Global Journal of Advanced Engineering Technologies and Sciences

SEISMIC UPGRADING OF RC COUPLED SHEAR WALLS: STATE OF THE ART AND RESEARCH NEEDS

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Abstract

Coupled shear walls are one of the most efficient structural systems for resisting lateral loadings due to wind and earthquakes. Their performance relies predominantly on the coupling beams, which must be appropriately designed and detailed to provide enhanced ductility and energy-absorption capacity. Many existing buildings with coupled shear walls were designed according to previous generations of codes and standards. Therefore, they are not up to modern, more stringent seismic codes and standards. Retrofitting coupling beams to improve their seismic performance can be a viable and cost-effective option. The objective of this paper is threefold: (i) to identify the deficiencies of existing coupling beams; (ii) to present a literature review of different techniques and methods for retrofitting coupling beams to enhance their seismic performance; and (iii) to highlight the advantages and disadvantages of these techniques. In addition, some strengthening techniques used for beam-wall joints, which play an important role in providing shear capacity for coupled shear walls, are also presented. Finally, research needs for a new and practical retrofit method with FRP sheets to improve the seismic performance of existing coupled shear walls are outlined.

Keywords: Coupled shear walls, coupling beams, retrofit of coupling beams, seismic performance, beam-wall joints.

Introduction

Past earthquakes have shown that most building structures collapse due to excessive deformation. Therefore, deformation should be kept within acceptable limits to avoid instability. Shear walls can be an effective system for resisting lateral forces. However, they should have adequate strength and stiffness to reach their full potential to resist wind and earthquake loadings. In this context, coupled shear walls (CSWs) are very effective systems for controlling deflection and inter-story drift within acceptable limits. CSWs are generally used for medium-high rise buildings of 10 to 20 stories. Unlike solid single walls, which behave like a cantilever beam that resists lateral loads through shear and moment at the base (see Figure 1a), CSWs resist lateral forces not only through the shear and moment resistance of their wall segments, but also and most importantly through the action of their coupling beams (CBs). As illustrated in Figure 1b, CBs transfer axial loads, P (tension to tension wall and compression to compression wall), which translate into a substantial additional moment resistance (Pl) at the base. This additional moment depends on the rigidity of the CBs with respect to that of the wall segments, which is often expressed in terms of the so-called degree of coupling (DC) as follows:

$$DC = \frac{Pl}{M_1 + M_2 + Pl},\qquad(1)$$

where P is the magnitude of the tension (or compression) force resulting from the coupling action; l is the length of the lever arm between the wall pier centroids; and M_1, M_2 are the moments resisted by wall segments 1 and 2 respectively. Therefore, DC is an important parameter when designing CSWs for seismic loading. For instance, Canadian Standard CSA-A23.3-04 (CSA 2004) links the value of DC to the ductility factor (R_d) as follows: $R_d = 3.5$ for $DC \le 2/3$ and $R_d = 4$ for $DC > 2/3$.

In fact, the lower the rigidity of the CBs and hence the smaller the DC (i.e., $DC \ll 2/3$), the less will be the coupling benefit; ultimately, the CSWs will behave as two separate single walls. In contrast, a very high DC ($DC \gg 2/3$) will lead the CSWs to behave like a pierced wall, that is, a wide single wall with a width equal to the overall width of the CSWs (i.e., $L_W + L_b + L_W$ in Figure 1b). However, for the coupling effect of CSWs to provide benefits, they should behave neither as two separate walls nor as a pierced wall.

Different research studies were conducted to assess the load – deformation behavior of coupling beams (Paulay 1969; Binney 1972; Santhakumar 1974; Tassios et. al. 1996; Galano and Vignoli 2000; Kwan and Zhao 2002; Fortney 2006) in which the primary test variables were the beam aspect ratio (ratio of the beam clear span to the beam total depth) and the beam reinforcement layouts mainly conventional versus diagonal. However, some retrofit techniques were also proposed to improve the seismic performance of coupled shear walls in terms of ductility, energy dissipation, shear/flexural strength and hysteretic behavior. Application of steel plates on one side of coupling beams (Harries, 1995), upgrading the degree of coupling of CSWs (Chaallal and Nollet 1997), bolting steel plates on to the vertical faces of CBs (Su and Zhu 2005; Su and Cheng 2011) and application of fiber-reinforced polymer sheet (Riazi et al. 2007) are among those techniques which will be reviewed in details in this paper.

Importance Of Coupling Beams In Coupled Shear Walls
A simplified numerical analysis of coupling beams in which the coupling beam is replaced by an equivalent continuous medium using a mathematical model originates from Chitty's solution (1947) for dowelled cantilever problem. This mathematical model converts a highly statically indeterminate problem to a simple one in which the indeterminate shearing forces of beams are calculated through a continuous function. Later, research studies by Beck (1962), Eriksson (1961) and Rosman (1964) extended the analysis by considering the finite width of the wall, wall system with multiple bands of openings and wall systems with various foundation conditions. The first experimental investigations on coupling beams were carried out by Paulay (1969), Binney (1972), and Santhakumar (1974).

Prior to the first CSA standard in 1959, the only requirements for the design of reinforced concrete walls were specified in the NBCC code with no specific provisions for coupled shear walls. Design requirements for coupled shear walls were considered for the first time in the 1984 standard CSA A23.3-M84 (CSA 1984). Prior to 1984, the shear walls could be designed according to the ACI 318 Building code, which introduced provisions for the design of CSWs in 1963.

Table *1* summarizes the evolution of the first introduction of the design provisions for single shear walls (SSWs) and coupled shear walls to codes and standards.

When linking *DC* to the ductility of CSWs, Canadian standard CSA-A23.3-04 (CSA 2004) and other modern codes encourage use of coupling beams with the required rigidity to attract the greater shear forces that generate greater moment resistance for the CSWs. However, such a philosophy implies that CBs should be designed and detailed to resist load reversals without loss of rigidity or strength to maintain this coupling effect during an earthquake. Failure of CBs leads the CSWs to behave as two separate walls with the maximum shear and moment concentrated at their bases. If seismic demand is greater than supply (i.e., the shear and moment resistance of the wall segments), then hinging at the base followed by instability and collapse will occur.

Ideally, CSWs should be designed and detailed to ensure that: (i) plastic hinging occurs in the CBs before the walls (Figure 2); (ii) the CBs do not show strength or stiffness degradation with load reversal; and (iii) the CBs should be the primary energy-dissipation elements by featuring stable energy-absorbing hysteresis loops without pinching. However, designing and detailing CBs with all these features was not possible before the 1970s. This is particularly true for energy-absorbing hysteresis without stiffness and strength degradation, where pioneering work led by Paulay's team (Binney, 1972) was successfully completed.

Deficiencies Of Existing Csws
Existing shear walls suffer deficiencies for two main reasons: (i) inadequate design for seismic loads, given the evolution of code requirements; and (ii) inadequate seismic design and detailing to achieve the desired characteristics and behavior outlined earlier. Deterioration of reinforced concrete elements, poor concrete quality, poor confinement of boundary regions, inadequate lap splices in longitudinal reinforcement, and ineffective reinforcement layout in coupling beams are among the deficiencies often observed (El-Sokkary 2012; Layssi and Mitchell 2012; Woods 2014).

Evolution of Seismic Loading
Many existing RC buildings with CSW systems that are located in seismically active zones were designed according to older design codes in which ductility requirements were not emphasized. The seismic performance of these buildings will be undermined in case of earthquake due to lack of strength, ductility, and energy dissipation, which are important features of modern seismic design codes. The minimum lateral earthquake design force at the base according to the National Building Code of Canada has evolved from 1941 to 2010 as indicated in Table 2 and Table 3, highlighting the differences between old (prior to the 70's) and modern seismic design codes. More stringent design requirements are specified in NBCC 2010 for enhanced performance and ductility of RC structures. For example, the base shear calculated using NBCC 2010 would be much greater than that obtained using NBCC 1941. Therefore, buildings designed according to old codes have less ductility and weaker seismic performance. Therefore, they have insufficient flexural capacity above the plastic hinge region and inadequate shear strength over their height (Mitchell *et al.* 2010).

