- Technical Paper -

THE STRENGTH OF STEEL COUPLING BEAM EMBEDDED CONCRETE SHEAR WALL

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ABSTRACT

No specific guidelines are available for computing the bearing strength of connection between steel coupling beam and reinforced concrete shear wall in a hybrid wall system. There were carried out analytical and experimental studies on connection between steel coupling beam and concrete shear wall in a hybrid wall system. The test variables included the reinforcement details that confer a ductile behavior in connection between steel coupling beam and shear wall, *i.e.*, the auxiliary stud bolts attached to the steel beam flanges. The proposed equations in this study were in good agreement with both our test results and other test data from the literature.

Keywords: Steel coupling beams, Bearing strength, Stud bolt

1. INTRODUCTION

Structural steel coupling beams are a useful alternative to conventional reinforced beam designs, as dimension changes in these beams can be incorporated according to the desired proportions. The primary advantages gained from the use of embedded steel coupling beam sections arise from their compactness and the simplicity of detailing. The main design issues involving steel coupling beams are: (1) proportioning and detailing of the steel coupling beam, and (2) the steel coupling beam-wall connections. In particular, the transfer of forces between the steel coupling beams and the shear wall is not well understood. Several researchers [1-4] have investigated novel approaches to improve the ductility and energy absorption of reinforced concrete coupling beams. A number of recent studies have focused on examining the seismic response of concrete, steel, and composite coupling beams. However, since no specific equations are available for computing the bearing strength of steel coupling beam-wall connections, it is necessary to develop such strength equations. In this study, we set out to develop the strength equations of steel coupling beam-wall connections in a hybrid wall system, and analytical and experimental studies on steel coupling beam-concrete wall connections were carried out.

2. ANALYTICAL STUDY

No specific guidelines are available for computing the bearing strength of connection between steel coupling beam and reinforced concrete shear wall, but references to previous studies show the adequacy of four models proposed by the Prestressed Concrete Institute (PCI), Chicago, USA [5, 6], Kriz and Raths [7], Williams [8], and Mattock and Gaafar [9]. These four models were originally developed for the design of precast, bracket, corbel, and beam-column joint, respectively, and have been used to propose equations describing the strength of connection between steel coupling beam and reinforced concrete shear wall.

2.1 Bearing strength in the embedment

Figure 1 shows actual and assumed

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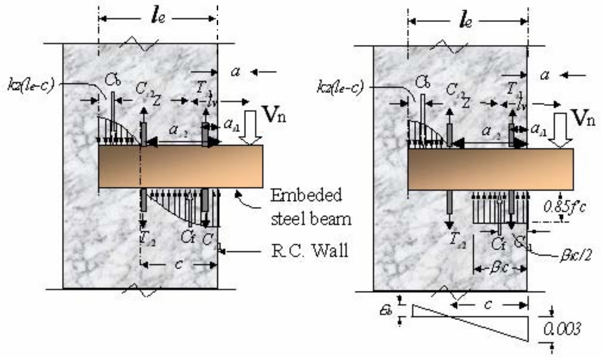
stresses and strains for connection between steel coupling beam and reinforced concrete shear wall. The compressive stresses in the concrete above and below the embedded steel section caused by the load, V_n, acting on the section at a given distance from the face of the concrete shear walls are shown in Fig. 1(a). The applied shear (V_n) is resisted by mobilizing an internal moment arm between the bearing forces, C_f and C_b. For calculation purposes, the stresses in the concrete at the ultimate stress are assumed to be as shown in Fig. 1(b). The parabolic compressive stress distribution below the embedded steel coupling beam section has been replaced by the equivalent rectangular stress distribution, equal to $0.85f_c'$, which is defined in Section 10.2.7 of the ACI 318-02 report [10]. The parabolic distribution of bearing stresses above the embedded steel coupling beam section is assumed to obey the following stress-strain relationship proposed by Kent and Park [11]

$$f_c = f_c'\left[\frac{2\varepsilon_c}{0.002} - \left(\frac{\varepsilon_c}{0.002}\right)^2\right] \text{ (MPa)} \qquad (1)$$

and is also assumed that there is a linear relationship between the compressive strains above and below the steel coupling beam section, as shown in Fig. 1(b). The assumed stress-strain relationship for concrete above the embedded steel coupling beam section corresponds to a parabola with a maximum stress of f_c' at a strain = 0.002. The factor, k_2, defining the location of the resultant compressive force, C_b, is given by

$$k_2 = \left[\frac{1 - 0.375\left(\dfrac{l_e - c}{c}\right)}{3 - 1.50\left(\dfrac{l_e - c}{c}\right)}\right] \qquad (2)$$



