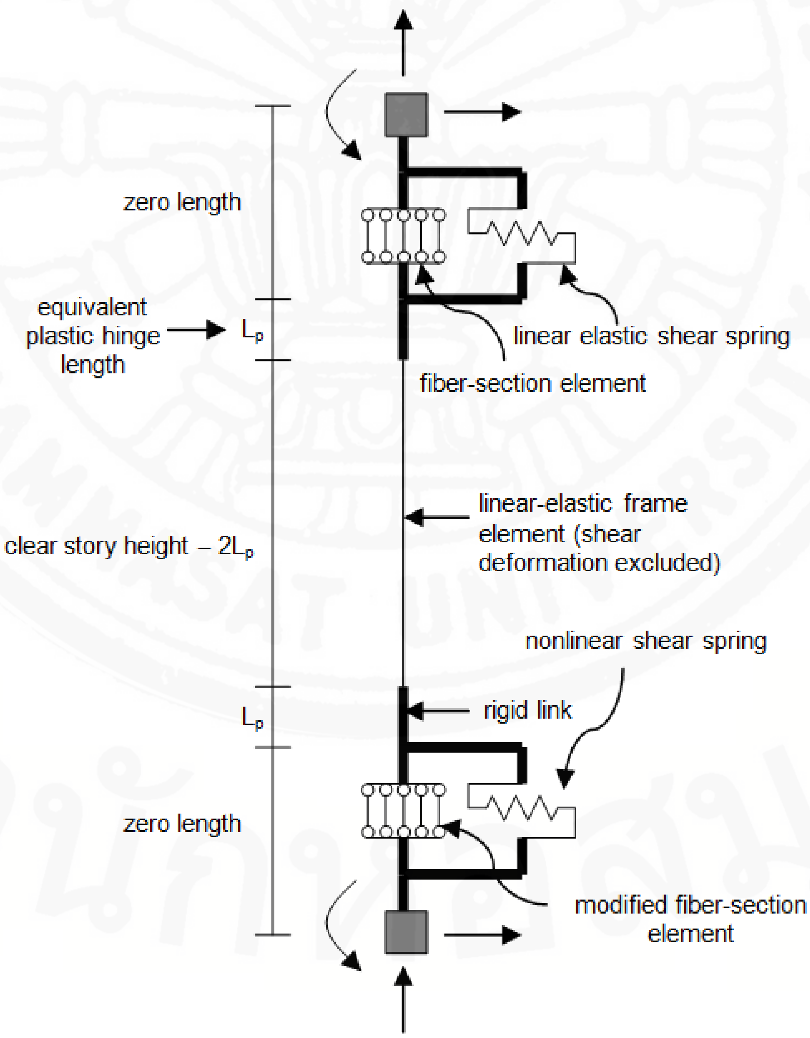

Fig 2.18:RC column model, Matrin (2007)