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Modeling Test of Shear Connectors between Precast Concrete Members to Infill the Gap Interface

*Sung-Yong Yu[1], Geun-Taeck Song[2], Soo-Kyoung Ha[3], Hyun-Seok Song[4] and Guk-Won Son[3]

[1], [3] *Department of Architectural Engineering, Dongguk University, 30, Pildong-ro 1-gil, Jung-gu, Seoul, Korea*
[2], [4] *Department of Technical Engineering, Hilti (Korea) Ltd., SEI Tower, 39, Eonju-ro 30-gil, Gangnam-gu, Seoul, Korea*
[1] *ysy@dongguk.edu*
[2] *geuntaeck.song@hilti.com*

ABSTRACT

High strength concrete shear connectors of adjustable widths between precast concrete members were investigated to infill the gap interface, which can tolerate unpredictable construction errors. The shear connection of this study is composed of three layered concrete to adjust the width against construction errors. Experiments on studded and welded steel panel connection were performed to determine the shear strength of these connections.

Accordingly, this study aimed at predicting the shear strength of narrow connections with two stud groups. Each stud group was cast in a straight line with no space in between them. All studs had one end fastened to precast concrete and the other end fastened to the concrete connection to be subjected to external force due to shear deformation. In other words, one stud group is subjected to the same deformation and movement because its one side is fastened in the same manner to withstand the load altogether. According to ACI 318M-11 Appendix D, when an stud group is cast in a straight line, it results in the same side break-out strength regardless of the number of studs in the group. However, when side break-out failure occurs simultaneously in this stud group, the connection model of this study will manifest higher break-out strength than the break-out strength in the air predicted by the ACI equation (cf. Fig. 5 and Fig. 6). The connection of this study is different from the connection suggested by ACI. Shear modeling test on this unique connection was carried out in this study, and the result was compared with the prediction equation of ACI analysis result. The final result was close to the experimental values when the shear resistance force of each stud end was computed separately and then summed up.

[1] Professor
[2] Engineering Director
[3] Student
[4] Technical Engineer

1. INTRODUCTION

A great deal of researches have been carried on masonry-infilled and cast-in wall infilled reinforced concrete frame in Korea and overseas since early 1960s for existing reinforced concrete beam-column structure. Nonetheless, researches on seismic test of the existing reinforced concrete frame with Precast Concrete Wall Panel (PC Wall Panel) and the connections of it are very scarce. When PC wall panel is inserted to the existing RC beam-column structure to increase its seismic strength, a connection to deal with the deformation of existing aged RC structure due to construction error and to allow for the deformation after the construction is required. Thus, this study focused on the high strength concrete shear connection, of which the gap interface may be infilled by adjusting the size of the connection between the PC members. The stud connection and welded steel panel connection model of this study were investigated to examine the shear performance of the connection between PC members. The stud connection and welded steel panel connection in this study are flexible in the size of width and can also withstand a large amount of shear load.

2. MATERIAL

The specimen of this study is composed of two types of concretes. As shown in Table 1, the central and external PC concretes have target strength of 35 MPa after 28 days, and the connection part of the concrete (hereafter, connection concrete) was placed with target strength of 50 MPa. The welded studs of a test specimen (cf. Table 2 (1)) were fastened to PC layer and were designed to withstand shear force from the connection concrete layer. The properties of the steel box material for specimen P1 and the steel plate material for specimen P2 are listed in Table 2 (2).

Table 1 Cylinder strength of concrete

Specimens	PC[1)] f_{ck}[MPa]	Connection[2)] f_{ck}[MPa]
P1	33.0	55.3
P2	33.0	44.8

1) strength of precast concrete member
2) strength of concrete at connection

Table 2 Strength of steel

Steel	Yield strength [MPa]	Tensile strength [MPa]	Effective section area [mm^2]
1) Cast-in M24	350	450	353
2) SM400*	235	400	-

* Material of welded steel plate

3. DESIGN OF THE SHEAR STUD CONNECTION MODEL

The center part, external part, and the connection part of the shear stud connection specimen-P1 in Fig. 1 are composed of concrete panels of 250mm thickness. The studs used in this study can be grouped into two groups of one stud group fastened to central PC panel and the other stud group fastened to external PC panel. Each stud group was cast in a straight line. All studs had one end fastened to PC and the other end fastened to the connection concrete.

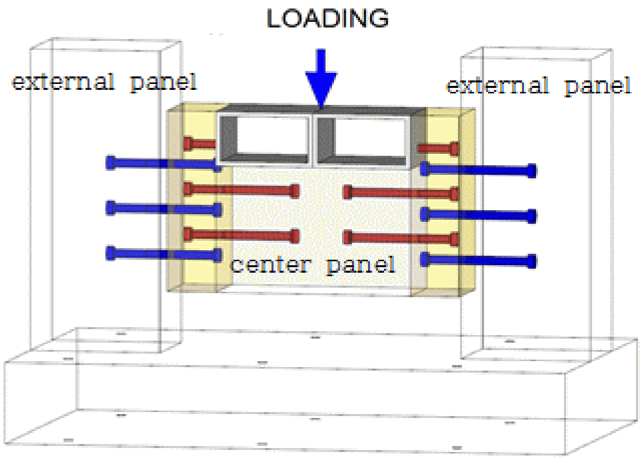

Fig. 1 Shear test of studded connection

This test computed the shear capacity of both sides of the connection specimen as depicted in Fig. 2 by loading from top to bottom of the steel box. If the test modeling of this study is followed, the shear failure in lower part concrete will not be observed due to the compression force coming down from the steel box, but shear failure will be exhibited only in connection part and external part. Thus, for these reasons, shear test is carried out with same variables (concrete strength of central and external structures, depth of installation, type of stud, and shear reinforcement spacing). Then, it is expected that the shear capacity of the cast-in studs in the connection part and the external part can be obtained.