Design Evolution of CSWs
Another problem associated with old coupling beams (CBs) is related to their conventional reinforcement layout, which features top and bottom longitudinal bars to resist flexure and closed vertical ties or stirrups distributed along the length to provide shear resistance and some confinement of the cross section (Figure 3a). After a number of post-elastic load cycles, severe cracks occur at beam-wall interfaces, leading to significant strength degradation of the CBs, which ultimately can no longer transfer shear forces to the walls through aggregate interlocking in the compression zone (Paulay 1969, Lam *et al.* 2001). Most conventional CBs behave in a non-ductile manner and exhibit either diagonal tension failure in case of insufficient reinforcement or sliding shear failure at the beam-wall joints if sufficient shear reinforcement is provided (Kwan and Zhao 2002).

There are some requirements for using conventional reinforcement layout in different design codes such as *ACI 318-11 Building Code Requirements for Structural Concrete*, *NZS 3101 Concrete Structures* Standard, and *CSA A23.3-14 Design of Concrete Structures*. For example, It must be noted that CBs with conventional reinforcement are allowed by CSA A23.3-14, but only if the shear stress resulting from factored loads is less than $0.1(l_u/d)\sqrt{f_c'}$, where l_u is the clear span of the coupling beam (effective length), d is the effective depth, that is the distance from the extreme compression fiber to centroid of longitudinal tension reinforcement, and f_c' the compressive strength of concrete.

The load-displacement curves of conventionally reinforced CBs, especially at large deflection amplitude, exhibit considerable pinching, which causes rapid stiffness degradation and hence relatively low energy dissipation (Figure 3b). This may be attributed to widening of shear and flexural cracks, which leads to excessive inelastic deflection of conventionally reinforced coupling beams (Kwan and Zhao 2002).

The above-mentioned problems of conventionally reinforced concrete coupling beams have prompted the development of new diagonal reinforcement configuration for coupling beams (Paulay and Binney 1974). More details of diagonally reinforced concrete coupling beams are provided in the following section.

Diagonal Reinforcement Concept For Coupling Beams
The pioneering work led by Paulay's team and others (Binney (1972), Santhakumar (1974), Shiu *et al.* (1978), Tassios *et al.* (1996), Galano and Vignoli (2000), and Kwan and Zhao (2002)) on the subject of CSWs and CBs opened a whole new era for the design of such structural elements, in particular the development of CBs with diagonal reinforcement (Figure 3c) as opposed to conventional reinforcement. Diagonally reinforced CBs showed highly satisfactory behavior under cyclic loading and achieved all the desired strength, stiffness, ductility, and hysteresis stability characteristics (Figure 3d). Therefore, the concept has been accepted and adopted worldwide. It is now part of most modern seismic design codes and guidelines.

The diagonal reinforcement extends through the entire CB. It provides both flexural and shear resistance, greatly improving CB ductility. In such CBs, shear force is transferred from one wall to the other, dividing itself into diagonal tension and compression forces which intersect at mid-span where there is no moment (Figure 4). Extending diagonal reinforcement beyond the beam ends improves hysteretic behavior by preventing sliding shear and by spreading the hinging regions away from the wall face (Paulay 1974). This translates into a more stable load-displacement hysteresis

without undesirable pinching effects. Opening and closing of cracks in the concrete have little effect on CB lateral resistance because this lateral resistance does not rely on the beam compression developed in the concrete (Kwan and Zhao 2002). However, sudden failure of the coupling beam is possible due to buckling of the diagonal reinforcing bars. This is the main concern when designing CSWs with diagonally reinforced CBs. Therefore, to keep the surrounding concrete in place and delay or prevent buckling failure, sufficient lateral hoops should be provided along the diagonal bars (Binney 1972). However, compaction of concrete near the bottom may be difficult to achieve because of the presence of ties around the main flexural steel. Experimental tests have also demonstrated that for higher span-to-depth ratios (between 2.5 and 5), diagonal reinforcement is not as efficient due to its lower angle of inclination, which leads to a reduced contribution to shear resistance (Harries *et al.* 2000). It has been noted that anchorage and confinement requirements often make these diagonally reinforced CBs difficult to assemble due to congestion at the center of the beam and at the wall faces.

In ductile CSWs, the coupling beams are the first to yield, dissipating most of the seismic energy input. However, as a second energy-absorbing line of defense, the walls should also be detailed to accommodate plastic hinging at the base without excessive loss of strength to avoid collapse after all the CBs have yielded. In this context, the pioneering work of Park (1975) has shown that walls with concentrated longitudinal reinforcement have greatly enhanced ductile behavior compared to walls with uniformly distributed reinforcement.

Wall segments with concentrated confined steel reinforcement and diagonally reinforced CBs have been accepted and adopted worldwide. They are now part of most seismic design codes and guidelines.

Failure Modes Of Coupled Shear Walls
The deficiencies of existing CSWs as described above must be addressed to improve their seismic performance. This can be achieved using retrofit or upgrade techniques. However, for each of the potential failure modes, strengthening configurations should be assessed and optimized to select an appropriate retrofit method for existing CSWs. The most common failure modes of coupled shear walls are described in the following paragraphs.

Flexural Failure Mode
In this failure mode, flexural cracks form first in the tension wall. However, flexural cracks also develop at the junctions of the walls and the CBs, particularly at high stress levels. As the load is increased, new flexural cracks may develop along the height of the wall and may also spread to more CBs, as illustrated in Figure 5a. Finally, crushing of the compression wall at the highly stressed corner and spreading of flexural cracks in most of the coupling beams lead to failure of the wall (Subedi 1991).

Shear Failure Mode
This failure mode, which is common in CSWs with moderate to deep reinforced CBs, starts with formation of flexural cracks in the tension wall, with some minor flexural cracks at wall junctions with CBs at high stress levels (Subedi 1991). However, the main feature of this failure mode is the formation of diagonal cracks which initiate near the center of the CBs and spread across the compression diagonal. As the load is increased, new flexural cracks form along the height of the wall simultaneously with the spread of shear cracks into other coupling beams. Finally, failure of the CSWs occurs by shear failure in most of the CBs and by crushing of the compression wall, as indicated in Figure 5b.

There are two possible shear failure modes: shear tension and shear sliding. The shear-tension mode of failure is characterized by: (i) formation of numerous diagonal cracks in the CB, (ii) yielding of the shear reinforcement before failure, and (iii) opening up of diagonal cracks until complete failure. In contrast, the shear-sliding mode of failure is characterized by: (i) formation of deep flexural cracks at the beam-wall joints, (ii) sliding movement along cracks at the beam-wall joints during failure, and (iii) reliance on the dowel action of the longitudinal reinforcing bars at beam-wall joints for residual shear strength in the post-peak stage. Although both modes of shear failure are brittle in nature, the brittleness of the shear-sliding failure mode is more severe because it is not preceded by yielding of the shear reinforcement, unlike the shear-tension mode. Therefore, when designing deep CBs, sufficient shear reinforcement should be provided to prevent shear-tension failure. However, this reinforcement should not be excessive because it could lead to undesirable shear-sliding failure. Between these two modes of shear failure, the one associated with lower failure load will occur first. If the failure loads of the two shear failure modes are very close, then either failure mode can happen (Zhao *et al.* 2004).

Rigid Action
This failure mode occurs when the coupling beams are very much stiffer than the walls (e.g., *DC*>>2/3). A large number of cracks form in the tension wall, with only partial damage to the coupling beams (Subedi 1991). The failure of the wall is similar to that of a simple cantilever beam (Figure 5c).

Review Of Retrofit And Upgrading Methods For Csws
Retrofitting methods have been developed in recent years, mainly for (i) coupling beams and (ii) coupling beam-wall joints.

