CHAPTER 2

LITERATURE REVIEW

2.1 Research on seismic evaluation methods

2.1.1 Review of seismic evaluation handbook and guideline

One of the most effective ways of minimizing potential earthquake-related losses is to conduct reliable assessments of the vulnerability of existing structures and to develop and implement effective ways to upgrade structures identified as hazardous (Miranda, 1991). There are many evaluation methods conducted in US. In 1987, Applied Technology Council (ATC) published a report on “Evaluating the Seismic Resistance of Existing Buildings” and it is named “ATC-14”. The methodology introduced in this report was developed on the basis of the state-of-the art/practice collected from various sources such as earthquake damage reports, existing and proposed provisions and consultant interviews.

In 1982, Building Seismic Safety Council (BSSC), the general guideline introduced in this report can be used to evaluate wood, steel, concrete and masonry buildings. The analysis procedures in this report base on calculating capacities and demands on certain structural and nonstructural element and checking against the capacity-demand ratio provided in the statement. These element capacities are calculated using appropriate building code provisions for the structural material being evaluated (e.g. UBC, AISC). The element demands are calculated by using equivalent lateral force procedures that reduced lateral force computed through linear elastic analysis referred to as linear static procedure or dynamic lateral force procedures using elastic response spectra referred to as linear dynamic procedure.

In 1992, Building Seismic Safety Council (BSSC) published the NEHRP handbook for the seismic evaluation of existing building, FEMMA-178. This report is prepared under the National Earthquake Hazards Reduction Programs (NEHRP). The evaluation method developed in this handbook basically based on the procedure recommended in ATC-14. In this handbook, the evaluation statements were improved by taking into account the actual mechanisms and failure modes of each type of structure. The analytical procedures were mostly the same as recommended in ATC-14.

More realistic approach was introduced in ATC-40 which was published in 1996. The analytical procedures shown in this report account for inelastic deformation of the structure by using nonlinear static pushover analysis. Inelastic earthquake demands are calculated by reducing the 5% damped elastic response spectrum. The spectral reduction factors are given in terms of effective damping. An approximate effective damping is calculated based on the shape of the capacity curve, the estimated displacement demand, and the resulting hysteretic loop. Moreover, this approach also presents rules for developing analytical models of existing concrete buildings which address the full range of concrete element and component behavior considering cracking, hinging, potential degradation and loss of gravity resistance. This report provides the numerical values of modeling parameters related with the potential degradation of each type of elements and components assembled in a building. In addition, this report also provides the guideline to model some factors that normally neglected in the analysis and design such steel.

In 1997, the Federal Emergency Management Agency (FEMA) published a "NEHRP guidelines for the seismic rehabilitation of buildings" and it is named "FEMA-273". The focusing on the rehabilitation of existing buildings in this report can be applied to steel, concrete, wood and masonry structures. There are four analysis procedures, linear static procedure (LSP), linear dynamic procedure (LDP), nonlinear static procedure (NSP) and nonlinear dynamic procedure (NDP), provided in this report and used to evaluate the existing buildings. The results of the linear procedures can be very inaccurate when applied to buildings with highly irregular structural systems, unless the building is capable of responding to the design earthquakes in a nearly elastic manner. Nonlinear procedures are especially recommended for analysis of buildings having irregularities. The nonlinear static procedure (NSP) is mainly suitable for buildings without significant higher-mode response. The modeling parameters and acceptance criteria of concrete buildings provided in this report are similar to ATC-40.

2.1.2 Simple seismic evaluation

There are many research conducted to study simple seismic evaluation. The method just use only simple data such as building configuration, dimension of column, beam and building area. By processing the data with testing criteria, seismic evaluation results can be obtained. The interesting researchs on simple seismic evaluation are as follow.

Hassan and Sozen (1997) propose a method to help select the buildings with higher seismic vulnerability in an inventory of low-rise monolithic reinforced concrete buildings located in the same region. The method requires only the dimensions of the structure as input and is based on defining the position of building on a two-dimensional plot using the wall and column indices. The wall index is the ratio of the effective wall area at the base of the building to the total floor area above base. The column index is the ratio of the effective column area at base to the total floor area above base. In the computation of the effective areas, 100 percent of reinforced concrete walls, 10 percent of nonreinforced infill walls, and 5 percent of column area are considered effective. The function of the proposed method is to rank a group of buildings with respect to the expected amount of earthquake damage. In the rehabilitation process, the ranking may be modified by the importance of the building and other knowledge about building properties. The salient attribute of the method is that it requires minimum of information and computation for filtering large inventory of low-rise monolithic reinforced concrete buildings to identify the fraction that should have priority for remedial action.

Yakut (2004) propose a preliminary procedure to assess rapidly the likely seismic performance of existing reinforced concrete buildings is presented. In this procedure, a Capacity Index is computed considering the orientation, size and material properties of the components comprising the lateral load resisting structural system. This index is then modified by several coefficients that reflect the quality of workmanship and materials, and architectural features. The procedure has been tested and calibrated based on the data compiled from damage surveys conducted after the earthquakes that occurred within the last decade in Turkey. The method classifies the buildings either as safe, meaning the building might suffer no severe damage or as unsafe, indicating that life safety performance level would not be met.

2.1.3 Analytical seismic evaluation methods

The four categories of building analysis methods are consisted of linear static (LS), linear dynamic (LD), nonlinear static (NS) and nonlinear dynamic (ND). The most simply method is linear analysis, LS and linear dynamic, LD, which are normally used by practicing structural engineers. However, these methods can not foretell the inelastic response of the building because the building components have infinite strength and constant stiffness during the analysis. For nonlinear analysis, NS and ND, a nonlinear building model simulating the strength and stiffness degradation of the buildings can be predicted realistically seismic behavior of building. However, nonlinear dynamic (ND) analysis method is lengthy complex (Miranda, 1996).

A nonlinear static or "pushover" analysis is one of the building analysis methods. This method requires that a nonlinear mathematical model of the building be subjected to monotonically increasing lateral forces until reaching a predetermined target displacement. The nonlinear static pushover analysis is mainly a performance evaluation procedure which is used as a tool to obtain further insight into the seismic behavior of structures that can not obtained from elastic static or dynamic analysis (Krawinkler and Seneviratna, 1997). The primary objective of pushover analysis is to obtain estimations of the global lateral strength, the global displacement ductility and the failure mechanism of a structure which is likely to experience in an earthquake ground motion. The obtained results are used to assess the integrity of the building system (Lawson et al., 1994). In addition, this analysis method is also applicable in determining the yielding distribution hierarchy and when it is used together with the demand spectrum, it may be used to predict the maximum deformation in the structure under the seismic action (Krawinkler, 1995). Recently, the versatility of using this approach was confirmed by Nagao et al. (2000), FEMA-273 (1997), ATC-40 (1996), Miranda and Bertero (1996) and Faella (1996).

The capacity spectrum method (CSM) was originally developed by Freeman (1975). Its concept has been introduced in several US guidelines for seismic evaluation and retrofit of existing buildings such as ATC-40 and FEMA-273. The capacity spectrum method incorporates the inelastic response of the structure in the analysis, but it is based on a quasi static approach which is amenable to current engineering practice (Deierlein and Hsieh, 1990). However, the capacity spectrum method is not well adapted for the analysis of structure.

In 1999, Chopra and Goel proposed a new method for calculating deformation demand of inelastic systems. They found that the deformation of an inelastic system estimated by ATC-40 did not converge for some of the systems analyzed and yielded inaccurate results compared with the deformation determined by nonlinear time history analysis. Therefore, they proposed an improved procedure that eliminated the errors and discrepancies in the ATC-40 procedures. The improved procedures use the well known constant ductility design spectrum for the demand spectrum, instead of the elastic design spectrum for equivalent linear systems in ATC-40 procedures. The inelastic design spectrum is plotted in the ADRS format to obtain the corresponding demand spectrum. The capacity spectrum computed from nonlinear static pushover.

In 2001 Kiattivisanchai (2001) studied and modified nonlinear static pushover to evaluate seismic capacity of existing building in Bangkok. They formulated building model with the site-specific features such as the stiffness and strength of pile foundations in Bangkok's soil, the mechanical properties of masonry infill walls, the over strength of reinforcing steels. With the seismic demand spectra of Bangkok's earthquake ground motions, the seismic capacity of 5 and 9 story buildings represented Bangkok's existing building can be determined. The evaluation results indicated that despite designed without any consideration on seismic loading, the building has sufficient capacity to withstand the highest intensity earthquake ground motions which was expected in the Bangkok area. A cost effective scheme to improve the seismic capacity of the building is also identified by using the evaluation method. Therefore the study of strengthening existing building in low to moderate seismic risk is needed.

2.2 Research on behavior of beam-column joint under seismic load

Beam column joints were identified as critical part under seismic load. Critical details of lightly reinforced RC frames were analyzed and their effects on the seismic behavior were studied by many researchers around the world. Through their reviews of detailing manuals and design codes from the past five decades, and their consultation with practicing engineers, they found typical and potentially critical to the safety of gravity load-designed structures in an earthquake. The interesting researches on seismic behavior of beam-column joint are as follow.

