

## DESIGN OF PLATE-REINFORCED COMPOSITE COUPLING BEAMS FOR TALL BUILDINGS

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### ABSTRACT

Plate-reinforced composite (PRC) coupling beams are fabricated by embedding a vertical steel plate into conventional reinforced concrete coupling beams to enhance their strength and deformability. Shear studs are weld on the surfaces of the steel plate to transfer forces between the concrete and steel plate. Based on extensive experimental studies and numerical simulation of PRC coupling beams, a design procedure based on Hong Kong design codes of practice is proposed to aid engineers in designing this new type of beams. The proposed design guidelines consist of four main parts, which are (1) estimation of ultimate shear capacity of beam, (2) design of RC component and steel plate, (3) shear stud arrangement in beam span, and (4) design of plate anchorage in wall piers. An example is given to illustrate the use of the guidelines for the design of PRC coupling beams.

**Keywords:** Coupling Beams, Embedded Steel Plate, Shear Studs, Design Guidelines, Steel and Concrete Composite

## 1. INTRODUCTION

Coupling beams in coupled shear walls are often the most critical members in tall buildings subject to earthquake or wind loads. To ensure the survival of shear walls under high-intensity cyclic loading, these beams, which normally have limited dimensions, should possess high deformability and good energy absorption while being able to resist large shear forces. In view of that, the author and his co-workers developed plate reinforced composite (PRC) coupling beams which are fabricated by embedding a steel plate in a conventional RC beam and using shear studs to couple the steel plate and the concrete. PRC coupling beams are flexible in design and easy to construct. The insertion of steel plate has the least disturbance to reinforcement details, so that reinforcement from walls, slabs and coupling beams respectively can be harmoniously integrated together. The vertical arrangement of steel plate allows concrete to be placed and compacted easily, so honeycomb type of defects can be avoided. Furthermore, the cast-in steel plate can naturally be protected by the surrounding concrete against fire and lateral buckling. This new approach is a simple and economic design solution for high strength coupling beams. Extensive experimental and numerical studies [1-6] have been conducted to investigate the load-deformation behaviour and load-carrying capacity of PRC coupling beams. Based on the results and findings from these studies, design guidelines for this new type of beams according to British Standards were established [7]. To facilitate local engineers to conduct the design, the guidelines presented in this paper are revised based on the local construction practice [8, 9]. An example is given to illustrate the use of the guidelines for design of PRC coupling beams.

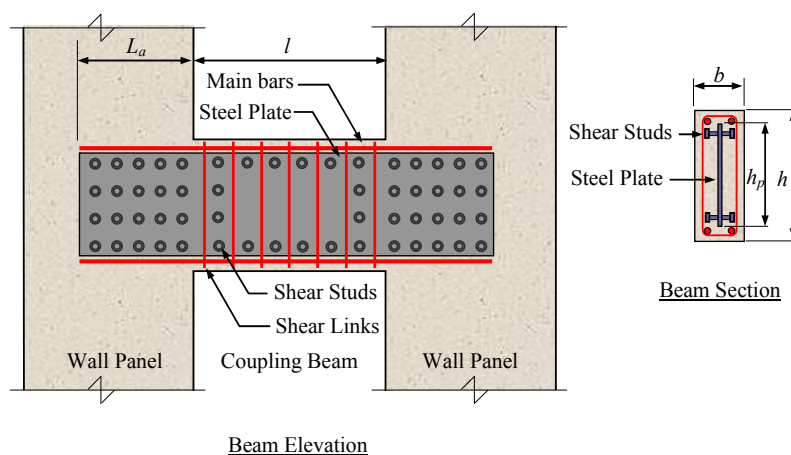


Figure 1. Typical Arrangement of PRC Coupling Beams

## 2. FEATURES OF PRC COUPLING BEAMS

Fig. 1 shows a typical arrangement and geometry of a PRC coupling beam. In this design, a steel plate is vertically embedded into the conventional RC beam section across the whole span. Throughout the span, shear studs are welded on both vertical faces of the plate along the top and the bottom longitudinal reinforcement to enhance the plate/ RC composite action. The plate is anchored in the wall piers and shear studs are provided in these regions to increase the plate bearing strength. With the embedded steel plate of a PRC coupling beam framing into the wall piers, a continuous shear transfer medium far less affected by concrete cracking at the beam-wall interfaces during inelastic stage is provided, thus preventing brittle failure and increasing the rotational deformability of the beam. The experimental study [5]

further indicated that the steel plate is effective in taking both shear and bending forces for deep coupling beams.

PRC coupling beams are flexible in design and easy to construct. By using different amounts of longitudinal reinforcement and steel plate, the flexural capacity of the beam can be easily adjusted to suit different magnitudes of design moment. Unlike other approaches, such as embedding steel sections in coupling beams, the insertion of steel plate has the least disturbance to reinforcement details, so that vertical, lateral and longitudinal reinforcement from walls, slabs and coupling beams respectively can be harmoniously integrated together. The vertical arrangement of steel plate allows concrete to be placed and compacted easily, so honeycomb type of defects can be avoided. Furthermore, the cast-in steel plate can naturally be protected by the surrounding concrete against fire and lateral buckling. Small holes through the plate to accommodate pipes and conduits are also possible. As shear studs are welded on the plate to couple the concrete element and the steel plate, it is much simpler, faster and economical than fabricating compound steel sections.

### 3. PROPOSED DESIGN PROCEDURES

The design procedure of PRC coupling beams described in this section is applicable to normal practical ranges of span/depth ratios ( $1 \leq l/h \leq 4$ ) and plate depth/ beam depth ratios ( $0.95 > h_p/h > 0.8$ )

#### 3.1 Ultimate Shear Capacity of PRC Coupling Beams

PRC coupling beams were recommended to be designed for shear stresses not exceeding 12MPa for grade 60 concrete. For other concrete grades lower than grade 60 concrete, the following ultimate shear stress capacity of PRC coupling beam should be adopted,

$$v_u = V_u / (bd) \leq 1.5\sqrt{f_{cu}} \leq 12\text{MPa} \quad (1)$$

where  $V_u$  is the ultimate shear force resisted by the beam,  $b$  is the beam width,  $d$  is the effective depth measured from top fiber to centre of longitudinal tensile steel and  $f_{cu}$  is the characteristic concrete cube strength. It should be noted that the material partial safety factor (1.25) of shear has been incorporated in Equation (1).

