CHAPTER 2

Theoretical Background and Literature Review

2.1 Basic concepts

Connection is one of the most important components in prefabricated structures. The behavior of a whole precast building mainly depends on the response of the connections. There are two important factors in seismic design (Englekirk, 2003), ductility and shear transferred mechanism, reflecting the structural behavior during earthquake event.

Park (1988) described the ductility factor defined as the ability of a structure after the maximum capacity under cyclic large deformations, without a significant reduction in loading capacity. The factor also performs to significantly reduce the transmitted energy during those cyclic deformations, exhibiting as dissipating energy. Furthermore, the effect of ductile characteristic to the structural system has traditionally been used to design an elastic assumption of the seismic structure such as static linear and dynamic linear procedures.

Because any external load applying to a structure must pass through structural components by transferring at the connections between them. Shear transfer mechanisms are important required at the connection between precast elements. It is critical to the effective use of precast concrete structure under seismic load path. It is not appropriate in the prefabricated concrete structures. An understanding of their response, mechanisms and limit states is essential to develop the load paths in the precast concrete systems. To limit states identified, the experimental program under equivalent seismic loading need to be tested in each precast detail.

2.2 General

In general, precast concrete structure could be currently categorized into moment resisting frame, portal frame and wall panel frame. The study focus on the moment resisting frame because it is the most challenging among the precast structural categorization, both architecturally and structurally. In the moment resisting frame, it is assembled by several precast elements such as beams and columns. Those elements are suitably connected by using connections of each element, to form the space-frame for resisting gravity and lateral load.

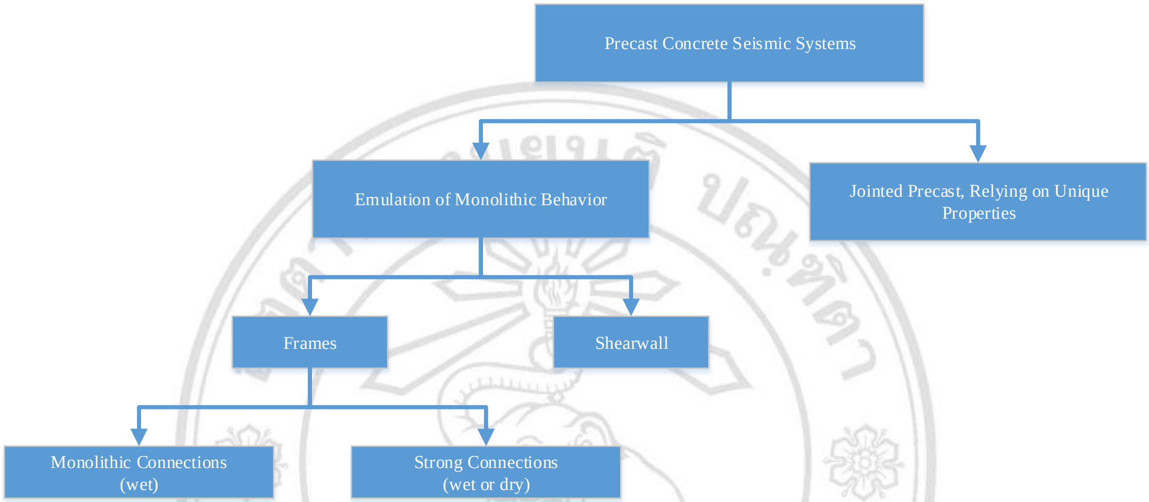


Figure 2.1 Precast concrete seismic systems.

Normally, precast connections in the concrete moment resisting frame can be divided into two fundamental groups (Ataköy, 1998); wet and dry connections, depending on the installation process. Wet connection is constructed to join the precast elements together using cast-in-place concrete. For dry connection system, the precast elements can be connected using bolting, post-tensioning or welding with steel plates or other steel inserts embedded into the edge of each precast concrete element. Considering the overall behavior of the precast concrete structures, the connections are the most important part because the behavior of a whole precast building greatly depends on the response of their connections. The NEHRP provision (1994), UBC1997 (1997) and FIB bulletin 27 (2003) classify the categories of precast concrete connection for the moment resisting frame into the equivalent monolithic system and the jointed system as shown in Figure 2.1. The equivalent monolithic is emulated of behavior of monolithic reinforced concrete structure while the other one considers the unique properties of each precast element interconnected by using the dry connection.

In 1997, Ghosh *et al.*(1997) proposed a strong connection concept according to UBC1997 design provisions of precast structures in seismic zones. The strong connection is sub-division of the emulating behavior of monolithic reinforced concrete. For the design concept of the connection, it is considered to remain elastic behavior while the other elements are designed in nonlinear response under designed ground motion. The connection has to be prevented from inelastic action such as yield and slip. Therefore, both nominal flexural and shear strength must be greater than the actual bending moment and shear force, respectively. Furthermore, the nominal strengths of the connection must be higher than the other elements. Furthermore, ACI 550.1R-01 (2001) proposed to design the emulating cast-in-place detailing in the precast concrete structures in the seismic region. For special attention, it is directed to detailing the joints and splices between precast elements.

Regarding the use of precast concrete connection in multi-storey building in Thailand, the connection with design philosophy in PCI handbooks (1985) have been widely used. It is well known that there are two important factors e.g., shear transfer mechanisms and ductility, for effective seismic design of monolithic concrete and precast concrete structures. Currently, the emulation of monolithic concept have been normally used for the precast construction. Most precast connections were designed to specifically support the gravity load without considering seismic effect. Furthermore, the design criteria of those beam-colunm connection is assumed to be a simple beam span or determinately structural system having the only shear mechanisms. There is no ductile consideration in the precast structure subjected to cyclic lateral force, resulting to the abrupt collapse during a strong ground motion. Figure 2.2 shows an example of precast connection which has been used to construct small buildings in Thailand seismic region.

(a) Welding type of precast connection

(b) Precast beam-column connection

Figure 2.2 Example the precast connections in seismic region of Thailand

2.3 Precast connection under cyclic loading

There have been several researchers who have made new discoveries to the precast concrete connections in the last decade. Welded connections have been widely used because of their easy application and lower cost advantages. Pillai and Kirk (1981), Bhatt and Kirk (1985) studies showed that the welded precast concrete connections tested in these studied, from considerations of strength, stiffness, ductility and energy-dissipating capacity, performed satisfactorily and comparable to the performance of similar monolithic connections. In this detailing as presented in Figure 2.3, T section was used in the column and the anchor bars were welded to the horizontal leg of the T section.

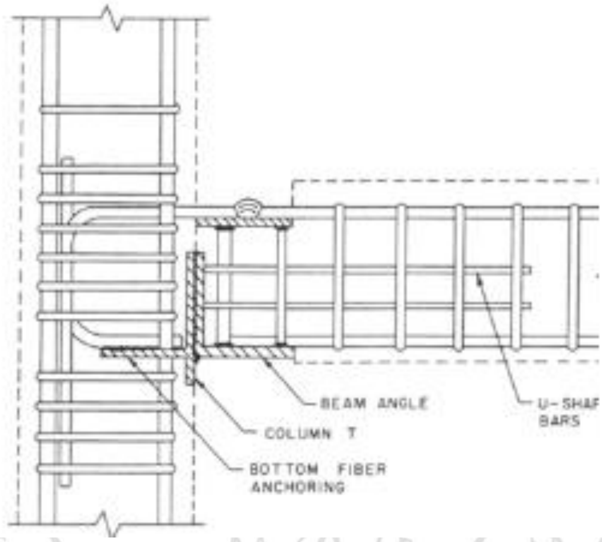
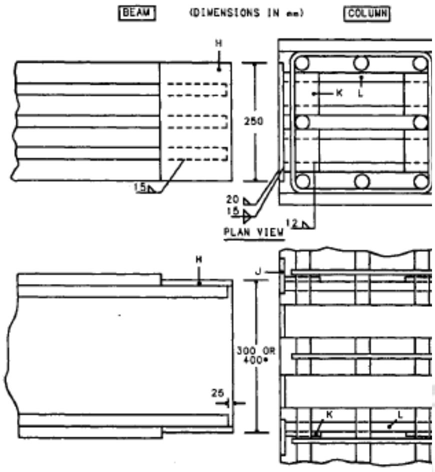
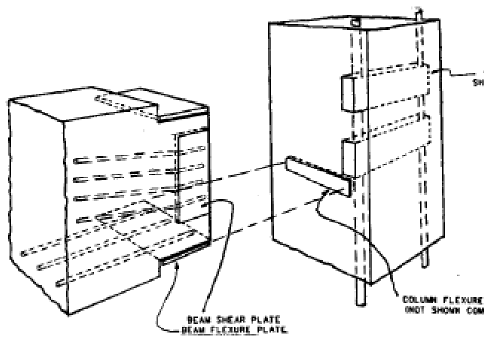


