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Under seismic loading, experimental investigation was conducted on the punching shear behavior of RC slab-column connections including shear slabs.

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ABSTRACT

The practicality of reinforced concrete flat slab-column constructions makes them commonly employed. Nevertheless, punching-shear failure in the slab-column connections is a real possibility in these kinds of buildings. The slab-column connection is vulnerable to brittle punching failure in the absence of shear reinforcement, particularly in seismic zones where lateral stress is common. A excellent kind of transverse reinforcement that prevents punching failure, shear studs are efficient. The purpose of this study is to investigate the usage of shear studs to enhance the ductility and punching shear capacity of inner slab-column connections subjected to both continuous vertical load and cyclic loading. The FP-GR-CTRL specimen was the only one tested vertically without cyclic loading; the others, FP-VR-0.4, FP-VR-0.6, FP-VR-0.8, FP-SS-0.4, FP-SS-0.6, and FP-SS-0.8, were tested consistently vertically with an increasing amount of reversed cyclic loading until punching shear failure was reached. Discussing how the shear studs affect the punching shear behavior is the primary goal. Lastly, the experimental outcomes are evaluated and contrasted with global standards like the European Union's EC2-2004 and the American Code ACI318-14. We provide some first findings based on these data.

KEYWORDS: Seismic loading, displacement control, punching shear, shear studs, interior slab-column connections, and the gravity shear ratio are all important concepts.

1. INTRODUCTION

An extremely common kind of reinforced concrete construction is the flat slab structure, which uses a flat plate and columns to support the slab rather than beams between the columns. This sort of structure's design and behavior are the focus of the investigation. Seismic zone slab-column connections' punching shear behavior is the main focus. Reduced building story height, ease of formwork setup, simplicity for utilities placement, and nice slab look are some of the reasons flat slab-column structural systems are popular. Nevertheless, brittle punching shear failure is a common problem for structures of this sort. Cracks will form within the slab near the column when the flat slab-column connections are loaded vertically with high loads. These fissures eventually reach the base of the slab after cutting through its thickness at an angle of twenty to forty-five degrees. As a result, the slab's fissures may experience punched shear failure. The slab-column connection is more prone to punching shear failure when exposed to seismic lateral loads because shear stresses in the slab rise as a result of an unbalanced force caused by horizontal cyclic loading.

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2. EXPERIMENTAL PROGRAM

A total of seven full scale specimens were tested. The specimen can be regarded as part a prototype structure of which the flat concrete slab spans 4.5 m between columns. The slab thickness is 200 mm. Figure 1 shows the concrete dimensions of all tested specimens while Figure 2 shows layout of shear studs for specimens with shear studs. The specimens represent interior slab-column connections, which are isolated specimens with dimensions correspondingto the lines of contra flexure under gravity loads. The control reference specimen FP-GR- CTRL, was subjected to a monotonic vertical load up to punching shear failure. Specimens without shear studs FP-VR-0.4, FP-VR-0.6 and FP-VR-0.8 were subjected to a constant verticalload V=230, 295 and 393 KN respectively, in addition to a reversed displacement controlled cyclic loading which was increased up to punching shear failure. Finally, all tested specimens with shear studs FP-SS-0.4, FP-SS-0.6 and FP-SS-0.8 were subjected to a constant vertical loadV=195, 295 and 393 KN respectively, in addition to a reversed displacement controlled cyclicloading up to failure. The specimens FP-VR-0.4, FP-VR-0.6, FP-VR-0.8, FP-SS-0.4, FP-SS-0.6 and FP-SS-0.8 were subjected to constant gravity shear ratio V/V_0 equal 0.4, 0.6, 0.8, 0.4, 0.6 and 0.8 respectively, where V is the applied vertical load and V_0 is the vertical load causing punching shear failure according to ACI318-14 [16].

In the tension side of the concrete slab, the flexural reinforcement ratio is 1.62% within a width of 824 mm from the center of the slab as shown in Figure 3. The reinforcing ratio in thecompression side of the slab is 0.6%. The reinforcement is designed by using first principles toensure punching shear failure of these connections and not failure due to flexure. The reinforcing ratio of the columns is 4.86% and closed ties are used in order to make the columnstrong enough to transfer the axial load and unbalanced moment to the slab. Figure 3 shows thereinforcement details of all tested specimens. For control purposes, standard concrete cubes andcylinders were cast alongside the specimens and were tested at the same day as the specimen. The compressive cylinder concrete strengths for specimens FP-GR-CTRL, FP-VR-0.4, FP-VR-0.6, FP-VR-0.8, FP-SS-0.4, FP-SS-0.6, and FP-SS-0.8 equal 27, 35, 25, 25, 25, 25 and 25 N/mm^2 respectively.

