ENGINEERING RESEARCH JOURNAL (ERJ)
Vol. 51, No. 4 October. 2022, pp.8-18
Journal Homepage: erj.bu.edu.eg

PROPOSED CORRECTION FACTORS for PUNCHING SHEAR CAPACITY of RC FLAT SLABS with SHEAR REINFORCEMENT

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Abstract: Punching shear is the main problem facing reinforced concrete (RC) flat slabs. There are many studies that investigate the punching shear of flat slabs in the normal cases, while there are very few studies that investigate circular flat slabs supported on circular columns. This paper investigates the effect of: (1) steel shear studs; (2) steel stirrups; (3) main reinforcement details and (4) shear reinforcement arrangement on the punching shear resistance of circular RC flat slabs. An experimental program was conducted by the authors and the obtained results are compared with the numerical results using ANSYS V. 15, as well as the analytical results using the American and the Egyptian codes of standards. The obtained results show that punching shear strength increased when using steel shear reinforcement (studs and stirrups) by 9% to 57%.Toughness and displacement ductility improved by 31% to 333% and 85 to 121%, respectively. Both codes overestimate the ultimate failure load for the range of the studied variables in this research and not in general.

KEYWORDS: **:** Punching shear; Flat slabs; Circular slabs; Circular columns; Shear studs; Stirrups; Shear reinforcement, Numerical modeling, Correction factors.

1. INTRODUCTION

Flat slab systems are commonly used in construction projects such as multi-story buildings, bridges, car parks, halls and circular water tanks because it saves time, money and gives flexibility for architectural design. Punching causes brittle failure of flat slabs and causes disasters, so punching is a great problem for flat slabs. There are many factors that affect punching behavior such as slab thickness, column dimensions, column shape and the slab shear reinforcement. Previous researches and codes provisions concluded that punching shear resistance of circular flat slabs increased by using drop panel, using column head, increasing column dimensions and use of steel shear reinforcement.

Cheng et al. [1]used Concrete-Filled Steel Tube (CFST) and steel plate to improve punching resistance of slabs according to ACI 318-19 [2].Using steel plate increased the value (Vpeak/$f_c^{0.5}$) more than CFST column by 10-25%.Augustinet al. [3] concluded that the best value for Vtest/$V_{R.C}$ ratio equal one to resistance punching shear according to Eurocode 2 [4]. Issaet al. [5] concluded that ultimate load increased by 10% due to increasing compressive reinforcement ratio per Egyptian code ECP 203-2020 [6].Yousef et al. [7] concluded that the punching shear strength calculated using equations of JSCE-2010 [8] for High Strength Concrete (HSC) specimens were unsafe and for Normal Strength Concrete (NSC) specimens were safe.Ferreiraet al. [9] tested flat slabs with unbalanced moments and concluded that Eurocode 2 [4] presented the better results while ACI318-19 [2] and FIB Model Code 2010[10] showed over conservative results.

El-Kashifet al. [11] used Carbon Fiber Reinforced Polymer (CFRP) and concluded that experimental results showed remarkable agreement with ECP 203-2020 code [6], ACI-318-19 code [2] and JSCE-2010code [8]. Said et al. [12] used Glass Fiber Reinforced Polymer (GFRP), bolts and steel bars. The predictions of ACI 318-19code [2] were underestimated while Euro code 2[4] showed good agreement with experimental results. Salamaet al.[13]studied punching-shear equation of EN1992–1-1–05 [14] using the tensile properties of FRP bars instead of steel. Abbood and AL-Bayati [15] used Steel Fiber Reinforced Concrete (SFRC) and concluded that the calculations of punching shear were underestimated for Euro code 2 [4],ACI-318-19[2] and BS8110 [16]codes because these codes did not consider the strength contribution of steel fibers.

