

The Effect of Anchorage of Shear Reinforcement in Flat Plate Slab

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ABSTRACT: The punching shear on the flat plate slab-column connection can bring about the reason of the brittle punching shear failure which may result of collapsing the whole structure. From the development of residential flat plate system, the shear reinforcement is developed for preventing the punching shear. For making sure of the punching shear capacity, developed for shear reinforcement in slab-column connection, the structural test is performed. The dimension of the slabs was 2620*2725*180mm with square column(600*800mm). The slabs were tested up to failure monotonic vertical shear forces. The presences of S/S bar and wire mesh substantially increased the punching shear capacity and the ductility of the slab-column connections.

Keywords: punching failure, shear reinforcement, gravity force

1. INTRODUCTION

Reinforced concrete flat plate system is much more economical than other types of structural system such as frames and shear walls. This system was restrictively used in underground parking lots in Korea. However, it has become popular to use this system in high-rise residential buildings these days.

Flat plate system has some structural weaknesses compared to frame system. Punching shear failure near the slab-column connection is major concern of flat plate system, since this failure mechanism could lead to collapse of the whole structure. Directly transferred load from the slab to the column causes stress concentration around

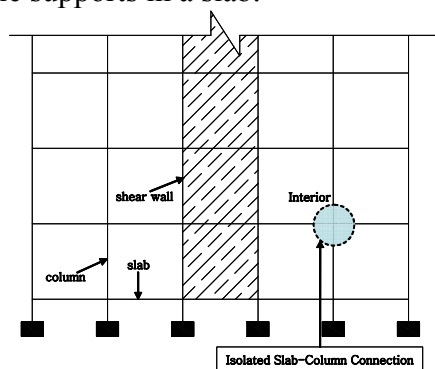
the column, and this stress causes punching shear failure. To prevent punching shear failure three types of new shear reinforcements are proposed. These shear reinforcements are economical because manufacture process is simple and applications at the construction site are excellent.

The purpose of this study is to investigate performance of the slab-column connection of flat plate system which has newly developed three shear reinforcements. To accomplish this four full-scale slab-column connections were tested and analytical studies were conducted. The major difference of three reinforcements was material and shape.

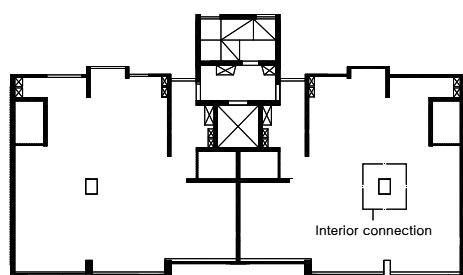
2. EXPERIMENTAL PROGRAM

2.1 Design of Specimen.

To evaluate shear capacity of proposed shear reinforcements, one specimen without shear reinforcement and three reinforced specimens which have different shear reinforcement were tested. In order to ease the production of specimens and execution of tests, slabs of the test specimens were cut one forth point of a slab from a column assuming the inflection point was generated at the point when the specimens were subjected to relatively low levels of gravity load. Both sides of a slab were enlarged as much as 150mm to meet the condition of simple supports in a slab.



(a) Prototype building structure



(b) prototype building plan

Figure 1: Prototype Building

2.2 Material Tests

The test specimens were designed based on the compressive strength of concrete of 21 MPa. To obtain the properties of concrete, test cylinders were produced and cured based on KSF 2404 specification. The test results of concrete compressive strength are shown in Table 1. Properties of steel reinforcement were obtained based on the experiment using coupons meeting KS B

0801 specification and test procedure meeting KS B 0802 specification. The test results of steel tensile strength are shown in Table 2.