The main reinforcement and shear studs were made of deformed steel bars (Grade 40/60) of actual yield stress (F_Y) of 400 MPa, ultimate tensile strength f_u of 600 MPa and modulus of elasticity E_s of 200 GPA.

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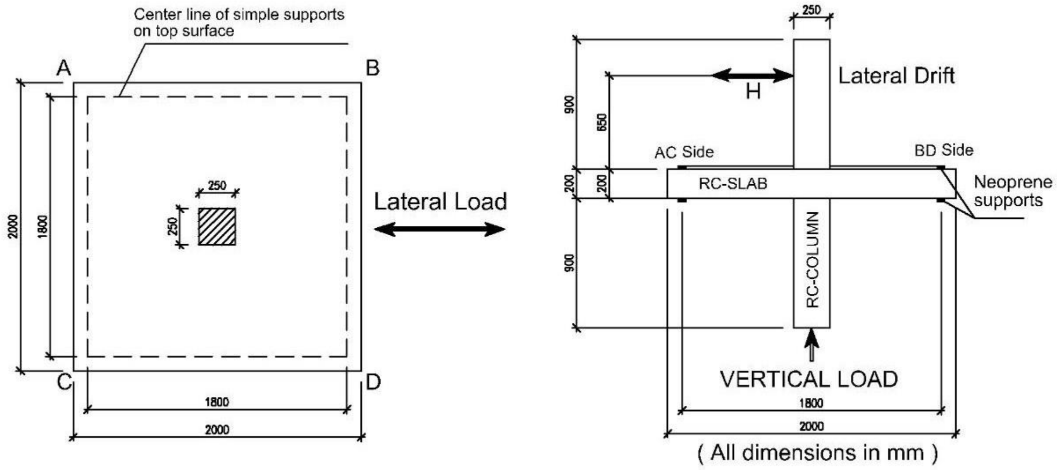


Figure 1: Concrete dimensions of all tested specimens

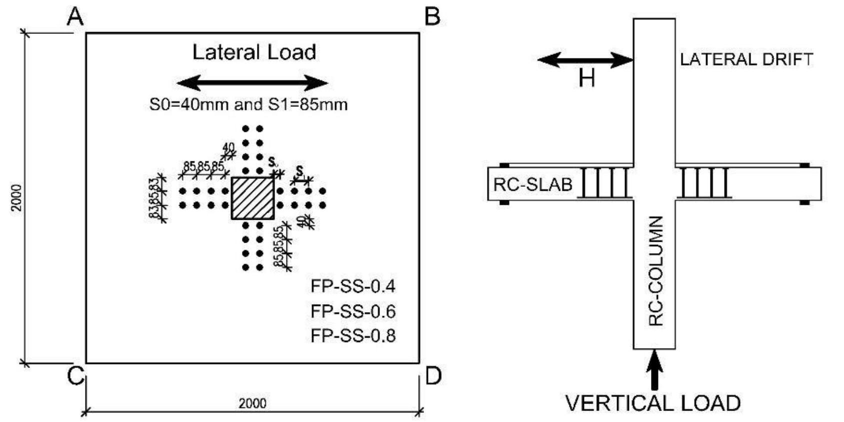
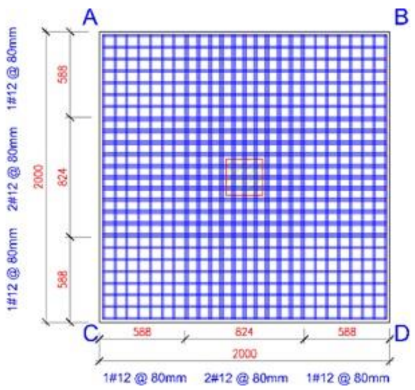
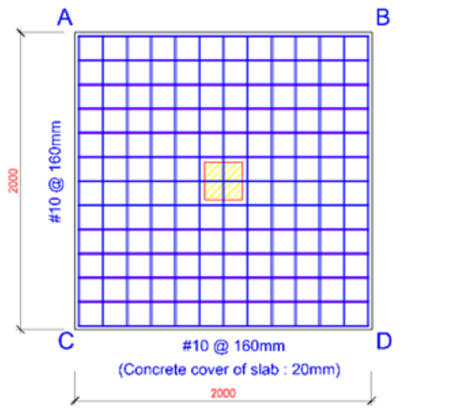