Figure 2.3 Welded connection at column face (Bhatt and Kirk, 1985)

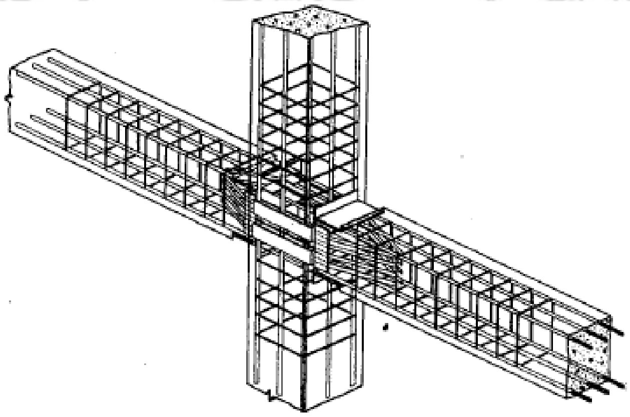
In 1990, Seckin and Fu (1990) presented an experiment investigation that conducted the behavior of semi-rigid precast beam-to-column connections subjected to simulated seismic forces. The four full-scale interior beam-to-column subassemblies were tested to observe a seismic behavior. The four specimens were consisted of one monolithic specimen and three precast specimens. For the monolithic one, ACI 352R-85(1985) was considered to design and detail the joint reinforcement according to Type2 joint definition. For the precast specimens, the connection detail as show in Figure 2.4 was consisted of top and bottom horizontal steel plates to resist flexural stresses and vertical middle plates to resist shearing forces. The beam and column elements were assembled by welding the shear and flexural plates to the exposed column plates. The test results were found that precast beam-to-column connection exhibited better stiffness characteristics and energy dissipation and negligible slippage of beam bars.



(a) Flexural connector



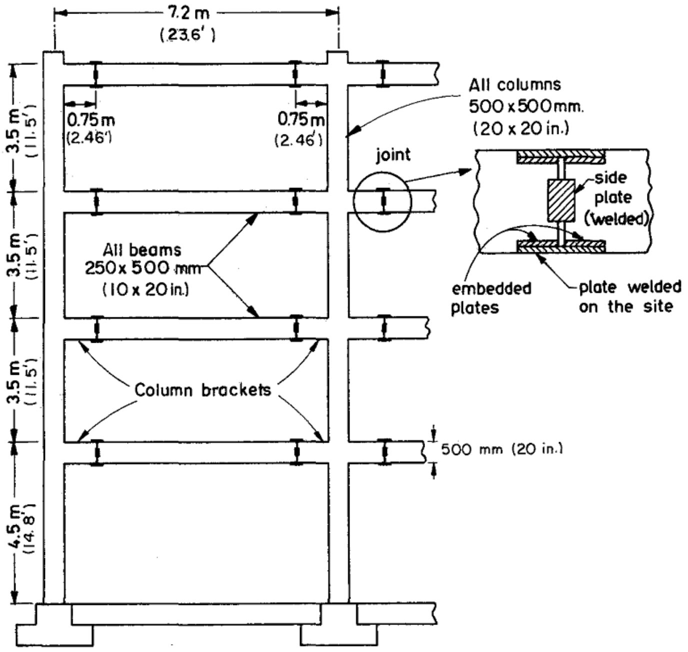
(b) Connection details



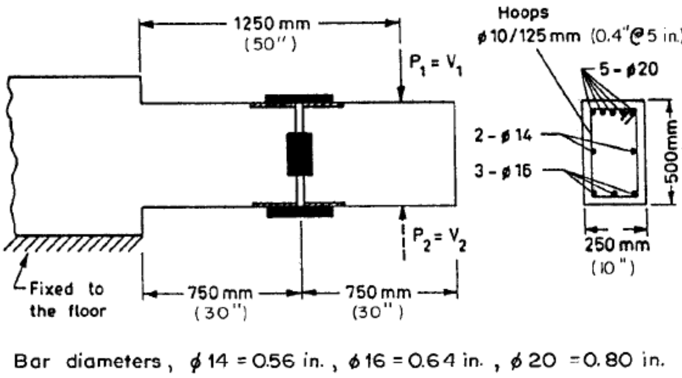
(c) Precast beam-column specimen

Figure 2.4 Test specimens of Seckin and Fu (Seckin and Fu, 1990)

For previous study concerning a beam-to-beam connection, Uğur and Tuğrul (1993) conducted the performance of precast concrete beams with dry joint. The five precast specimens and two monolithic reference specimens were tested under reversed cyclic loading to study their response under seismic action. For the precast connection, it was intended to transfer both shear force and bending moment. The main connection detail consisted of two steel plate, one at the top and other at the bottom as shown in Figure 2.5. The plates were welded to the anchored steel plates into the column and the beam. Also, added side plates were installed in only three specimens to compare the seismic response in the precast specimens with/without the side plate. The concrete joint width was varied in the program. The test result revealed that the width of the joint was an important parameter during subjected to reversed cyclic bending, especially. The proposed joints had adequate stiffness and behaved satisfactorily. For the side plates, it was mandatory to be subjected to reversed cyclic loading. The precast specimen with the side plates showed the reduction of the deformation and improvement in the loading capacity, compared with the precast specimens without the side plates.



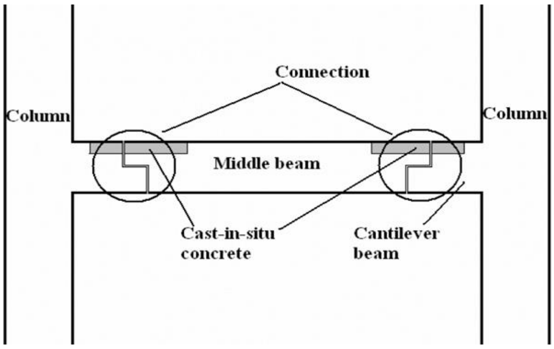
(a) Prototype of precast concrete structure



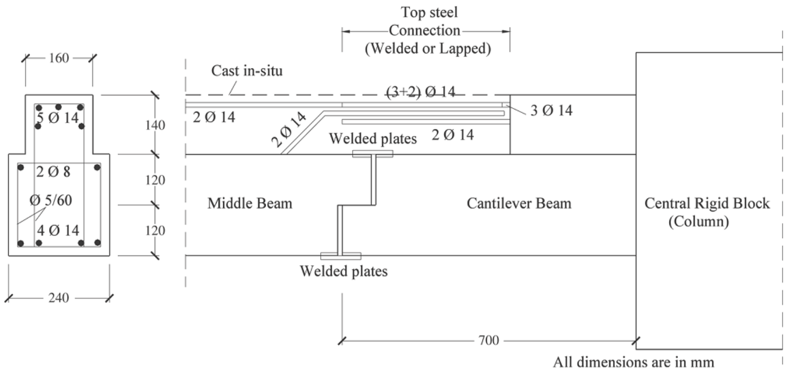
(b) Connection detail of test specimen

Figure 2.5 Study program of Uğur and Tuğrul (1993)