Retrofit Methods for Coupling Beams
It has been found that many existing CBs are deficient in shear. Therefore, under earthquake loading, these CBs tend to fail in a brittle manner, compromising the energy-dissipation ability and the structural safety of the entire building. Various methods have been developed and documented to increase the deformability and energy-dissipation capacity of CBs, as presented in the following sections. In addition, during the last few decades, alternative coupling-beam designs have been suggested to improve the seismic performance of coupled shear walls.

Table *4* presents various retrofit methods for RC coupled shear walls as well as alternative designs of coupling beams.

Application Of Steel Plates To One Side Of Shear-Deficient Reinforced Cbs
Following the successful use of steel plates bonded to structural RC members to increase flexural and shear capacity, Harries (1995) extended this method to retrofitting of coupling beams in order to enhance their shear capacity without a significant increase of their flexural capacity. In this approach, steel plates are bonded to the accessible side of the CBs. As indicated in Figure 6, three methods were considered for attaching the steel plates to the CBs, as follows: 1) epoxied steel plates, 2) epoxied and bolted steel plates, and 3) epoxied and bolted steel plates extending onto the walls. These techniques were aimed at improving the shear capacity of the beams with the least possible effect on their flexural capacity. This approach was taken because an increase in the ultimate flexural capacity of CBs may lead to strengthening the walls and foundations, which is not desirable. Harries (1995) tested one full-scale control specimen (not retrofitted) and three specimens with a span-to-depth ratio of three, which were retrofitted with steel plates attached to one side of the beams using structural epoxy and mechanical anchor bolts. The results indicated that the retrofitted plates improved the strength, stiffness, displacement capacity, and energy absorption of shear-deficient RC CBs. In addition, this retrofit method caused the least disruption of architectural appearances. It was also observed that attaching the steel plate with epoxy caused failure in the concrete cover and that the steel plate was prone to peeling and debonding under cyclic loading. In contrast, anchor bolts prevented the complete separation of the steel plate from the concrete cover and enabled the retrofitted plate to contribute to the post-peak response of the coupling beams. However, out-of-plane buckling of steel plates may occur and may lead to loss of the additional capacity provided by the steel plate.

Upgrading the degree of coupling of coupled shear walls
Chaallal and Nollet (1997) proposed upgrading the degree of coupling for partial CSWs where coupling is insufficient. To this end, a small number of deep coupling beams were added to increase the stiffness and strength of the CSWs and hence the degree of coupling. To achieve this desirable behavior, the number and location of the added deep beams were optimized. In this case, the new axial force, *N*, was generated by the shear forces of the newly retrofitted CBs in addition to the existing regular beams and can therefore be determined as follows (Chaallal and Nollet 1997; Nollet and Chaallal 2002):

$$N = \int_0^H \upsilon(x)dx + \sum_{i=1}^{n} N_i , \qquad (4)$$

where *v(x)* is the shear force intensity in the regular coupling system and N_i is the axial force related to the retrofitted CBs. The advantage of this method is that by optimizing the number and location of the new attached coupling beams, the solution can be made cost-effective. Furthermore, using this retrofit method results in minimal reduction of the clearance for passage of services along corridors.

Attaching external steel plates to the side faces of CBs
To strengthen CBs for shear, Su and Zhu (2005) used steel plates bolted onto their side faces (Figure 7). Thereby, the bending moments and shear forces were transferred from the steel plates to the wall using appropriate bolt positions. To evaluate the performance of this retrofit method, three RC CBs with a span length-to-depth ratio of 2.5 and different steel plate arrangements were tested under cyclic loading. The first specimen was considered as a control beam with conventional reinforcement layout, whereas the second and third specimens were retrofitted with 3-mm and 6-mm thick steel plates respectively. The test results revealed that the steel plates increased the stiffness, strength, and deformability of the CBs. However, ultimate failure was due to crushing of concrete and excessive deformation. In addition, by attaching ductile steel plates, the maximum nominal ductility factor ($\mu_n = \theta_u / \theta_{yn}$) and the maximum ductility factor ($\mu = \theta_u / \theta_y$) were reduced due to an increase in yield rotation (θ_y) much greater than the ultimate rotation angle (θ_u). Moreover, local buckling instability of the plate was observed near the beam-wall joints, indicating that the applied diagonal compressive forces resulting from a combination of bending, shear, and axial forces were greater than the critical limit (Su and Zhu 2005).

Due to buckling of steel plates in the retrofit method proposed by Su and Zhu (2005) and lack of research into strengthening of CBs with span-to-depth ratios less than two, Su and Cheng (2011) proposed the addition of a buckling restraint device to control plate buckling and investigated the performance of deep CBs with a low span-to-depth ratio of 1.11 retrofitted with a bolted steel plate. The buckling restraint device does not increase the stiffness of CBs, unlike stiffeners which lead to brittle failure of CSWs under strong seismic loads because they attract greater lateral seismic loads. The four specimens tested by Su and Cheng under reverse cyclic loading indicated that adding an external plate improved the shear capacity, energy dissipation and rotation deformability of deep RC CBs. In addition, attaching a buckling restraint device resulted in more ductile failure behavior, less pinching, higher energy dissipation, and more stable energy absorption. It was also found that specimens with a sufficient number of bolts within the anchorage zones featured a more stable response and better inelastic performance under reverse cyclic loads (Su and Cheng 2011).

Application of fiber-reinforced polymer sheet
In the past few decades, fiber-reinforced polymer (FRP) composite materials have been widely used for strengthening and retrofit of RC structural members due to the advantages they offer, including high strength, high elastic modulus, light weight, ease of application, and high corrosion resistance. The three most used fiber types for structural retrofits are glass, carbon, and aramid. The choice depends on the required strength and stiffness, durability considerations, cost, and availability of the FRP materials. Moreover, using FRP composites is a faster and easier retrofitting method in special cases where evacuation of the entire building is not feasible.

Riazi *et al*. (2007) investigated the behavior of conventional RC CBs in shear walls strengthened with externally bonded CFRP sheets to improve their shear capacity. After test failure of four coupling beams having different reinforcements, but with similar shear strength, two of them were rehabilitated, strengthened with CFRP sheets, and retested. The test results indicated that the CBs rehabilitated with CFRP sheets achieved enhanced strength in comparison with the original beams.

Meftah *et al*. (2013) strengthened both sides of CBs using CFRP plates to investigate the dynamic behavior of RC CSWs. They developed new finite-element models for both the walls and the strengthened coupling beams and carried out various analyses, including static and free vibration analysis and dynamic analysis under El Centro and Northridge earthquake accelerations. The results of comparing the maximum top lateral deflection responses of strengthened and unstrengthened RC CSWs indicated that the geometric characteristics of the shear wall structure and the dominant range frequencies of the input earthquake accelerations affected the mitigation of seismic behavior achieved by strengthened RC coupled shear walls.

Yeghnem *et al*. (2013) investigated the effect of creep and shrinkage of RC coupled shear-wall structures strengthened using CFRP sheets with different spacings bonded to the bottom of the CSWs. A finite-element lateral stiffness model was presented and used to analyze a 25-story CSW under two recorded earthquake accelerations from Algeria to verify

the accuracy of the proposed method. It was concluded that bonding CFRP sheets at the wall edges resulted in improved displacement response. However, the predominant actions of creep and shrinkage resulted in an increase in lateral displacement with time.