Figure 2: Layout of shear studs for specimens FP-SS-0.4, FP-SS-0.6 and FP-SS-0.8



(a) Tensile reinforcement mesh (upper)



(b) Compression reinforcement mesh (lower)

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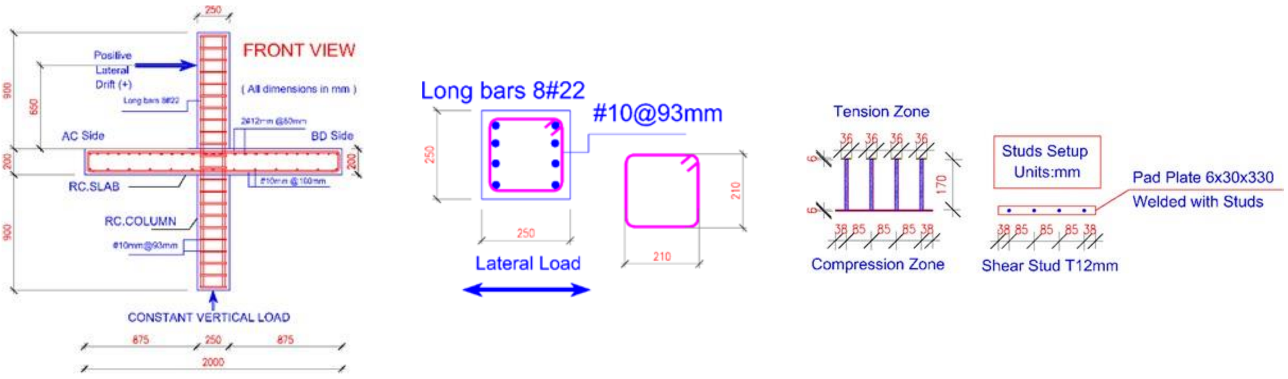

(c) Specimen reinforcement details (d) Column reinforcement (e) Shear studs properties

Figure 3: Reinforcement details of all tested specimens

3 TEST SETUP, BOUNDARY CONDITIONS AND LOADING SCHEME

Specimen FP-GR-CTRL was tested under vertical load only while all other specimens were tested under a constant vertical load, in addition to reversed cyclic load up to punching shear failure. Figure 4 shows a schematic of test set up of all specimens and the protocol cyclic loading path of tested specimens. Figure 5 shows the horizontal and vertical actuators as well as the digital recording computer system used to record the drift ratio versus the applied lateralload