Table 1: concrete compressive strength test

Concrete Compressive Strength (MPa)	Young factor (GPa)	Poisson's ratio
21.8	21.92	0.15

Table 2: Steel bar tensile strength test

Steel bar	Yield Strength (MPa)	Yield Strain ($\times 10^{-6}$)	Ultimate Strength (MPa)
D13	491	2864	665
D10	555	4948	774
D22	484	2871	773
D10	519	3500	778
$\emptyset 6$	375	1923	375

2.3 Reinforcement detail

The proposed shear reinforcements are shown in Figure 2. These reinforcements play a role of spacers when arranging rebars, and they enable to secure effective depth between top and bottom longitudinal bars, which secures structural stability. By using the reinforcements, additional spacers are not required. The shear reinforcements were installed at every 75mm intervals, which is a half length of the depth of the slab ($0.5d$), from column face to a length of 5 times the depth of the slab ($5d$).

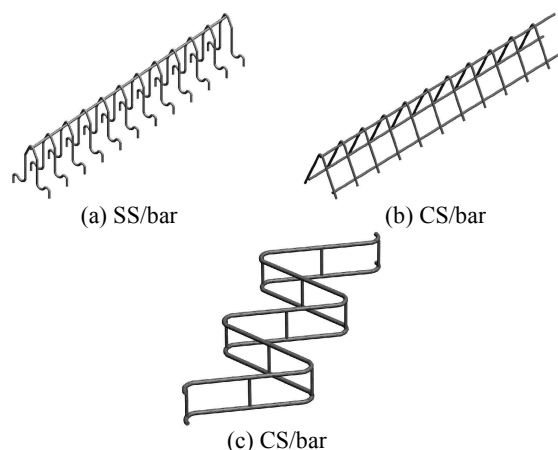
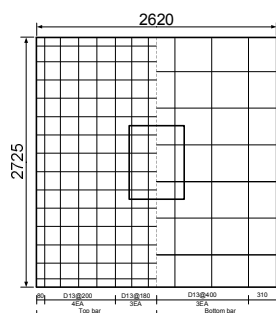


Figure 2: Reinforcement

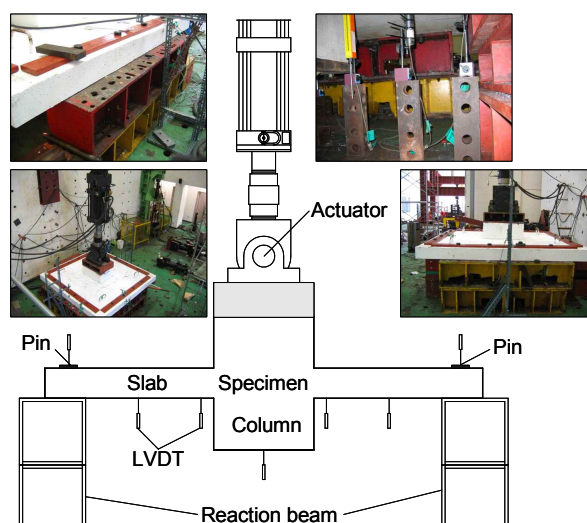
Table 3: Dimension of Specimen

Specimen	L1 (mm)	L2 (mm)	c1 (mm)	c2 (mm)	h (mm)	d (mm)	b0 (mm)	fck] (MPa)	f _y (MPa)	p(c+3h)	p _t
control	2620	2725	600	800	180	150	3400	21.8	489	0.0055	0.002
FP-S/S bar	2620	2725	600	800	180	150	3400	21.8	489	0.0055	0.002
FP-CS-bar	2620	2725	600	800	180	150	3400	21.8	489	0.0055	0.002
FP-wiremesh	2620	2725	600	800	180	150	3400	21.8	489	0.0055	0.002



2.4 Test Setup

The test setup of a specimen is shown in Figure 4. Reaction steel beams were installed to support the specimen and were tied to the specimen. An actuator was linked to the reaction frame and the column of the specimen, and the specimen was loaded by pull-out system. Although deflection and punching shear failure of the slab were supposed to be caused by uniformly distributed loads to meet design condition, this experiment was carried out simulating the loading condition of actual construction. Two LVDTs on the top of the slab was installed to measure the movement of slab when loading was applied and LVDTs under the column was installed to measure maximum deflection of specimen. To estimate movement of slab, LVDTs were installed at the point of 0.5d and 10d from x direction, and 0.5d and 6d from y direction. The experiment was executed using the load control system. The movement of each rebar and shear reinforcement was measured by installing four strain gauges at the point of 0.5d, d, 3d, and 5d from x, y direction of upper longitudinal rebar and shear reinforcement in the slab.