Kormaz and Tankut (2005) improved the seismic performance of the precast concrete beam-to-beam connection in highly seismic regions as shown in Figure 2.6. Six beam-beam connection subassemblies scaled to approximately 1/2.5 were investigated under reversal cyclic loading. The first specimen as monolithic concrete was used to be a reference specimen. The second specimen was a precast specimen with connection detail commonly used in Turkey, proposed by the collaborating company. The others with different connection detail were modified according to the test results of the formerly specimens. In additionally, the beam reinforcements were varied in the case of reasonable ($\rho \approx 0.015$) and heavily ($\rho \approx 0.020$) reinforced beams. Especially, the connection details were composed mainly of welded plates and lap splices. The test results showed that the connecting of the top reinforcement via welding is able to solve the problem of anchorage. In regard to the bottom connectors, the original detail appeared to be insufficient. The modified bottom connectors were improved by strengthening, and it exhibited a satisfactory test results. There was a similar study by considering a composite section at connection region. Yang *et al.* (2010) developed hybrid precast concrete beams with H-steel beams at both ends to create a simple ductile connection, particularly useful for precast concrete structures. Three precast concrete beams were tested under two-point concentrated loads to explore the effectiveness and limitations of the developed hybrid beam system in transferring an applied flexure to a supporting column. Also, effect of pre-stressing force on the flexural behavior of the beam was observed in the study. The results showed that the pre-stressing force in longitudinal tension reinforcement significantly improved the ductility and flexural strength of the hybrid precast concrete beams. Furthermore, no slippage at the interface between two concretes casted at different time was developed; and no crake was observed around the composite section region. In 2006, Khoo *et al.* introduced a modified assembled configuration for emulative precast concrete frames with wet connection on the beam elements, to approach the "strong connection". As shown in Figure 2.8, the connections were kept away from the face of columns, to avoid coinciding with the plastic hinge regions under seismic loading.



(a) Precast frame and connection under study of Kormaz and Tankut



(b) Dimensions and reinforcement detail of the test specimen

Figure 2.6 Study program of Kormaz and Tankut (2005)

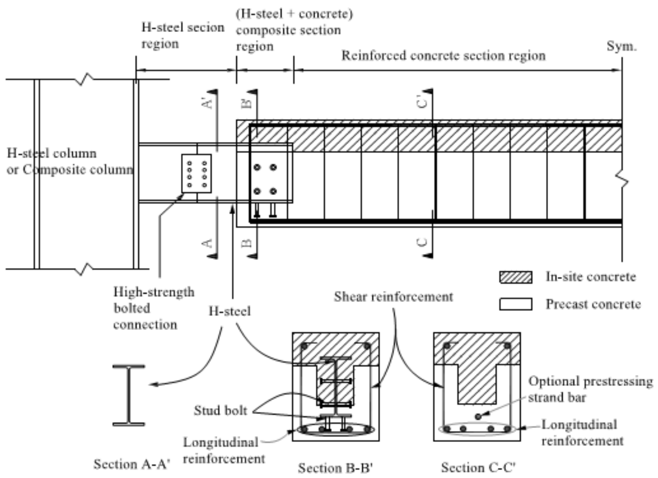
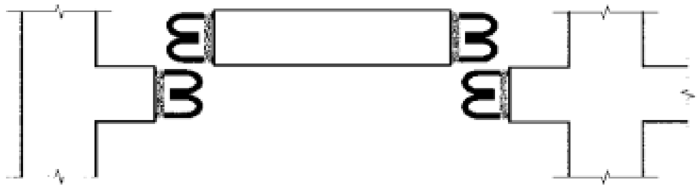
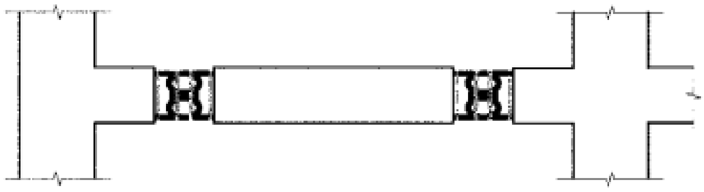


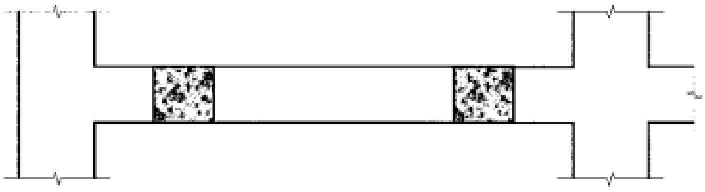
Figure 2.7 Detail of developed hybrid precast concrete beam system (Yang *et al.*, 2010)



(a) Precast components



(b) Connections established through overlapping hooks



(c) Cast-in-place with concrete

Figure 2.8 Precast concrete frame with modified assembling configuration

(Khoo *et al.*, 2006)

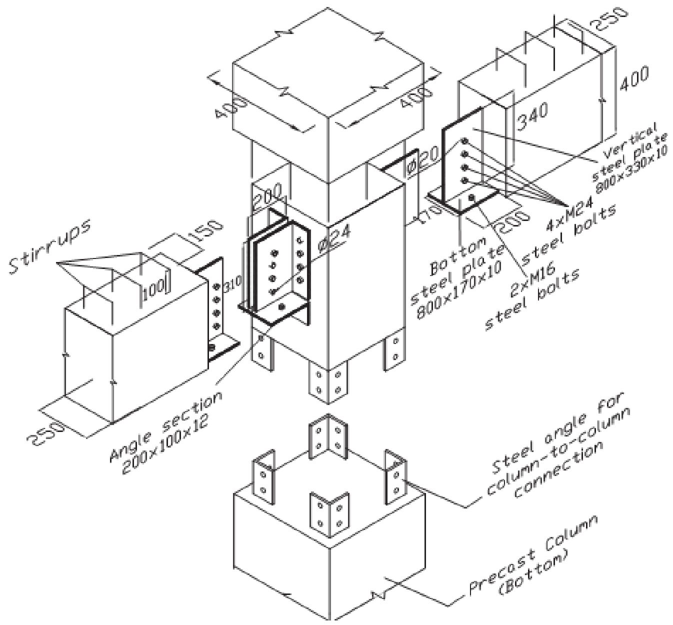
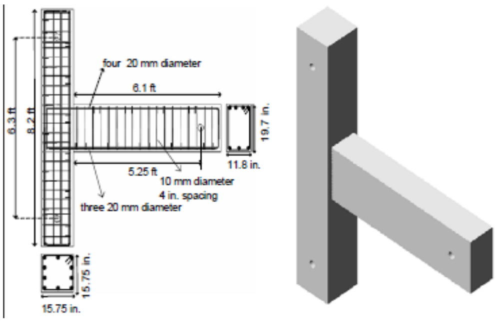
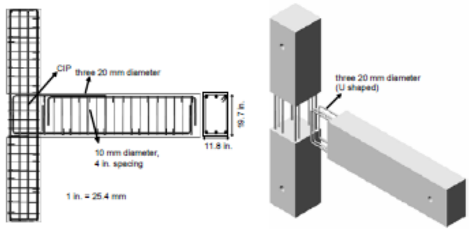


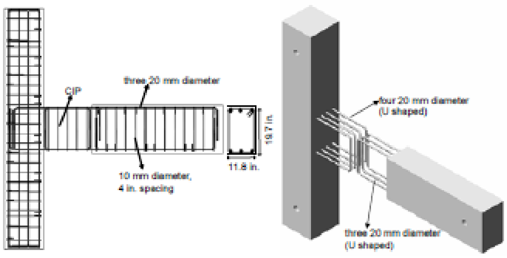
Figure 2.9 Typical connection detail (Li *et al.*, 2009)