(a) Actual stresses (b) Assumed stresses

Fig. 1 Actual and assumed stresses and strains in the embedment

Therefore, V_n may be obtained by taking moments about the line of action at point C_b, as

$$V_n = 0.85 f_c' \beta_1 b l_e \left(\frac{c}{l_e}\right)\left[\frac{1 - k_2\left(1 - \dfrac{c}{l_e}\right) - \dfrac{\beta_1}{2}\left(\dfrac{c}{l_e}\right)}{1 - k_2\left(1 - \dfrac{c}{l_e}\right) + \dfrac{a}{l_e}}\right] \qquad (3)$$

The value of c/l_e was corresponded to the values of a/l_e = 0.5–2.7 for 20.7< f_c'/MPa< 55.2, i.e., for β_1 = 0.85–0.79. Figure 2 shows that the value of c/l_e has only a small variation from its average value. As shown in Fig. 2, the average value of c/l_e was 0.66, and the coefficient of variation was 3.5% for normal-strength concrete. Therefore, the value of c/l_e was assumed to be c/l_e = 0.66. It follows from Equation (2) that k_2 = 0.36. Then, $V_{n(theory)}$ is given by

$$V_{n(theory)} = 0.85 f_c' \beta_1 b l_e \left[\frac{0.85 - 0.22\beta_1}{0.88 + a/l_e}\right] \text{ (N)} \qquad (4)$$

Figure 3 shows comparisons between the predicted values from the theoretical

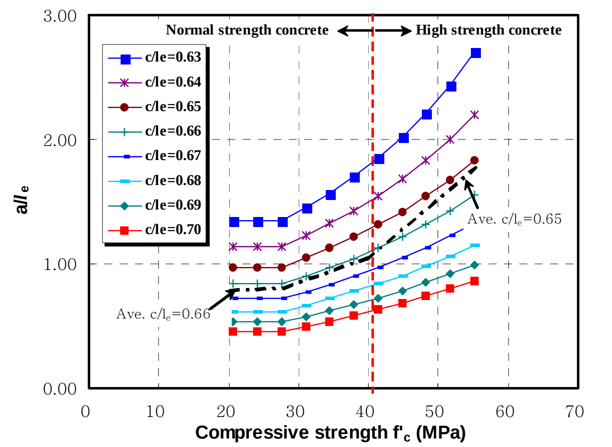


Fig. 2 c/l_e versus concrete compressive strength and a/l_e

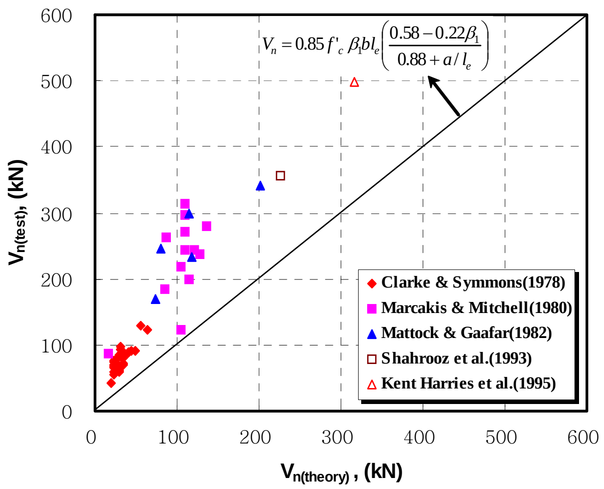


Fig. 3 Comparison of values predicted by theoretical Eq. and observed strength

equations and the observed strength. As shown in Fig. 3, the predicted values from the theoretical equations underestimate the observed strength.

2.2 Contribution of stud and horizontal ties

Based on the test results from a previous study [12], stud bolts on the top and bottom flange of an embedded steel coupling beam section, as shown in Fig. 1, were specified in an effort to improve the stiffness, and to improve the transfer of the flange bearing force to the surrounding concrete. By taking moments about the line of action, C_b, the additional strength due to the internal moment arm among the stud bolts can be computed using Equation (5)

$$V_s = \frac{2(0.88 - a/l_e)\sum_{i=1}^{n} A_{si} f_{si}}{0.88 + a/l_e} \quad \text{(N)} \qquad (5)$$

A_{si} = cross-sectional area of the auxiliary bar, *i*, inside the joint, and f_{si} = stud stresses in the auxiliary bar, *i*, inside the joint.