#### 3.2 Shear Resistance Design of Steel Plate

In the design of PRC coupling beams, a steel plate is cast in a conventional RC coupling beam to supplement the RC component for resisting shear. The steel plate is required to take up the additional shear when the design ultimate shear  $V_u$  exceeds the maximum allowable shear in the RC component  $V_{RC,allow}$ , which varies depending on the concrete grade. Numerical investigation [6] revealed that the load shared by the steel plate should not be more than  $0.45V_u$  even for beams with a small span/depth ratio and embedded with a thick plate. Thus the plate shear demand  $V_{p,req}$  is expressed as

$$V_{p,req} = V_u - V_{RC,allow} \leq 0.45V_u \quad (2)$$

According to the Code of Practice for the Structural Use of Steel 2005 [9], there are two possible cases for the design shear in a steel plate, low shear load and high shear load. Low shear load is defined as

$$V_{p,req} \leq 0.6p_y(0.9t_p h_p)/\sqrt{3} = 0.312p_y t_p h_p \quad (3)$$

where  $p_y$  is the design strength of the steel plate and  $t_p$  is the thickness of the steel plate. This load case usually corresponds to medium-length and long coupling beams with span-to-depth ratios,  $l/h \geq 1.5$ .

Alternatively, high shear load is defined as

$$V_{p,req} > 0.312p_y t_p h_p \quad (4)$$

This loading case is more often associated with short coupling beams ( $l/h < 1.5$ ). As the embedded steel plate is restrained from buckling, plastic hinge can be developed with sufficient rotation capacity, thus Class 1 plastic section is assumed in the design. In this case, bending strength of the steel plate have to be reduced by a factor  $1-\rho_1$ , where  $\rho_1$  has been given in the Code of Practice for the Structural Use of Steel 2005 [9] and is expressed as,

$$\rho_1 = 0.81 \left( \frac{2V_{p,req}}{V_p} - 1 \right)^2 \quad (5)$$

It is noted that in the cases of low shear load,  $\rho_1=0$ .

Based on Structural Use of Steel 2005 [9], the shear capacity  $V_p$  of steel plates can be obtained as:

for low shear load condition

$$V_p = 0.312p_y t_p h_p \geq V_{p,req} \quad (6a)$$

for high shear load condition

$$V_p = 0.519p_y t_p h_p \geq V_{p,req} \quad (6b)$$

At the beam-wall joints, the plate is subject to combined bending, axial and shear forces. In order to reserve sufficient load-carrying capacity for the steel plate to resist the bending and axial forces, for the cases of high shear load, the design shear load is advised to be

$$V_{p,req} \leq 0.8V_p \quad (7)$$

such that the stress reduction factor  $\rho_1$  is less than 0.3.

For sizing of plate, the depth of plate  $h_p$  can be determined geometrically. Based on Equations (2), (6) and (7), a suitable plate thickness  $t_p$  can be selected for short coupling beams ( $l/h \leq 1.5$ ), of which the design is controlled by shear force only. For the cases of long coupling beams where the design is controlled by bending, sizing of steel plate and reinforcement will be described in the next section.

### 3.3 Bending Resistance Design of Steel Plate and RC Section

In typical RC coupling beam designs, the same amount of top and bottom reinforcements is often provided as both reinforcements are required for taking tension under reversing cyclic loads. Also, because of the plate/RC interaction in a PRC coupling beam, the RC component will be under an axial compression and the standard design procedure for RC beams in Code of Practice for Structural Use of Concrete 2004 [8] cannot be applied to determine the required longitudinal reinforcement. A new design procedure for design PRC coupling beams under bending is proposed herein.

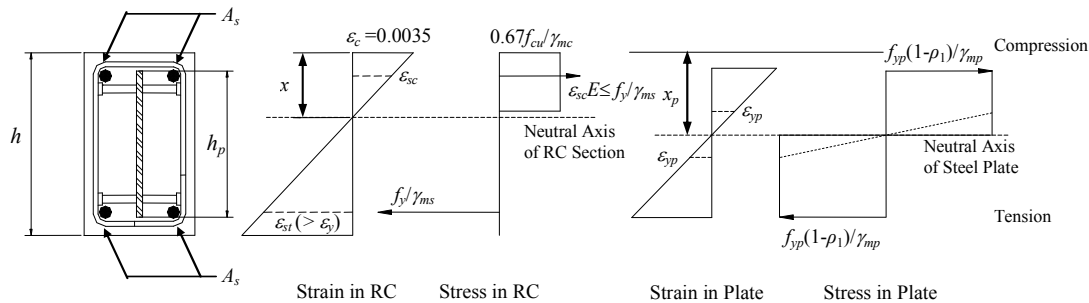


Figure 2. Strain and Simplified Stress Diagrams of Beam Section under Ultimate Sagging Moment

Under partial plate/RC composite action, the flexural strains of the concrete and the steel plate will not be the same. Horizontal forces  $F_x$  would be exerted on the RC part and the steel plate respectively in equal magnitudes but at opposite directions. The previous numerical and experimental investigations found that beam-wall joints are the most critical location for the plate design and yielding often occurs at the ultimate loading stage. The simplified stress blocks of member forces at beam-wall joints are shown in Fig. 2. The force of longitudinal compression steel can be expressed as

$$C_s = \varepsilon_{sc} EA_s = \frac{x - (h - d)}{x} \cdot \varepsilon_c EA_s \leq \frac{f_y A_s}{\gamma_{ms}} \quad (8)$$

where  $\varepsilon_{sc}$  is the strain of longitudinal compression steel,  $\varepsilon_c$  is the ultimate compressive strain of concrete,  $E$  is the Young's modulus of steel bars,  $x$  is the neutral axis depth,  $f_y$  is the yield strength of reinforcement,  $A_s$  is the area of longitudinal tensile or compressive reinforcement and  $\gamma_{ms}$  is the partial safety factor of reinforcement. Using the simplified rectangular stress block, the compression of concrete can be obtained and expressed in Equation (9).