Replacing of RC coupling beams with steel or steel-concrete composite coupling beams

Replacing RC coupling beams with steel or steel-concrete composite coupling beams were proposed to enhance their seismic performance particularly their shear capacity. These types of coupling beams have shown a better performance in resisting lateral loads compared to conventional CBs. Therefore, the technique can be a potential retrofit method. The benefits of these alternative designs for coupling beams are described in the following sections:

Steel coupling beams with and without stiffeners

Based on the concept of linked steel beams in an eccentrically braced frame with regard to ductility and energy-absorption capability, Harries (1995) suggested using steel coupling beams with their ends embedded in reinforced concrete walls (Figure 8a). Four specimens were considered to evaluate this method. Three of these were designed as shear-critical steel beams in which the ultimate shear capacity was developed while the beams remained elastic in flexure. For the second shear-critical specimen, some stiffeners were attached to the embedded region of the coupling beam in addition to its clear span. The fourth specimen was designed as a flexure-critical coupling beam such that the beam remained elastic in shear while flexural hinges occurred at either wall face. The test results indicated that flexure-critical steel CBs were superior to conventionally reinforced CBs due to their greater energy-absorbing capability, achieving a ductility level at least equal to that of conventionally reinforced CBs, but without strength or stiffness degradation (Harries 1995; Harries *et al*. 2000). It was also concluded that the shear-critical steel CBs exhibited better ductility and energy-absorption features than diagonally reinforced CBs. For the first specimen without stiffeners in the embedded region, insufficient shear and local buckling resistance in the embedment region caused high concentrations of compressive stress at the wall faces and inelastic deformation in which both shear yielding and web crippling occurred.

Using this method, beams of small dimensions can be constructed and used easily. However, detailing of wall reinforcement around the embedment region of the coupling beam remains a challenging task. In addition, cutting openings for service ducts is difficult at the slab level due to the presence of the vertical steel plate (Lam *et al*. 2005).

Concrete-filled steel-tube coupling beams

Teng *et al*. (1999) proposed concrete-filled rectangular steel tubes (Figure 8b) as an alternative design for coupling beams with high ductility and energy-absorbing capacity. Experimental results for four rectangular tubes under cyclic loading indicated that the one without concrete infill had low ductility and rapid strength degradation because it failed by flange buckling. In contrast, the other coupling beams with concrete infill had higher ultimate strength and failed by tensile cracks in the flanges. However, slip at the steel-concrete interface or formation of shear cracks due to concrete deterioration may cause strength and stiffness degradation. Although bonding between concrete and steel may be difficult to achieve using this method, the presence of concrete infill prevents buckling failure of beams at low loads.

Steel coupling beams encased in reinforced concrete members

In this retrofitting method, steel coupling I-beams are encased in reinforced concrete members (Figure 8c), thereby avoiding welding and bolted connections. Coupling forces are transferred from embedded steel sections to shear walls through a bearing mechanism. In this type of beam, a sufficient embedment length of the steel section creates a dependable transfer of forces from the beam to the walls and affects the strength of the beam-to-wall connection. These steel-composite coupling beams are an appropriate choice for cases in which deep reinforced concrete beams cannot be used due to height restrictions, or where the required capacities and stiffness cannot be provided economically by a concrete beam.

The performance of this design method was investigated by Gong and Shahrooz (2001a and 2001b) and Motter *et al*. (2012). The effects of various parameters were studied, including the effects of encasement, the amount of web stiffener in the steel beam, the presence or absence of face bearing plates at the wall-beam interface, the level of shear force, and the nature of the floor slab around the coupling beam. It was observed that web buckling and flange instability could be prevented by encasement around steel CBs, so that web stiffeners are not required (Gong and Shahrooz (2001a and 2001b)). However, the concrete encasement causes extra strength and stiffness, leading to over-coupling and hence greater forces in the walls. Consequently, the failure sequence may change and become

undesirable. Therefore, the embedment length is an important parameter due to its strong effect on strength and ductility degradation (Motter *et al*. 2012).

Embedded steel-composite coupling beam with shear studs
Lam *et al*. (2001) proposed a steel-composite coupling beam in which shear studs are welded onto the top and bottom of both sides of the plate to improve horizontal shear transfer and bonding of the steel plate and the concrete (Figure 8d). The results of experimental investigations performed by Lam *et al*. (2005) indicated that embedded steel plates improved the shear strength and stiffness of coupling beams. Adding shear studs enhanced the plate/reinforced concrete interaction and resulted in satisfactory inelastic performance under large imposed shear deformations (Lam *et al*. 2005).

Retrofit of Beam-Wall Joints
Similarly to beam-column joints, CB-wall joints are also critical elements in structural design and play an important role in resisting seismic loading because their failure may lead to excessive lateral drift and collapse. One of the important problems in CB-wall joints is local deformation due to stress concentration in both the elastic and inelastic stages (Kwan and Zhao 2002). With the increase in applied load and the occurrence of cracks near the CB-wall joints, bond-slip of the longitudinal bars and inelastic deformation in the walls near the joints leads to additional local deformation, resulting in significant increases in lateral deflection and rotation of the CBs (Kwan and Zhao 2002).

Therefore, retrofit of CB-wall joints is an important issue. However, despite its relevance, this problem has not received much attention. This contrasts with the comprehensive research studies that have been devoted to seismic retrofit and strengthening of beam-column joints due to their importance in the performance of reinforced-concrete framed structures. On the basis of the concept of "strong column, weak beam", seismic strength and ductility demands are resisted through hinging mechanisms in ductile beams rather than column hinging or brittle-joint shear failures. Due to the similarity of desirable failure sequences, this concept can also be used when retrofitting CB-wall joints. Epoxy injection, shotcreting, steel plate adhesion, steel jacketing, and externally bonded (EB) FRP are among the documented retrofit methods for beam-column joints. However, this paper concentrates on research studies devoted to retrofit of beam-column joints using EB FRP. A comprehensive review of 54 tests undertaken from 1998 to 2008 on seismic rehabilitation of RC frame beam-column joints with FRP can be found in Bousselham (2009). To evaluate the influence of FRP on shear capacity, the specimens were designed so that shear failure would occur at joints in most experiments. Other retrofit configurations using FRP were considered, as well as the effects of various parameters, including the effectiveness of strips versus sheets; the number of strips or of sheet layers; mechanical anchorages; type of fibers; level of axial load in the column; damage to the joint before strengthening; and the effect of transverse beams.

The results of experimental studies such as those of Ghobarah and Said (2002), Antonopoulos *et al*. (2003), Ghobarah and El-Amoury (2005), and Pantelides *et al*. (2008) indicated that a greater number of FRP layers results in a significant increase in strength and energy-dissipation capacity. Moreover, flexible sheets were found to be more effective than strips for the same reinforcement ratio. In addition, mechanical anchorages enhanced the contribution of both FRP strips and sheets. It was concluded that joint shear reinforcement is required to prevent joint shear failure and also to maintain concrete integrity in the anchorage region. Furthermore, the retrofit method favored the formation of plastic hinges in the beams away from the joint region and resulted in an increase in inelastic rotation capacity.

Li and Kai (2010) proposed a method for retrofitting beam-wide column joints using FRP. To evaluate this retrofit method, four interior beam-column joints were considered as control specimens in two series (1 and 2) with column-to-beam width ratios equal to 3.56 and 7 respectively. First, these specimens were tested under cyclic lateral displacement, and then all damaged specimens were repaired using CFRP and GFRP sheets according to two schemes based on failure mode and location of plastic hinges with the purpose of restoring the original strength and drift capacity. Generally, it was concluded that both FRP sheet configurations were effective in recovering the performance of specimens in the first series. However, neither was able to improve the seismic performance of specimens in the second series.

Parvin *et al*. (2014) tested two full-scale beam-column joint specimens designed and built with pre-1970s deficiencies, including widely spaced column ties and hence inadequate confinement of concrete, lack of transverse reinforcement in the joint region, and construction joints above and below the joint core. One of the specimens was retrofitted using CFRP sheets in a specified configuration. The specimens were tested under reverse cyclic displacement. Results

indicated that joint failure occurred in the control specimen, with considerable pinching in hysteretic loops. In contrast, the retrofitted specimen featured an increase in maximum load capacity and an improvement in hysteretic behavior with neither pinching nor strength degradation. It was also found that this retrofit configuration changed the failure sequence from the joint region to the formation of plastic hinges in the beam.