3. EXPERIMENTAL PROGRAM

Two test specimens were employed, included on wall pier with the other two being steel coupling beams. The test variables used are summarized in Table 1. The observed material properties are reported in Tables 2 and 3. A schematic diagram of the test apparatus is shown in Fig. 4. The test specimens were loaded using two hydraulic jacks: a pair of 2,000 kN hydraulic jack for the wall, and a 1,000 kN hydraulic jack for the steel coupling beams. The observed displacement history during the tests is shown in Fig. 5.

3.1 Experimental results

All the specimens experienced similar damage patterns, consisting of cracking and spalling between the top and bottom flanges, as shown in Fig. 6. For all the specimens, an initial cracking at the steel coupling beam flange-concrete interface was observed during Load stage 1, corresponding to a load of about ± $0.5\delta_y$. On completion of the tests, cracks with a width of up to 3 mm around the top and bottom flanges could be observed. These cracks were approximately 40 mm deep, as shown in Figs. 6(a)–10(c). Finally, spalling of the concrete below the embedded steel coupling beam section began at a load of about 92% of the ultimate load for all the specimens.

Table 1. Variables of test specimens

Item / Specimens	Stud bolts	Wall reinforcements: In wall	Wall reinforcements: In connections	Eccentricity e (mm)
SCB-ST	None	HD13 @230	HD13 @230	+150
SCB-SB	12-ϕ19			

Table 2. Material properties

Item / Specimens	Compressive strength (MPa)	Ultimate strain (μ)	Slump (mm)	Elastic modulus (GPa)	Poisson's ratio
SCB Series	34.0	2,340	145	26,200	0.11

∗ At the time of testing

Table 3. Properties of steel

Item		Yield strength f_y, (MPa)	Yield strain ε_y, ($\times 10^{-6}$)	Elastic modulus E_s, (GPa)	Ultimate strength f_{su}, (MPa)
Reinforcement	10mm diameter deformed bar	398	2,325	171.2	566
	13mm diameter deformed bar	400	2,533	157.9	555
Steel	beam web	339	1,682	201.2	461
	beam flange	352	1,827	192.7	489
Stud bolts	ϕ19	362	1,701	215.8	449

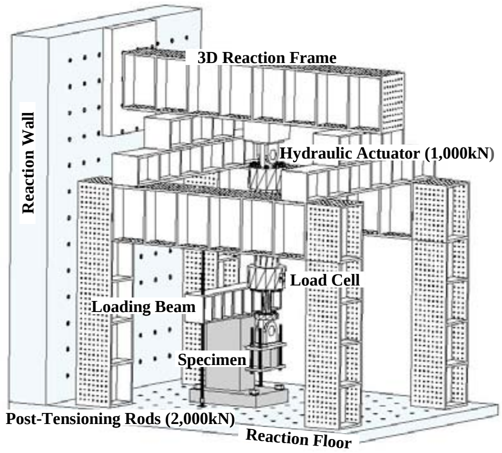


Fig. 4 Test setup

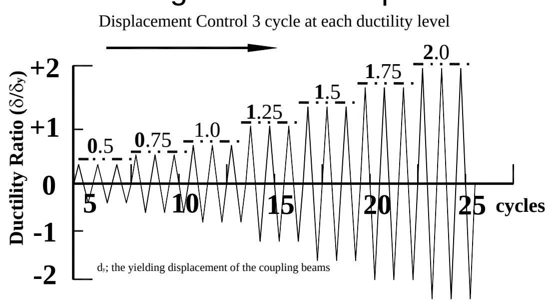


Fig. 5 Loading history

Figure 6 shows a plot of the applied load versus the steel coupling beam-rotation angle. The bearing strengths of Specimens SCB-ST and SCB-SB could develop a bearing force 313 and 428.3 kN, respectively, in the compression cycles (beam push down). In particular, in specimen SCB-ST, the steel coupling beam did not reach the plastic moment capacity, because of wall spalling and bearing failure. As shown in Fig. 7, in specimen SCB-SB, the average strain of the stud bolts on the top and bottom flanges at the ultimate load was equal to about 0.000366, 0.000496, and 0.000903 for the two specimens studied. Specimen SCB-SB was reinforced by stud bolts on the top and bottom flanges, and this increased the bearing strength compared with that of specimen SCB-ST by approximately 36.7%.