$$C_c = \frac{0.67 f_{cu} \times 0.9(b - t_p)x}{\gamma_{mc}} = \frac{0.603 f_{cu} (b - t_p)x}{\gamma_{mc}} \quad (9)$$

where  $\gamma_{mc}$  is the partial safety factor of concrete. When the concrete compressive strain has reached its ultimate value ( $\epsilon_c = 0.0035$  for  $f_{cu} \leq 60\text{MPa}$ ), the deformation of the longitudinal tensile steel has usually exceeded the yield limit and the tensile force of the reinforcement can be calculated by Equation (10).

$$T_s = A_s f_y / \gamma_{ms} \tag{10}$$

Due to the horizontal force interaction between the RC part and the steel plate, a net compressive force  $F_x$  is exerted on the RC section at the beam-wall joints. In the parametric study [6], the tensile force  $F_x$  was found to be dependent on the steel ratio of the longitudinal bars, the plate thickness to beam width ratio, and the span/depth ratio of the beam. The plots of  $F_x / V_u$  against  $l/h$  with various steel ratios  $\rho_s$  are reproduced in Fig. 3 for reference.

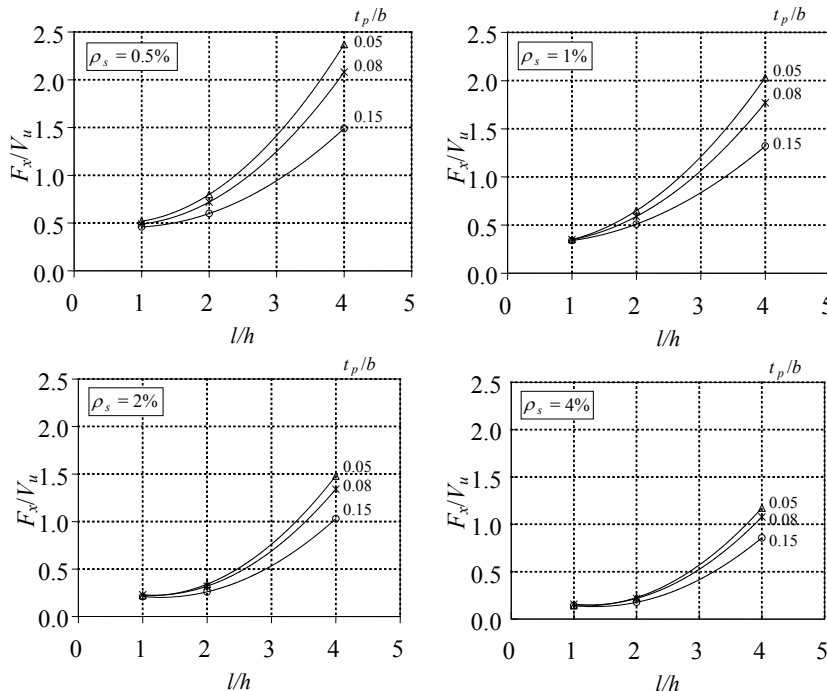


Figure 3. Axial Force of Steel Plate at Beam-Wall Joint

When  $\epsilon_{sc} < \epsilon_y$ , where  $\epsilon_y$  is the yield strain of reinforcement, the neutral axis depth of RC section is derived as,

$$x = \frac{F_x + \left( \frac{f_y}{\gamma_{ms}} - \epsilon_c E \right) A_s + \sqrt{\left[ F_x + \left( \frac{f_y}{\gamma_{ms}} - \epsilon_c E \right) A_s \right]^2 + 2.412 \frac{f_{cu}}{\gamma_{mc}} (b - t_p) (h - d) \epsilon_c E A_s}}{1.206 \frac{f_{cu}}{\gamma_{mc}} (b - t_p)} \tag{11a}$$

Alternatively, when  $\epsilon_{sc} \geq \epsilon_y$ , the neutral axis depth can be simplified to

$$x = \frac{\gamma_{mc} F_x}{0.603 f_{cu} (b - t_p)} \tag{11b}$$

From the force equilibrium, the neutral axis depth of the steel plate can be expressed as

$$x_p = \frac{h}{2} - \frac{F_x}{2p_y t_p (1 - \rho_1)} \quad (12)$$

The compression of steel plate is equal to

$$C_p = \frac{p_y (1 - \rho_1) h_p t_p}{2} - \frac{F_x}{2} \quad (13)$$

And the tension of steel plate is

$$T_p = \frac{p_y (1 - \rho_1) h_p t_p}{2} + \frac{F_x}{2} \quad (14)$$

Taking moment at the neutral axis of the RC section, the bending moment capacities of RC section and steel plate are expressed, respectively, as

$$M_{RC} = T_s (d - x) + C_s (x - (h - d)) + 0.55 C_c x \quad (15)$$

$$M_p = T_p \left( x_p - x + \frac{h_p}{4} + \frac{F_x}{4t_p p_y (1 - \rho_1)} \right) + C_p \left( x - x_p + \frac{h_p}{4} - \frac{F_x}{4t_p p_y (1 - \rho_1)} \right) \quad (16)$$

By choosing a suitable steel ratio of longitudinal reinforcement  $A_s$  and plate thickness  $t_p$ , the total bending capacity of the section can be designed to be greater than the ultimate design moment  $M_u$ , i.e.

$$M_{RC} + M_p \geq M_u \quad (17)$$

### 3.4 Shear Resistance Design of RC Section

After determining the steel area of reinforcement  $A_s$ , the shear reinforcement can be provided to resist the shear force  $V_u - V_{p,req}$  and the corresponding shear stress, i.e.

$$v_{RC,req} = \frac{V_u - V_{p,req}}{(b - t_p)d} \quad (18)$$

where  $v_{RC,req}$  is the required shear stress in RC part. The design concrete shear strength according to Structural Use of Concrete 2004 [8] is

$$v_c = \frac{0.79}{\gamma_{mv}} \cdot \left( \frac{100A_s}{(b - t_p)d} \right)^{1/3} \left( \frac{400}{d} \right)^{1/4} \left( \frac{f_{cu}}{25} \right)^{1/3} \quad (19)$$

where  $\frac{100A_s}{(b - t_p)d} \leq 3$  and  $\frac{400}{d} \geq 1$ .