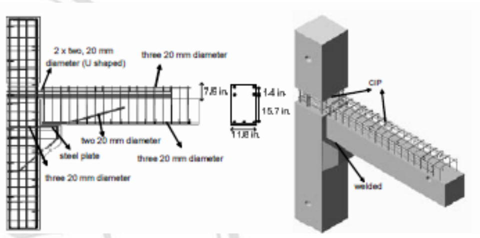
(a) Specimen M



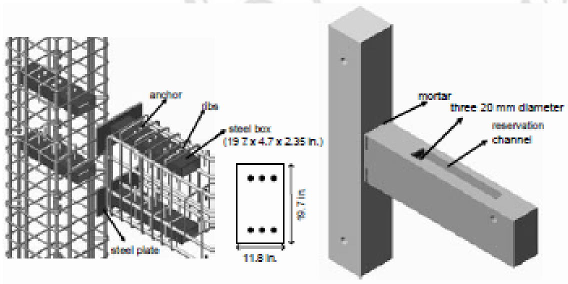
(b) Specimen CIPC



I Specimen CIPB



(d) Specimen GOK_R

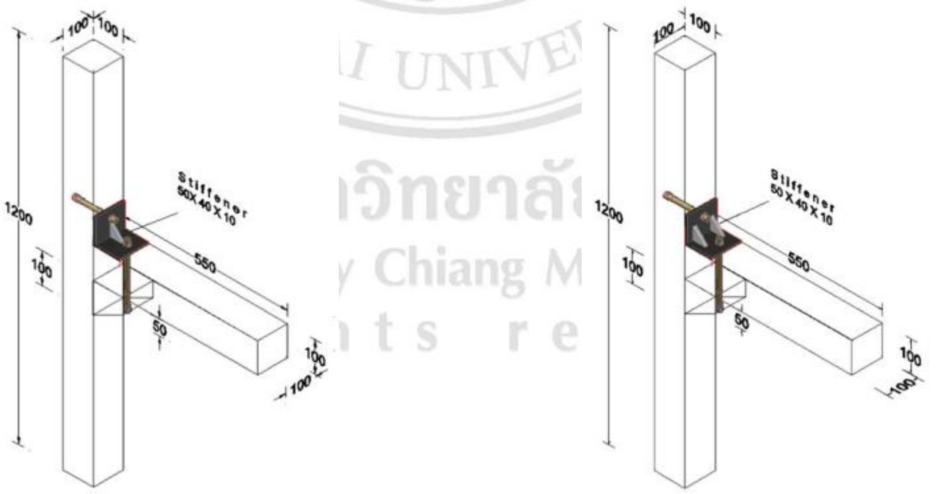


(e) Specimen Mod-B

Figure 2.10. Test precast specimens of Onur *et al.* (2006)

Regarding studies of beam-column precast connections in moment-resisting frames, Li *et al.* (2009) studied the seismic performance of precast hybrid-steel concrete connections under cyclic loading. Four full-scale specimens, one monolithic and three precast specimens, were tested. The typical connection detail in all precast connection is shown in Figure 2.9. The column-to-column connection was connected by using four angle sections partially embedded in the column. For assembling of the beam to joint core, unequal angle section of size 200x100x12 mm with partially embedded steel plate as 800x330x10 mm and 800x170x10 mm for vertical and bottom plates, respectively, were used to connect by using bolts. The experimental observations showed the precast connections without abrupt damage within the joint core region, exhibiting adequate ductile behavior and were considered acceptable in comparison to the monolithic one. Embedment of the steel sections in the joint greatly enhanced the strength of the joint-core leading to the increase of ductility factor to 3.50.

Furthermore, six exterior precast beam-column connections were tested at Boğaziçi University in Turkey by Onur *et al.*(2006). As shown in Figure 2.10, the six connection details could be subdivided into three groups, namely cast-in-place, composite with welding, and bolted. They were observed the seismic performance under cyclic loading pattern according to ACI T1.1-01(2001). During the assembly process of the composite connection, the corbel and beam were connected by welding together the embedded steel plates which was continuous the beam bottom reinforcement. The hairline concrete cracks parallel to the bar axes in the vicinity of the weld location were observed, leading to the bond damage at the location. The test results reveal that the hysteresis behavior of cast-in-place connections are similar to those of monolithic specimen. While the composite with welding connection is inferior the other types of connection because excessive welding adversely affects the mechanical properties of the reinforcement.

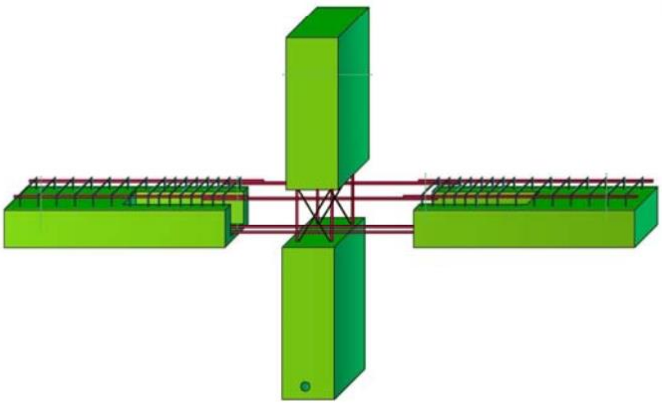


(a) Precast connection with single stiffener (b) Precast connection with double stiffener

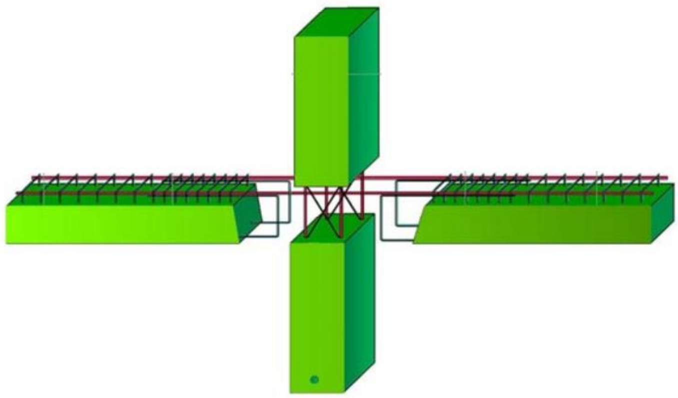
Figure 2.11 Precast specimen with corbel and stiffener using steel cleats

(Vidjeapriya and Jaya, 2013)

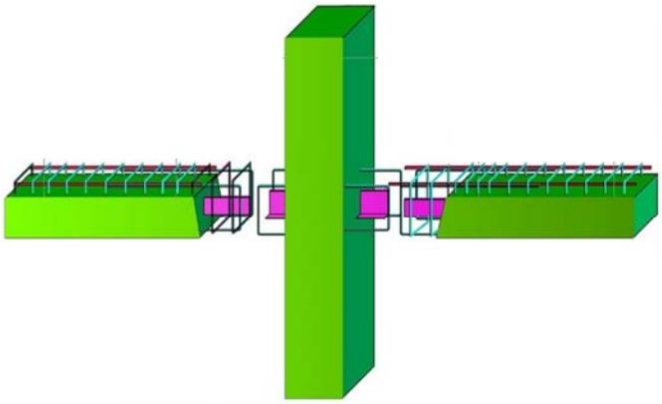
Vidjeapriya and Jaya (2013) presented the experimental results of two precast concrete beam-column connections compared to the monolithic connection under cyclic loading. The precast beam was connected to the column with corbel using a cleat with a single stiffener for the first precast specimen, and double stiffener for the second specimen as shown in Figure. The test results revealed that the ultimate loading of the monolithic specimen was better than the other precast specimens. The energy dissipation and ductility of both precast specimens exhibited satisfactory behavior. In a similar study, Shariatmadar and Beydokhti (2014) investigated a seismic response of three precast interior subassemblies to compare with a monolithic connection. All precast joints were assembled by using cast-in-place connections with different details; namely straight sliced, U-shaped, and U-shaped with a steel plate. Comparisons of seismic performance showed that the seismic performance of the precast specimen with straight sliced and the monolithic specimen were similar and can be suitable for use in high seismic zones.