The experimental program conducted by Pessiki et al. (1990) and Beres et al (1991) included testing of twenty interior and fourteen exterior full-scale beam-column joints under cyclic static loading, and shake table tests on a 1/8-scale three-story building. In interior joints having continuous beam bottom reinforcement, failure was due to the heavy damage in the joint and in the column splice region. In the case of discontinuous beam bottom reinforcement, cracks appeared in the embedment region, and later the cracks either merged with diagonal joint cracks. The beam bars bulled out at about two-thirds of their yield stress. In the exterior joints, initial cracks around the embedment region proceeded diagonally toward the column bar splice region and extended downwards to the bottom column, causing spalling of a large column piece and prying of the beam top bar. Increase in column axial load resulted in an increase in peak strength of both interior and exterior joints, while it reduced strength degradation in exterior specimens. The main conclusion from shake table tests on the 1/8-scale building was that lightly reinforced RC structures are very flexible and may show significant P-Δ effects. Floor slabs played a major role in increasing the capacity of beams, thus leading to a soft story column failure.

Hakuto et al. (2000) reported on the performance of 6 beam column joints designed according to pre-1970's practice in New Zealand. The beam bottom bars were continuous through the interior joints, the beam stirrups were widely spaced, and the hooks of the longitudinal beam bars were bent out of the joint core in one of the exterior joints. The experimental results exhibited that brittle shear failure in joint and bond deterioration were failure mode of these specimen.

The study of Li et al. (2002) was the experimental of narrow beam-wide column joints normally found in Malaysia and Singapore. The beam column joint was non-seismically detail same as Thailand. The result showed that all specimens exhibited severe joint diagonal cracking after testing. The addition of 15 and 24 percent of the joint transverse reinforcement required by NZS 3101:1995 did not increase the strength but improved the ductility and energy dissipation.

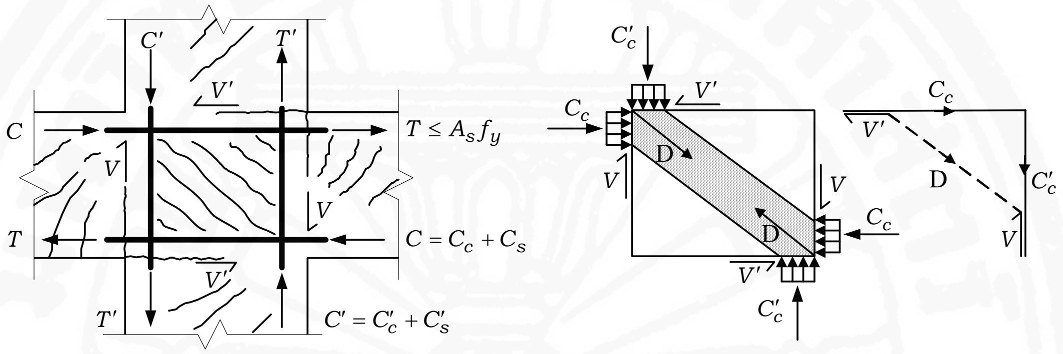
Supaviriyakit and Pimanmas (2008) do an experiment to compare performance of sub-standard interior reinforced concrete beam-column connection with various joint reinforcing details. They found that substandard existing beam column joint designed for gravity load only and without ductile reinforcing detail, the beam-column joint in substandard existing frame may perform in a moderately ductile flexural mode if the size of column is large. They also found that the provision of substantial horizontal joint reinforcement may not prevent joint shear failure even though the bond was not lost. The ACI minimum joint shear reinforcement may thus not be adequate for moderately ductile joint performance.

There are many researches on analytical and finite element analysis for RC beam-column joint such as Hegger et al. (2004), Baglin et al. (2000), Cervenka et al. (1991) and Yamaguchi and Chen (1990) but the notable research is Maekawa K. and Okamura H. from concrete laboratory, Tokyo University. Their 20 year researches on behavior of reinforced concrete were constructed in nonlinear constitutive model. The models are combined in the nonlinear finite element program "WCOMD" and "COM3". Thus, the program is wildly used in many researches to verify experimental results and to construct analytical model.

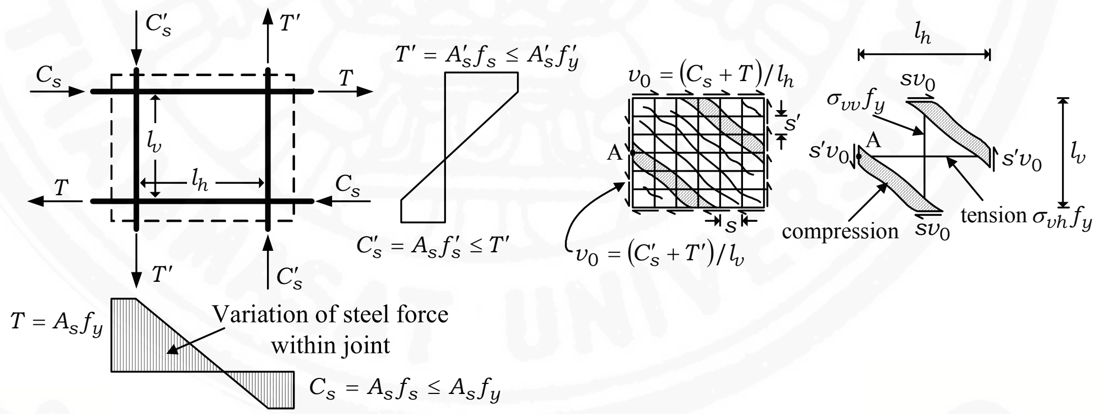
The famous model for joint shear strength in beam column joint is propose by Park and Paulay (1975). This model was already adapted in New Zealand Standard. The strut and truss is basic mechanism for shear force transferring through beam column joint. Compression strut is formed by all compressive forces which is carried by the concrete and combined by equilibrating each other through a single, broad diagonal strut across the joint (Fig.2.1a) When yielding in the flexural reinforcement occurs, it is appropriate to assume that the whole shear force in each of the adjoining members is introduced to the joint core through the concrete compression zones in the beams and columns, respectively. Truss mechanism is constructed for self-equilibrated elements in panel zone as illustrated in Fig. 2.1b. The diagonal compression forces could be supplied by concrete struts, formed between diagonal cracks. The tensile forces would require a mash or well-anchored horizontal and vertical bars, where the bond forces are introduced. This mechanism can only work well in two conditions, which are, the presence of a good bond along the beam bars and transverse reinforcement at the joint.

Another joint shear model is proposed by Shiohara (2001). In this model, shear deformation in the connection is not uniformly distributed and is due to the rotation of the four triangular concrete segments and the crack opening. The rotational movement of the segments cause uneven opening of the cracks like flexural cracks at the diagonal boundaries of the segments. As shown in Fig. 2.2, the vertical and horizontal springs connecting the segments prevent them from breaking into pieces. In a conventional beam-column connection, a longitudinal reinforcing bar in the beams and columns, passing through the connection and transverse reinforcement in the connection, functions as the springs. When moment is applied to the segments from adjacent beams and columns, rotation of the segments occurs. If there were no springs, the segments consisting of the beam-column connection cannot resist to any moment. Thus, this system is advantageous as a model for the moment resistance of the beam-column connection. This system can also resist joint shear force. Therefore, this behavior model meets the requirements to account for the real behavior of the joint shear failure, in which shear deformation of the beam-column connection increases while the remaining shear-resisting capacity of the connection is reserved.

Hwang and Lee (2000) also proposed rational model based on strut and tie. In their model, a simple compression strut and tie at maximum response is used for constructing joint shear force equation. By inputting data such as concrete compressive strength, reinforcement data and column axial force in mathematic model, joint shear force and failure mode are obtained. In their research, a comparison between 56 tested beam column joint and analytic joint shear force is compared. It shows that, analytical prediction give close result with average 20% higher than tested value.



(a) Internal action and crack pattern (b) Shear transfer by compression mechanism



(c) Forces in the reinforcement only (d) Shear transfer by truss mechanism

Fig. 2.1 Idealized behavior of interior beam-column joints
(Source: Park and Paulay, 1975)

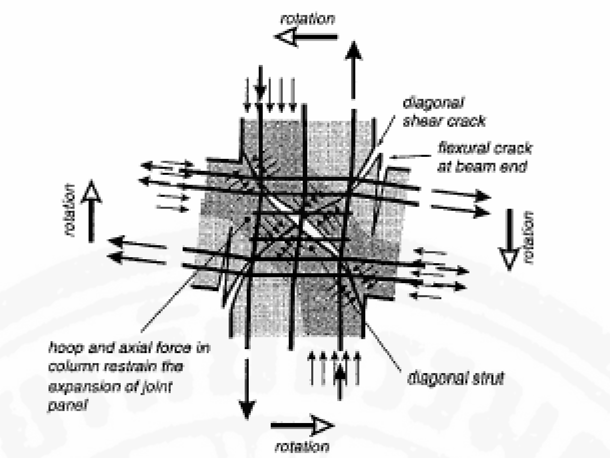


Fig. 2.2 Behavior model for joint shear failure and force flow in beam-column connection (Source: Shiohara, 2001)

2.3 Research on repair and strengthening beam-column joints

Recent earthquakes have highlighted the urgency and importance of rehabilitating seismically deficient structures to achieve an acceptable level of performance. This can be achieved, in part, by reducing the load effect input to the existing structures. Or by improving the strength, stiffness, and /or ductility of the existing structures. Over the past 20 years, significant advancements have been made in the research and development of innovative materials and technologies for improving the seismic performance of existing structures through rehabilitation processes (Liu and Driver 2005).