For span/depth ratio  $l/h \leq 4$ , shear enhancement described in the clause 6.1.2.5, Structural Use of Concrete 2004 [8] may be used. The design shear stress may be increased by  $2d/a_v$ , where  $a_v$  is the distance measured from the concentrated load to the support and it may be taken as half of the span length of the coupling beam. In any case, the design shear stress should not be higher than  $0.8\sqrt{f_{cu}}$  or 7MPa [8].

$$\frac{A_{sv}}{s_v} \geq \frac{(b - t_p) \cdot (v_{RC,req} - v_c)}{f_{yv} \gamma_{mv}} \quad (20)$$

where  $A_{sv}$ ,  $s_v$  and  $f_{yv}$  are the area, the spacing and the yield strength of transverse reinforcement respectively. It should be noted that Equation (20) has only considered the shear area from the shear links. The shear area contributed from the vertical steel plate is ignored conservatively.

### 3.5 Shear Stud Arrangement in Beam Span

Shear studs are required in the beam span to serve four functions as illustrated in Fig. 4. Based on the observations from the parametric study [6], design equations for estimating the required shear connection strengths, and the numbers of shear studs required in turn, for these functions are listed below.

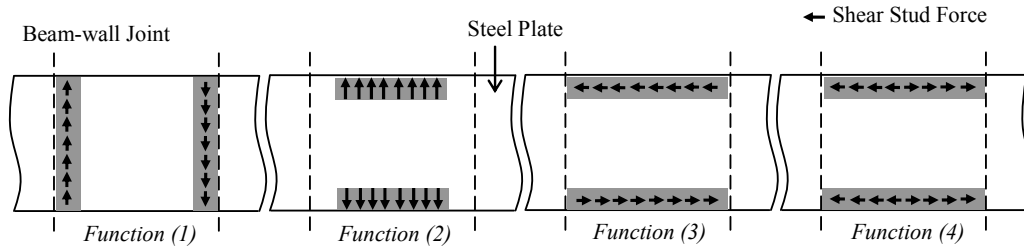


Figure 4. Functions of Shear Studs in Beam Span

### 3.6 Function 1: Vertical Stud Forces for Inducing Shear on Steel Plate

Shear studs should be provided within a width of  $0.3 h_p$  away from the beam-wall joint at each beam end for transferring the plate shear force. The required transverse shear connection force for inducing shear on the steel plate  $P_{t1,req}$  is

$$P_{t1,req} = 0.065 \frac{\rho_p^{0.75} V_u}{\rho_s^{0.1}} \quad (21)$$

where  $\rho_p = 100h_p t_p / (hb)$  [%] is the ratio of plate sectional area to the beam sectional area and  $\rho_s = 100A_s / (hb)$  [%] is the steel ratio.

### 3.7 Function 2: Vertical Stud Forces for Maintaining Tension Tie Effect of Steel Plate

Such forces are provided within the central  $(1-0.6h_p)$  region near the top and the bottom plate fibres, and the required transverse shear connection strength for providing tension tie effect  $P_{t2,req}$  was found to be



$$P_{t2,req} = 0.3(l/h)^{0.65} \rho_s^{0.1} V_u \tag{22}$$

**3.8 Function 3: Horizontal Stud Forces for Inducing Moment on Steel Plate**

It was proposed that the required longitudinal shear connection force within the beam span for transferring moment  $P_{11,req}$  would be

$$P_{11,req} = 0.165 \left(\frac{l}{h}\right)^{0.3} \rho_s^{0.45} \rho_p^{0.4} V_u \tag{23}$$

**3.9 Function 4: Horizontal Stud Forces for Inducing Axial Force on Steel Plate**

The required longitudinal shear connection forces within the beam span for inducing axial force on steel plate  $P_{12,req}$  was expressed as

$$P_{12,req} = \max \left\{ \begin{array}{l} 0.0217 \left(\frac{l}{h}\right)^{2.2} \frac{M_u}{\rho_s^{0.75} \rho_p^{0.15} h_p} \\ 0.106 \left(\frac{h}{l}\right)^{0.4} \frac{V_u}{\rho_s^{0.07} \rho_p^{0.2}} \end{array} \right. \tag{24}$$

It should be noted that the contribution of natural plate/RC bonding to transfer shear forces is ignored in the design.

The numerical results [6] showed that shear studs provided in the central beam region could not be effectively mobilized. By arranging all shear studs near the beam-wall joints and near the top and the bottom fibres of the steel plate, where shear studs could be effectively mobilized, a high degree of shear stud mobilization could be assumed.

According to Structural Use of Steel 2005 [9], the shear stud force  $Q$  is taken as  $0.8Q_k$  and  $0.6Q_k$  under positive and negative moments respectively when designing conventional composite beams with RC slabs and structural steel beams interconnected by shear studs. As the plate/RC interface slips in a PRC coupling beam are unlikely to be as large as in the case of conventional composite beams under positive moments,  $Q = 0.6Q_k$  is considered when calculating the numbers of shear studs required in different regions in the beam span.

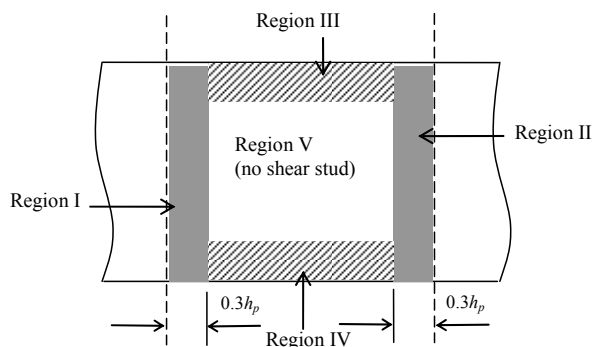


Figure 5. Regions of Steel Plate in Beam Span for Shear Stud Arrangement

The steel plate of a PRC coupling beam can be divided into five regions in the beam span

according to the different shear stud arrangements (Fig. 5). Based on the above proposals, and assuming the width of Regions I and II to be  $0.3h_p$ , the required numbers of shear studs ( $n_{req}$ ) in different regions are expressed as follows:

Region I or Region II:

$$n_{req} = \frac{\sqrt{(0.5P_{t1,req})^2 + [0.3(h_p / l)(P_{l1,req} + P_{l2,req})]^2}}{0.6Q_k} \quad (25)$$

Region III or Region IV:

$$n_{req} = \frac{\sqrt{(0.5P_{t2,req})^2 + [(0.5 - 0.3h_p / l)(P_{l1,req} + P_{l2,req})]^2}}{0.6Q_k} \quad (26)$$

### 3.10 Design of Plate Anchorage in Wall Piers

The simplified plate anchorage design model adopted in this paper is depicted in Fig. 6. Design plate anchorage loads near the beam-wall joints

The plate anchors of a PRC coupling beam are designed to take up the ultimate plate moment  $M'_p$  and part of the ultimate plate shear  $F'_y$  (as part of the shear transfer will take place in the beam span). They also need to resist an axial force  $F'_x$  jointly induced by the plate/RC interaction under bending and the beam elongation upon cracking of concrete.