(a) Straight spliced specimen



(b) U-shaped rebar specimen

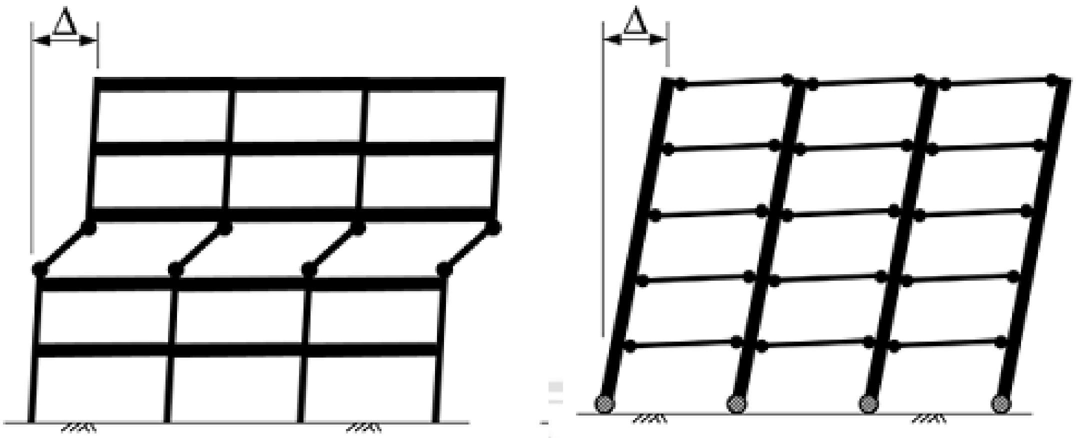


(c) U- shaped rebar with plate specimen

Figure 2.12 Test precast specimen (Shariatmadar and Beydokhti, 2014)

2.4 Relocating plastic hinge in concrete beam and strong column-weak beam frame

During earthquake phenomena, the building is immediately swayed from backward and forward as dynamic pattern. The seismic loading distribution of the structural mainly depends on the stiffness in each elements and damage distribution. Most of traditional structural systems, the stiffness and loading capacity of the beam elements are less than the column and the beam-column joint. When the structure sways under earthquake ground motion, the story drift concentrically tends in a few stories, leading to exceed the story drift capacity and to generate the nonlinear plastic hinge of the columns as shown in Figure 2.13(a). On the other hand, if the columns spine over the building height are prepared to be stronger in terms of stiffness and strength than the beams, the distribution of inter-story drift is more uniform as shown in Figure 2.13(b), resulting in the reduction of localized damage. It’s well known that a role of column elements in each story has to carry the gravitational weight of the entire building above the story. Consequently, the column failure is of greater importance than the beam failure. Thus, current building codes for considering seismic design provide the strong column-weak beam principle which the columns should be stronger than the beams, to achieving safe behavior of the whole structure during strong earthquake ground motion.



(a) Story mechanism (b) Beam mechanism

Figure 2.13 Design of special moment frames (Moehle J.P. *et al.*, 2008)

In the seismic design provision such as NZS 3101-2006(2006), ACI 318-14(2014), GB50011-2010(2010), and etc., the strength requirement of beams and column have adopted to examine the strong column-weak beam that summation of column flexural capacity have to exceed the sum of beam flexural capacity at the beam-column joint of a special moment frame. There are the study of Kuntz and Browning (2003), and Moehle (2014) that showed the full structural mechanism of Figure 3(b) by defining the column-to-beam strength ratio which should be greater than 1.20. The strength ratio has been widely used for controling a potentail plastic hinge generated in the beam elements.

In current design code, the plastic hinges are usually formed in the beam regions adjacent to the beam-column joint. The inelastic deformations at the column face such as high straining of longitudinal bar and cracking concrete are able to penetrate into the joint, severely leading to bond strength deterioration between the reinforcing bar and the surrounding concrete (Zhao and Sritharan 2007, Elmenshawi *et al.* 2012, Yan and Au 2010; Rutledge *et al.* 2013). There were several previous studies (Park and Milburn, 1983, Al-Haddad 1990, Paulay and Priestley 1992, Al-Ayed *et al.* 1993, Galunic *et al.*, 1977, Scribner and Weight, 1978, Briss, 1978 and Wight and Al-Haddad, 1987, Yi *et al.* 1996, and Derecho and Kianoush, 2001) to solve the beam-column connection problem by technique of moving the beam plastic hinge far away from the column face as shown in Figure 2.14. In general, the expected critical section in beam elements is mostly provided at one beam depths away from the column face. Additional reinforcement of cross-diagonal rebar which is one of the relocating techniques has been often installed to move the plastic hinge in concrete structural elements as shown in Figure 2.14(a). the use of the bent and the straight reinforcements (Figure 2.14(b) and (c)) are also used to additionally generate the stiffness of the concrete joint and the beam ends. The nominal flexural strength of beam at the location is diametrically increased around 1.25 times the nominal strength of the conventional section. Figure 2.14 (d) shows the non-prismatic section by increasing depth of the beam ends near the column which is used to generate higher flexural capacity in the section remaining elastic behavior. While the inelastic hinge is formed at the critical section. A strengthening method for enhancing shear and moment capacities is capable of achieving the plastic hinge relocation by wrapping Fiber Reinforced Polymer (FRP) sheet on the concrete joint and the beam sections near the column face as shown in Figure 2.14(e). The wrapped section remains elastic region while the unwrapped section reaches the inelastic region by forming the plastic hinge at interface between both sections. The technique of using Single Slotted Beam (SSB) was early proposed by Oudah and El-Hacha(2016) as shown in Figure (f). The virtical slot was installed away from the concrete joint to relocate the center of rotation of the beam.

The use of the intermediate longitudinal beam reinforcement for relocating the potential plastic hinge in monolithic subassemblages was introduced in the studies of Scribner and Weight, Wight and Al-Haddad, and Yi *et al.* (1996). The stuies can be concluded that the ratio of one layer of intermediate bar to the tesion reinforcement (A_i/A_s) should be approximately 0.30 to 0.35, to avoid large shear forces in the joint. The stiffness of beam at the additional reinforcing region was signifigantly improved. Moreover, small-diameter bar should be used as the intermidiate reinforcement to distribute the flexural-shear cracks over a wide lenght of the beam. Regarding another study similar to those studies, Chutarat and Aboutaha (2003) proposed a use of headed bars, which were reinforced in the beam through the concrete joint region as shown in Figure 2.16. The additional headed bars were used to move the expected plastic hinge away from the column face during equivalent sesimic force. The study can be concluded that the use of Headed bars is capable to relocate the beam plastic hinge, resulting in an increase in beam shear demand.