Rahman *et al.* (2014) investigated the effect of CFRP sheets on the performance of four full-scale beam-column joints in two sets, one designed to fail in flexure and the other designed to fail in shear. In each set, one specimen was retrofitted using CFRP sheets. The specimens were tested under axial load on the column and lateral load on the beam under cyclic displacement. The results indicated that the specimens which had deficiencies in flexure failed in flexure and that the retrofit configuration using CFRP sheets for joint strengthening did not result in any significant increase in load capacity. However, the load capacity of the shear-deficient specimens retrofitted with CFRP sheets increased considerably, and their failure mechanism changed from shear to flexural failure in the beam.

Advantages and Disadvantages of Retrofit Methods and Perspectives for FRP Composites
The advantages and disadvantages of each retrofit method of coupling beams which were proposed to improve seismic performance of coupled shear walls, are summarized in Table 5.

The studies mentioned earlier confirmed that EB FRP composites have the potential to improve joint shear capacity and prevent shear failure. They also offer solutions to some of the problems encountered when using conventional retrofit methods, such as difficulties in construction and access or heavy and oversized jacketing. These studies also show the importance of surface preparation and use of mechanical anchorages to achieve reliable and durable retrofit performance.

Required Research
Despite the retrofit methods that have been proposed in the literature to improve the seismic performance of CSWs, major problems remain to be solved. Therefore, more research is still needed to develop new, suitable, and practical methods to strengthen existing CSWs. In recent years, considerable research has been devoted to strengthening and retrofitting concrete structures with EB FRP composites. As a result, many codes and design guidelines have been published in this area worldwide. Use of FRP sheets to strengthen structural elements such as slabs, beams, and columns is well documented. This is not the case for CBs and beam-wall joints of CSWs. The observed effectiveness and success of FRP composites for retrofitting buildings and bridges has led people to believe that their use can be extended successfully to retrofit CSWs. Because the behavior of CBs is distinct and different from that of flexural beams, special attention should be given to investigating and developing an appropriate, suitable, and effective retrofit method for these special elements. To study the various parameters involved, including the number of FRP sheet layers, the FRP configuration, and the effect of mechanical anchorages, more research is needed on this subject to develop a comprehensive technique for practical application. A number of important issues related to retrofit of CBs with FRP sheets should also be investigated. The most salient ones are: (i) identifying the parameters that influence the shear resistance mechanism of CBs; (ii) proposing retrofit configurations and strategies with EB FRP to improve the seismic performance of CBs; (iii) studying the effects of FRP sheets and FRP configurations on the behavior of beam-wall joints; (iv) studying the hysteresis behavior of CBs retrofitted using externally bonded FRP; and (v) studying the effects of FRP sheets on the ductility, flexural capacity, and shear capacity of CBs and the failure sequence in CSWs.

Conclusions
In this study, a literature review of different retrofit methods for CBs in CSWs has been presented. This important step makes it possible to identify the advantages and drawbacks of previously developed methods before trying to improve existing methods and develop new strengthening schemes. An appropriate retrofit method can be selected on the basis of the probable failure mode, the expected gains in terms of ductility and hysteretic behavior, and the budget available for the retrofit. However, the exploratory studies performed to investigate some of these retrofit methods, although useful, clearly remain very few and exploratory in nature. Therefore, they remain disconnected and fail to translate into sound approaches that can be used in engineering practice. It follows that more research studies and experimental investigations are needed to introduce comprehensive and targeted techniques for practical applications.

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Table 1. Consideration of design provisions for single shear walls and coupled shear walls in ACI 318 and CSA standard A23.3.

Standards	Year	Consideration of CSWs		Consideration of SSWs	
		Yes	No	Yes	No
ACI 318	1963		X	X	
	1971		X	X	
	1977		X	X	
	1983		X	X	
CSA A23.3	1959		X	X	
	1977		X	X	
	1984	X		X	
	1994	X		X	
	2004	X		X	
	2014	X		X	

Table 2. Evolution of seismic design forces in the NBCC: (a) from 1941 to 1970.

Code	Lateral force (*V*)	Total weight (*W*)	Seismic zoning map	Comments
1941	$V=CW$	DL+0.25SL	-	*C* varies from 0.02 to 0.05
1953	$V_i=C_iW_i$	DL+0.25DSL	1	*C* = horizontal force factor for minimum earthquake load; Zone 1: *C = 0.15/(N+4.5)*, Zone 2: *C = 0.30/(N+4.5)*, Zone 3: *C = 0.60/(N+4.5)*
1965	$V = RCIFSW$	DL+0.25DSL+LL	Same as 1953	*R* = seismic region factor (= 0, 1, 2, or 4 for earthquake intensity zones 0, 1, 2, or 3, respectively); *C* = type of construction factor (= 0.75 for moment-resisting space frame, 1.25 for non-ductile structures), *I* = importance factor (1 or 1.3); *F* = foundation factor, *S* = structural flexibility factor = *0.25/(N+9)*
1970	$V =1/4R(KCIFW)$	DL+0.25DSL+LL	1	*R, I, F* are the same as NBCC 1965; *K* = type of construction factor (values from 0.67 to 1.33 for buildings); *C* = structural flexibility factor = 0.05/T1/3≤0.10; *T* = fundamental period of the structure (0.05h$_n$/D1/2 or 0.10N); *h$_n$*= height of the structure in feet; *D* = dimension of the building in direction parallel to seismic force in feet; *N* = number of stories.

DL=Dead load, SL=Snow load, DSL=Design snow load, LL=Live load.

Table 3. Evolution of seismic design forces in the NBCC: (b) from 1975 to 2010.

| Code	Lateral force (V)	Total weight (W)	Seismic zoning map	Comments
1975	$V = ASKIFW$	DL+0.25DSL+LL	Same as 1970	I, F are the same as NBCC 1965; A= assigned horizontal design ground acceleration; S= seismic response factor ($0.5/T^{1/3} \leq 1$); K= numerical coefficient reflecting the influence of the type of construction on the damping, ductility, and (or) energy-absorption capacity of the structures (values range from 0.7 to 2 for buildings).
1980	$V = ASKIFW$	DL+0.25DSL+LL	Same as 1970	No major change
1985	$V = vSKIFW$	DL+0.25DSL+LL	2	New methodology in the calculation of seismic risk; a change in the probability level at which design ground motion is computed; use of both peak ground acceleration and peak ground velocity as ground motion parameter to represent the intensity of shaking; an increase in the number of seismic zones in Canada; K, I, F are the same as NBCC 1975; v = zonal velocity ratio; S = new seismic response factor depending on the periods of the structure.
1990 - 1995	$V = U(vSIFW)/R$	DL	2	U =0.6, calibration factor; R = force modification factor (ranging from 1 to 4); v = zonal velocity ratio; S = seismic response factor, I = importance factor (1, 1.3, 1.5); F = foundation on site factor.
2005	$V=S(T_a)M_vI_EW/R_dR_o$	DL+0.25SL	4	$S(T_a)$ = design spectral response acceleration at the fundamental period of vibration; I_E = importance factor (1, 1.3, 1.5); R_d = ductility factor ($1 \leq R_d \leq 5$) and R_o = over-strength factor ($1 < R_o < 1.7$); M_v = factor to account for higher mode effects on base shear.
2010	$V \geq S(4.0)M_vI_EW/R_dR_o$ $V \geq S(2.0)M_vI_EW/R_dR_o$	DL+0.25SL	4	Same as 2005

DL=Dead load, SL=Snow load, DSL=Design snow load, LL=Live load.