**Figure 4:** Test Setup

3. Test Result

Figure 4: Test Setup

3.1 Load-Displacement Relation

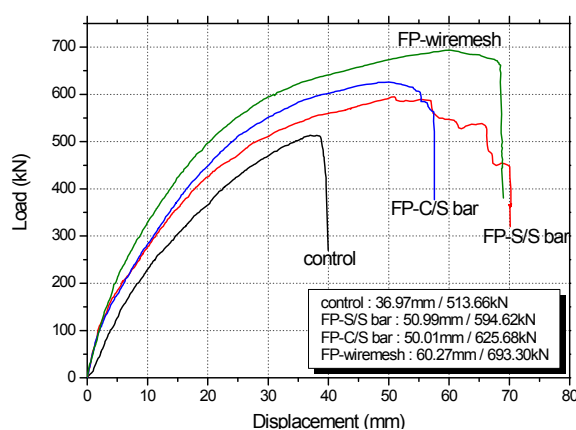
The displacement of specimen was measured with LVDT on the lower part of the column. Table 4 shows comparison between load and displacement relationship of the control specimen, which is the unreinforced specimen, and the reinforced specimen. The ductility rate on deflection of each specimen was estimated from the

Table 4: Test result

specimen	Yield load V_y (kN)	Ultimate load V_{max} (kN)	Yield deflection δ_y (mm)	Ultimate deflection δ_{max} (mm)	Strength ratio	Deflection ratio	Displacement ductility δ_{max}/δ_y
Control	440.1	513.66	22.45	36.97	-	-	1.65
FP-S/S bar	462.98	594.62	23.99	50.99	1.16	1.38	2.13
FP-CS/bar	518.94	625.68	25.88	50.01	1.22	1.35	1.93
FP-Wiremesh	532.26	693.30	23.10	60.27	1.35	1.63	2.61

relationship between ultimate displacement and yield displacement. To estimate yield displacement, offset method was used in this research.

For the control specimen ultimate strength of 513.66kN was recorded at displacement of 37mm, and sudden punching shear failure reduced the strength dramatically at displacement of 40mm. On the other hand, for FP-SS/bar specimen ultimate strength of 594.62kN was recorded at displacement of 51mm, and failure occurred after repeating the load cycles which reduced the strength gradually. For FP-CS/bar specimen ultimate strength of 625.68kN was recorded at displacement of 50mm and failure occurred after gradual reduction of the strength until displacement of 57mm. For FP-wiremesh specimen ultimate strength of 693.30kN was recorded at displacement of 38mm.

**Figure 5:** Load-Displacement Curve

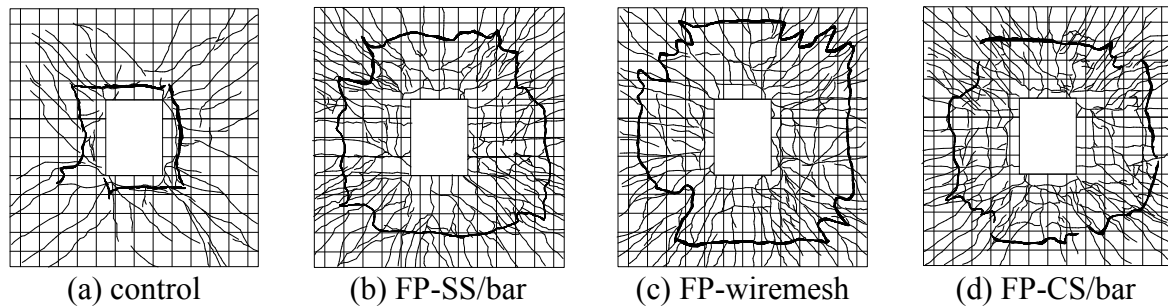
Based on observation of load-displacement curves of the specimens, the strength of the reinforced specimens increased on the average of 23% compared to the control specimen, and punching shear

resistance of Wiremesh was the highest among the reinforced specimens. This was the result of jointing the top and the bottom bars with the web bars, which caused truss action and dowel action on the top and the bottom bars and improved capacity of the connection.