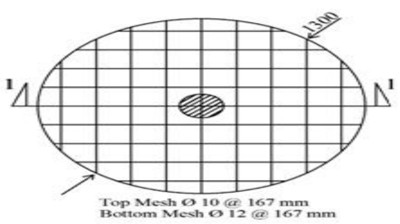
Ferreiraand Filho [17] used prefabricated truss bars and concluded that the predictions of ACI 318-19 [2], Eurocode 2 [4], and FIB Model Code 2010[10] were safe and underestimated. Tareshet al. [18] used steel angle plates and concluded that the results of ACI-318-19 [2] were underestimated and safe.Schmidtet al. [19] used stirrups and concluded that Eurocode 2 [4] depended on concrete contribution and shear reinforcement but FIB Model Code 2010 [10] depended on concrete and steel contribution together.Deifalla [20] used GFRP,CFRP and concluded that calculations of CSA [21] design code were more accurate than JSCE-2010 [8], ACI318-19 [2] codes and safety factor of JSCE-2010 code [8] was safer than CSA[21] and ACI318-19 [2] codes. Shataratand Salman [22] used rectangular and circular spiral stirrups and concluded that Eurocode 2 [4] and ACI 318-19 code [2] were conservative but the predictions of Eurocode 2 [4] were more estimated than ACI 318-19 code [2].

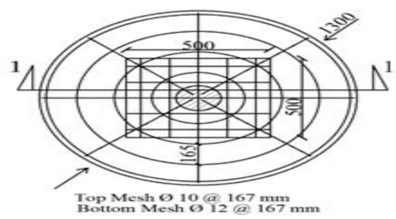
This research investigates numerically the modern techniques to overcome punching failure and increase punching resistance of circular RC slabs that have been tested experimentally by the authors [23]. Furthermore, the research studies predictions of punching strength calculated from different building codes and compares the results with the experimental and numerical results.

2. Experimental program for Circular Flat Slabs and Results

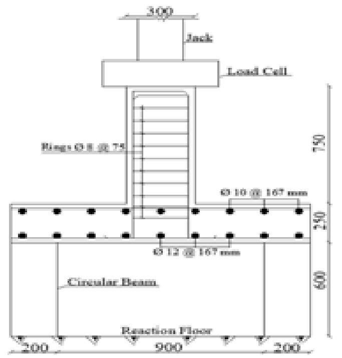
An experimental program was conducted by the authors [23] where seven circular specimens were cast and tested having thickness 250mm and diameter 1300mm, with central circular column 750mm height and with 300mm diameter. Specimen S1 with orthogonal main reinforcement mesh6□12/m and orthogonal secondary reinforcement mesh6□10/m. The other six specimens have radial and tangential main reinforcement mesh 6□12/m and secondary reinforcement mesh6□10/m and specimen S2 considered as a control specimen for this reinforcement details. Figure 1 presents concrete dimensions and details of the steel reinforcement for all tested specimens while Fig. 2 presents details of the internal strengthening technique and arrangement of shear reinforcement for all tested specimens.

The average cubic and cylindrical concrete compressive strength were 33 and 28 MPa, respectively. Average concrete tensile strength was 4 MPa and the Elastic modulus (E_c) was 17GPa.