The load-carrying capacity of PRC coupling beams and the plate anchorage design loads near the beam-wall joints are controlled by the shear and flexural capacities of the steel plate. The shear and flexural capacities of PRC coupling beams are expressed, respectively, in Equations (27) and (28).

Shear capacity  $V_{PRC} = V_{RC} + V_p \geq V_u$  (27)

Flexural capacity  $M_{PRC} = M_{RC} + M_p \geq M_u$  (28)

where  $V_{RC}$ ,  $V_p$ ,  $M_{RC}$  and  $M_p$  can be calculated from Equations (6), (15), (16) and (18) to (20).

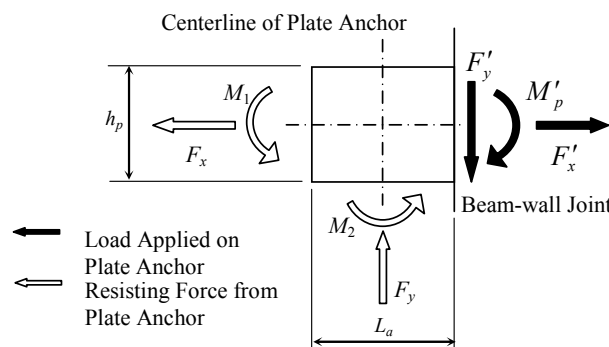


Figure 6. Simplified Plate Anchorage Design Model

The PRC coupling beam is flexural-controlled when  $V_{PRC} > 2M_{PRC} / l$  or shear-controlled

when  $V_{PRC} < 2M_{PRC}/l$ . When the beam is flexural-controlled, the plate has reserved shear but not flexural capacity. In such cases, the capacities of the steel plate, the plate anchorage and the PRC coupling beam are all governed by the yield moment of the steel plate. The design applied moment of the plate anchorage (equal to the yield moment) can be obtained by taking moment about the centroid of the plate, and is expressed in Equation (29).

$$M'_p = C_p \left[ \frac{(h_p + h)}{2} - x_p \right] \quad (29)$$

Assuming the plate has fully yielded with the stress distribution as shown in Fig. 2, the compression of steel plate can be determined and Equation (29) becomes

$$M'_p = \left[ \left( \frac{h_p}{2} \right)^2 - \left( \frac{h}{2} - x_p \right)^2 \right] p_y t_p (1 - \rho_1) \quad (30)$$

The shear load taken by the steel plate near the beam-wall joint, which is less than or equal to the shear capacity of the steel plate, is estimated by Equation (31).

$$V'_p = V_{PRC} \left( \frac{M'_p}{M_{PRC}} \right) \leq V_p \quad (31)$$

Conversely, when the beam is shear-controlled, the shear load taken by the steel plate near the beam-wall joint may be obtained from Equation (6), such that,  $V'_p = V_p$ . The design plate anchorage moment of the steel plate may then be estimated as,

$$M'_p = M_{PRC} \left( \frac{V'_p}{V_{PRC}} \right) \leq M_p \quad (32)$$

It was observed in the parametric study [6] that the plate anchor of a PRC coupling beam would take up about 50 to 75% of the design plate shear  $V'_p$ . As the shear studs in beam span near each beam-wall joint have been designed (in Equation 21) to transfer  $0.5P_{t1,req}$  to the plate, the remaining vertical force required to be taken by the plate anchor is

$$F'_y = V'_p - 0.5P_{t1,req} \quad (33)$$

The axial force  $F'_x$  induced on a plate anchor is equal to  $F_x$  acting on the plate and can be determined from Fig. 3. The design plate anchorage loads obtained ( $M'_p$ ,  $F'_x$  and  $F'_y$ ) will be used for calculating the bearing stress distribution and designing the shear stud arrangement in plate anchors.

### 3.11 Bearing Stress Distributions and Shear Stud Arrangements in Plate Anchors

Taking moment about the centroid of beam section at the beam-wall joint, the required resisting moments of the plate anchor can be expressed as:

$$M_1 + M_2 = 0.5F'_y L_a + M'_p \tag{34}$$

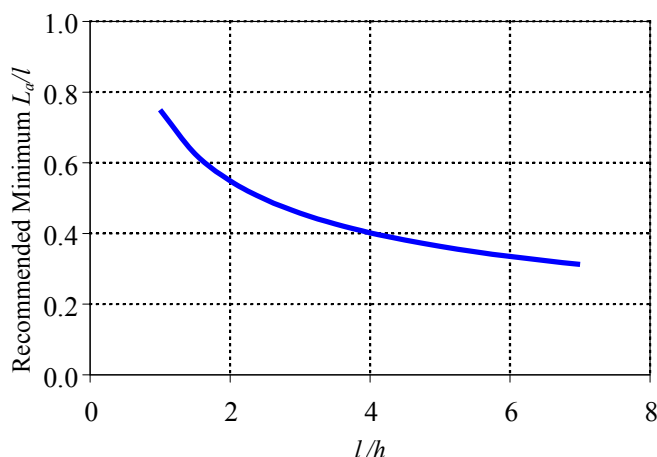


Figure 7. Recommended Minimum Plate Anchorage Length

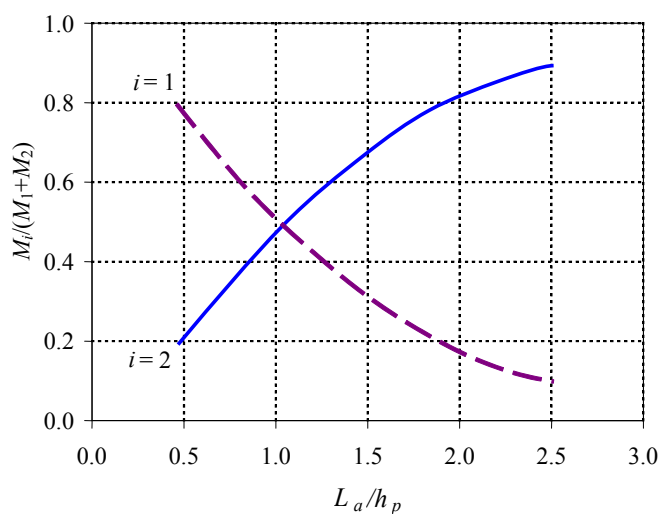


Figure 8. Distributions of Resisting Moments in Plate Anchor

The required moment resistance has to be determined in conjunction with the plate anchorage length  $L_a$  of which the minimum length is given in Figure 7. Note that slightly longer plate anchorage length than the recommended minimum value may be assumed first, as the recommendation is based on the arrangement of shear studs at minimum allowable spacing throughout the whole anchor, which is not necessarily the case in the design. The distributions of resisting moments, which depend on the geometry of the plate anchor, were investigated in the parametric study [3] and are plotted in Fig. 8.

Assuming high degrees of shear stud mobilizations (i.e.  $Q = 0.6Q_k$ ), the required numbers of shear studs  $n_{req}$  in different regions are calculated from Equations (35) and (36). The design envelopes for the bearing stress distributions in the vertical and the horizontal directions are

shown in Fig. 9 for arranging shear studs in Regions I and II in the plate anchors.

Region I (width  $L_I = F'_y L_a^2 / 6M_2$ ):

$$n_{req} = \frac{\sqrt{\left(\frac{L_I}{L_a}\right)^2 \left(\frac{w_x h_p}{2}\right)^2 + (w_{y1} L_I)^2}}{0.6Q_k} \quad (35)$$

Region II (width  $L_{II} = (1 - F'_y L_a / 6M_2) L_a$ ):

$$n_{req} = \frac{\sqrt{\left(\frac{L_{II} w_x h_p}{L_a}\right)^2 + (w_{y2} L_{II})^2}}{0.6Q_k} \quad (36)$$

where

$$w_x = \frac{6M_1 + 2F'_x h_p}{h_p^2} \quad (37)$$

$$w_{y1} = \frac{6M_2 + L_a F'_y}{L_a^2} \quad (38)$$

$$w_{y2} = \frac{6M_2 - L_a F'_y}{L_a^2} \quad (39)$$

Note that although the effects of shear studs in the shaded area in Fig.9 are ignored, evenly distributed shear studs are provided in Region II for simplicity. Furthermore, the code [9] states that the minimum allowable shear stud spacing is five times and four times the nominal shank diameter in the directions along and perpendicular to the major shear stud action respectively. As the major shear stud actions can either be in the horizontal or in the vertical directions, it is recommended that a minimum shear stud spacing of five times the nominal shank diameter be provided in all cases.

By setting the material partial safety factors equal to unity and using the aforementioned design procedure, the theoretical design capacity can be calculated. By comparing the predicted design capacity with the numerical and experimental results, the reliability and accuracy of the proposed design procedure following the British Standards have been verified [7]. Consistent results from the proposed design procedure and finite element analysis were observed over a wide range of span/ depth ratios, plate thicknesses and steel ratios of PRC coupling beams. In general, the predicted capacity underestimated that from the numerical analysis by around 10%.

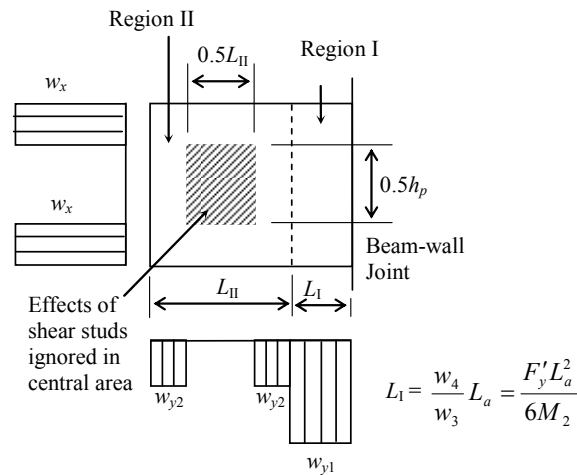


Figure 9 Regions of plate anchor with different shear stud arrangements and simplified design bearing stress blocks

#### 4. A DESIGN EXAMPLE

The design information of the long PRC coupling beam is listed as follows:

Dimensions:  $l = 2\text{m}$ ,  $b = 250\text{mm}$ ,  $h = 500\text{mm}$ ,  $\text{cover} = 30\text{mm}$

Material strengths:  $f_{cu} = 40\text{MPa}$ ,  $f_y = f_{yv} = 460\text{MPa}$ ,  $p_y = 345\text{MPa}$ ,  $E = 205000\text{MPa}$

Design ultimate loading:  $V_u = 800\text{kN}$ ,  $M_u = 800\text{kNm}$

Partial safety factors:  $\gamma_{mc} = 1.5$ ,  $\gamma_{mv} = 1.25$ ,  $\gamma_{ms} = 1.15$

Main longitudinal steel = T40 and steel plate thickness = 25mm

##### 4.1 Ultimate Shear Resistance

Assume using T40 longitudinal reinforcement and T12 stirrups,

$d \approx 500 - 30 - 12 - 20 = 438\text{mm}$

Ultimate shear resistance of the PRC coupling is

$$v_u = \frac{800 \times 10^3}{250 \times 438} = 7.3\text{N/mm}^2 \leq 1.5\sqrt{40} = 9.48\text{N/mm}^2 \leq 12\text{N/mm}^2$$

##### 4.2 Shear Resistance Design of Steel Plate

$d/h = 438/500 = 0.88 \approx 0.9$

$h_p \approx 500 - 30 - 30 - 12 - 12 - 15 = 401\text{mm}$

Adopt  $h_p = 400\text{mm}$

$h_p/h = 400/500 = 0.8$

Use Grade S355 steel plate, assuming  $16\text{mm} < t_p \leq 40\text{mm}$ ,  $p_y = 345\text{N/mm}^2$