3.2 Failure Mode

The control specimen showed the diagonal initial cracks developing from the edge of the column at the load of 162kN. As the load increased gradually, these cracks were concentrated on the critical section of the slab-column connection and eventually sudden punching shear failure occurred after increasing of crack width. On the other hand, three other reinforced specimens generated both flexural cracks and shear cracks at the same time, and failed outside of shear reinforced area after cracks were uniformly developed throughout the slabs.

The shear reinforced specimen which used curved wires showed the early crack on the edge of the column. As the load increased, numbers of cracks on the critical section of the slab-column connection increased and the cracks were spread toward outside of the slab. Later, the cracks were increased gradually outside of reinforced area. After the load was declined gently, failure was followed. The curved shear reinforcement prevented brittle failure due to redistribution of the load at ultimate strength after cracks occurred and showed increased ductile capacity of the connection. FP-wiremesh specimen had the widest destruction area and the highest shear reinforcement capacity.

**Figure 6:** Failure Mode

3.3 Flexural Reinforcement Strain

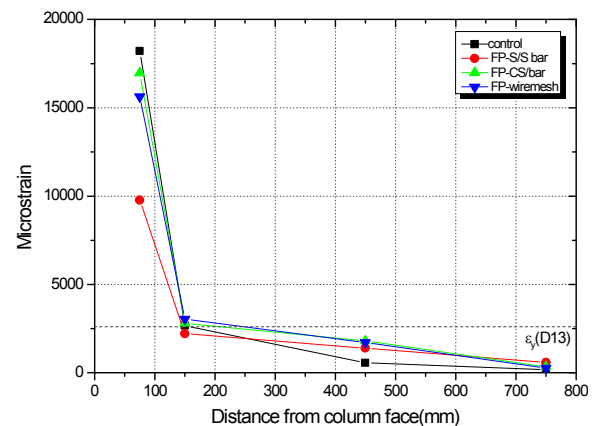
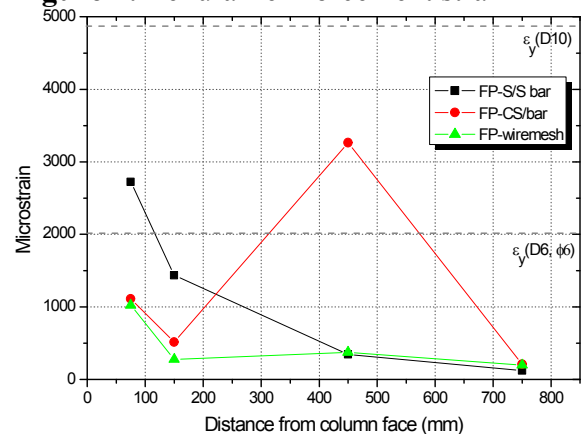
All specimen's top reinforcement strain turned out to be in inverse proportion to the distance from column face, and top bar strain at the distance of $0.5d$ from column face showed ultimate strain, which exceeded yield strain in every specimens. For the control specimen, although the critical section around the column and the top bar behaved well, brittle punching shear failure governed. Although other reinforced specimens had high strain at the top bar at the distance of d from column face, punching shear failure occurred outside of reinforced area. Punching shear failure was affected by flexural failure in the area of critical section, and beyond that area, punching shear failure occurred.

3.4 Reinforcement Strain

To measure the behavior of shear reinforcements in the slab, strain gauges were installed vertically on the shear element of the shear reinforcement. The strain gauges were installed on the shear reinforcement at the distance of $0.5d$, d , $3d$, and $5d$ for x and y directions, respectively. Fig 7 shows the graph of strain of the shear element of the shear reinforcement at the ultimate strength.

Except for FP-CS/bar specimen, other shear reinforcements represented the highest strain at the distance of $0.5d$, and strain became lower as shear reinforcement became more distant from the column. For FP-CS/bar, strain of shear element at the distance of 450mm from the column face was the highest. It was because inclined cracks influenced on the shear element

greatest at the distance of 450mm , and shear reinforcement resisted.