There are many seismic rehabilitation techniques available at the moment, depending upon the various types and conditions of structures. Relatively speaking, seismic rehabilitation of existing structures is still a fairly new and challenge activity for many practitioners. And since no two structures are exactly the same, it adds challenge to the rehabilitation process. Therefore, the selection of rehabilitation techniques is a complex process, and is governed by technical as well as economic and social aspect considerations. The following are some factors affecting the choice of various rehabilitation techniques (Bai and Hueste 2003):

- Cost versus importance of the structures
- Available workmanship
- Duration of work/disruption of use
- Fulfillment of the performance goals of the owner
- Functionally and aesthetically compatible and complementary to the existing building
- Reversibility of the rehabilitation
- Level of quality control
- Political and /or historical significance
- Structural compatibility with the existing structural system
- Irregularity of stiffness, strength and ductility
- Adequacy of local stiffness, strength and ductility
- Controlled damage to non-structural components
- Sufficient capacity of foundation system
- Repair materials and technology available

Research on the repair and strengthening of joints included epoxy repair, removal and replacement, reinforced or prestressed concrete jacketing, concrete masonry unit jacketing or partial masonry infills, steel jacketing and/or addition of external steel elements, and fiber-reinforced polymer (FRP) composite applications. Each technique required a different level of detailing and consideration of labor, cost, disruption of building occupancy, and range of applicability. The main objective was to establish a strength hierarchy between the columns, beams, and joints so that seismic strength and ductility demands could be accommodated through ductile beam hinging mechanisms instead of column hinging or brittle joint shear failures. In gravity load-designed structures, where beams are often stronger than columns, strengthening the column is generally not sufficient by itself since the joint then becomes the next weakest link due to either lack of transverse reinforcement, discontinuous beam bottom reinforcement, or other non-ductile detailing. Thus, the shear capacity and the effective confinement of joints must be improved. Achieving such an improvement is challenging in actual three-dimensional frames because of the presence of transverse beams and floor slab which limit the accessibility of the joint and because of the difficulties in developing the strength of externally placed reinforcements (i.e. steel plates, FRP sheets or rods) within the small area of the joint. At present, the techniques which have been tested either have not accounted for the three-dimensional geometry of the actual frame joints and are applicable in only special cases, or they resulted in architecturally undesirable configurations with bulky members. Review of seismic repair and retrofitting are as follow.

2.3.1 Epoxy repair

The conventional method for repairing concrete structures have long been using pressure injection of epoxy which was followed ACI 224-1999. Resent study of French et al. (1990) in using new method of epoxy repair is proposed. The method is vacuum impregnation. The study of both pressure injection and vacuum impregnation epoxy repair are compared with their moderately damaged interior beam column joint specimen. As shown in Fig 2.3, epoxy inlet ports were located at the bottom of each beam and at the base of the column repair region. The vacuum was applied through three hoses attached at the top of the repair region in the column. Both repair techniques were successful in restoring over 85 percent of the stiffness, strength, and energy dissipation characteristics of the original specimens. Severe bond deterioration in the repaired joints occurred only one half-cycle earlier than in the original specimens. The main conclusion was that vacuum impregnation presents an effective means of repairing large regions of damage at once and that it can be modified for joints with fewer accessible sides.

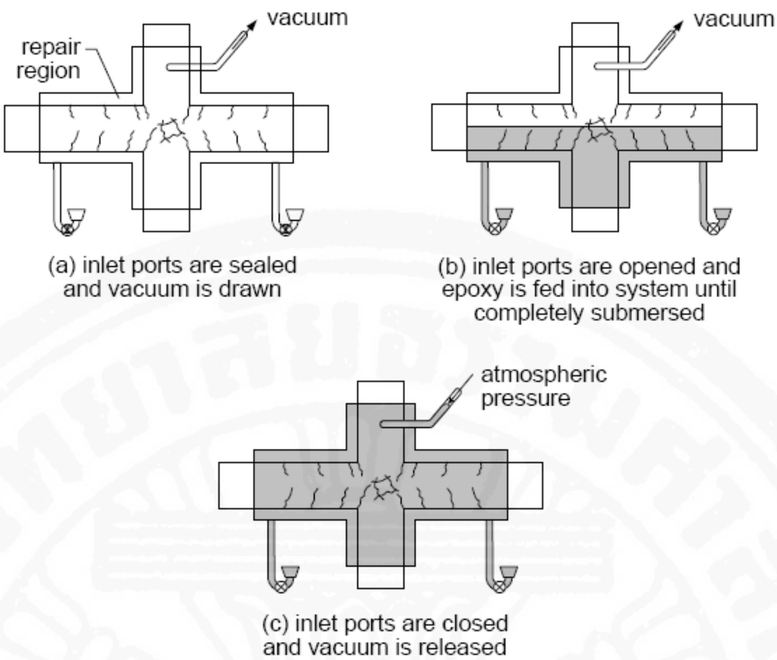


Fig. 2.3 Vacuum impregnation procedure applied by French et al (1990)

Others research on epoxy repair beam-column joint conducted by Corazao and Durrani (1989), Beres et al. (1992) Filiatrault and Lebrun (1996) and Karayannis et al. (1998) also shown similar result. The epoxy repair applications on beam-column joints have shown that the reliability of this technique in restoring the original characteristics of damaged joints is doubtful. The bond around the reinforcing bars once destroyed, does not seem to be completely restored by epoxy injection. This is evidenced by the partial recovery of stiffness and by the pinching in the hysteresis loops. It is also clear that the effectiveness of the epoxy repair is limited by the access to the joint and that epoxy cannot be effectively introduced into the joints surrounded by transverse beams and floor slab. This limitation can possibly be overcome by further advances in the vacuum impregnation technique. A high level of skill is required for satisfactory execution of such techniques, and application may be limited by the ambient temperature (ACI 224, 1999).

2.3.2 Removal and replacement

This method is suitable for severe damage of joints with crushed concrete, buckled longitudinal bar or ruptured ties. The preparation of temporary supported for existing damage structure is needed. Sometime existing concrete and may be removed. New longitudinal and ties may be add up to the damage condition. Generally, high-strength, low-shrinkage or non-shrinkage concrete is used for replacement. Special attention must be paid to achieving a good bond between the new and the existing concrete.

Corazao and Durrani (1989) studied a one-way, two-bay beam-column subassemblage by removing and replacing the concrete within the joints and the adjacent portions of beams and columns. The study shows that the specimen completely recovered its strength and stiffness but not the energy dissipation because high using of strengthen concrete reduced the rate of damage in the repairs. The researchers stated that, when shoring can be economically provided, this technique is appropriate for repairing localized damage such as flexural hinging in the beams, but replacing concrete in the joints in a real building may not be practical.

The experimental program conducted by Karayannis et al. (1998) and Tsnos (2001) also repair one-way beam-column joint by replacing the damage with low- and non-shrinkage cement paste. The aforementioned experiment results are in the same way that this technique can be used for strengthening even by itself if high-strength non-shrinkage concrete is used for replacement. However, this relies on the assumption that the damaged joint is readily accessible, which is rarely the case in actual buildings, and shoring can be economically provided. Also, Lee et al. (1977) stated that, if only the beam end is repaired with this technique, the high strength of the repair materials can cause the damage to move from the beam to the unrepaired joint and column.

2.3.3 Concrete jacket

Concrete jacket is a famous solution for rehabilitation of existing column and along with the joint region by encasing it with new concrete and additional longitudinal steel or tie. In some case slab perforation is required for inserting the longitudinal bar and the beams are also cored, making this method more labor intensive. However, this method is successful in restoring strength, stiffness and energy dissipation.

In tests conducted by Corazao and Durrani (1989), they strengthened three single (two exterior, one interior) and two multi-joint (two-bay) subassemblages some including a floor slab, by jacketing the column, the joint region, and sometimes a portion of the beam. Due to the difficulties experienced in in-situ bending of the cross tie hooks in the joint region, the additional joint reinforcement was modified to a set of dowels with a hook. The strength, stiffness, and energy dissipation of all three single-joint specimens were increased, except for the one-way exterior joint which dissipated less energy after jacketing. In two of these specimens, the damage was successfully moved away from the joint due to added beam bottom bars hooked both in the joint and at 25 cm from the column face.