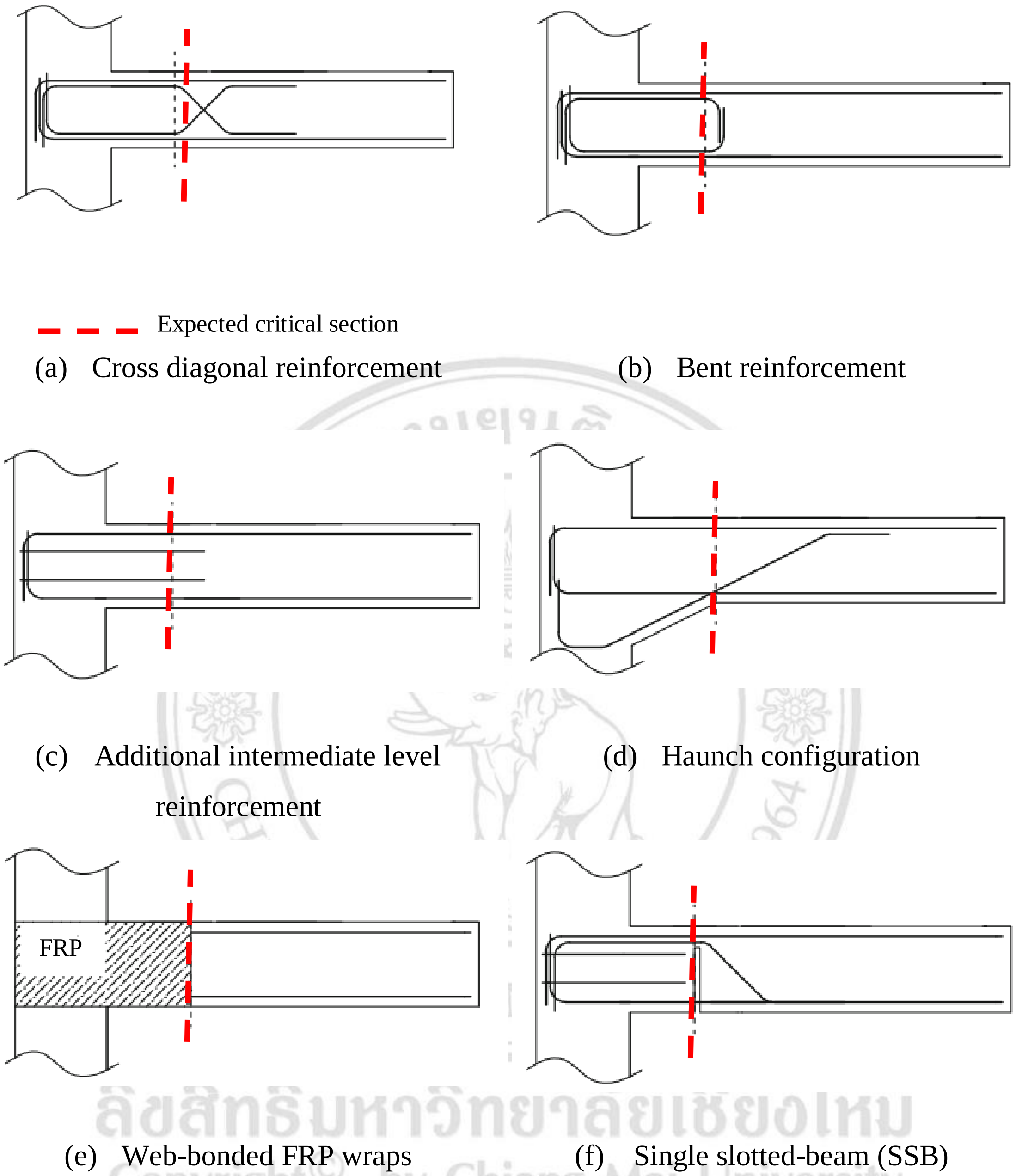
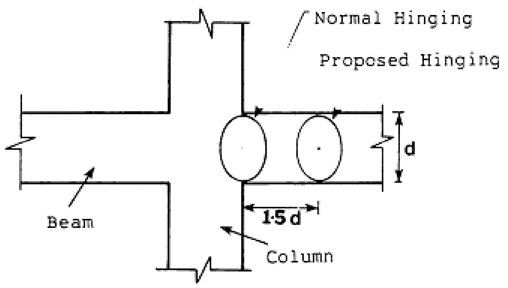
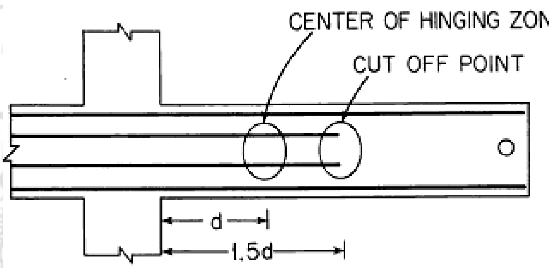


Figure 2.14 Techniques of plastic hinge relocation (Oudah and El-Hacha, 2016)

There was another concept to relocate the plastic hinge. The studies of Ohkubo *et al.* (1999) early intoduced a slotted beam as shown in Figure 2.17 – 2.19, to limit the cracking in the planned beam-end yield sections. The slotted beam, vertical narrow slot, was installed between the beam and the column. The slot was run from the bottom of beam for about ¾ of beam depth. Then, Oudah and El-Hacha(2016) developed single and double slotted-beam detailing technique (SSB and DSB), used to relocate the plastic hinge in concrete frame by moving the vertical slots away from the face of column as shown in Figure 2.20. Base on experimental results, the plastic hinge were succcessfully relocated at the expected location, resulting in the strain level and concrete cracking at the column face significantly reduced. It led to reduce permanent shear distorion and strain in the concrete joint. Forthermore, insignificant degradation in the stiffness causing pinchicing behaviour was observed.

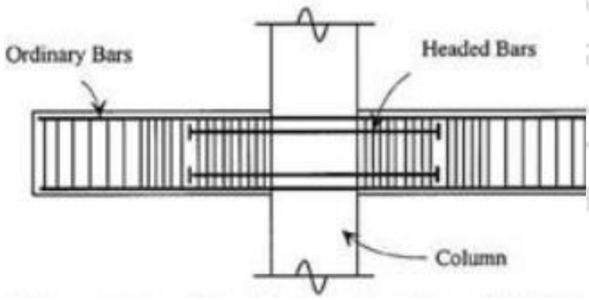




(a) Possible beam plastic hinge location (b) Arrange of intermediate beam reinforcement

Figure 2.15 Concept of moving beam plastic hinging zone

(Wight and Al-Haddad, 1987)



(a) Relocating plastic hinge in the beam by using Headed Bars (b) Headed Bar

Figure 2.16 Relocating plastic hing by using Headed Bars

(Chutarat and Aboutaha, 2003)

Furthermore, there were several studies (Mahini and Ronagh 2011, Dalalbashi *et al.* 2012, Eslami *et al.* 2013, Zarandi *et al.* 2015, and Behnam *et al.* 2015) presenting an instaliation of composite material such as Fiber Reinforced Polymer (FRP) and Carbon Fiber Reinforced Polymer (CFRP) for retrofitting of reinforced concrete jo int and relocating of inelastic beam hinge away from the column faces. The use of composite layer application at beam-colum joint is improvement of confining pressure and increasement of stiffness for the concrete structrure in the location. The studies show that the application is not only capable of improving the loading capacity of the joints and relocating the plastic hinges in the beam elements but also capable of preventing the typical joint failure.

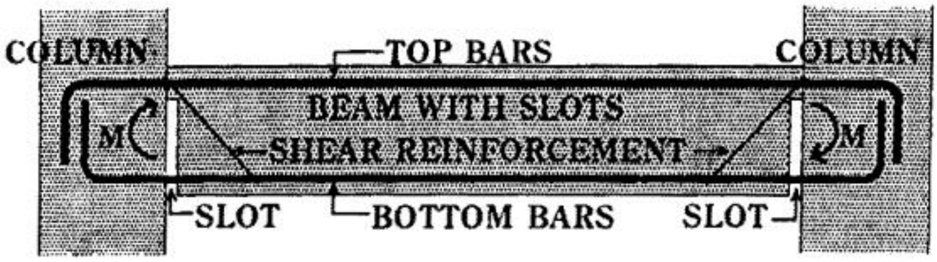


Figure 2.17 Conceptual illustration of the slotted beam (Ohkubo *et al.*, 1999)

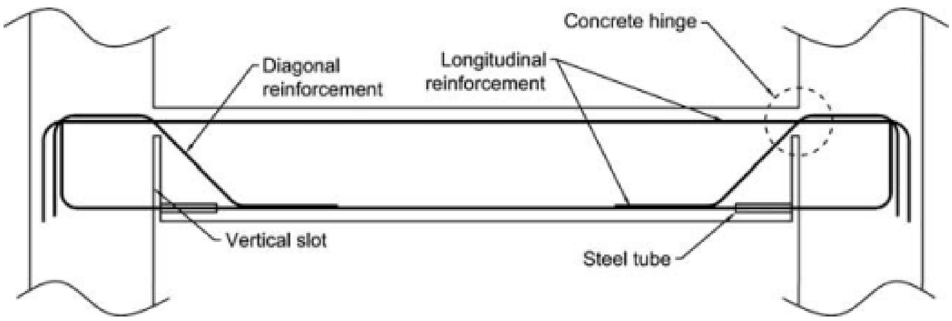


Figure 2.18 Single slotted-beam (SSB) (Oudah and El-Hacha, 2016)