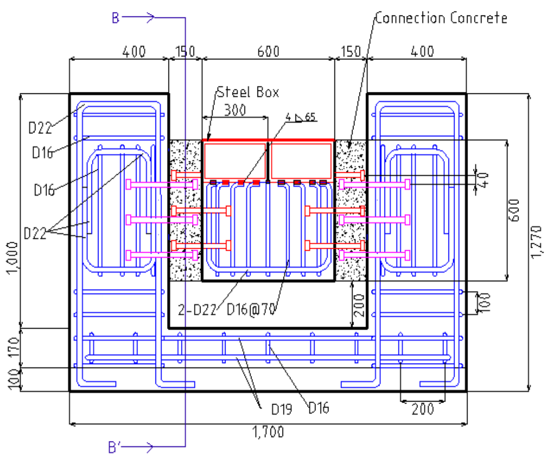

Fig. 2 Section of specimen P0

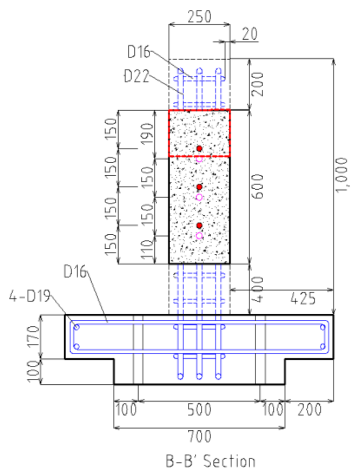

Fig. 3 Section B-B

Three studs were installed in the central panel as shown in Fig. 3. Two studs were fastened to PC, and the other one was welded on top of the steel box. The shear bearing capacity of the two studs cast in PC can be computed by an ACI Appendix D equation. The shear strength, T_{box}, of the vertical stud on top of the steel box can be computed by taking the smaller value of horizontal bearing capacity of the upper single stud or shear strength of the steel box.

3.1 Shear strength of stud (ACI Eq. (D-29))

1) Shear strength of stud in central PC panel

$$\begin{aligned} T_{box} &= \phi(0.6A_{pl}f_u) \\ &= 0.75(0.6 \times (180 \times 250 - 165 \times 235) \times 235 \times 10^{-3} = 658.3 \text{ kN} \\ &> 0.75(0.6 \times 1 \times 353 \times 500) \times 10^{-3} = 79.4 \text{ kN} \end{aligned} \tag{1}$$

$$\begin{aligned} \phi V_{sa} &= \phi\left(0.6n_1A_{se,V}f_{uta}\right) + T_{box} \\ &= 0.75(0.6 \times 2 \times 353 \times 450) \times 10^{-3} + T_{box} \\ &= 143.0 + 79.4 = 222.4 \text{ kN} \end{aligned} \tag{2}$$

Where, A_{pl} represents section area of steel box (mm^2), f_y is specified yield strength of reinforcement (MPa), n_1 denotes number of a group of studs , $A_{se,V}$ means effective cross-sectional area of stud in shear(mm^2), and f_{uta} means specified tensile strength of stud steel (MPa).

2) Shear strength of stud in external PC panel

$$\begin{aligned} \phi V_{sa} &= \phi\left(0.6n_1A_{se,V}f_{uta}\right) \\ &= 0.75(0.6 \times 3 \times 353 \times 500) \times 10^{-3} = 238.3 \text{ kN} \end{aligned} \tag{3}$$

3.2 Pry-out strength

1) Pry-out strength of central stud (ACI Eq. (D-41))

According to the Eq. (D-41) of ACI Appendix D found in ACI Committee 318 (2011), pry-out strength is obtained by embedding one end of the stud in the concrete and then exposing the other end to the air to be subjected to loading in the air to compute the resistance found in ACI Committee 355 (2011) and Eligehausen (2006) of the concrete against the rotation force of the stud as shown in Fig. 4. The connection part of the specimen of this study is composed of three layers of concrete. The pry-out strength of the stud in PC concrete and connection concrete layer of this study is obtained as illustrated in Fig. 5. Since each stud has both ends embedded in different concretes as shown in Fig. 6, the effects are added and listed in Table 3 of pry-out strength.

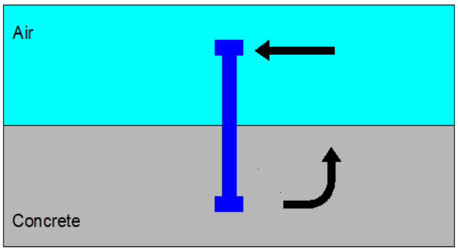

Fig. 4 Pry-out strength of ACI

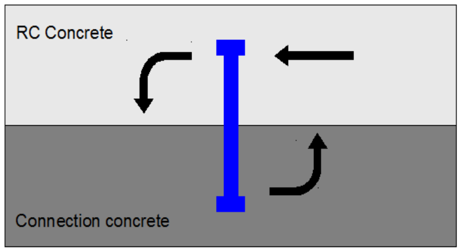

Fig. 5 Pry-out strength of specimens

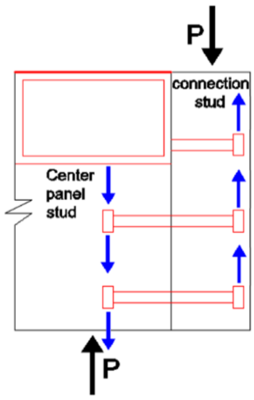

Fig. 6 Multi-studs

a) Pry-out strength of the central panel

Since eight D16 rebars are subjected to shear force upon pry-out loading on the steel box, the minimum pry-out strength of steel box, V_{box} (kN), was calculated by using the shear force of the rebars.

$$V_{box} = 0.6 n_2 A_{se} f_y = 0.6 \times 8 \times 198.6 \times 235 \times 10^{-3} = 224.0 \text{ kN} \tag{4}$$

$$\phi V_{cpg} = \phi[k_{cp}\left(\frac{A_{Nc}}{A_{Nco}}\right)\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b + V_{box}]$$
$$= 0.75[2\left(\frac{105{,}000}{396{,}900} \times 1.0 \times 0.82 \times 1.25 \times 174.8\right) + 224.0] = 239.1 \text{ kN} \tag{5}$$