$V_{RC,allow} \approx 5 \times (250 - 25) \times 438/1000 = 492\text{kN}$

$V_{p,req} = V_u - 492 = 308\text{kN} \leq 0.45V_u = 360\text{kN}$

Hence, for low shear load

$V_p = 0.312 \times 345 \times 400 \times 25/1000 = 1076\text{kN} \geq 308\text{kN}$

### 4.3 Bending Resistance Design of Steel Plate and RC Section

$$\rho_s = 2 \times 1256 \times 100 / (500 \times 250) = 2.01\%, \quad t_p / b = 25 / 250 = 0.1$$

From Fig. 3,  $F_x = 1.2V_u = 1.2 \times 800 = 960\text{kN}$

According to Equation (11a)

$$F_x + \left( \frac{f_y}{\gamma_{ms}} - \varepsilon_c E \right) A_s = 960000 + \left( \frac{460}{1.15} - 0.0035 \times 205000 \right) 1256 \times 2 = 162,440\text{N}$$

Assume  $\varepsilon_{sc} < \varepsilon_y$ , then

$$x = \frac{162440 + \sqrt{162440^2 + 2.412 \frac{40}{1.5} (250 - 25)(500 - 438) 0.0035 \times 205000 \times 2512}}{1.206 \frac{40}{1.5} (250 - 25)}$$

$$= 199.6\text{mm}$$

From Equation (8),

$$C_s = \frac{199.6 - (500 - 438)}{199.6} \cdot 0.0035 \times 205000 \times 2512 / 1000 = 1242 > \frac{460 \times 2512}{1.15 \times 1000} = 1005\text{kN}$$

Hence the compressive reinforcement has been yielded and from Equation (11b)

$$x = \frac{1.5 \times 960000}{0.603 \times 40(250 - 25)} = 265\text{mm}$$

$$C_s = T_s = \frac{460 \times 2512}{1.15 \times 1000} = 1005\text{kN}$$

$$C_c = \frac{0.603 \times 40(250 - 25)265}{1.5 \times 1000} = 960\text{kN}$$

Hence the moment resistance of RC part according to Equation (15) is

$$M_{RC} = [1005(438 - 265) + 1005(265 - (500 - 438)) + 0.55 \times 960 \times 265] / 1000 = 518\text{kNm}$$

By Equations (12) to (14)

$$x_p = \frac{500}{2} - \frac{960000}{2 \times 345 \times 25(1 - 0)} = 194\text{mm}$$

$$C_p = \frac{345(1 - 0)}{1000} \frac{400 \times 25}{2} - \frac{960}{2} = 1245\text{kN}$$

$$T_p = \frac{345(1 - 0)}{1000} \frac{400 \times 25}{2} + \frac{960}{2} = 2205\text{kN}$$

From Equation (16), the moment resistance of the steel plate is

$$M_p = \left[ 2205 \left( 194 - 265 + \frac{400}{4} + \frac{960000}{4 \times 25 \times 345(1 - 0)} \right) + 1245 \left( 265 - 194 + \frac{400}{4} - \frac{960000}{4 \times 25 \times 345(1 - 0)} \right) \right] / 1000 = 304\text{kNm}$$

Total moment resistance of the composite beam is

$$M_{RC} + M_p = 518 + 304 = 822\text{kNm} \geq 800\text{kNm}$$

#### 4.4 Shear Resistance Design of RC Section

As the span-to-depth ratio of the PRC coupling beam is  $4 \gg 1.5$ , the beam is flexural-controlled. The design moment of the steel plate, according to Equation (29), is

$$M'_p = 1245 \left[ \frac{(0.4 + 0.5)}{2} - 0.194 \right] = 318 \text{ kNm}$$

The design shear load in the steel plate near the beam-wall joint may be estimated by Equation (31)

$$V'_p = 800 \left( \frac{318}{800} \right) = 318 \text{ kN}$$

$$v_{RC,req} = \frac{(800 - 318) \times 10^3}{(250 - 25)438} = 4.89 \text{ N/mm}^2$$

$$v_c = \frac{0.79}{1.25} \cdot \left( \frac{100 \times 2512}{(250 - 25)438} \right)^{1/3} \left( \frac{40}{25} \right)^{1/3} = 1.01 \text{ N/mm}^2$$

$$\frac{A_{sv}}{s_v} \geq \frac{(250 - 25) \cdot (4.89 - 1.01)}{460/1.15} = 1.65 \text{ mm}^2/\text{mm}$$

Provide **T12-125-S.S.**,  $\frac{A_{sv}}{s_v}$  (provided) = 1.81 mm<sup>2</sup>/mm

#### 4.5 Shear Stud Arrangement in Beam Span

Use shear studs of 16mm shank diameter,  $Q_k = 66.5 \text{ kN}$ , minimum spacing = 80mm, as-welded height = 70mm,  $\rho_p = 8\%$ ,  $\rho_s = 2.01\%$ ,

$$P_{t1,req} = 0.065 \frac{8^{0.75}}{2.01^{0.1}} 800 = 230 \text{ kN}$$

$$P_{t2,req} = 0.3 \times 4^{0.65} \times 2.01^{0.1} \times 800 = 633 \text{ kN}$$

$$P_{l1,req} = 0.165 \times 4^{0.3} \times 2.01^{0.45} \times 8^{0.4} \times 800 = 629 \text{ kN}$$

$$P_{l2,req} = \max \left\{ \begin{array}{l} 0.0217 \times 4^{2.2} \frac{800 \times 10^3}{0.4 \times 2.01^{0.75} \times 8^{0.15} \times 10^3} = 397 \text{ kN} \\ 0.106 \times 0.25^{0.4} \times \frac{800}{2.01^{0.07} \times 8^{0.2}} = 31 \text{ kN} \end{array} \right. = 397 \text{ kN}$$

For Region I or Region II,

$$n_{req} = \frac{\sqrt{(0.5 \times 230)^2 + [0.3(400/2000)(629 + 397)]^2}}{0.6 \times 66.5} = 3.2$$

For Region III or Region IV,

$$n_{req} = \frac{\sqrt{(0.5 \times 633)^2 + [(0.5 - 0.3 \times 400/2000)(629 + 397)]^2}}{0.6 \times 66.5} = 13.8$$

Provide **2 shear studs in one column in Region I or Region II**, and **7 shear studs in one row in Region III or Region IV** on each side of plate.