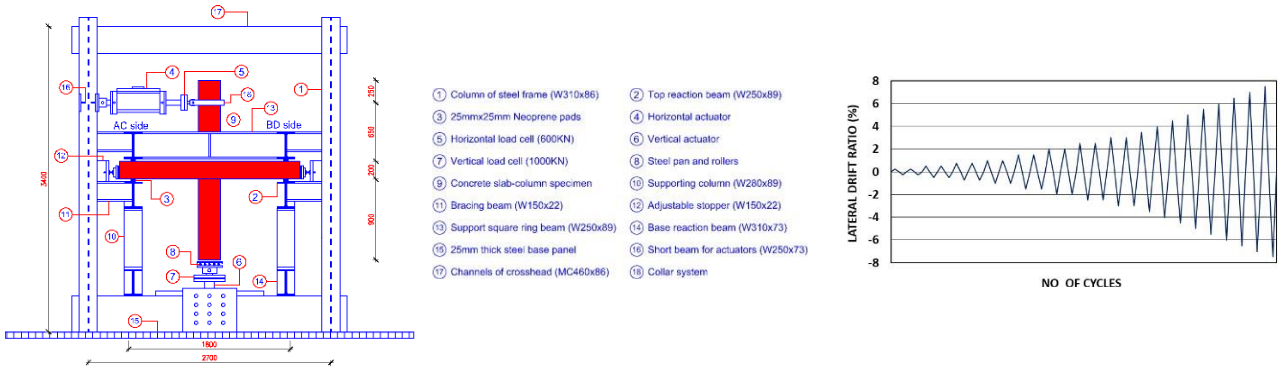

Figure 4: Schematic of test setup of tested specimens and cyclic loading path

Figure 5: The horizontal-vertical actuator and digital recording computer system

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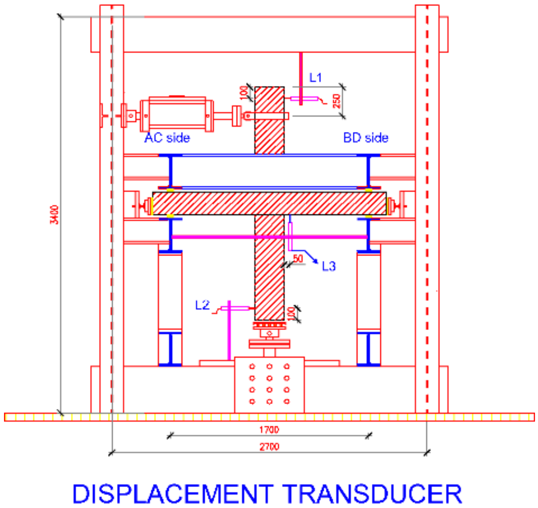
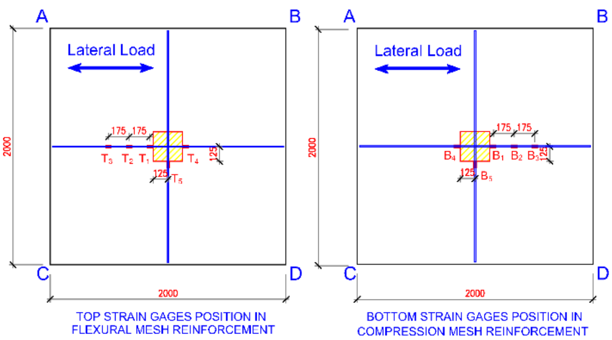
4. INSTRUMENTATION

Measurements were made thoroughly for displacements and steel strains at key locations of the tested specimens as shown as in Figures 6. All LVDTs and strain gauges were connected to a computer /controlled data acquisition system. The crack pattern was monitored and marked on the specimen with the associated load level indicated next to it.



(a) LVDT to measure of Lateral drift of top and bottom column



(b) Strain gages positions in flexural mesh reinforcement

Figure 6: Instrumentation scheme for all tested specimens

5 EXPERIMENTAL RESULTS

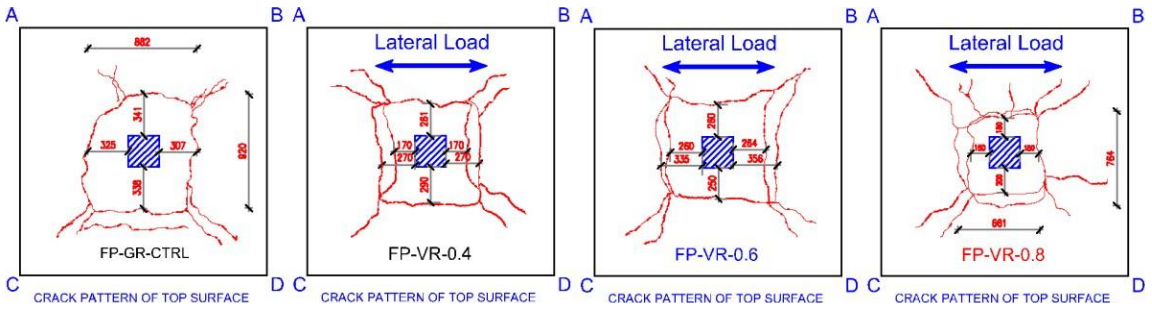
5.1. Observations and Crack Pattern for Specimens without Shear Studs

For specimen FP-GR-CTRL the first crack at the compression surface of the slab was observed at about a vertical gravity load equal 290 KN (about 43.5% of the failure load). For specimen FP-VR-0.4 cracks on slab surface started from corners of the column at the tension side, first on the top slab surface (which was subjected to tension from gravity load equal 230 KN) and then on the bottom surface. First crack at the bottom of the slab was observed at about 0.6~0.8% drift ratio. For specimen FP-VR-0.6 cracks on slab surface started from corners of the column at the tension side, first on the top slab surface (which was subjected to tension from gravity load equal 295 KN) and then on bottom surface. First crack at the bottom of the slab was observed at about 0.5~0.6% drift ratio. For specimen FP-VR-0.8 cracks on slab surface started from corners of the column at the tension side, first on the top slab surface (which was subjected to tension from gravity load equal 393 KN) and then on bottom surface. First crack at the bottom of the slab was observed at about 0.3~0.4% drift ratio. The final crack patterns of tension slab surfaces for tested specimens without shear studs are shown in Figure 7. All specimens failed in punching.