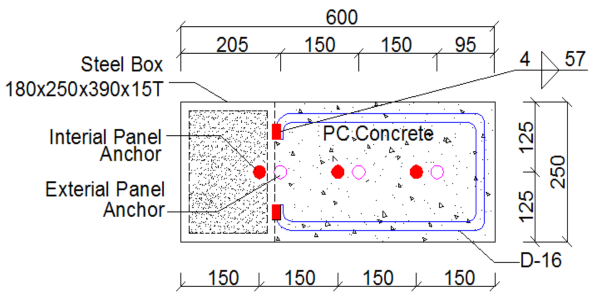

Fig. 7 Horizontal section of center panel of specimen P1

The bearing capacities of the two studs fastened to PC are computed after computing the values of A_{Nc}, A_{Nco}, $\psi_{ec,N}$, $\psi_{ed,N}$, N_b, V_{box} by the following equations. A_{Nc} is computed according to ACI Fig. RD.5.2.1 as shown in Fig. 6.

$$A_{Nc} = (125 \times 2)(150 \times 2 + 300 - 180) = 105{,}000 \ \text{mm}^2 \tag{6}$$

$$A_{Nco} = 9{h_{ef}}^2 = 9 \times 210^2 = 396{,}900 \ \text{mm}^2 \tag{7}$$

$$\psi_{ec,N} = \left(1/(1 + 2 \times \frac{e'_N}{3h_{ef}})\right) = \left(1/(1 + 2 \times \frac{0}{3 \times 210})\right) = 1.0 \tag{8}$$

$$\psi_{ed,N} = 0.7 + 0.3 \times \frac{C_{a,min}}{1.5h_{ef}} = 0.7 + 0.3 \times 125/(210 \times 1.5) = 0.82 \tag{9}$$

$$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} = 10 \times 1 \times \sqrt{33} \times 210^{1.5} \times 10^{-3} = 174.8 \ \text{kN} \tag{10}$$

Where, ϕ is the strength reduction factor(0.75). V_{cpg} represents nominal concrete pry-out strength of a group of studs (kN). k_{cp} denotes a coefficient for pry-out strength, having the value of 2 when h_{ef} is 65mm or greater. A_{Nc} indicates a projected concrete failure area of a single stud or group of studs for calculation of tensile strength (mm^2), A_{Nco} refers to projected concrete failure area of a single stud for calculation of tensile strength if not limited by edge distance or spacing (mm^2). $\psi_{ec,N}$ is a factor used to modify tensile strength of studs based on eccentricity of applied loads, and $\psi_{c,N}$ is a factor used to modify tensile strength of studs based on presence or absence of cracks in concrete. N_b is basic concrete breakout strength in tension of a single stud in cracked concrete (kN), e'_N is the distance between resultant tension load on a group of studs loaded in tension and the centroid of the group of studs loaded with tension, $C_{a,min}$ represents minimum distance from center of an stud shaft to the edge of concrete (mm), and k_c refers to coefficient for basic concrete breakout strength in tension. λ_a denotes modification factor reflecting the reduced mechanical properties of lightweight concrete in certain concrete studage applications. f'_c represents specified compressive strength of concrete (MPa).

b) Pry-out strength of the connection

The vertical spacing of stud connection stretches 150 mm as shown in Fig. 3. The stud protruding from PC is installed in a straight line at the center section. Additionally, the embedded depth, h_{ef}, is computed to be 85 mm by subtracting head height of 10 mm and gap depth of 55mm from the connection depth, 150 mm. The pry-out strength was calculated in accordance with the equation proposed by ACI 7.1.3, and the result is listed in Table 5 (3).

A_{Nc}, A_{Nco}, $\psi_{ec,N}$, $\psi_{ed,N}$, N_b are computed according to the following Eq. (12)~(16) and are substituted to the following Eq. (11).

$$\phi V_{cpg} = \phi[k_{cp}\left(\frac{A_{Nc}}{A_{Nco}}\right)\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b]$$
$$= 0.75[2\left(\frac{138{,}750}{65{,}025} \times 0.79 \times 0.99 \times 1.25 \times 58.3\right)] = 182.4 \text{ kN} \tag{11}$$

$$A_{Nc} = (125 \times 2)(150 \times 2 + 2 \times 85 \times 1.5) = 138{,}750 \text{ mm}^2 \tag{12}$$

$$A_{Nco} = 9{h_{ef}}^2 = 9 \times 85^2 = 65{,}025 \text{ mm}^2 \tag{13}$$

$$\psi_{ec,N} = \left(1/(1 + 2 \times \frac{e_N}{3h_{ef}})\right) = \left(1/(1 + 2 \times \frac{0}{3 \times 85})\right) = 1.0 \tag{14}$$

$$\psi_{ed,N} = 0.7 + 0.3 \times \frac{C_{a,min}}{1.5h_{ef}} = 0.7 + 0.3 \times 125/(85 \times 1.5) = 0.99 \tag{15}$$

$$N_b = k_c\lambda_a\sqrt{f'_c}h_{ef}^{1.5} = 10 \times 1 \times \sqrt{55.3} \times 85^{1.5} \times 10^{-3} = 58.3 \text{ kN} \tag{16}$$

2) External stud pry-out strength

a) Pry-out strength in the external panel

Likewise, A_{Nc}, A_{Nco}, $\psi_{ec,N}$, $\psi_{ed,N}$, N_b for computation of pry-out strength of stud in external panel are obtained by the following Eq. (18)~(22) to be substituted to the following Eq. (17).

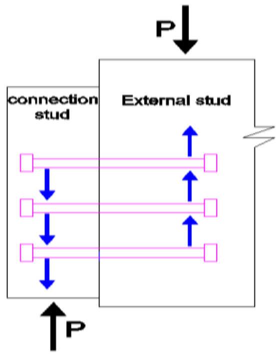

Fig. 8 Multi-studs

$$\phi V_{cpg} = \phi[k_{cp}\left(\frac{A_{Nc}}{A_{Nco}}\right)\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b]$$
$$= 0.75[2\left(\tfrac{150,000}{396,900} \times 1.0 \times 0.82 \times 1.25 \times 174.8\right)] = 101.6 \text{ kN} \tag{17}$$

$$A_{Nc} = (125 \times 2)(150 \times 2 + 95 + 205) = 150,000 \text{ mm}^2 \tag{18}$$

$$A_{Nco} = 9{h_{ef}}^2 = 9 \times 210^2 = 396,900 \text{ mm}^2 \tag{19}$$

$$\psi_{ec,N} = \left(1/(1 + 2 \times \frac{e_N}{3h_{ef}})\right) = \left(1/(1 + 2 \times \frac{0}{3 \times 210})\right) = 1.0 \tag{20}$$

$$\psi_{ed,N} = 0.7 + 0.3 \times \frac{C_{a,min}}{1.5h_{ef}} = 0.7 + 0.3 \times 125/(210 \times 1.5) = 0.82 \tag{21}$$

$$N_b = k_c\lambda_a\sqrt{f'_c}h_{ef}^{1.5} = 10 \times 1 \times \sqrt{33} \times 210^{1.5} \times 10^{-3} = 174.8 \text{ kN} \tag{22}$$

b) Pry-out strength of the connection

A_{Nc}, A_{Nco}, $\psi_{ec,N}$, $\psi_{ed,N}$, N_b are computed by the following Eq. (24)~(28) to be substituted to the following Eq. (23).