*Table 4. Different retrofit methods and alternative design of CBs.*

Retrofit techniques		Method proposed by
Upgrading and retrofit methods	Steel plates on one side of shear-deficient CBs	Harries (1995)
	Upgrading the degree of coupling of CSWs	Chaallal and Nollet (1997)
	Bolting steel plates onto the vertical faces of CBs	Su and Zhu (2005), Su and Cheng (2011)
	Application of fiber-reinforced polymer sheet	Riazi *et al.* (2007)
Alternative designs of coupling beams	Steel CBs with and without stiffeners	Harries (1995)
	Concrete-filled steel-tube coupling beams	Teng *et al.* (1999)
	Steel CBs encased in reinforced concrete members	Gong and Shahrooz (2001a and 2001b)
	Embedded steel coupling beam with shear studs	Lam *et al.* (2001)

Table 5. Advantages and disadvantages of retrofit methods for coupling beams.

Retrofit method for coupling beams	Advantages	Disadvantages
Application of steel plates on one side of coupling beams	Improvement in strength, stiffness, displacement capacity, energy absorption, and hysteretic behavior. Less disruption of architectural appearance.	Debonding and peeling of steel plate. Possible out-of-plane buckling of steel plates.
Attaching external steel plates on the vertical faces of the coupling beams	Increase in stiffness, strength, and deformability of coupling beams.	Difficulty in determining the number of bolts. Weakens the concrete due to drilling of bolt holes. Decrease in ductility factor. Steel plate buckling.
Application of fiber-reinforced polymer sheet	High strength, high elastic modulus, and light weight of sheets; easy to install. Increase in dissipated energy, displacement ductility, and shear capacity.	Debonding of FRP sheets is a major problem which may cause complete loss of composite action between concrete and FRP.
Upgrading the degree of coupling of coupled shear walls	Optimum number and location of new coupling beams. Cost-effective. Minimum reduction of clearance for the passage of services. Useful for both retrofit and new construction.	Construction of new rigid coupling beams.
Replacing of coupling beams with steel or steel-concrete composite ones	Greater ductility level, stiffness, and energy-absorbing capability. Smaller beam dimensions and easy construction.	High concentration of compressive stress at the wall faces. Difficult to detail the wall reinforcement around the embedment region. Determination of appropriate embedment length and encasement strength and stiffness. Probable undesirable failure sequence
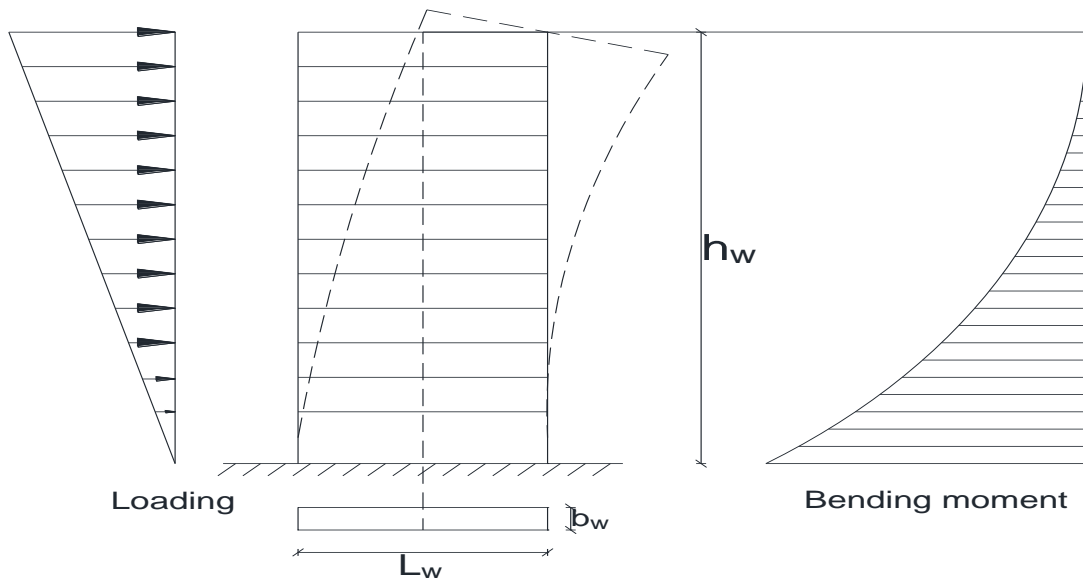
a)   Cantilever single shear wall subjected to lateral load
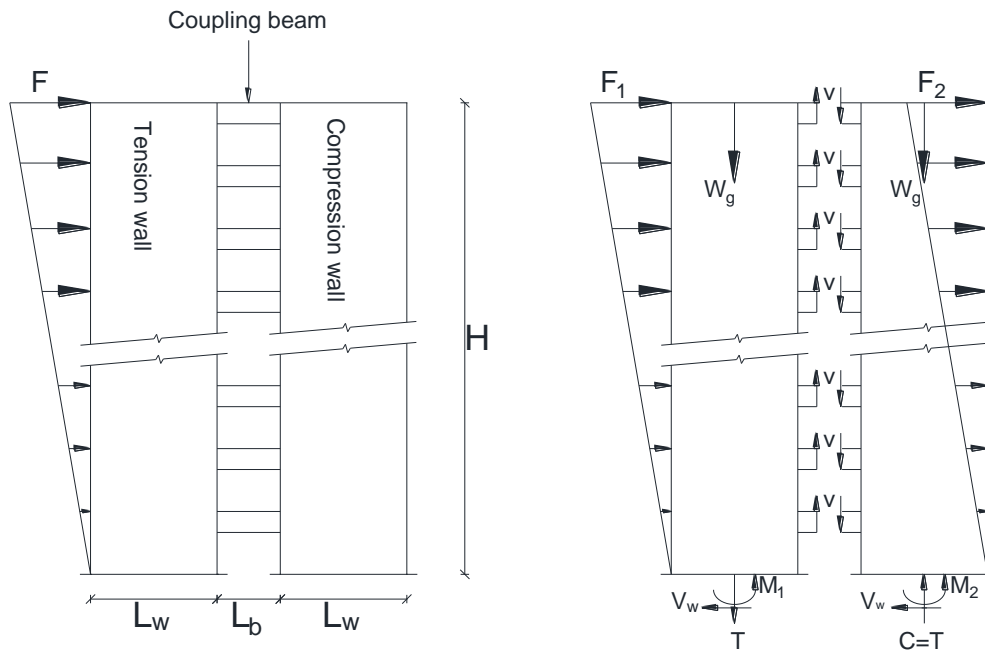
b)   Coupled shear walls subjected to lateral load
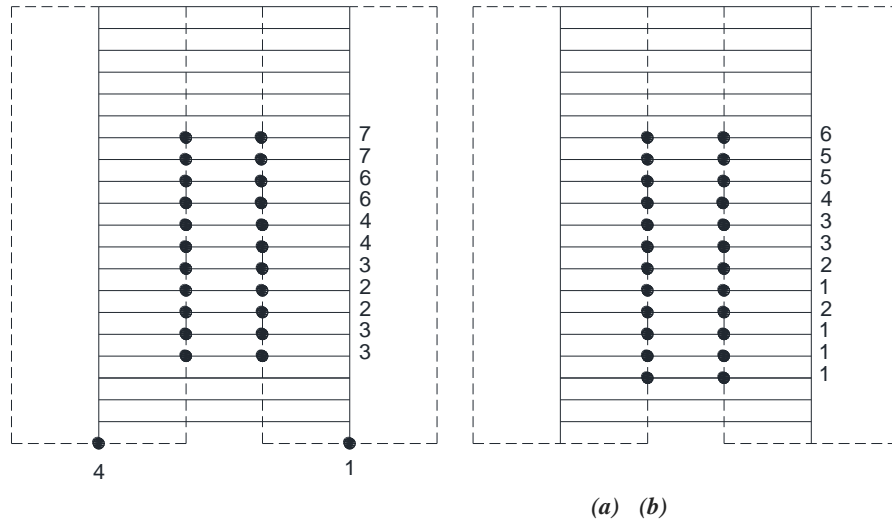***Figure 1. Shear walls under lateral load: a) Single shear wall, b) Coupled shear walls.***

(a)   (b)