5.2. Observations and Crack Pattern for Specimens with Shear Studs

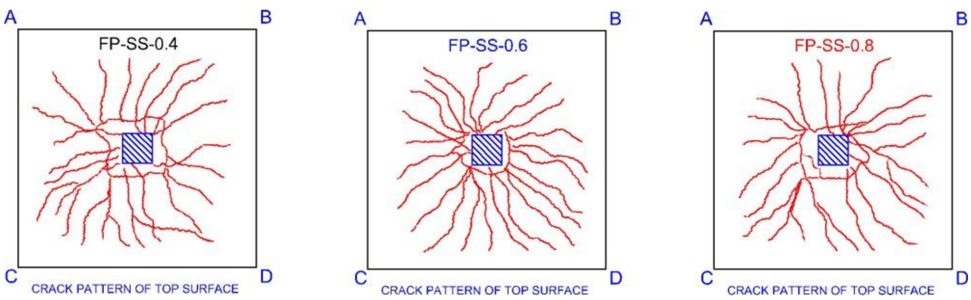
For specimen FP-SS-0.4 (concrete compressive cylinder strength f_c: *25MPa*) cracks on slab surface started from corners of the column at the tension side, first on the top slab surface (which was subjected to tension from gravity load equal 195 KN) and then on bottom surface. First crack at the bottom of the slab was observed at about 1.6~2.4% drift ratio. For specimen FP-SS-0.6 (concrete compressive cylinder strength f_c: *25MPa*) cracks on slab surface started from corners of the column at the tension side, first on the top slab surface (which was subjected to tension from gravity load equal 295 KN) and then on bottom surface.

First crack at the bottom of the slab was observed at about .11~1.6% drift ratio. Finally, for specimen FP-SS-0.8 (concrete compressive cylinder strength f_c: *25MPa*) cracks on slab surface started from corners of the column at the tension side, first on the top slab surface (which was subjected to tension from gravity load equal 395 KN) and then on bottom surface. First crack at the bottom of the slab was observed at about 0.6~0.87% drift ratio. The final crack patterns of tension slab surfaces for tested specimens with shear studs are shown in Figure 8. All specimens failed in flexure.



(a) FP-GR-CTRL (b) FP-VR-0.4 (c) FP-VR-0.6 (d) FP-VR-0.8

Figure 7: Crack pattern for each specimen without shear reinforcement (shear studs)



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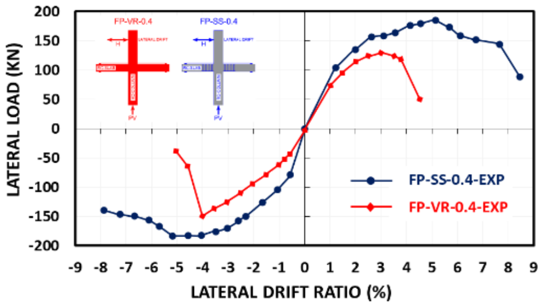
(a) FP-SS-0.4 (b) FP-SS-0.6 (c) FP-SS-0.8

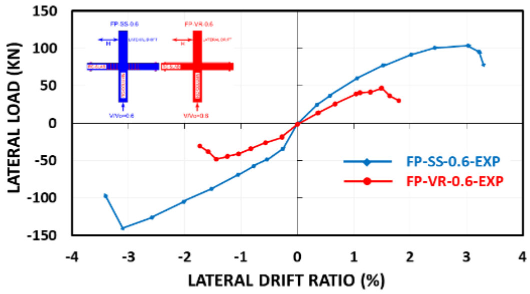
Figure 8: Crack pattern for each specimen with shear reinforcement (shear studs)