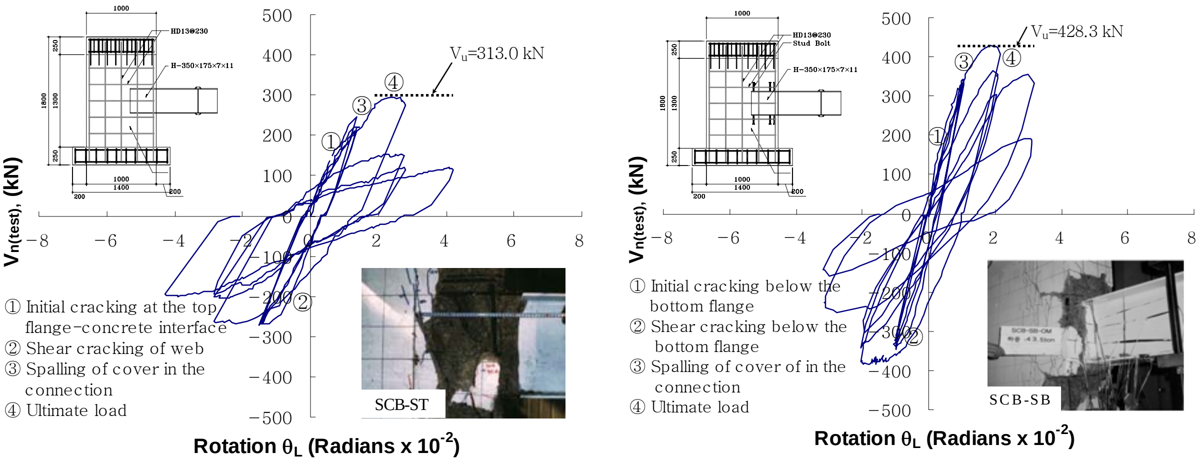
3.2 Revision of influential factors

(1) Bearing strength

The maximum loads carried by the specimens are listed as the values of $V_{n(test)}$ in Table 4. Also listed in this table are the calculated ultimate loads: $V_{n(PCI)}$, using the PCI equation, and $V_{n(theory)}$ using Equation (4) developed in this study. Both equations yield over-conservative estimates of the ultimate strengths of the specimens. The values from the PCI equation are about 40% more conservative than those determined using Equation (4). The degree of conservatism of Equation (4) increases as the width of the embedded steel coupling beam section decreases. This increase in conservatism must be due to an increase in the concrete bearing stress as the ratio of the width of the embedded steel coupling beam, b, to the thickness of the shear wall decreases. Similar behavior has been found in tests on column heads subjected to strip loading [4-6]. The ultimate strength is proportional to the bearing stress, f_b, that was assumed to be equal to $0.85 f_c'$ when calculating $V_{n(theory)}$. Therefore, we can write

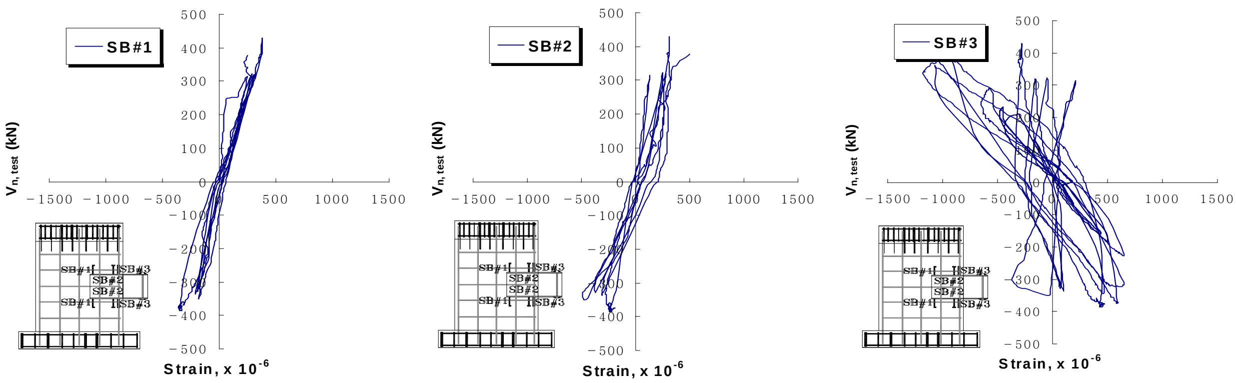
$$V_{n(test)} / f_b = V_{n(theory)} / 0.85 f_c' \qquad (6)$$