$$\phi V_{cpg} = \phi[k_{cp}\left(\frac{A_{Nc}}{A_{Nco}}\right)\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b]$$
$$= 0.75[2\left(\tfrac{130,625}{65,025} \times 1.0 \times 0.92 \times 1.25 \times 45.0\right)] = 155.9 \text{ kN} \tag{23}$$

$$A_{Nc} = (125 \times 2)(150 \times 2 + 95 + 1.5 \times 85) = 130,625 \text{ mm}^2 \tag{24}$$

$$A_{Nco} = 9{h_{ef}}^2 = 9 \times 85^2 = 65,025 \text{ mm}^2 \tag{25}$$

$$\psi_{ec,N} = \left(1/(1 + 2 \times \frac{e'_N}{3h_{ef}})\right) = \left(1/(1 + 2 \times \frac{0}{3 \times 85})\right) = 1.0 \tag{26}$$

$$\psi_{ed,N} = 0.7 + 0.3 \times \frac{C_{a,min}}{1.5h_{ef}} = 0.7 + 0.3 \times 95/(85 \times 1.5) = 0.92 \tag{27}$$

$$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} = 10 \times 1 \times \sqrt{33} \times 85^{1.5} \times 10^{-3} = 45.0 \text{ kN} \quad (28)$$

Since all studs are connected to PC panel and connection part, pry-out strength of the stud was calculated by adding the shear strengths of PC panel and connection.

Table 3 Pry-out strength of specimen P1

Item		1) Shear strength of panel part	2) Shear strength of connection part	1)+2) sum
P1	center panel stud	236.6	182.4	419.0
	exterior panel stud	101.6	155.9	257.5

3.3 Break-out strength of concrete(ACI Eq. (D-31))

1) Break-out strength of central stud

a) Break-out strength of the central panel

Breakout strength of steel box in central PC panel was computed by using the shear capacity of the 8-D16 rebars welded to the steel box in Fig. 7.

$$V_{box} = 0.6 n_2 A_{se} f_u = 0.6 \times 8 \times 198.6 \times 235 \times 10^{-3} = 224.0 \text{ kN} \quad (29)$$

$$\phi V_{cbg,pc1} = \phi[n_1 \left(\frac{A_{Vc}}{A_{Vco}}\right) \psi_{ec,V} \psi_{ed,\mathrm{V}} \psi_{c,\mathrm{V}} \psi_{parallel,\mathrm{V}} V_b + V_{box}]$$
$$= 0.75[2 \times (\tfrac{100{,}125}{320{,}800}) \times 1.0 \times 0.794 \times 1.4 \times 1.0 \times 1.0 \times 95.5 + 224.0] = 217.7 \text{ Kn} \quad (30)$$

$$A_{vc} = (1.5 \times C_{a1}) \times 250 = (1.5 \times 267) \times 250 = 100{,}125 \text{ mm}^2 \quad (31)$$

$$A_{vco} = 4.5 {C_{a1}}^2 = 4.5 \times 267^2 = 320{,}800 \text{ mm}^2 \quad (32)$$

$$\psi_{ec,\mathrm{V}} = \left(1/(1 + 2 \times \frac{e'_V}{3C_{a1}})\right) = \left(\frac{1}{1 + 2 \times \frac{0}{3 \times 267}}\right) = 1.0 \leq 1.0 \quad (33)$$

$$\psi_{ed,\mathrm{V}} = 0.7 + 0.3 \times \frac{C_{a2}}{1.5 C_{a1}} 0.7 + 0.3 \times 125/(267 \times 1.5) = 0.794 \leq 1.0 \quad (34)$$

$$\psi_{c,\mathrm{V}} = 1.4 \qquad \rightarrow \mathrm{ACI318-11D6.2.7} \tag{35}$$

$$\psi_{h,\mathrm{V}} = \sqrt{\left(\frac{1.5C_{a1}}{h_a}\right)} = \sqrt{(1.5 \times \frac{267}{400})} = 1.0 \geq 1.0 \tag{36}$$

$$\psi_{parallel,V} = 1.0 \tag{37}$$

$$V_b = 3.7\lambda_a\sqrt{f'_c}C_{a1}^{1.5} = 3.7 \times 1.0\sqrt{35} \times 267^{1.5} \times 10^{-3} = 95.5\ \text{kN} \tag{38}$$

b) Break-out strength of the connection

Since there are three studs in the connection part, break-out strength of the connection found in ACI Committee 318 (2011) was computed by the following Eq. (39).