Alcocer and Jirsa (1993) studied four three-dimensional beam-column-slab subassemblages subjected to bidirectional loading. They used welding steel cage around the joint (Fig 2.4b) instead of drilling holes through the beams for placing joint confinement. The cage consisted of steel angles designed to resist the lateral expansion of the joint and flat bars connecting the angles. The studied variables were jacketing the columns only or both beams and columns, jacketing after or prior to first damage, and using bundles or distributed vertical reinforcement (Fig. 2.4a) around the column. The critical section was within the jacket for the specimens with column jackets only, while the failure zone moved outside the cage when the beams were jacketed as well. The result showed that the steel cage and the corner ties confined the joint satisfactorily up to 4 percent drift, at which time severe crushing and spalling occurred. They recommended that the ACI 352R-76 provisions on joint strength and bond could be used to proportion the jacket and that distributed bars through the slab perforations should be preferred to bundles because bar development can be a problem for joints with smaller column-to-beam strength ratios when bundles are used.

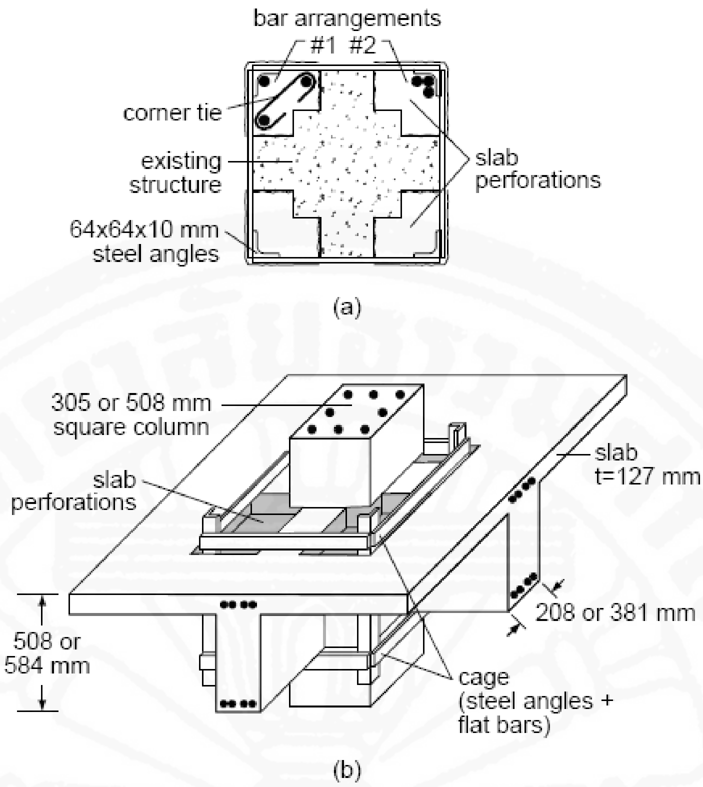


Fig. 2.4 Concrete jacketing technique studied by Alcocer and Jirsa (1993)
(a) plan (b) perspective

Concrete jacketing method studied by Choudhuri et al., (1992) and Bracci et al., (1995) employed post-tensioning of the additional column reinforcement placed in a high-strength concrete jacket and a reinforced concrete fillet built around the unreinforced beam-column joint (Figure 2.5a-c). The bottom half of the first story columns were conventionally jacketed with bonded longitudinal reinforcement and adequate transverse hoops in order to limit the strength enhancement due to post-tensioning and to ensure adequate energy dissipation in the event of an earthquake. Dimensions of the fillet were designed based on the required development length of the discontinuous beam bottom reinforcement and the desired beam hinge locations. As shown in Figure 2.5c, triangular segments of the slab were removed at the four corners of the column to permit placement of the fillets and vertical reinforcement, and all beams were drilled to place additional horizontal joint reinforcement. The interior columns were strengthened and provided with partial base fixity, and a series of shake table tests were conducted on the frame structure. Both the subassemblage test and the shake table tests showed that the original soft-story mechanism was avoided and that flexural hinges occurred at beam ends adjacent to the newly cast joint fillets with no noticeable damage to the columns.

Other study on concrete jacketing beam-column joint but a little different in detail were studied by Hakuto et al. (2000), Dogan et al. (2000), Shannag et al. (2002) and Tsonos (1999, 2001, 2002). Their results trend to be the same that the strengthening specimen with concrete jacketing could be shifted the joint shear failure to the beam after repair; the stiffness was restored, the strength and the dissipated energy increased.

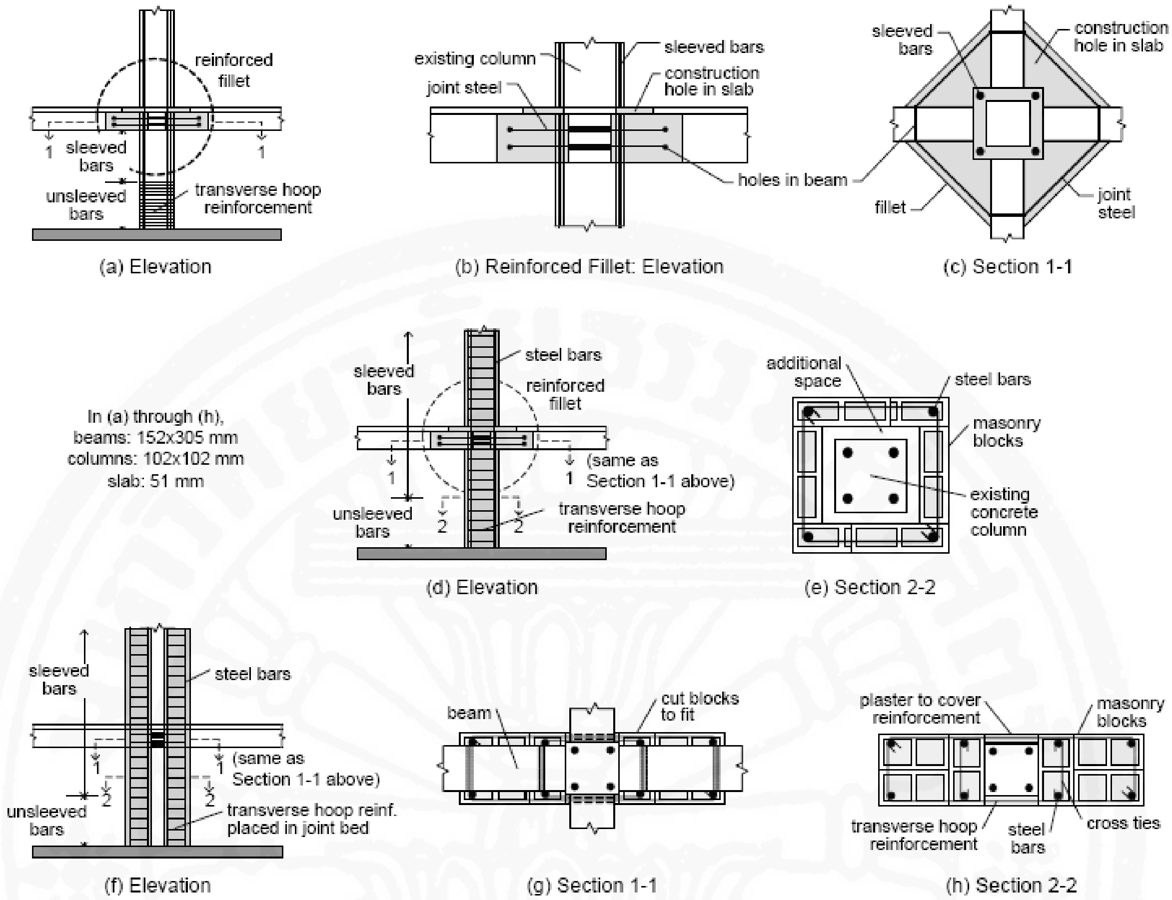


Fig. 2.5 Retrofit techniques studied by Bracci et al. (1995): (a, b, c) prestressed concrete jacketing, (d, e) masonry block jacketing, (f, g, h) partial masonry infill.

An apparent disadvantage of concrete jacketing techniques is labor practical such as drilling through the beams, perforating the floor slab, and sometimes in-situ bending of the added joint transverse reinforcement. Concrete jacketing results in pretty appearance. It increases the member sizes which reduces the available floor space and increases mass. The construction procedures also disrupt building occupants, which may well add to the overall cost of the rehabilitation. Finally, such jacketing techniques alter the dynamic characteristics of the building (for example, a 120 percent increase in first mode period and 73 percent increase in base shear capacity was reported by Bracci et al. (1995)). Changed dynamics may cause increased demands at unintended locations, and may require careful reanalysis. Nevertheless, concrete jacketing techniques did provide increased joint strength, shifted the failure to the beam, and increased overall lateral strength and energy dissipation.