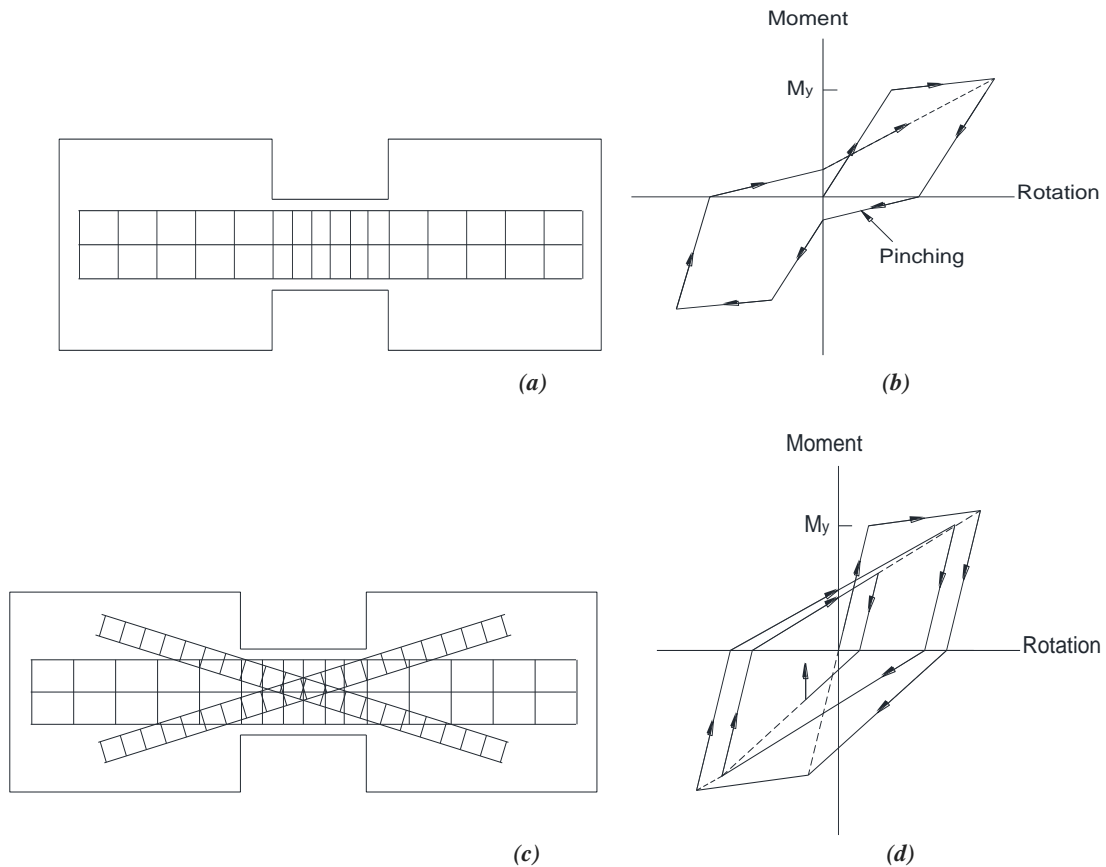*Figure 2. Plastic hinging sequence in CSWs: (a) Not desirable; (b) Desirable.*

(a) *(b)*

(c) *(d)*

Figure 3. Coupling beams: (a) Conventionally reinforced CB, (b) Hysteresis behavior of conventional CB, (c) Diagonally reinforced CB, (d) Hysteresis behavior of diagonal CB.
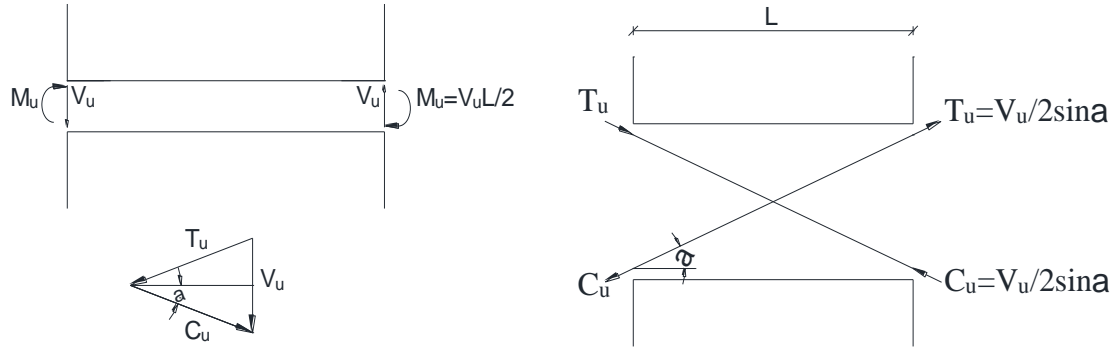
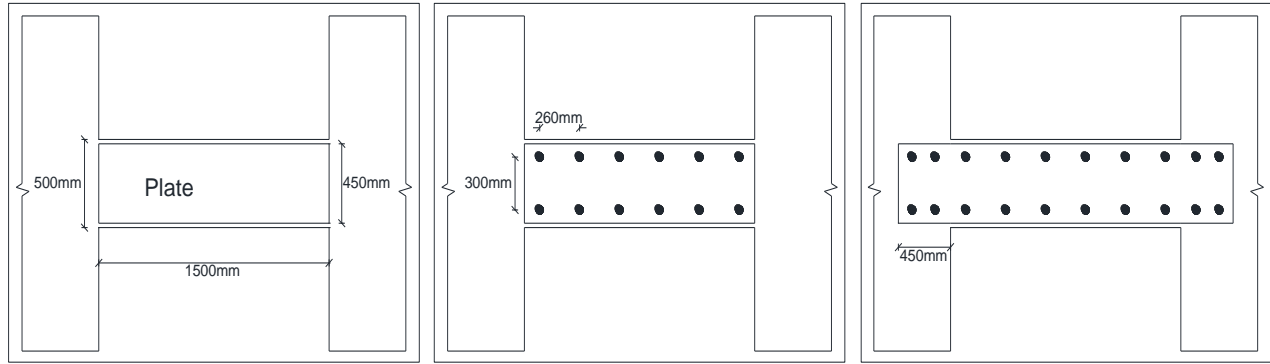*Figure 4. Distribution of forces in diagonal reinforcements (adapted from Harries 1995).*