5.3. Load-deflection Curves

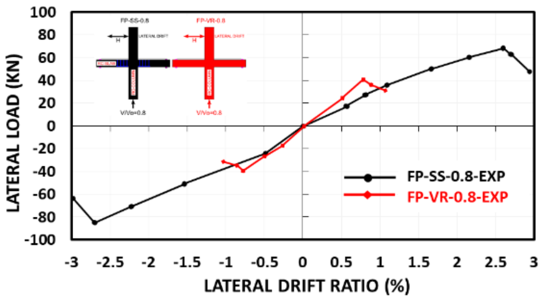
Figure 9 displays the backbone curve of the hysteresis curve for all specimens that were exposed to cyclic loading, with respect to the horizontal lateral load applied at the top column end and its horizontal lateral drift ratio. The nominal punched shear capacity of concrete, Vo= 0.333√fc bo d (ACI 318-14, in metric units), is shown in Table 1, together with the peak load and drift ductility of all specimens that were tested. Specimens with shear studs had a much better ductility capacity than those without, indicating that the studs help enhance ductility behavior and avoid abrupt brittle failure after the ultimate stress.For each specimen that was examined, Figure 10 displays the results of comparing the lateral drift ratio with the lateral load, as well as the influence of the gravity shear ratio level on both metrics. The peak points at the beginning of each set of same-drift cycles are connected to produce the backbone curve. Since the connection drift capacity is a function of the gravity shear ratio, we may deduce that a higher ratio significantly lowers displacement ductility.



(a) FP-VR-0.4 versus FP-SS-0.4



(b) FP-VR-0.6 versus FP-SS-0.6



(b) FP-VR-0.8 versus FP-SS-0.8

Figure 9: backbone curve of the hysteresis curve of the horizontal load versus horizontal driftmeasured for all tested specimens without and with shear studs under cyclic loading

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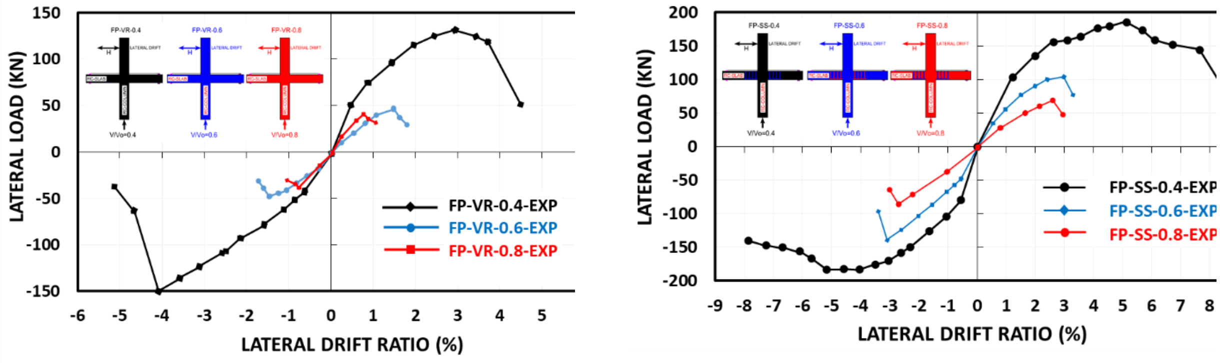

(a) All tested specimens without shear studs (b) All tested specimens with shear studs

Figure 10: comparisons between lateral drift ratio and lateral load,

Table 1: Peak load and drift ductility of all tested specimens under cyclic loading

SPECIMEN	V (KN)	PEAK LATERAL LOAD (KN)		HORIZONTAL DRIFT RATIO AT PEAK LATERAL LOAD (%)		YIELD DRIFT RATIO (%)		DRIFT DUCTILITY AT PEAK LATERAL LOAD μ Peak		DRIFT DUCTILITY AT 95% post PEAK LATERAL LOAD μ 0.95		DRIFTDUCTILITY AT 80% post PEAK LATERALLOAD μ 0.8	
		-	+	-	+	-	+	-	+	-	+	-	+
FP-VR-0.4	230	148	132	4	3.05	3.4	1.9	1.18	1.61	1.21	1.79	1.24	2.08
FP-VR-0.6	295	47.5	46	1.45	1.5	1.16	1.25	1.25	1.2	1.284	1.216	1.371	1.328
FP-VR-0.8	393	38	41	0.78	0.78	0.73	0.66	1.068	1.182	1.137	1.227	1.425	1.561
FP-SS-0.4	195	187	185	5.15	5.13	2.35	2.6	2.191	1.973	2.277	2.154	2.864	2.769
FP-SS-0.6	295	139	105	3.08	3	2.57	1.98	1.198	1.515	1.266	1.561	1.288	1.636
FP-SS-0.8	393	85	68.5	2.7	2.6	2.5	2.2	1.08	1.182	1.1	1.205	1.18	1.286

5.4. Comparison between shear stress at failure from experimental results and theoretical calculations from international codes such as ACI318-14 and EC2-2004

Figure 11 shows the critical punching shear perimeter according to ACI code and Euro code. The total shear stress ν due to direct shear V and the moment M is calculated according to Eqs. (1) ACI-318 and (2) EC2 [17]. The nominal punching shear stress resistance ν_c for slab – column connections without shear studs according to ACI 318-14 Eq (3) and EC2-2004.