**Figure 7:** flexural reinforcement strain**Figure 8:** Reinforcement Strain

3.5 Displacement ductility

Yield displacements of all specimens were closed to each other since yield displacement of the reinforced specimens showed maximum yield displacement increase of 17% compared to that of the control specimen. However, deflection at the ultimate strength was increased as much as 38% in FP-SS/bar, 35% in FP-CS/bar, and 63% FP-wiremesh. Especially, FP-wiremesh specimen represented higher

increase in strength and displacement than those of other specimen. This is because of deformed shear reinforcements increased bonding strength and welding of the web reinforcements and the top and the bottom longitudinal reinforcements increase of dowel action.

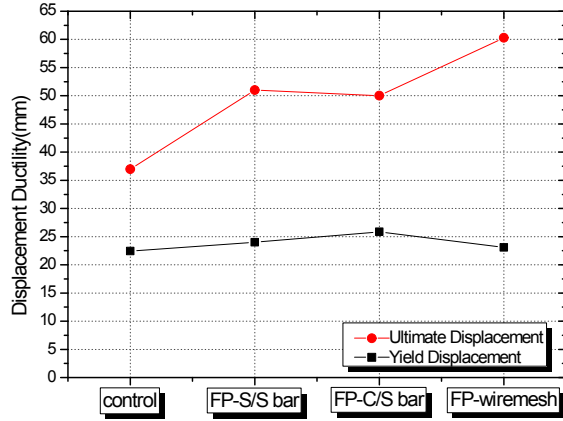


Figure 9: Displacement Ductility

4. Result Analysis

4.1 Analysis based on KCI-2007

The shear design is based on

$$V_u < \phi V_n \quad (1)$$

According to KCI-2007, shear strength, V_c , of two-way slab is calculated using minimum value among following three formula.

$$V_c = \frac{1}{3} \sqrt{f_{ck}} b_o d \quad (2)$$

$$V_c = \frac{1}{6} \left(1 + \frac{\alpha_s d}{2b_o} \right) \sqrt{f_{ck}} b_o d \quad (3)$$

$$V_c = \frac{1}{6} \left(1 + \frac{2}{\beta_c} \right) \sqrt{f_{ck}} b_o d \quad (4)$$

For slabs with Shear reinforcement the nominal shear strength, V_n , is expressed by

$$V_n = V_c + V_s \quad (5)$$

In the presence of shear reinforcement the shear strength of concrete, V_c , is reduced to $V_c = \sqrt{\frac{f_{ck}}{6}} b_o d$

The shear strength of the shear reinforcement, V_s , is computed as

$$V_s = \frac{A_s f_y d}{s} \quad (6)$$

By applying test results to the equations,

$$V_n = V_c + V_s$$

$$V_c = \sqrt{\frac{f_{ck}}{6}} b_o d$$

$$= \frac{1}{6} \sqrt{21.8} \times 3400 \times 150 \times 10^{-3}$$

$$= 337.34 \text{ kN}$$

$$V_s = \frac{A_s f_y d}{s} = \frac{71.3 \times 3 \times 4 \times 400 \times 150}{75} \times 10^{-3}$$

$$= 684.5 \text{ kN}$$

$$V_n = V_c + V_s$$

$$= 337.34 + 684.5 = 1021.84 \text{ kN}$$

Design strength of two-way shear strength of shear reinforced slab of 1021.84kN was computed. But the test result recorded was 625.68kN which was 60% of the design value.

$$V_n = V_c + K \times V_s$$

$$= 337.34 + 0.42 \times 684.5$$

$$= 624.83 \text{ kN}$$

The shear strength of shear reinforcement showed approximately 42% of computed value of equation (6). When the design value was compared to the test result, computed test result was lower than design value for the following reasons;

- According to previous studies, the slab with thickness smaller than 250mm can not perform effectively due to insufficient development length of shear reinforcement in the slab.

- In case of proposed reinforcements, since the shear reinforcements were installed between the top bar and the bottom bar, shear elements generated slip failure before they reached yield strength.