2.3.4 Reinforced masonry blocks

Bracci et al. (1995) analyzed (but did not test) strengthening using reinforced concrete masonry units (CMUs). The first method required the existing interior columns to be jacketed by CMUs with additional longitudinal reinforcement within the corner cores extending continuously through the slabs and later posttensioned (Fig. 2.5d,e). Any space between the units and the existing column was then grouted. The shear capacity was increased by providing wire mesh in the mortar bed joints. A reinforced concrete fillet (Figure 2.5b,c) was built around the joints. In a second method, partial masonry infills reinforced with posttensioned vertical reinforcement were constructed on each side of existing columns as shown in Fig. 2.5f-h. The exact number of units was governed by the development length of the discontinuous beam bottom reinforcement. The beam-column joints were strengthened in shear by wrapping with rectangular hoops passing through holes drilled in beams. Nonlinear dynamic analyses on the 1/3-scale three-story GLD model (Bracci et al., 1995), incorporating the results from previous component tests (Aycardi et al., 1994), showed that strong column-weak beam behavior was enforced and that adequate control of interstory drifts was achieved. For the case in which all columns in the model were strengthened, a beam hinging mechanism was dominant. When only interior columns were strengthened, a predominant beam hinge mechanism was accompanied by some yielding in upper story exterior columns.

The same limitations mentioned earlier for concrete jacketing also apply to CMU jacketing. In the case of partial masonry infills, an added functional disadvantage is an increased loss of internal space between the bays.

2.3.5 Steel jackets and external steel elements

Various configurations of steel jackets, plates, or shapes have been used to increase the strength and ductility of deficient beam-column joints. Steel jackets consist of flat or corrugated steel plates, or rectangular or circular steel tubes prefabricated in parts, welded in-situ. The space between the jacket and RC frame is grouted with non-shrink or expansive cement mortar. Steel parts are often mechanically anchored to the concrete to improve confinement. Attaching plates to selected faces of the members using adhesives and bolts, and connecting these plates using rolled shapes (e.g. angles) has also been attempted.

Migliacci et al. (1983) strengthened four exterior joints with a steel skeleton made of angles epoxy-bonded to the beam and column corners. Steel straps were welded to the angles (Fig. 2.6). They reported that the steel straps of two of the specimens were prestressed by "first welding, then heating to a temperature of about $850\,°C$." Without preheating, the strength and energy dissipation capacity of the strengthened specimens were restored to their original level, while those of the preheated specimens were increased up to 35 percent.

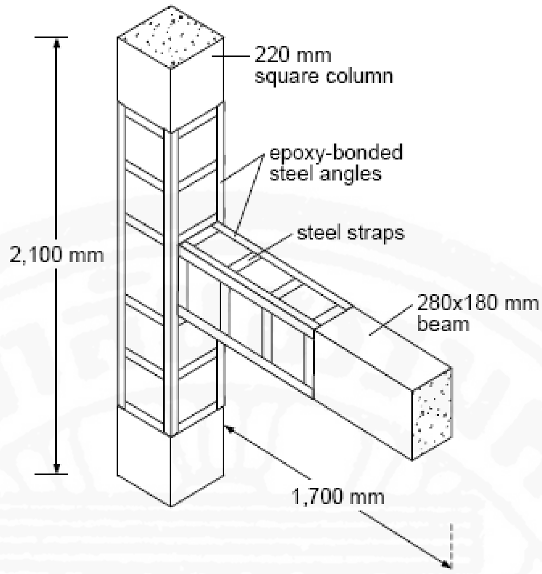


Fig. 2.6 Typical exterior joint strengthened with a steel skeleton by Migliacci et al (1983).

Corazao and Durrani (1989) strengthened one exterior and one interior two-way beam-column-slab subassemblage by bolting and epoxy-bonding external steel plates on each column face, welding steel angles to the plates, and by enlarging the joint region with a concrete fillet. As shown in Figure 2.7, the joint enlargement was similar to that used by Bracci et al. (1995) (Figure 2.5c) except that the continuous joint hoops were replaced with dowels with a hook. The steel plates bonded at each face of the upper and lower columns were bolted to the existing concrete near the joint and connected to each other by welded angles continuous through the slab. In the case of the interior joint, a plate was also bonded and bolted to the underside of the enlarged joint. For both specimens, cracking near the joint observed before retrofit was successfully moved to the end of the enlarged joint region after retrofit, and there was no evidence of damage in the column or its external reinforcement. The strength, initial stiffness, and energy dissipation of the beam-column joint were increased.

Beres et al. (1992) considered two different external plate configurations for strengthening one of their interior joints with discontinuous beam bottom reinforcement and for one of their exterior joints. To prevent pull-out of the beam bottom bars, the interior joint was strengthened by bolting two steel channel sections to the underside of the beams and connecting them by two steel tie-bars running alongside the column (Fig. 2.8a). The damage was transferred from the joint embedment zone to other parts of the joint; a 20 percent increase in peak strength, 10 to 20 percent increase in stiffness, and no significant change in energy dissipation were observed. The objective of the exterior joint retrofit was to force the flexural hinges to form in the beam and to increase the joint confinement. External steel plates placed along the opposite faces of the upper and lower columns were connected with threaded rods (Figure 2.8b). This retrofit prevented the cracks from extending into the column bar splice region. A flexural hinge formed in the joint panel close to the beam, which was followed by the pull-out of the beam bottom bars. The increase in the peak strength and the initial stiffness were 33 percent and 12 percent, respectively, with a higher rate of degradation than in the unstrengthened specimen. A notable increase in energy dissipation was observed in the final stages of loading.

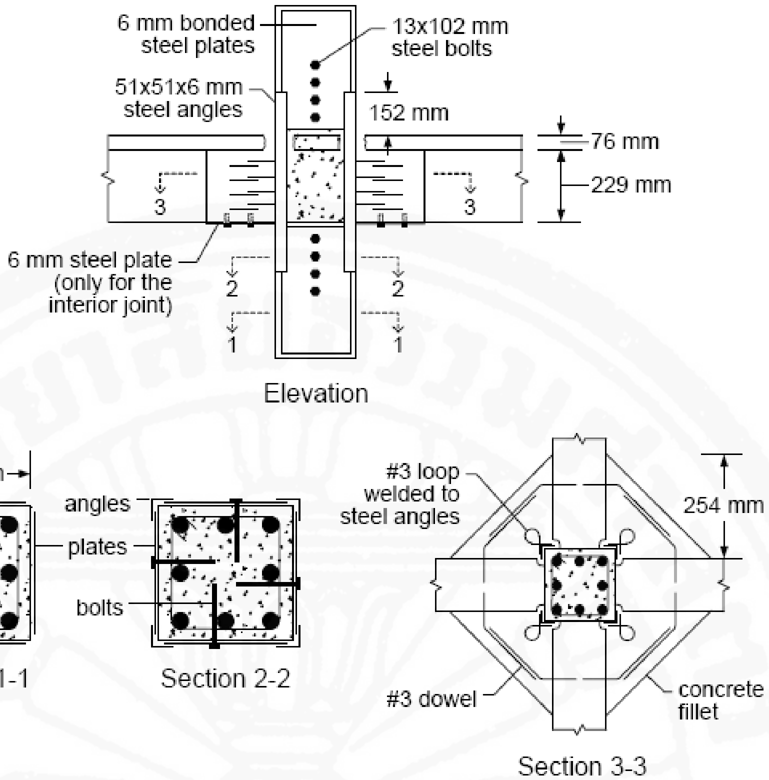


Fig. 2.7 External steel configurations studied by Corazao and Durrani (1989).

In a strengthening technique proposed by Adin et al. (1993), epoxy was injected through ports at the bottom of thin steel plates attached on two opposite sides of the joint until epoxy came out of the air outlet holes at the top (Figure 2.9). The circumferences of the plates were sealed prior to epoxy injection. Four T-shaped one-way joints were loaded to yield and retested after retrofit with two types of steel plates: one taking into account the presence of transverse beams (Figure 2.9a), and the second simulating the repair of two-dimensional joints (Figure 2.9b). The column was posttensioned by two unbonded 7 mm bars to account for column axial load due to gravity loading. With both repair schemes, the strengths of the original specimens were restored, and their stiffnesses were increased after repair. The second repair scheme, in which a steel plate was bonded in the joint panel (Figure 2.9b), provided considerably more energy dissipation than the first scheme. The epoxy-sealed cracks did not reopen; however, new shear cracks developed at the surfaces perpendicular to steel plates.

Hoffschild et al. (1995) studied the viability of using circular and square grouted steel tubes to encase one-way exterior joints. Four specimens were strengthened and tested in damaged or undamaged condition. Specimens without jackets exhibited rapid degradation in stiffness and strength due to extensive damage in the joint region. During upward loading of the beam, the beam bottom bars with hooks bent into the joint core yielded, and a plastic hinge formed in the beam near the column face. During downward loading, the beam top bars did not yield, and the maximum moment was limited by slippage of these bars within the joint. A typical joint encased in a circular tube is shown in Figure 2.10. The lengths of the retrofit were made equal to the total depth of the grouted jacket (*d*) along the columns and twice the total depth (2*d*) along the beam. A 25 mm gap was left at a distance *d* from the column face along the beam to cause a flexural hinge at that location. For both circular and square jackets, the hinge formed at the gap when the beam was in positive moment (bottom bars in tension). When the beam was in negative moment, however, unexpected increase in moment capacity caused failure to occur outside the jacket (beam shear failure for square jacket, hinge yielding in upper column for circular jacket). In the second stage of testing, these failure modes were prevented, and desirable hysteretic behavior (high energy dissipation with very little pinching) was obtained by jacketing the entire length of the beam or by providing two more gaps at distances of *d/3* and *2d/3* from the column face. Only the circular jackets, which are more difficult and expensive to fabricate, were effective in producing significant confinement.