(a)   (b)   (c)
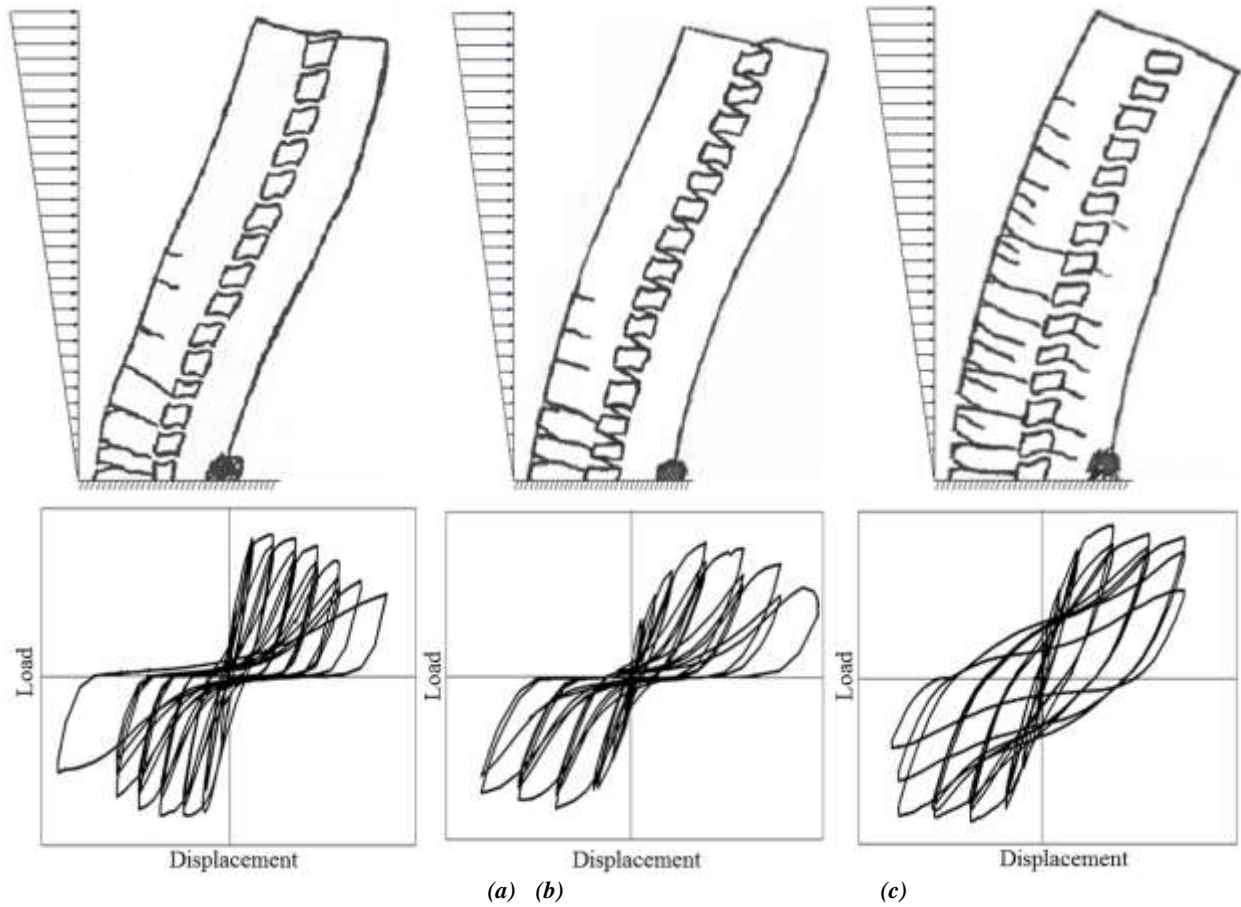
Figure 5. Modes of failure of CBs and their schematic sketch of load-displacement curve: a) Flexural failure, b) Shear failure, c) Rigid action (adapted from Subedi 1991).

(a)   *(b)*   *(c)*

Figure 6. Methods of attaching steel plate to CBs: a) Epoxied steel plate, b) Epoxied and bolted steel plate, c) Steel plate extended to walls (adapted from Harries 1995).
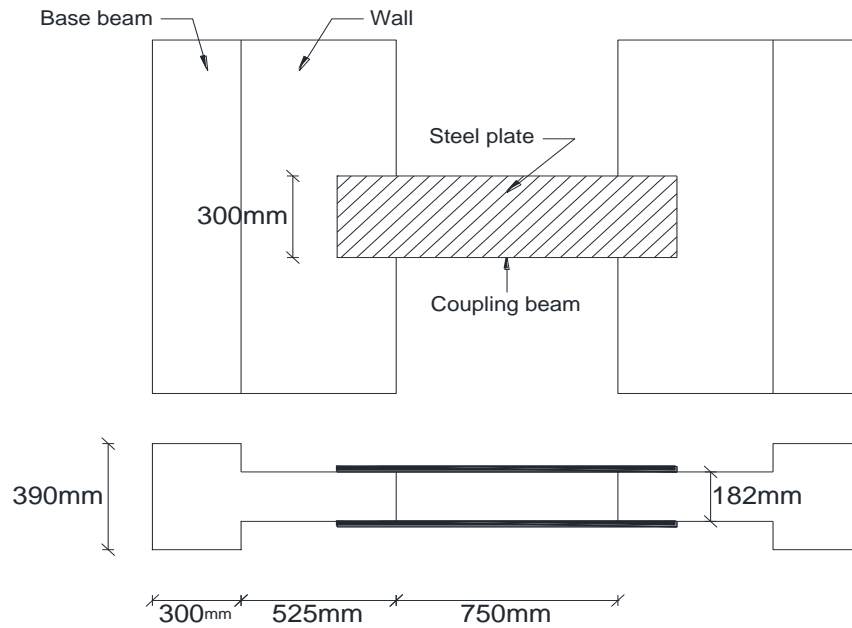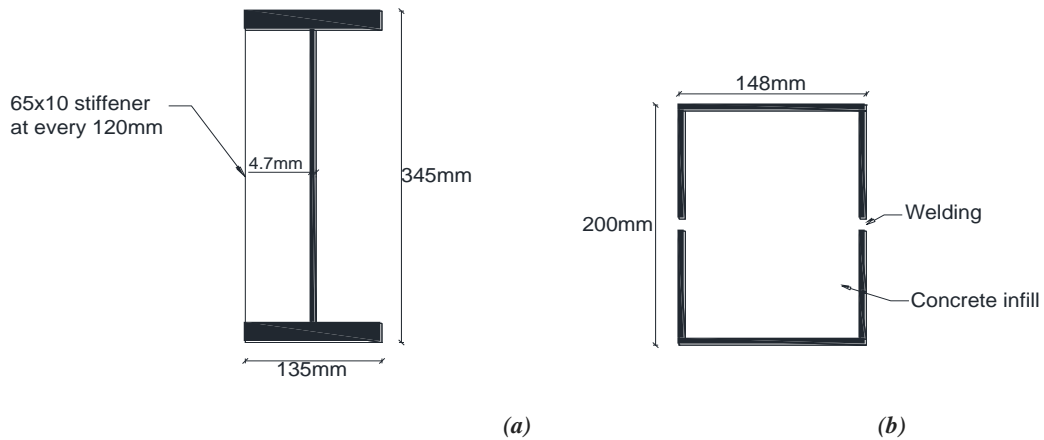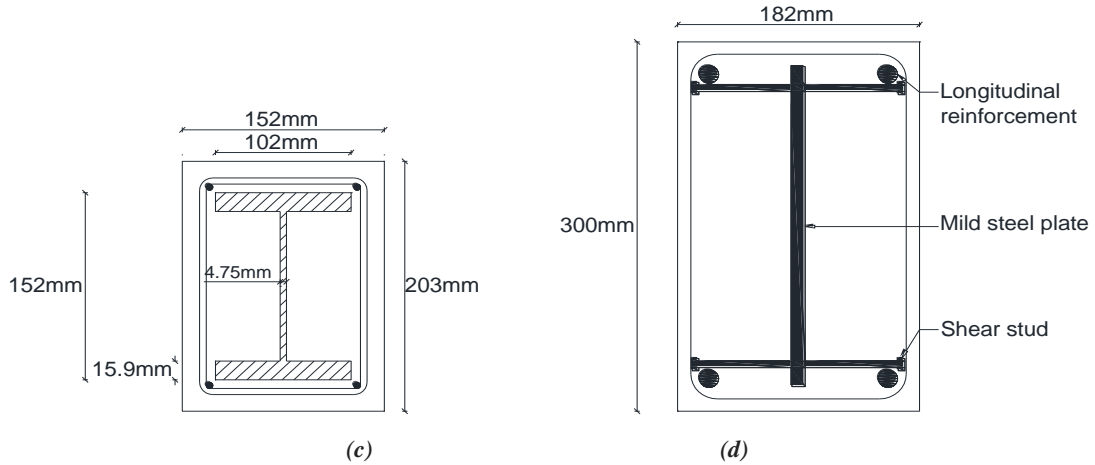

Figure 7. Configuration of specimens (adapted from Su and Zhu 2005).

(a)   *(b)*

(c)   *(d)*

1.   *Figure 8. Alternative designs of CBs: a) Steel coupling I-beam with stiffeners; b) Steel CB with concrete encasement; c) Concrete-filled steel-tube CB; d) Steel composite CB with shear studs (adapted from Lam et al. 2001).*