4.2 Proposal of K factor

V_s which is provided by design code can be achieved when shear reinforcement is sufficiently bonded. But the proposed reinforcement was not expected to provide adequate development length due to thin

slab. For this reason V_s formula proposed in the previous design was not appropriate to apply. K factor was proposed for the variable considering effect of shear reinforcement due to slab thickness and concrete compressive strength.

K value was evaluated from forty three test specimens of 11 papers which studied on shear reinforcement. It was found that as the thickness of slab increased the K factor also increased. It implied that longer development length provided better shear reinforcement.

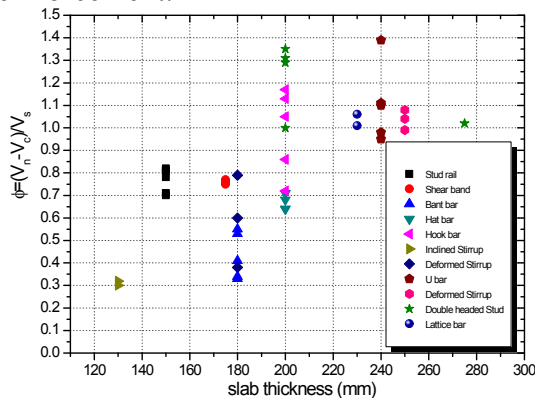


Figure 10: Previous Study (anchorage effect)

4.3 Development effect according to anchorage length

The proposed reinforced specimens were spot welded between transverse reinforcement and shear element to improve development. Finite element analysis method (FEM) was performed to examine accurate development effect for the reinforced specimens. For concrete and bond requirement for steel and concrete, Hognestad's model and CEB-FIP bond-slip model were chosen for the analysis, respectively, and ANSYS program were used for the analysis.

Development effect with slab thickness and concrete compressive strength was considered in the spot welded specimen. To estimate accurate shear strength formula of three proposed reinforced specimens, K factor was used.

$$K = [1 + 0.6(f_{ck} - 21)](0.0077d - 1.04)$$

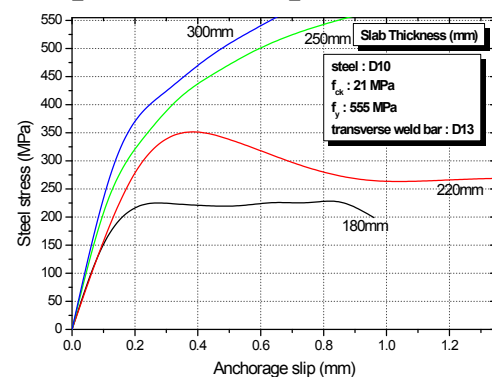


Figure 11: Effect of Anchorage Length

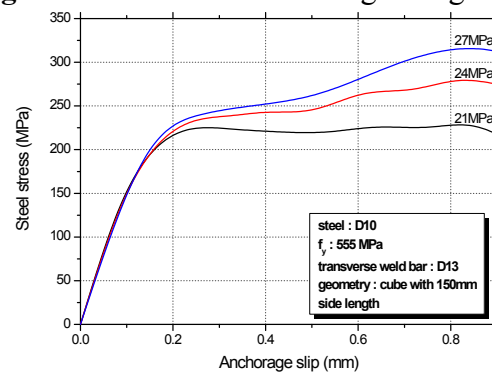


Figure 12: Effect of Concrete strength

5. Conclusions

Conclusions from the experimental results and the analytical studies are summarized as follows;

1. For the shear reinforced specimens, ultimate strength was increased as much as 16~35% and deflection was increased as much as 35~63% compared to control specimen. This shows that proposed shear reinforcements are effective reinforcements and improve shear strength and deformation capacity on the slab-column connection.

2. After cracks were generated at the critical sections of the slab, the width of the cracks did not increase while these cracks were spread throughout the slab. This happened by avoiding stress concentration and distributing stress equally throughout the whole slab, while stress concentration at the critical sections occurred at the control specimen.

3. The anchorage behavior of links can be improved if transverse welded bars were placed.

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