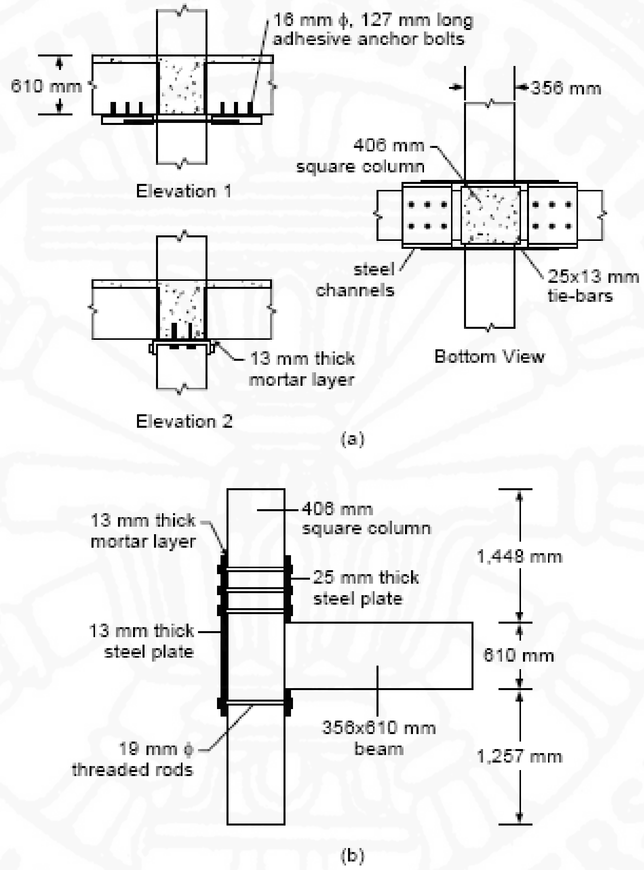


Fig. 2.8 External steel configurations studied by Beres et al (1992).

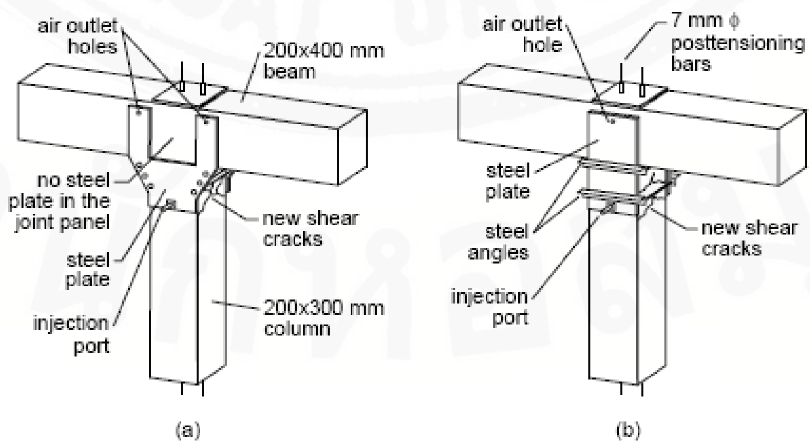


Fig. 2.9 External steel configurations studied by Adin et al. (1993):
(a) repair of 3-D joints, (b) repair of 2-D joints.

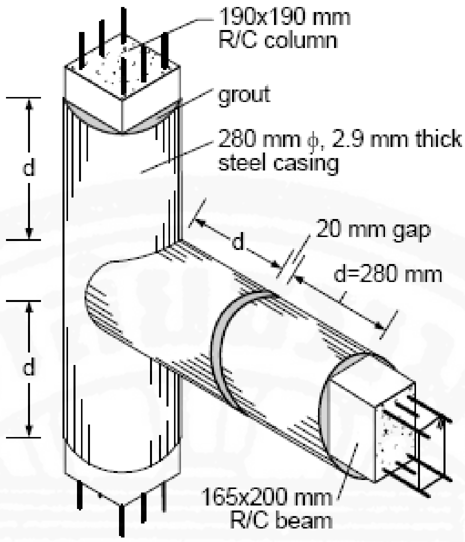


Fig. 2.10 Grouted steel tube technique studied by Hoffschild et al. (1995).

Ghobarah et al. (1997) and Biddah et al. (1997) proposed the use of corrugated steel shapes to provide high out-of-plane stiffness. The grouted corrugated steel jacket was intended to provide early lateral confinement effect in the elastic range of the RC column as well as additional shear resistance in the column, beam, and joint. The cross-section of the corrugated steel plates and of the two-part jackets before and after installation are shown in Figure 2.11. In addition to the in-situ welding, the joint jacket was also anchored to the concrete using two steel angles and anchor bolts (Figure 2.11a). A 20 mm gap was provided between the end of the beam jacket and the column face to minimize the flexural strength enhancement. Tests on four one-way exterior joints showed that the proposed system could change the joint shear failure mode to a ductile flexural mode in the beam when both the column and the beam were jacketed (Grobarah et al., 1997). Effective confinement was achieved up to a 5 percent drift by increasing the ultimate compressive strain of concrete. Biddah et al. (1997) added to this study by testing two exterior joint specimens with discontinuous beam bottom bars. One of them was a reference specimen, and the second was strengthened with a corrugated steel jacket around the column only in addition to two steel plates bolted to the beam and to the joint to prevent pullout of beam bottom bars. This strengthening system could not resist bottom bar pullout observed in the reference specimen, and the bolts failed in shear; but the system did provide an increase of about 38 percent in strength and 180 percent in energy dissipation capacity. A design methodology for calculating the required thicknesses of the corrugated steel jackets and the grout was also proposed (Grobarah et al., 1997).

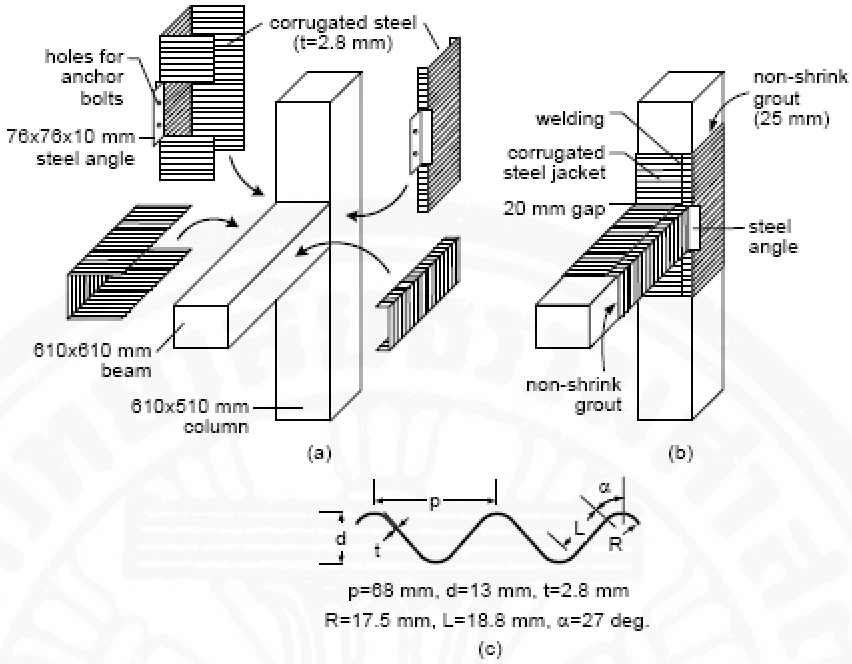


Fig. 2.11 Corrugated steel jacketing technique proposed by Ghobarah et al. (1997) and Biddah et al. (1997): (a) before installation, (b) after installation, (c) cross-section of the corrugated steel plates.

When compared with concrete and masonry jackets, the use of steel jackets can significantly reduce the construction time due to prefabrication. However, disadvantages such as difficulty in handling the heavy steel plates, the potential for corrosion, objectionable aesthetics in the case of corrugated steel shapes, and loss of floor space in the case of grouted steel tubes cannot be overlooked. Steel jackets may result in excessive capacity increases, even where only confinement effect is intended, and create unexpected failure modes (Hoffschild et al., 1995). Even if these disadvantages are ignored, it seems difficult to apply these schemes to actual three-dimensional joints. The presence of a floor slab, for instance, makes it difficult, if not infeasible, to install the beam jackets shown in Fig. 2.10 and Fig. 2.11. Although different two-part corrugated steel jackets have been proposed (Biddah et al., 1997) for interior, exterior, and corner joints with floor slab, there are no available data to validate their performance. Prestressing by pre-heating of externally attached steel straps in a repair scheme has been useful, but should not be relied on since it is difficult to control in the field.