$$f_b / f_c' = 0.85 V_{n(test)} / V_{n(theory)} \qquad (7)$$



(a) SCB-ST (b) SCB-SB

Fig. 6 Load versus beam rotation angle hysteretic response



(a) US1 (b) US2 (c) US3

Fig. 7 Strain of stud bolts at ultimate load; Specimen SCB-SB

The values of f_b / f'_c calculated using Equation (7) are given in Table 4. The values of f_b / f'_c for specimens SCB-ST and SCB-SB and other test data are plotted against the ratio of b/t in Fig. 8, where t is the thickness of the shear walls (or width of column). A point corresponding to the case where f_b / f'_c is equal to 0.85 when b/t is unity is also plotted, *i.e.*, a bearing on the full thickness of the shear walls. For a member without any horizontal ties, it can be seen that the variation of f_b / f'_c with b/t can be represented closely by

$$f_b / f'_c = 0.85\left(\frac{b}{t}\right)^{-0.55} \quad (8)$$

Table 4. Test results

Specimens	$V_{n(test)}$ (kN)	$V_{n(PCI)}$ (kN)	$\frac{V_{n(test)}}{V_{n(PCI)}}$	$V_{n(theory)}$ (kN)	$\frac{V_{n(test)}}{V_{n(theory)}}$	$\frac{f_b}{f'_c}$	Failure mode
SCB-ST	313.0	190.9	1.64	258.9	1.21	1.03	BF*
SCB-SB	428.3	256.9	1.66	258.9	1.65	1.41	BF

* Bearing failure, 1mm=0.03937in., 1MPa=145.14psi., 1kN=0.2248kip.

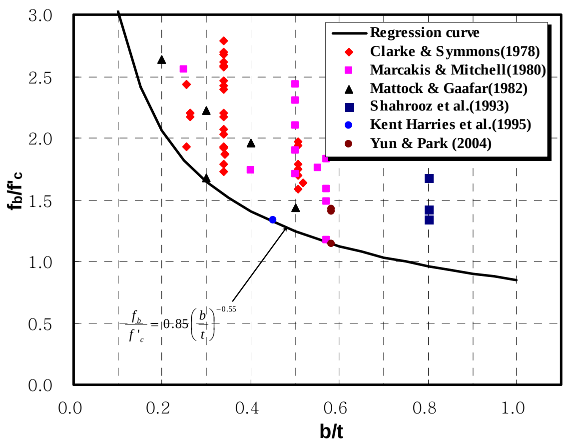


Fig. 8 Variation of bearing stress at ultimate load with ratio b/t

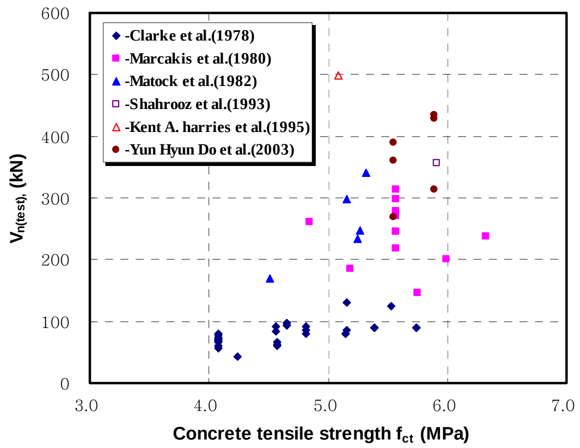


Fig. 9 Revision of influential factors

$$f_b = 28.9\left(\frac{b}{t}\right)^{-0.55} \text{ (MPa)} \quad (9)$$

for this group of specimens with an average value of $f'_c = 34.0$ MPa.

(2) Tensile strength

The studies in References [7] and [8] found that the concrete bearing strength under strip loading was proportional to the concrete tensile strength, f_{ct}, rather than to the compressive strength, f'_c. The authors of References 7 and 8 assumed that f_{ct} was proportional to $\sqrt{f'_c}$ and proposed equations of the following form, as shown in Fig. 9