### 4.6 Plate Anchorage Design

From Fig. 3,  $F'_x = F_x = 1.2V_u = 960\text{kN}$ .

From Equation (33),  $F'_y = 318 - 0.5 \times 230 = 203\text{kN}$

Referring to Fig. 7, assume  $L_a = 0.45l = 900\text{mm}$

$$\frac{L_a}{h_p} = \frac{900}{400} = 2.25$$

From Fig. 8,

$$\frac{M_1}{M_1 + M_2} = 0.14 \text{ and } \frac{M_2}{M_1 + M_2} = 0.86$$

From Equation (34),  $M_1 + M_2 = 0.5F'_yL_a + M'_p = 0.5 \times 203 \times 0.9 + 318 = 409\text{kNm}$

$M_1 = 57\text{kNm}$  and  $M_2 = 352\text{kNm}$

$$L_I = \frac{203 \times 0.9^2 \times 1000}{6 \times 352} = 77\text{mm} \text{ and } L_{II} = 900 - 77 = 823\text{mm}$$

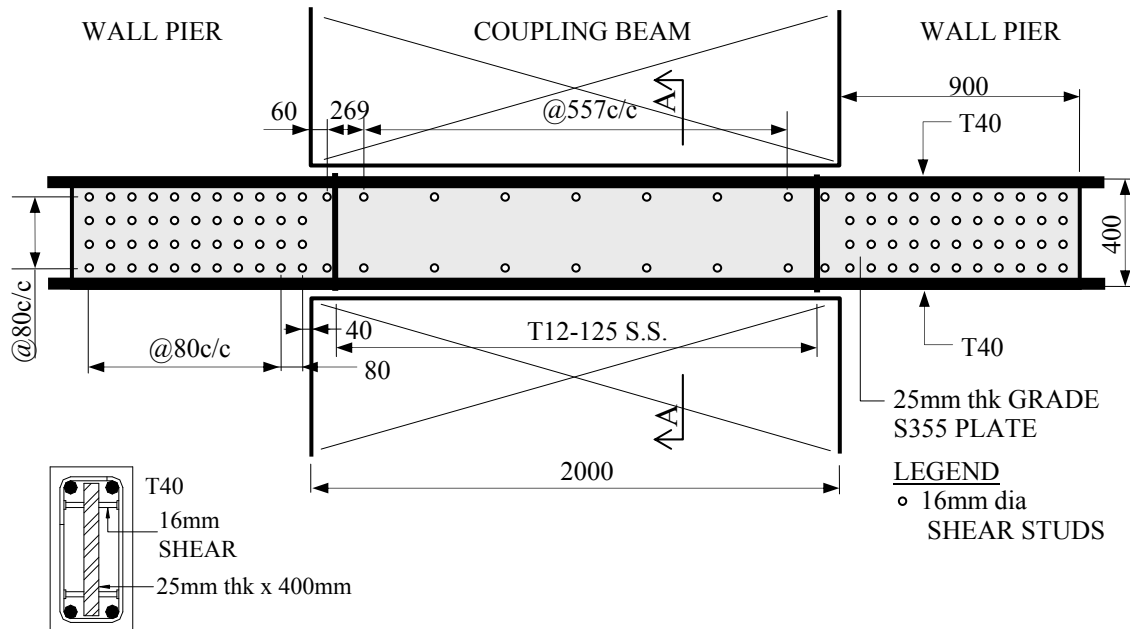


Figure 10. Steel Plate and Shear Stud Arrangements of the PRC Coupling Beam

Furthermore, use shear studs of 16mm shank diameter,  $Q_k = 66.5\text{kN}$ , minimum spacing = 80mm

$$w_x = \frac{6M_1 + 2F'_x h_p}{h_p^2} = \frac{6 \times 57 + 2 \times 960 \times 0.4}{0.4^2} = 6937\text{kN/m}$$

$$w_{y1} = \frac{6M_2 + L_a F'_y}{L_a^2} = \frac{6 \times 352 + 0.9 \times 203}{0.9^2} = 2833\text{kN/m}$$

$$w_{y2} = \frac{6M_2 - L_a F'_y}{L_a^2} = \frac{6 \times 352 - 0.9 \times 203}{0.9^2} = 2382\text{kN/m}$$

For Region I (width =  $L_I = 77\text{mm}$ ),

$$n_{req} = \frac{\sqrt{\left(\frac{0.077}{0.9}\right)^2 \left(\frac{6937 \times 0.4}{2}\right)^2 + (2833 \times 0.077)^2}}{0.6 \times 66.5} = 6.2$$

For Region II (width =  $L_{II} = 823\text{mm}$ ),

$$n_{req} = \frac{\sqrt{\left(\frac{0.823 \times 6937 \times 0.4}{0.9}\right)^2 + (2382 \times 0.823)^2}}{0.6 \times 66.5} = 80.3$$

Provide **4 shear studs in one column in Region I**, and **40 shear studs in 10 columns in Region II** on each side of plate.

Figure 10 shows the detailing of the PRC coupling beam in this example.

## 5. CONCLUSIONS

With the aim of providing the construction industry with a practical, effective and economical coupling beam to resist high shear force and large rotational demand from large wind or seismic loading, plate-reinforced composite (PRC) coupling beam was developed. The effectiveness and efficiency of this new form of beams were demonstrated by extensive experimental studies and numerical simulations. The practicality of PRC coupling beams in terms of integration of steel plate together with neighbouring reinforcement, easy of concreting and no special requirement for protecting steel plate against fire and lateral buckling was highlighted. Furthermore, original design guidelines compiled with local construction practice are proposed to ensure proper beam detailing for desirable performances of PRC coupling beams. The guidelines consist of four main parts, which are (1) estimation of ultimate shear capacity of beam, (2) design of RC component and steel plate, (3) shear stud arrangement in beam span, and (4) design of plate anchorage in wall piers. An example is given to illustrate the use of the guidelines for designing a PRC coupling beam.

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