2.3.6 Fiber-reinforced polymeric composites

Since 1998, research efforts on upgrading existing beam-column joints have focused on the use of fiber-reinforced polymer (FRP) composites in the form of epoxy-bonded flexible sheets, shop-manufactured strips, or near surface-mounted rods. The relatively higher initial cost of FRPs is purportedly outweighed by their advantages such as high strength-to-weight ratios, corrosion resistance, ease of application including limited disruption to building occupancy, low labor costs, and no significant increase in member sizes (Antonopoulos and Triantafillou, 2002; Ghobarah and Said, 2002). They are most attractive for their tailorability; the fiber orientation in each ply can be adjusted so that specific strengthening objectives such as increasing the strength only, confinement only, or both, can be achieved. An externally bonded FRP system requires that the concrete surface be thoroughly cleaned (all loose materials removed, and cracks epoxy-injected in damaged structures), a penetrating epoxy primer be applied, and each ply be placed between two coats of resin. Zureick and Kahn (2001) postulated that the primer and the resin should only be applied when the ambient temperature is between 5 and $32\,°C$, the relative humidity is less than 90 percent, the concrete surface temperature is more than $2\,°C$ above the dew point, and the concrete moisture content is no greater than 4 percent. They also suggested that the glass transition temperature of the resin should be at least $30\,°C$ above the maximum operating temperature and that elapsed time between mixing and application of the first ply and between any two successive plies should be within a time period not exceeding the gel time of the resin.

Gergely et al. (1998) designed a carbon fiber (CFRP) retrofit scheme for deteriorated bridge bents using nonlinear pushover analysis, and they tested two full-scale bridge bents with or without retrofit. Unidirectional CFRP sheets placed at $\pm 45°$ directions at the joints were secured by wrapping the beam ends; a 50 mm gap was left between the bottom of the cap beam and the partial column wrap (Figure 2.10a). Typical joint shear failure in the reference specimens was delayed in the strengthened specimens until tensile capacity of the wraps at beam ends was reached, after the wrap failed and the joint concrete crushed. The lateral load capacity and the maximum displacement ductility were increased by 20 percent and 76 percent, respectively. This retrofit scheme was later validated by Pantelides et al. (1999) and Pantelides and Gergely (2002) through in-situ testing of two strengthened bridge bents prior to demolition. In order to decrease the stress demand on the column longitudinal reinforcement, two U-shaped straps were also bonded around the joint in the direction of column axis (Figure 2.12b). The two failure modes of the CFRP composite were (1) delamination of the wraps on the column, bent cap, and joint, and (2) tensile failure of the U-straps. The system displacement ductility, the joint shear strength, and the lateral load capacity were increased.

In a related paper, Gergely et al. (2000) studied the effects of concrete surface preparation, fiber orientation, and elevated temperature cure on CFRP-strengthened bridge bents by testing fourteen 1/3-scale T-joints with no joint reinforcement (Fig. 2.12c). The carbon sheets were applied to the sides and bottom of the cap beam. The joint shear damage observed in the reference specimens was delayed but not totally prevented in the strengthened specimens due to delamination starting at the uncovered face of the beam, at a stress level of only one-fifth of the composite's capacity.

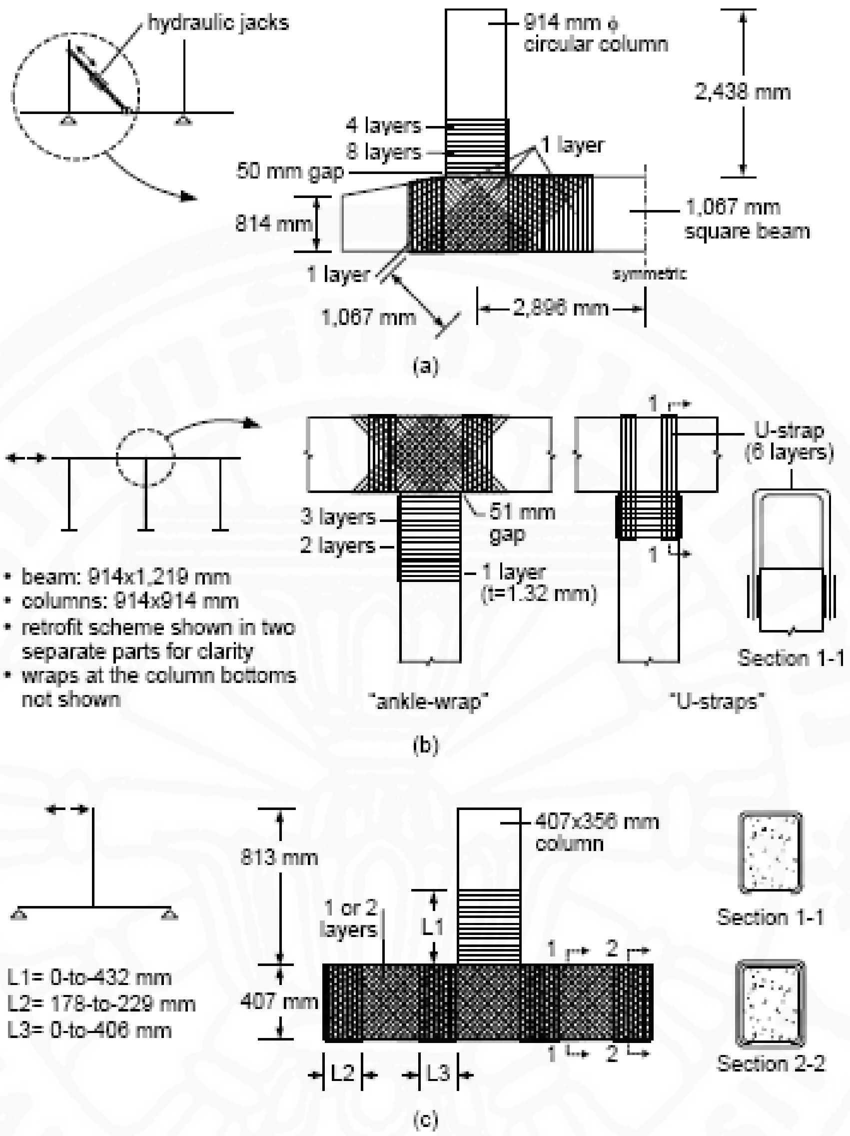


Fig. 2.12 CFRP-strengthened bridge bents tested by (a) Gergely et al. (1998), inverted bridge bent laboratory test, (b) Pantelides et al. (1999) and Pantelides and Gergely (2002), bridge bent tested in the field, (c) Gergely et al.(2000), inverted laboratory test.

Prota et al. (2001; 2002) used CFRP rods in combination with externally bonded sheets (Fig. 2.13a) to upgrade and test eleven one-way interior joints with three different levels of column axial load in an attempt to shift the failure first from the column to the joint, then from the joint to the beam. The CFRP rods were placed in epoxy-filled grooves prepared near the surface (Figure 2.13b). The failure modes could not be controlled as intended, and a ductile beam failure was not achieved. Type 2 scheme moved the failure from the compression to the tension side of the column for low column axial load, while, for high axial load, a combined column-joint failure occurred. The addition of CFRP rods as flexural reinforcement along the column (Type 3) led to a joint shear failure. When the joint panel was also strengthened (Type 4), the column-joint interface failed, which was attributed to termination of the FRP sheet reinforcement at that location to account for the presence of a floor system.

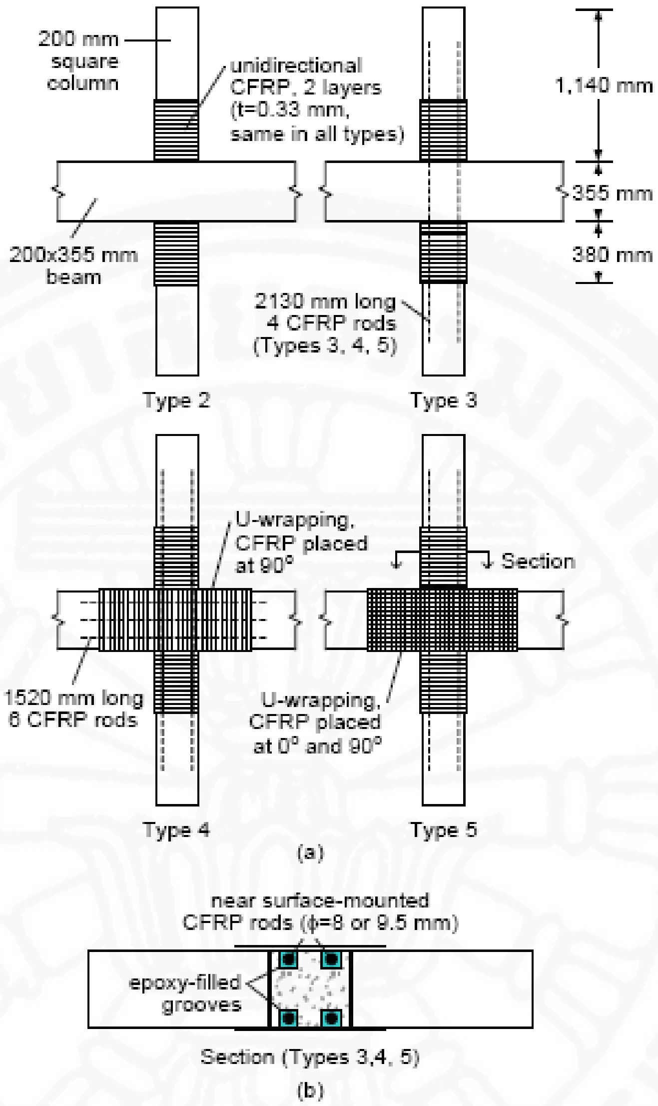


Fig. 2.13 Specimens strengthened with CFRP sheets and/or rods, tested by Prota et al. (2001; 2002): (a) elevation, (b) plan.