Eq (4). The nominal punching shear stress resistance ν_R for slab–column connections with shear studs according to ACI 318-14 Eq (5) and EC2-2004 Eq (6).

$$\nu = \frac{V}{} + \frac{\gamma_v M e}{} \quad \text{Eq (1)}$$

$$\nu = \frac{V}{}\beta \quad \text{Eq (2)}$$

$$\beta = \left(1 + \frac{\gamma M u_1}{V_f W_1}\right)$$

(a) Total shear stress due to direct shear and Moment according to ACI318-14

(b) Total shear stress due to direct shear and Moment according to EC2-2004

where γ_v is the fraction of M transferred by shear, e is the distance from the centroid of the critical section to the point where shear stress is calculated, J is the analogous polar moment of inertia of the shear critical section, β is the increase in shear stresses due to the unbalanced moment for one direction moment, b_o and u_1 are the perimeters of the critical shear section, γ is a coefficient based on the ratio between the column dimension for a square column γ = 0.6) and W_1 corresponds to a distribution of shear and is a function of the basic perimeter. For a rectangular column W_1 is given as:

$$W_1 = \frac{c_1^2}{2} + c_1 c_2 + 4c_2 d + 16d^2 + 2\pi d c_1$$

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Where, C_1 is the column dimension parallel to the eccentricity of the load and C_2 is the dimension of the column that is perpendicular to the eccentricity of the load.

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$$v_c = 0.333\sqrt{fc} \qquad \text{Eq (3)}$$

$$v_c = 0.18k(100\rho f_{ck})^{\frac{1}{3}} \qquad \text{Eq (4)}$$

(c) Nominal punching shear capacity without Shear studs according to ACI318-14

(d) Nominal punching shear capacity without Shear studs according to EC2-2004

$$v_R = 0.5Vc + \frac{Astuds*fys}{bo*s} \leq 0.66\sqrt{fc} \qquad \text{Eq (5)}$$

$$v_R = 0.75Vc + 1.5\frac{Astuds*fywd,ef}{bo*s}$$
$$K = 1 + (200/d)^{\frac{1}{2}} \qquad \text{Eq (6)}$$
$$fywd,ef = (250 + 0.25d) \leq fys$$

(e) Nominal punching shear capacity with Shear studs according to ACI318-14

(f) Nominal punching shear capacity with Shear studs according to EC2-2004

Where, A_{studs} is the area of one row of the vertical shear reinforcement, s is the spacing between the perimeters of shear studs, f_{ys} is the yield strength of the shear studs. and $f_{ywd,}$ is the effective design strength of the shear studs

(a) According to ACI318-14

(b) According to EC2-2004

Figure 11: Punching shear perimeter according to ACI code and Euro code

Table 2 compares the experimental results with the code provisions. It can be concluded that for the specimen without applied moment the codes accurately predict the shear resistance. However, for the specimens with moment transfer the codes (especially EC2) overestimates the resistance of the tested specimens.

Table 2: Failure shear stress (N/mm^2) according to **ACI318-14** and **EC2-2004**

Specimen	f'_c **(N/mm^2)**	**ACI318-14**			**EC2-2004**		
		v_r **(N/mm^2)**	v_{test} **(N/mm^2)**	**Ratio** v_{test}/v_r	v_r **(N/mm^2)**	v_{test} **(N/mm^2)**	**Ratio** v_{test}/v_r
FP-GR-CTRL	**27**	**1.732**	**2.26**	**1.305**	**1.268**	**1.203**	**0.949**
FP-VR-0.4	**35**	**1.972**	**1.801**	**0.913**	**1.383**	**0.788**	**0.570**
FP-VR-0.6	**25**	**1.667**	**1.317**	**0.790**	**1.236**	**0.648**	**0.524**
FP-VR-0.8	**25**	**1.667**	**1.608**	**0.965**	**1.236**	**0.809**	**0.655**
FP-SS-0.4	**25**	**3.334**	**1.952**	**0.585**	**2.397**	**0.823**	**0.344**
FP-SS-0.6	**25**	**3.334**	**2.035**	**0.610**	**2.397**	**0.911**	**0.380**
FP-SS-0.8	**25**	**3.334**	**1.918**	**0.575**	**2.397**	**0.923**	**0.385**