$$\phi V_{cbg,con1} = \phi[n_1\left(\frac{A_{Vc}}{A_{Vco}}\right)\psi_{ec,V}\psi_{ed,\mathrm{V}}\psi_{c,\mathrm{V}}\psi_{parallel,\mathrm{V}}V_b]$$
$$= 0.75[2 \times 37{,}500/45{,}000 \times 1.0 \times 0.95 \times 1.4 \times 1.0 \times 1.0 \times 26.2] = 65.3\ \text{kN} \tag{39}$$

$$A_{vc} = (150 \times C_{a1}) \times 250 = (1.5 \times 100) \times 250 = 37{,}500\ \text{mm}^2 \tag{40}$$

$$A_{vco} = 4.5{C_{a1}}^2 = 4.5 \times 100^2 = 45{,}000\ \text{mm}^2 \tag{41}$$

$$\psi_{ec,\mathrm{V}} = \left(1/(1 + 2 \times \frac{e'_V}{3C_{a1}})\right) = \left(1/(1 + 2 \times \frac{0}{3 \times 100})\right) = 1.0 \leq 1.0 \tag{42}$$

$$\psi_{ed,\mathrm{V}} = 0.7 + 0.3 \times \frac{C_{a2}}{1.5C_{a1}} 0.7 + 0.3 \times 125/(100 \times 1.5) = 0.95 \leq 1.0 \tag{43}$$

$$\psi_{c,\mathrm{V}} = 1.4 \qquad \rightarrow \mathrm{ACI318-11D6.2.7} \tag{44}$$

$$\psi_{h,\mathrm{V}} = \sqrt{\left(\frac{1.5C_{a1}}{h_a}\right)} = \sqrt{(1.5 \times \frac{100}{150})} = 1.0 \geq 1.0 \tag{45}$$

$$\psi_{parallel,V} = 1.0 \tag{46}$$

$$V_b = 3.7\lambda_a\sqrt{f'_c}C_{a1}^{1.5} = 3.7 \times 1.0\sqrt{50} \times 100^{1.5} \times 10^{-3} = 26.2 \text{ kN} \tag{47}$$

c) Sum of break-out strength

All studs fastened to central PC and were also anchored to connection part. In central PC, two studs were fastened to the PC panel concrete, and one stud was fastened to the steel box as shown in Fig. 9. Three studs were fastened in the connection part. Thus, the breakout strengths of central PC part and connection part can be summed and obtained as it follows in Eligehausen (2006).

$$\phi V_{cbg} = \phi V_{cbg,pc1} + \phi V_{cbg,con1} = 217.7 + 65.3 = 283.0 \text{ kN} \tag{48}$$

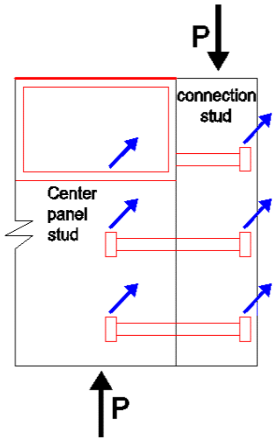

Fig. 9 Multi-studs

2) External stud break-out strength

a) Break-out strength of the external panel

Since there were three cast-in studs in the external panel, break-out strength of the external panel is computed as the following.

$$\phi V_{cbg,pc2} = \phi[n_1\left(\frac{A_{Vc}}{A_{Vco}}\right)\psi_{ec,V}\psi_{ed,V}\psi_{c,V}\psi_{parallel,V}V_b]$$
$$= 0.75[3 \times 122{,}513/122{,}513 \times 1.0 \times 1.0 \times 1.4 \times 1.0 \times 2.0 \times 43.6] = 274.7 \text{ kN} \tag{49}$$

$$A_{vc} = 4.5{C_{a1}}^2 = 4.5 \times 165^2 = 122{,}513 \text{ mm}^2 \tag{50}$$

$$A_{vco} = 4.5{C_{a1}}^2 = 4.5 \times 165^2 = 122{,}513 \text{ mm}^2 \tag{51}$$

$$\psi_{ec,\mathrm{V}} = \left(1/(1 + 2 \times \frac{e'_V}{3C_{a1}})\right) = \left(1/(1 + 2 \times \frac{0}{3 \times 165})\right) = 1.0 \leq 1.0 \tag{52}$$

$$\psi_{ed,\mathrm{V}} = 0.7 + 0.3 \times \frac{C_{a2}}{1.5C_{a1}} \leq 1.0$$
$$= 0.7 + 0.3 \times 390/(165 \times 1.5) = 1.17 \rightarrow 1.0 \tag{53}$$

$$\psi_{c,\mathrm{V}} = 1.4 \qquad \rightarrow \mathrm{ACI318-11D6.2.7} \tag{54}$$

$$\psi_{h,\mathrm{V}} = \sqrt{\left(\frac{1.5C_{a1}}{h_a}\right)} \geq 1.0$$
$$= \sqrt{(1.5 \times \frac{165}{400})} = 0.79 \rightarrow 1.0 \tag{55}$$

$$\psi_{parallel,V} = 2.0 \tag{56}$$

$$V_b = (0.6(l_e/d_a)^{0.2}\sqrt{d_a})\lambda_a\sqrt{f_{ck}}C_{a1}^{1.5}$$
$$= (0.6 \times \left(\frac{200}{24}\right)^{0.2}\sqrt{24}) \times 1.0\sqrt{21} \times 165^{1.5} \times 10^{-3} = 43.6 \text{ kN} \tag{57}$$

b) Break-out strength in the connection
There were also three studs in the connection part.

$$\phi V_{cbg,con2} = \phi[n_1\left(\frac{A_{Vc}}{A_{Vco}}\right)\psi_{ec,V}\psi_{ed,\mathrm{V}}\psi_{c,\mathrm{V}}\psi_{h,V}\psi_{parallel,\mathrm{V}}V_b]$$
$$= 0.75[3 \times 49{,}500/54{,}450 \times 1.0 \times 1.0 \times 1.4 \times 1.048 \times 1.0 \times 31.9] = 95.8 \text{ kN} \tag{58}$$

$$A_{vc} = (1.5 \times 110 \times 2) \times 150 = (330) \times 150 = 49{,}500 \text{ mm}^2 \tag{59}$$

$$A_{vco} = 4.5{C_{a1}}^2 = 4.5 \times 110^2 = 54{,}450 \text{ mm}^2 \tag{60}$$

$$\psi_{ec,\mathrm{V}} = \left(1/(1 + 2 \times \frac{e'_V}{3C_{a1}})\right) = \left(1/(1 + 2 \times \frac{0}{3 \times 100})\right) = 1.0 \leq 1.0 \tag{61}$$

$$\psi_{ed,\mathrm{V}} = 0.7 + 0.3 \times \frac{C_{a2}}{1.5C_{a1}} = 0.7 + 0.3 \times 165/(110 \times 1.5) = 1.0 \leq 1.0 \tag{62}$$

$$\psi_{c,\mathrm{V}} = 1.4 \qquad \rightarrow \mathrm{ACI318-11D6.2.7} \tag{63}$$