Ghobarah and Said (2002) tested four one-way exterior joints (Figure 2.14a), originally designed to fail in joint shear, with or without strengthening by unidirectional or bidirectional ($\pm 45°$) glass fiber reinforced polymer (GFRP) sheets. Specimens T1R and T2R, previously damaged in the joint region and repaired, were provided with mechanical anchorage using steel plates and threaded rods core-drilled through the joint. While the GFRP sheet anchored through the joint in Specimen T1R was effective until it failed in tension, it provided no improvement in Specimen T4 due to lack of threaded-rod anchorage and the resulting early delamination. No debonding or joint shear cracking was observed in Specimen T2R; the failure was due to a beam plastic hinge. The placement of the diagonal unidirectional strips in Specimen T9 was facilitated by the triangular steel bars fitted at the four corners of the joint panel. This scheme could not prevent expansion of the joint concrete, which led to delamination and a simultaneous failure of the beam and joint. Overall, this study highlighted the importance of anchorage of composite sheets in developing the full fiber strength in a small joint area.

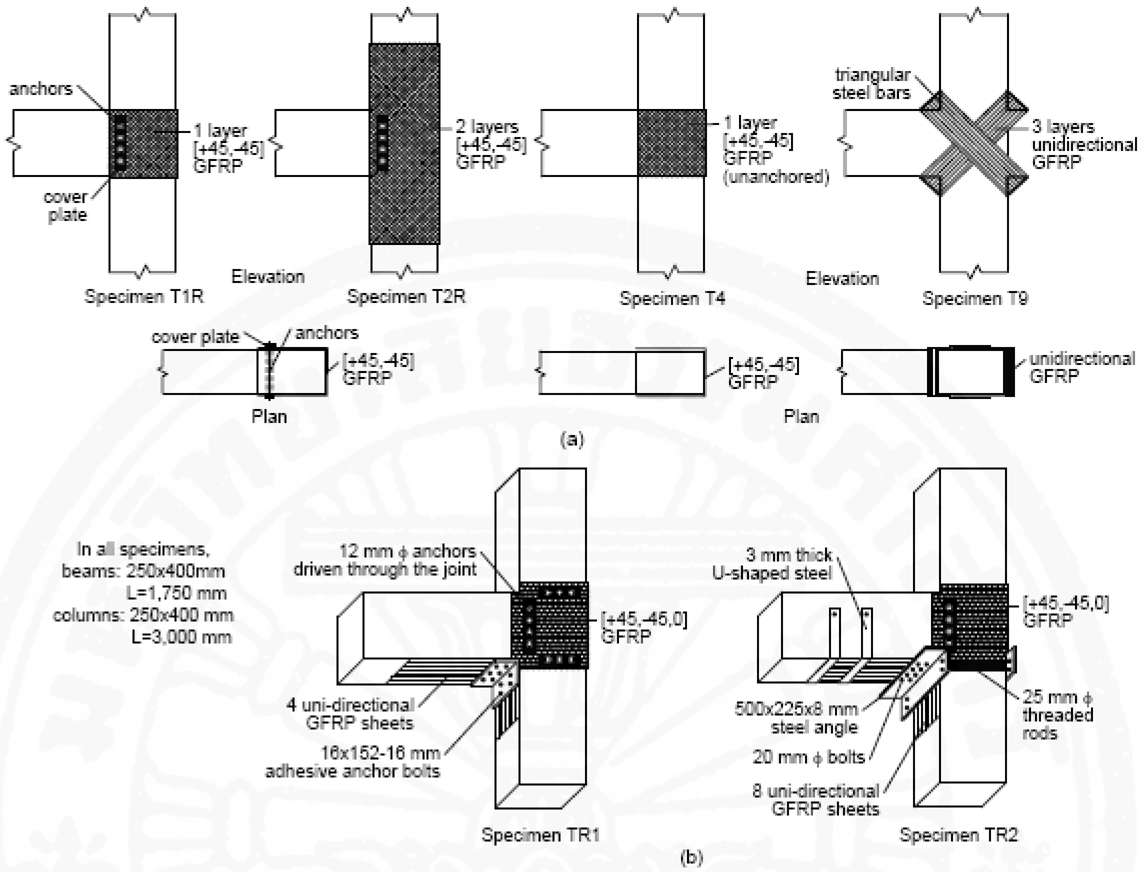


Fig. 2.14 GFRP-strengthened specimens tested by (a) Ghobarah and Said (2002), (b) El-Amoury and Ghobarah (2002)

El-Amoury and Ghobarah (2002) modified these GFRP schemes, as shown in Figure 2.14b, for strengthening joints with both inadequate anchorage of beam bottom bars and no hoop shear reinforcement. Both schemes resulted in around 100 percent increase in load carrying capacity; Specimen TR1 and TR2 dissipated three and six times the energy dissipated by the reference specimen, respectively. The failure of Specimen TR1 was due to complete debonding of the composites from the beam and column surfaces, and pull-out of the beam bottom bars led by fracture of the weld around the bolt heads. In Specimen TR2, the use of two U-shaped steel plates eliminated debonding of the GFRP and reduced the strength degradation; this specimen eventually failed in joint shear.

As part of the experimental program conducted by Clyde and Pantelides (2002), the performance of CFRP sheets on a single one-way exterior joint was investigated. With the CFRP layout shown in Figure 2.15, the joint shear failure in the original specimens was shifted to the beam-column interface with minimal damage in the CFRP wrap. The increases in joint shear strength, maximum drift, and energy dissipation capacity were 5 percent, 78 percent, and 200 percent, respectively.

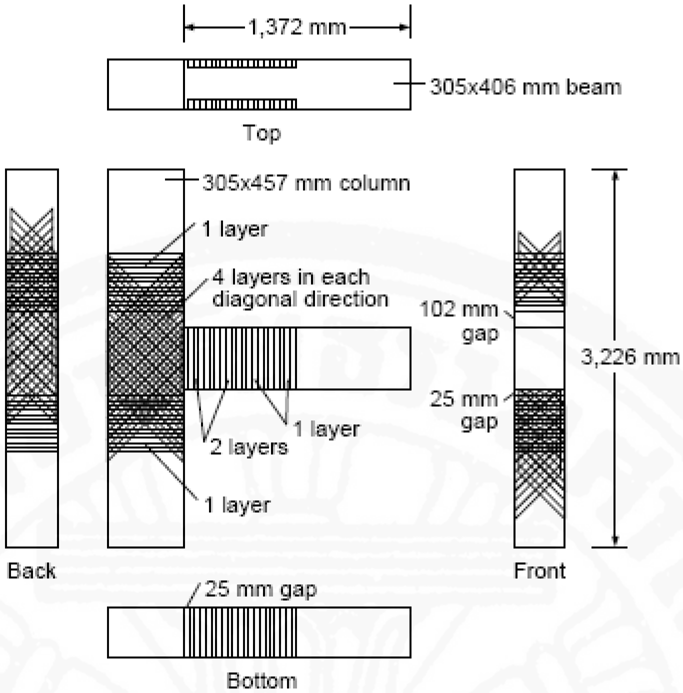


Fig. 2.15 CFRP-strengthened specimen tested by Cylde and Pantelides (2002)

The above survey of the literature indicates that externally bonded FRP composites can eliminate some of the important limitations (e.g. difficulties in construction, increase in member sizes) of other strengthening techniques, and still improve the joint shear capacity and shift the failure towards ductile beam hinging mechanisms. Such improvements have been achieved even with low quantities of FRP by placing the fibers in $\pm 45°$ directions in the joint region and by wrapping the member ends to clamp the $\pm 45°$ sheets and increase the confinement. Most studies have shown that the behavior is dominated by debonding of the composites from the concrete surface, and have indicated the need for a thorough surface preparation as well as for reliable mechanical anchorage methods that would lead to effective joint confinement and full development of fiber strength. The authors believe that the development of such anchorage methods can possibly create a potential for FRP-strengthened actual three-dimensional joints, which are yet to be tested. Though a high level of skill is not necessary, selection and application of FRP composites require careful consideration of the environmental conditions (e.g. temperature, humidity) present at the time of application, and likely during the service life (Zureick, and Kahn 2001).