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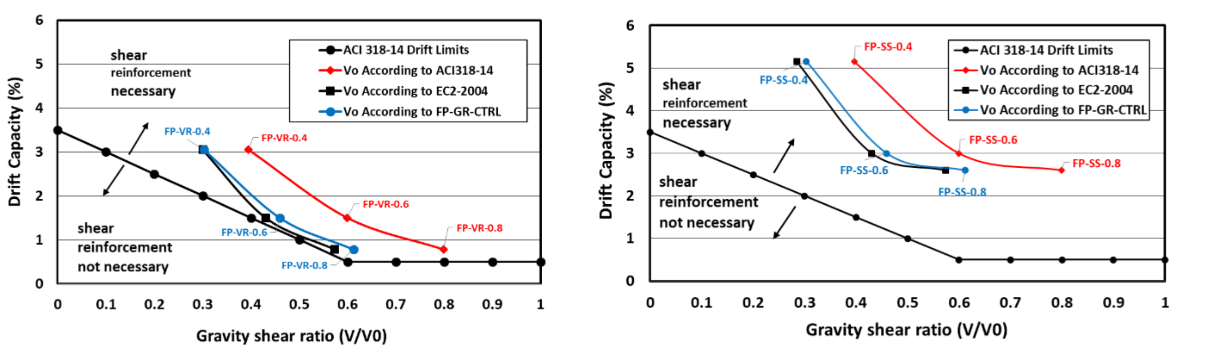
Equation (Eq.7) of ACI318-14 determines the zone in which shear reinforcement is required as a relation between the ultimate design drift ratio DR_u and the gravity shear ratio V/V_0.

$DR_u = 0.035 - 0.05*(V/V_o)$ for $(V/Vo \leq 0.6)$, $DR_u = 0.005$ for $(V/V_o \geq 0.6)$ Eq (7)

Fig. 12 includes drift results of all tested specimens as a function of V/Vo as well as Eq. 7. The gravity shear resistance V_o was calculated in three different ways: according to ACI 314, EC 2 and from the control test FP-GR-CTRL. The results indicate that the drift ratio was achieved without shear reinforcement in contrary to the ACI requirements. Equation 7 of ACI 318 seems to be in a better match with the shear stresses calculated according to EC 2.

Experimental Investigation on The Punching Shear Behaviour of RC Slab-Column Connections Containing Shear Studs Under Seismic Loading



(a) All specimens without shear studs (b) All specimens with shear studs

Figure 12: Maximum drift ratio DR_u achieved for interior slab-column connections tested without and with shear reinforcement (shear studs)

6. CONCLUSION

7. The impact of shear studs on the punched shear response of inner slab-column connections subjected to seismic loads is experimentally investigated and reported in this work. Here are the primary findings.
1-Diminishing displacement ductility is a direct result of an increase in the gravity shear ratio, which in turn decreases the connection drift capacity.
2-Experimental findings shown that lateral cyclic loading, in addition to vertical stress, reduces the punching shear resistance of slab-column connections.
3-For reference control specimens exclusively exposed to gravity shear, the code provisions of ACI318-14 and EC2-2004 properly anticipated the failure loads. However, for all other specimens subjected to reversed cyclic moments, the failure loads were underestimated.
4- According to cyclic moment tests, the expected drift ratio may be reached without shear reinforcement, which suggests that ACI 318-14 is conservative in this area.
5-For connections that are exposed to both gravity shear and cyclic moments, the test findings corroborated the trend of the ACI equation that relates the drift capacity to the amount of gravity shear. Applying EC 2 to determine the shear stresses improves the concordance between the ACI equation

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and the experimental findings.
6-The flat slab column connectors may have their failure modes changed by shear studs. Shear studs correctly installed in slabs prevent sudden punched shear failure and provide optimum flexural behavior.
7-The energy dissipation capacity is significantly enhanced in specimens with shear studs, as shown by their ability to tolerate more lateral drift cycles at high deformation.

8-Specimens with shear studs had a much better ductility capacity than those without, indicating that the studs help enhance ductility behavior and avoid brittle failure beyond the ultimate stress.

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