$$f_b = A\sqrt{f_c'}\left(\frac{t/2}{b}\right)^n \text{ (MPa)} \quad (10)$$

$$f_b = K\sqrt{f_c'}\left(\frac{t}{b}\right)^n \text{ (MPa)} \quad (11)$$

where b is the width of the steel coupling beam. Kriz and Raths [7] proposed values of A = 5.7 and n = 0.33, *i.e.*, $K = A/2^n = 4.5$, and Hawkins [13] suggested that for design purposes, the values of A and n proposed by Kriz and Raths [7] should be used. Williams proposed a value of n = 0.47. In view of the findings shown in References [7] and [8], we proposed that the bearing stress below embedded sections at ultimate load be expressed in the same form as Equation (11). For member without horizontal ties, by comparing Equations (9) and (11), n = 0.55 and $K\sqrt{f_c'} = 28.9$ MPa when $f_c' = 34.0$ MPa. Hence, the value of K = 4.9, which is very close to the value of A determined by Kriz and Raths [7]. Substituting the value of K = 4.5 proposed by Kriz and Raths [7] into Equation (11), the bearing strength of concrete for an embedded steel coupling beam section without horizontal ties can be calculated using

$$V_{n(revised)} = f_b \beta_1 b l_e \left(\frac{0.58 - 0.22\beta_1}{0.88 + a/l_e}\right) \text{ (N)} \quad (12)$$

$$f_b = 4.5\sqrt{f_c'}\left(\frac{t}{b}\right)^{0.55} \text{ (MPa)} \quad (13)$$

Until further test data are available, it is proposed that value of the ratio of t/b not be t/b > 2.2 when using Equations (12).

As governed by the bearings on the concrete, we proposed that the bearing

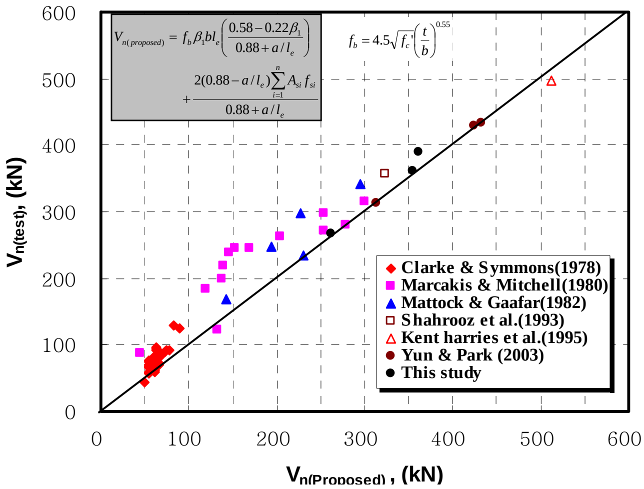


Fig. 10 Comparison of predicted and observed strength

strength of connection between steel coupling beam and reinforced concrete shear wall can be calculated using the following equation

$$V_{n(proposed)} = f_b \beta_1 b l_e \left(\frac{0.58 - 0.22\beta_1}{0.88 + a/l_e} \right) + \frac{2(0.88 - a/l_e)\sum_{i=1}^{n} A_{si} f_{si}}{0.88 + a/l_e} \quad \text{(N)} \qquad (14)$$

$$f_b = 4.5\sqrt{f_c'} \left(\frac{t}{b} \right)^{0.55} \quad \text{(MPa)} \qquad (15)$$

Figure 10 shows a comparison of the experimental and predicted data from the proposed equations for the connection between steel coupling beam and reinforced concrete shear wall. When Equation (14) was used to calculate the bearing strength of the specimens tested in this study, then the average values of the ratio of $V_{n(test)}/V_{n(proposed)}$ for specimens SCB-ST and SCB-SB of 1.00 and 1.02, respectively, were obtained, with standard deviations of 0.15 and 0.12, respectively. As shown in Fig. 10, the predicted values from the proposed equations are in good agreement with the measure strengths.

4. CONCLUSIONS

(1) In extracting the theoretical Equation (3) for the bearing strength of connection between steel coupling beam and reinforced concrete shear wall, the assumption of a constant value of c/l_e = 0.66 is reasonable.

(2) When calculating the bearing strength of a steel coupling beam section embedded in a shear wall, the PCI Code and other proposed models yield very conservative results. Therefore, from this study, the following equations are proposed to calculate the bearing strength of the connection between steel coupling beam and reinforced concrete shear wall

$$V_{n(proposed)} = f_b \beta_1 b l_e \left(\frac{0.58 - 0.22\beta_1}{0.88 + a/l_e} \right) + \frac{2(0.88 - a/l_e)\sum_{i=1}^{n} A_{si} f_{si}}{0.88 + a/l_e}$$

$$f_b = 4.5\sqrt{f_c'} \left(\frac{t}{b} \right)^{0.55} \quad \text{(MPa)}$$

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