$$\psi_{h,\mathrm{V}} = \sqrt{\left(\frac{1.5C_{a1}}{h_a}\right)} = \sqrt{(1.5 \times \frac{110}{150})} = 1.048 \geq 1.0 \tag{64}$$

$$\psi_{parallel,V} = 1.0 \tag{65}$$

$$\begin{aligned} V_b &= (0.6(l_e/d_a)^{0.2}\sqrt{d_a})\lambda_a\sqrt{f_{ck}}C_{a1}^{1.5} \\ &= (0.6 \times \left(\frac{100}{24}\right)^{0.2}\sqrt{24}) \times 1.0\sqrt{50} \times 110^{1.5} \times 10^{-3} = 31.9 \ \text{kN} \end{aligned} \tag{66}$$

c) Sum of break-out strength

The shear strength of studs embedded in the central PC was calculated by assuming that they share the same strength and condition. Thus, the shear strength of the multiple studs was calculated by multiplying the number of the studs n_1, to the strength of single stud. Since the anchors were also fastened to external PC and connection part, the breakout strengths of the external PC and connection part can be summed and obtained as it follows ACI Committee 355 (2011) and the result is inserted in Table 4.

$$\phi V_{cbg} = \phi V_{cbg,pc2} + \phi V_{cbg,con2} = 274.7 + 95.8 = 370.5 \ \text{kN} \tag{67}$$

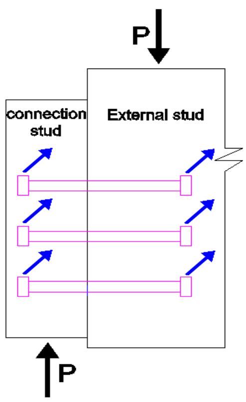

Fig. 10 Multi-studs

Table 4 Shear Strength of Top Connections for Specimen PR1-UA

Speci-mens	1) Interior panel stud strength [kN]			2) Exterior panel stud strength [kN]			3) Minimum chosen[kN]
	Stud shear strength	Concrete pry-out strength	Concrete breakout strength	Stud shear strength	Concrete pry-out strength	Concrete breakout strength	
P1	222.4	419.0	283.0	238.3	257.5	370.5	222.4

4. DESIGN OF THE WELDED STEEL PLATE CONNECTION MODEL

A specimen of welded steel plate connection was made of $\mathrm{H}\,400 \times 120 \times 15 \times 15$. The two steel plates of the connection part were welded in the site. The connection part was finished with high strength concrete (design strength of 50MPa).

4.1 Shear strength of embedded H-beam

An experiment to find out the shear resistance capacity of the embedded H-beam was carried out in order to obtain the shear strength of welded steel plate connection. The length of the welding was maximized to prevent the failure of the welding part. The shear reinforcement of central PC panel and external PC panel were similarly installed. When loading is applied from the center top of the specimen of Fig. 11 to bottom, shear failure can occur in the connection part or external panel, in which H-beam is embedded, to obtain the shear strength in these parts.

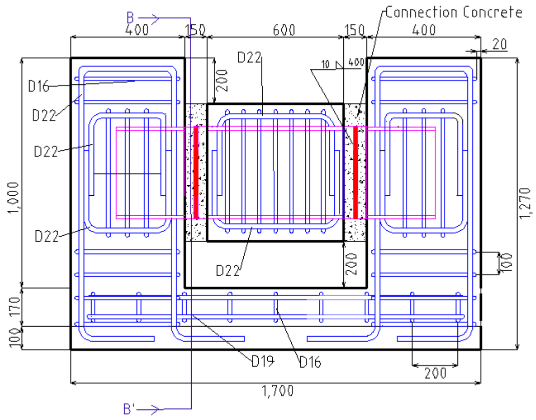

Fig. 11 Section of studded connection

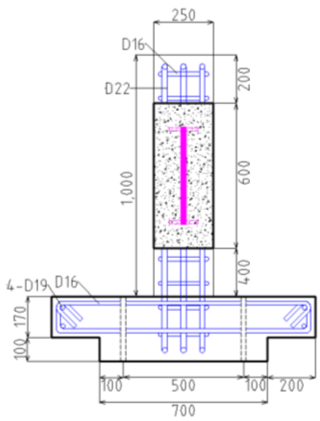

Fig. 12 Section B-B

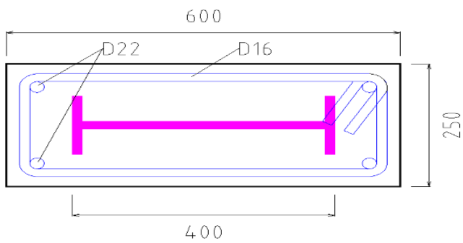

Fig. 13 Section of inner panel

The eccentricity distance, e, from the center of the load to the center of the connection steel plate can be computed by the following equation.

$$\mathrm{e} = \mathrm{a} + l_e/2 = 60 + 300/2 = 210\ \mathrm{mm} \tag{68}$$

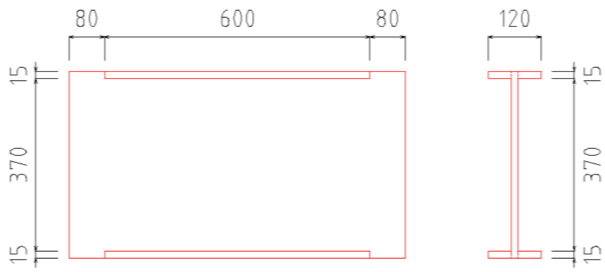

Fig. 14 Detail of embeded H-beam

Where, a denotes shear span (mm), and l_e represents effective moment of inertia for computation of deflection that is 300mm in Fig. 14. b of Fig. 14 represents width of compression face of member, ω refers to width of flange (mm), and A_s denotes area of longitudinal tension reinforcement (mm^2). The length of b is computed as shown below.

$$\mathrm{b} = 2.5\omega = 2.5(120) = 300\ \mathrm{mm} \tag{69}$$

According to the theory of Marcakis, K., and Mitchell, D. (1980), the shear capacity of concrete, V_c, is expressed as in Eq. (71). β_1 is the factor relating depth of equivalent rectangular compressive stress block to neutral axis depth.