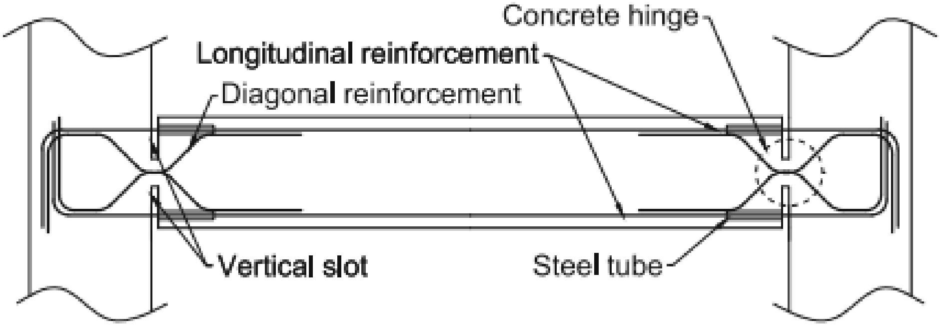
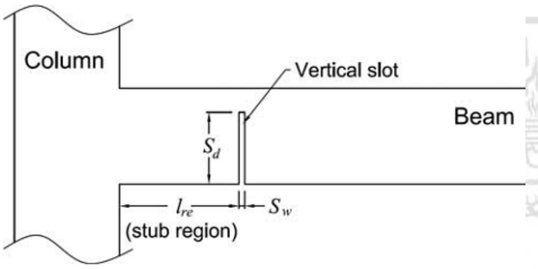
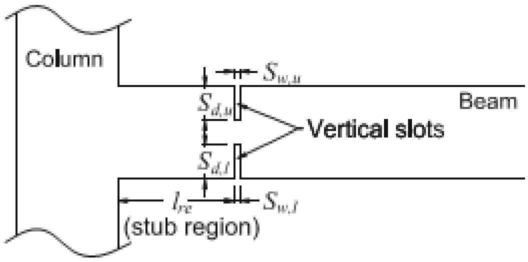


Figure 2.19 Double slotted-beam (DSB) (Oudah and El-Hacha, 2016)



(a) Single slotted-beam (SSB)

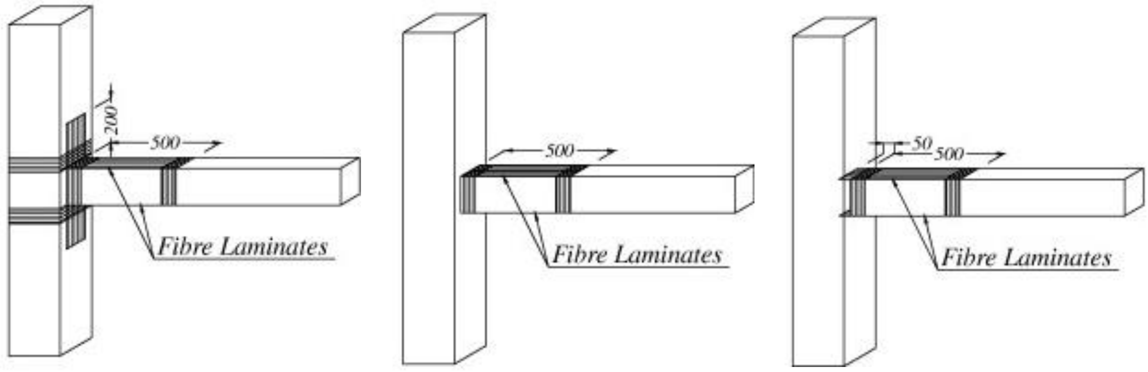


(b) Double slotted-beam (DSB)

Figure 2.20 Typical geometry with relocated plastic hinge

(Oudah and El-Hacha, 2016)

Regarding the study of plastic hinge relocation of emulative precast concrete structure, there was a research in the technique to relocate beam plastic hinges foe precast frame structure. Sucuoğlu (1995) presented a modified seismic design concept for moving beam plastic hinge away from the precast beam-column connection. The plastic hinges in beam are constructed by reducing yield capacities at the precast beam section as shown in Figure 2.22. It can be seen that the optimum distance for providing plastic hinge on beam element is the similarity both monolithic and precast structures. The beam plastic hinges should be located at one depths of beam away from the column face, to prevent the inelastic zone penetrating into the joint region. The relocating technique also effectively reduces the bonding stress within the joint.



(a) First design (b) Second design (c) Third design

Figure 2.21 Example of using the composite layers for relocating of inelastic beam hinge away from the column faces (Dalalbashi *et al.*, 2012)

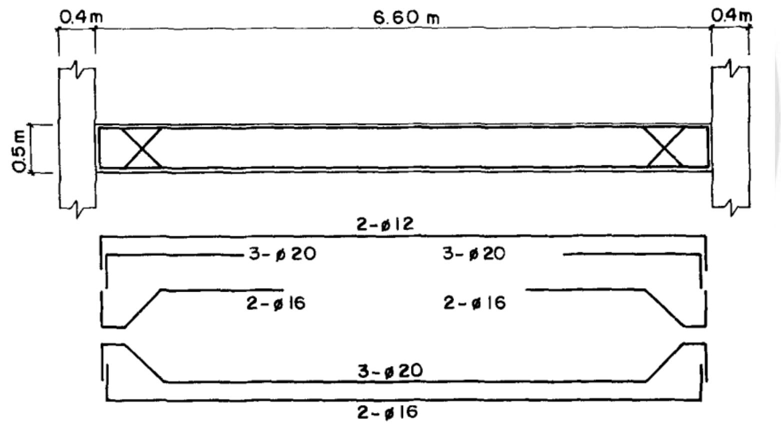
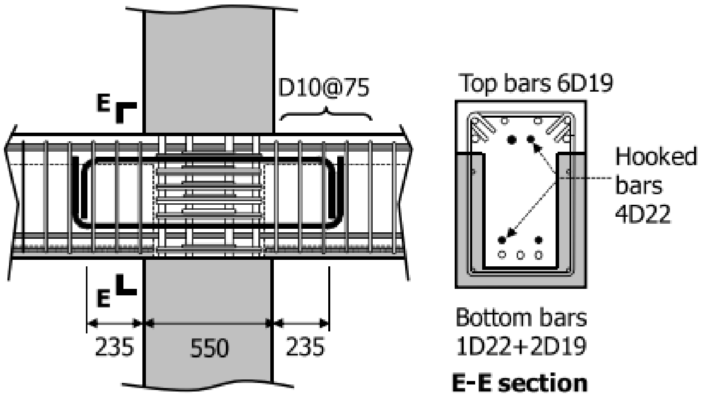
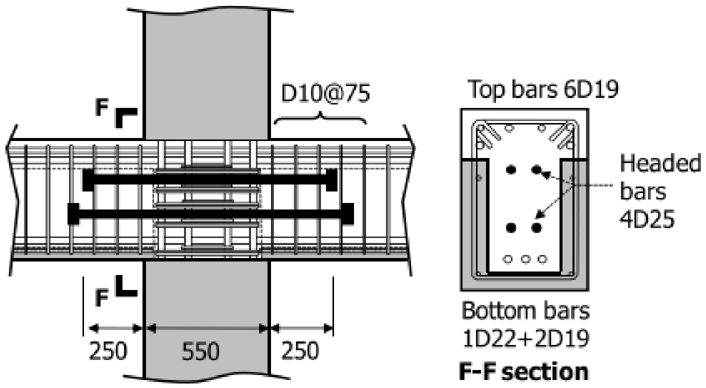


Figure 2.22 Construction the plastic hinge in a precast beam (Sucuoğlu, 1995)

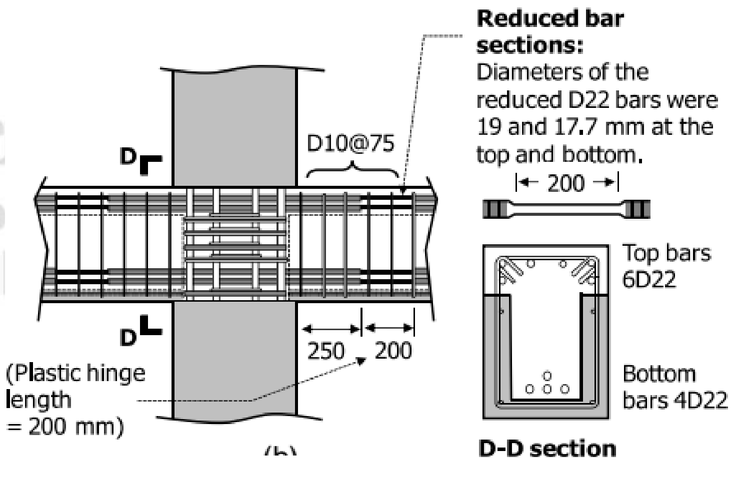
Eom *et al.* (2015) presented the techniques by using two strengthening methods (Hooked bars and Headed bars) and a weakening method (reduced beam bar section), to relocate the beam plastic hinging region away from the weak beam-column joint using U-shell PC beam as shown in Figure 2.23. Three precast specimens with the plastic hinge relocation were tested under cyclic loading and they were compared to the conventional RC and PC specimens without plastic hinge relocation. The study showed that the bond-slip of beam longitudinal reinforcements and joint shear cracking were significantly decreased due to the use of the relocating techniques. The energy dissipation increased, compared to the specimens without plastic hinge relocation.



(a) Strengthening method by using Hooked bars



(b) Strengthening method by using Headed bars



(c) Weakening method by reducing beam bar section

Figure 2.23 Plastic hinge relocation of PC beam-column connection by using U-shell beam (Eom *et al.*, 2015)

From the previous studies, it can be concluded about the major technique for relocating beam plastic hinging zone that the nominal flexural capacity of the beam section within a reasonable distance away from the column face have to be larger than the maximum anticipated moment capacity of the other beam section. The technique is able to avoid the inelastic deformations at the column face able to penetrate into the joint, severely leading to bond strength deterioration between the reinforcing bar and the surrounding concrete. Furthermore, the strong column-weak beam mechanism have to be provided in the concrete structure, to maintain the severe inelastic deformation in the column and joint elements. This study need to develop the relocating technique for precast concrete frame by using the information from these previous study as shown in the next chapters.

2.5 Finite element model

Numerical simulation has been widely adopted. This is due to the fact that the experimental investigation has been limited primarily due to its cost and lengthy time requirements. Due to the computational advantages compared to that of other modeling techniques, there were several studies (Taucer *et al.*1991, YU 2006, and Fragiadakis, Pinho and Antoniou 2008) performing fiber-based finite element modeling to predict hysteresis behavior and the load capacity of the reinforced concrete structures as shown in Figure 2.24. Normally, the stress actions over the cross-section can be classified into normal (consisting of normal forces and bending moments) and shear stress (consisting of shear and torsional forces). To separate total deformation into flexural and shear deformations, the normal stress causing flexural deformations can be computed on the basis of the Euler-Kirchoff hypothesis, i.e. a plane section remains plane. For the shear deformation resulting on the shear stress, it can be computed by relaxing the Euler-Bernoulli hypothesis. The validation of the assumptions had been experimentally verified for reinforced concrete structures (Ramamurthy 1966, and Furlong 1979). The cross-sections of the models were subdivided into longitudinal fibers with fiber area located in the local y,z reference system. The constitutive relations of specific materials were defined in the individual fibers, to observe the response of element cross-section under large deformation. Figure 2.25 shows the discretization of the a typical reinforced concrete cross-section.

Mander *et al.* (1988) was the first to introduce a sophisticated concrete model. It was a uniaxial nonlinear model including a constant confinement factor provided by transverse reinforcement. Martinez-Rueda and Einashai (1997) presented a new concrete model capable of predicting continuing cyclic degradation of strength and stiffness. To simulate the cyclic response of steel bars of reinforced concrete structures, Menegotto and Pinto (1973) proposed a uniaxial nonlinear model implemented in numerous programs intending to simulate the response of structures. In the early 1980s, the original reinforcing bar model was improved by Filippou *et al.* (1983 and 1983) to include the isotropic strain-hardening effects. The steel model has been widely used because it was capable (in numerical reinforcement concrete modelling) to reproduce the experimental results with acceptable accuracy.

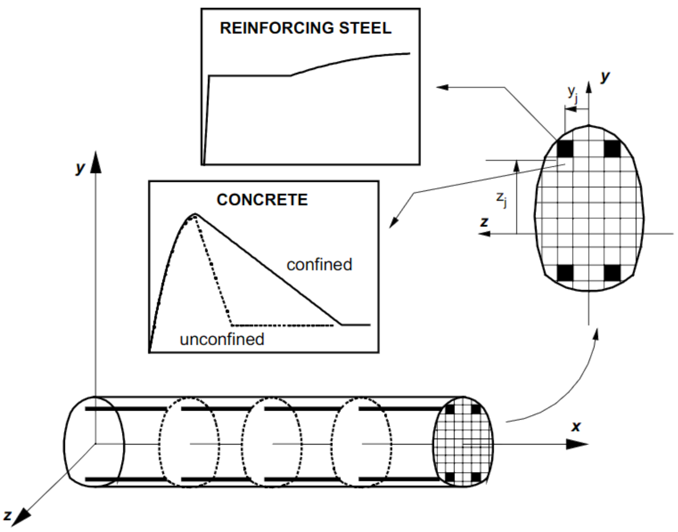


Figure 2.24 Fiber element: Distribution of control sections and section subdivision into fibers (Taucer, Spacone, and Filippou, 1991)

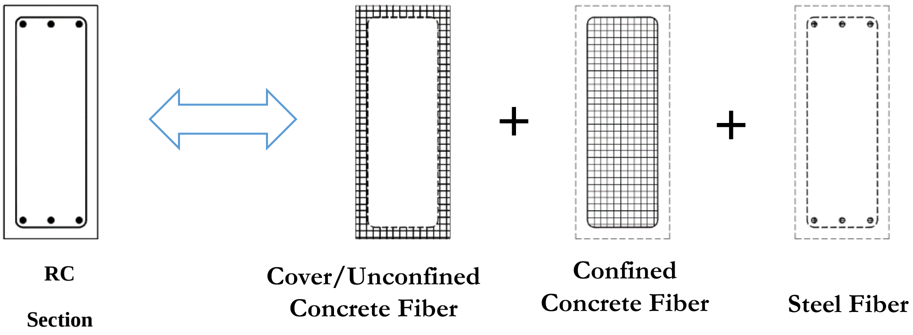


Figure 2.25 Discretization of a typical RC section

The beam and column elements are of obvious importance under seismic loading however, the joint zone is sometimes a more critical component to consider. To capture the failure in this zone, several studies developed the rational model of beam-column joint. Most concrete joint models performed in a numerical simulation have been proposed in consideration of the data from several empirical studies. For example, Alath and Kunnath (1995) presented a rigid link with a rotational spring model related to the rotation between the beam and the column. The spring models were calibrated by an inelastic shear-deformation relationship including degradation effect. A joint element consisting of a panel zone and four transition elements were proposed by Elmorsi *et al.* (1978). Moreover, bond-slip elements were provided to simulate the model responsible for connecting concrete material with reinforcing bars in both the panel zones and transition elements. However, the bond slip model was not considered for the effect of various confining ratios. Youssef and Ghobarah (2001) developed a beam-column joint model representing panel shear and bond-slip deformation. An equivalent moment rotational spring governing the relative rotation of beams and columns was proposed by Calvi *et al.* (2002). The rotational spring was adopted to simulate both linear and nonlinear behaviors of the beam-column joint. Beam and column elements were converged into the joint. In 2003, Lowes *et al.* (2003) presented an idealization of beam-column joint model capable of modeling the primary mechanisms for determining the inelastic behavior of RC beam-column joints under cyclic loading.