$$\beta_1 = 0.85 - 0.007(f_{ck} - 28) = 0.82 \tag{70}$$

$$\begin{aligned}\phi V_c &= \phi \beta_1 f_{ck} b l_e / \left(1 + \frac{3.6e}{l_e}\right) \\ &= 0.75 \times 0.82(32.3)(300)(300)(10^{-3})/(1 + 3.6(210/300))\end{aligned} \tag{71}$$

The width to thickness ratio of the steel plate is given by the following equation.

$$\begin{gathered}\frac{\mathrm{h}}{t_w} < 2.24\sqrt{E/F_y} \\ \frac{\mathrm{h}}{t_w} = \frac{800}{15} = 53.3 < 2.24\sqrt{200{,}000/235} = 65.3\end{gathered} \tag{72}$$

As shown in Fig. 18, h represents overall height of member (mm), and

t_w denotes thickness of web (mm). The shear capacity of the connection steel plate section is given by the following equation.

$$\phi V_n = \phi_v\left(0.6 f_y A_w C_v\right)$$
$$= 1.0(0.6 \times 235 \times 300 \times 20 \times 1.0) \times 10^{-3} = 846 \text{ kN} > 507.9 \text{ kN} \tag{73}$$

Where, A_w represents section area of shallow steel plate (mm2), and C_v is sheer buckling reduction factor.

4.2 Shear strength steel plate and welding at connection

The shear strength, V, on the protruding section of the steel plate, that was installed on the PC beam, is given by the following equation.

$$V = \phi\left(0.6 t l f_y\right)$$
$$= 0.75(0.6 \times 400 \times 15 \times 235) \times 10^{-3} = 634.5 \text{ kN} \tag{74}$$

Where, t represents thickness of shallow steel plate in the two welded steel plates at the connection, and l denotes length of steel plate, which is smaller in area at the connection.

t is 6 mm or greater, and the maximum size of the filet welding, s, is given by the next equation, $s = t - 2$ (mm). Since the thickness of the shallow plate for filet welding is 20 mm, the size of filet welding is given by the following equation and is computed to be 18 mm.

$$s_{max} = t - 2 = 20 - 2 = 18 \text{ mm} \tag{75}$$

$$F_w = 0.6 F_y = 0.6 \times 235 = 141 \text{ N/mm}^2 \tag{76}$$

$$a = 0.7s = 0.7 \times 18 = 12.6 \text{ mm} \tag{77}$$

$$l_e = 2 \times 200 = 400 \text{ mm} \tag{78}$$

$$A_w = a \times l_e = 12.6 \times 400 = 5{,}040 \text{ mm}^2 \tag{79}$$

Where, F_w represents nominal strength of welding (N/mm^2), A_w denotes effective area of welding (mm^2), and V_u means sheer force of welding.

$$V_u = \phi F_w A_w$$
$$= 0.9(141 \times 5{,}040) \times 10^{-3} = 639.6 \text{ kN} \tag{80}$$

Table 5 Shear strength of top Connections for specimen PR1-UP

Speci-mens	1) Cast-in H-Beam strength [kN]		2) Connection strength [kN]		3) Minimum chosen[kN]
	Embedded H-Beam shear Strength	Shear strength of H-Beam	Shear strength of steel plate at connection	Shear strength of welding	
PR1-UP	507.9	846.0	634.5	639.6	507.9

5. EXPERIMENTAL RESULTS

5.1 Load-displacement relationship

The connection parts of specimen P1, a stud-connected specimen, and P2, a steel plate welding connected specimen, were composed of two types of cast-in anchors and cast-in welded steel plate, respectively. Shear tests on the specimens P1 and P2 were carried out as depicted in Fig. 15.

Fig. 15 Image of specimen

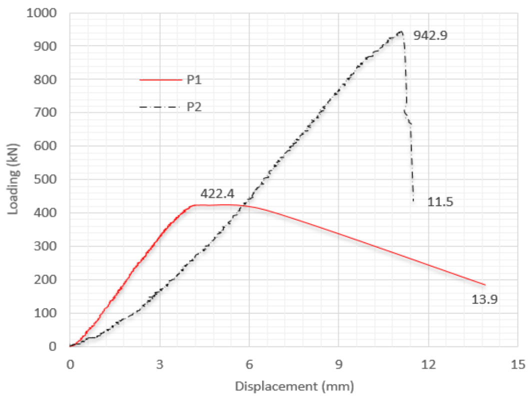

Fig. 16 Load-displacement relationship

Displacement of P1 increased linearly up to the maximum load of 422.4 kN as shown in Fig. 16 and then exhibited plastic deformation to 13.9 mm, at which it failed finally. On the contrary, the specimen P2 manifested its strength up to 942.9 kN, more than twice of the strength of P1, and exhibited brittle failure at the maximum load. The reason for the brittle failure of P2 is construed to be the lack of an alternative measure that can withstand the load additionally after tensile failure of the connection steel plate. Six anchors of specimen P1 sequentially failed or yielded to reach the ultimate load. It exhibited large deformation after the ultimate load and manifested final failure. Among the two specimens, specimen P1 showed greater initial stiffness.

Although the specimen P2 manifested brittle failure after the maximum load, the specimen P1 relatively exhibited ductile failure mode as shown in Fig. 16. The maximum loading of the analytical result was computed to be 6.5% greater than that of the experimental result on the average as shown in Table 6.

Table 6. Comparison between analytical and experimental results

	Analytical result	Experimental result			
	1) Analytical maximum loading for connection [kN]	2) Experimental maximum loading [kN]	3) Experimental maximum loading for connection [kN]	4) Experimental displacement at maximum loading [mm]	1) /3) Experimental [%]
P1	222.4	422.4	211.2	4.4	105.3
P2	507.9	942.9	471.5	11.1	107.7
Average	243.77	455.77	228.57	6.50	106.5

5.2 Cracking

The specimen P1 showed failure only in the connection part or between the connection part and panel, as shown in Fig. 17 and Fig. 18. It did not show any failure within the central panel itself or external panel. On the contrary, the cracking of P2 specimen occurred at just below the uniform loading points at both left and right sides as shown in Fig. 19 and Fig. 20. It is considered that the flange part of the embedded H-beam buckled to exhibit deformation and failure. Cracking of the specimen P2 was much less than P1, and it failed all at once by single cracking.

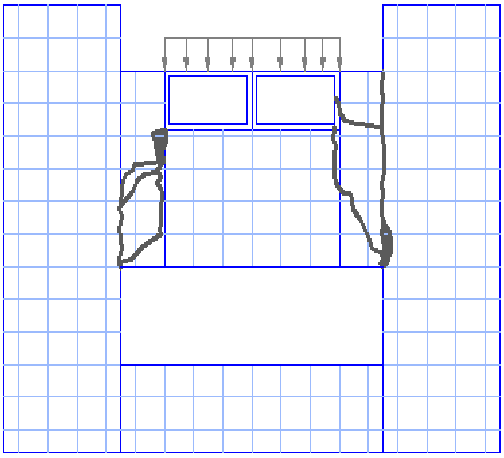

Fig. 17 Front view of P1 specimen

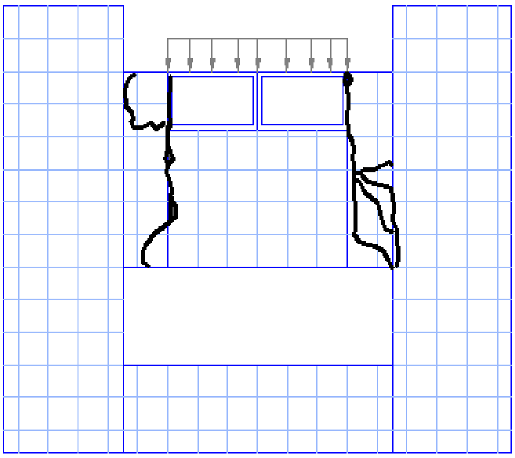

Fig. 18 Rear view of P1 specimen

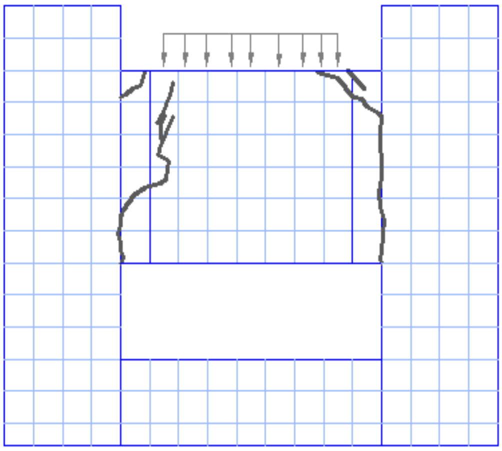
Fig. 19 Front view of P2 specimen

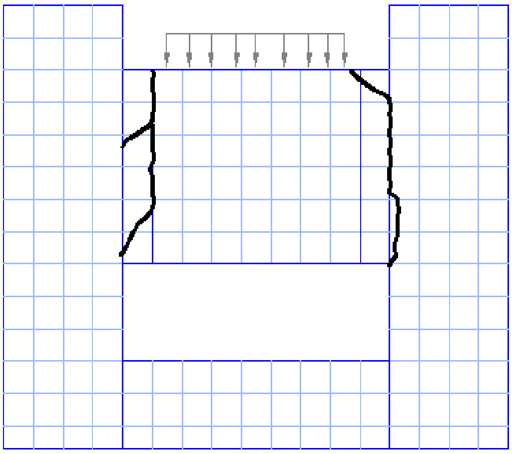
Fig. 20 Rear view of P2 specimen

5.3 Friction between PC concrete and connection concrete

PCI Handbook (7h Edition, pp. 5~52) found in PCI Design Handbook 7th edition (2010) specified that the shear resistance capacity, V_{fr}, should not surpass the limit of the following equation (81) in case of intentionally roughened and not including a tie or the case of including a minimum tie and not intentionally roughened.

$$\begin{aligned} V_{fr} &= \phi 0.55 b_v l_{vh} \\ &= 0.75(0.55\ \mathrm{MPa}) \times 250 \times 600 \times 10^{-3} = 62.1\ \text{kN} \end{aligned} \tag{81}$$

Where, ϕ denotes reduction factor, b_v represents the connection width of the prior concrete, and l_{vh} indicates the connection length.
Since this study did not intentionally roughened the surface around the PC part, the friction force between different concretes will be 60% of the value computed by the equation (81) for intentionally roughened case in accordance with the ACI 318M-11 11.6.4.3.

$$\begin{aligned} V_{Fr} &= 0.6 V_{fr} \\ &= 0.6 \times 62.1 = 37.3\ \text{kN} \end{aligned} \tag{82}$$

However, the bearing capacity due to the concrete friction, V_{Fr}, was relatively small, decreased rapidly after cracking, and reached failure due to shear of the anchor or threshold pry-out strength. In other words, the friction force exerted some influence initially, but it did not significantly affect the final failure strength. Thus, the friction force between concretes was disregarded in this study.

6. CONCLUSIONS

1. Displacement of P1, the anchor connected specimen, increased linearly up to the maximum load of 422.4 kN and then exhibited plastic deformation to 13.9 mm, at which it failed finally. On the contrary, the specimen P2 manifested its strength up to 942.9 kN, more than twice of the strength of P1, but exhibited brittle failure at the maximum load.

2. The specimen P1 showed failure only in the connection part or between the connection part and panel. It did not show any failure within the central panel itself or external panel. On the contrary, the cracking of P2 specimen occurred at just below the uniform loading points at both left and right sides. It is considered that the flange part of the embedded H-beam buckled to exhibit deformation and failure. The number of cracking of the specimen P2 was much less than P1, and it failed all at once by a couple of cracking.

3. The shear strength of stud connection part were computed with shear strength of stud, pry-out strength, and concrete breakout strength. Summing of each shear capacity at each stud end, which was used in this study, was carried out in computation of pry-out strength and concrete breakout strength. The analytic result was close to the experimental result at 106.5% greater than the latter.

4. Shear strength of embedded steel connection was analyzed with the equation for embedded H-beam of Marcakis, K., and Mitchell, D. The analytic result was close to the experimental result with the ratio of the analytic to experimental result of 107.7%.

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