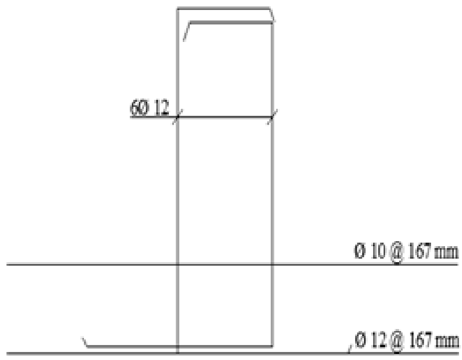




(a) RFT of specimen S1

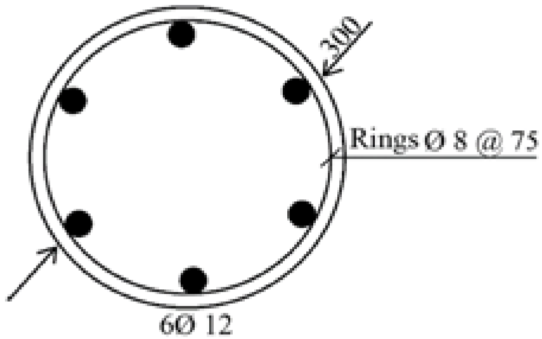


(b) RFT of specimens S2to S7



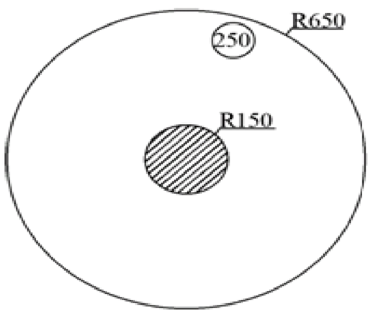
(c) Section 1-1

(d) RFT column details

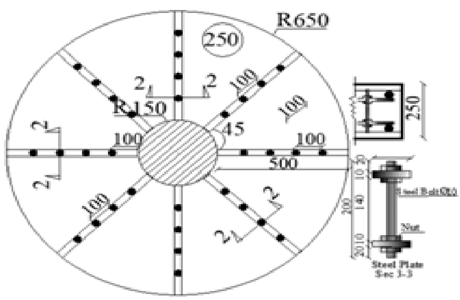


(e) Column cross section

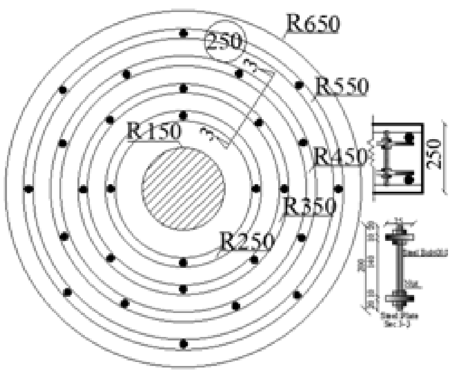
Fig. 1. Concrete dimensions and reinforcement details of tested specimens



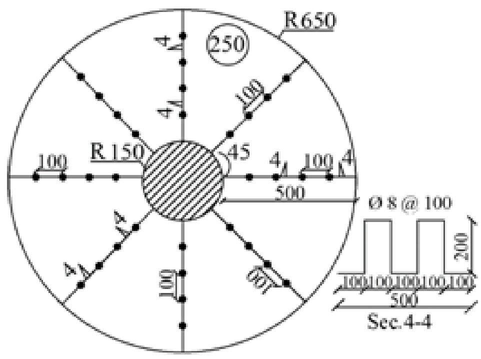
(a) Specimens S1&S2
Without shear reinfocement.



(b) Specimen S3
Steel shear studs arranged radially.

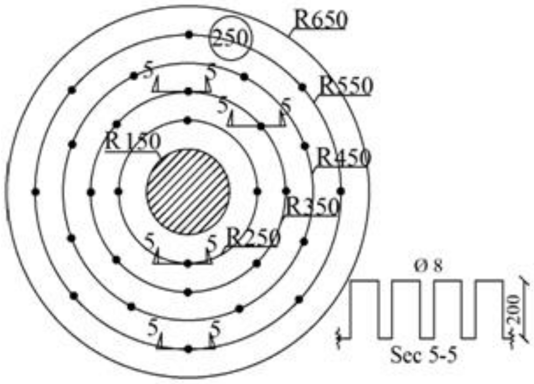


(c) Specimen S4

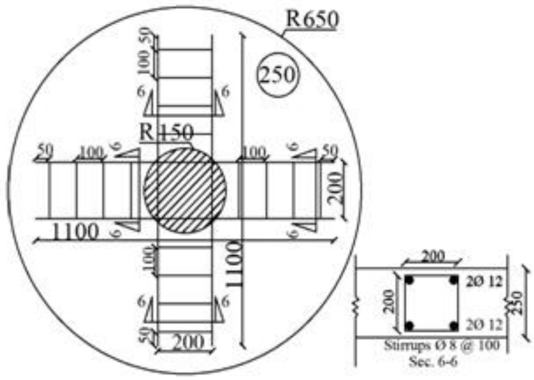


(d) Specimen S5
Single leg steel stirrups arranged radially.

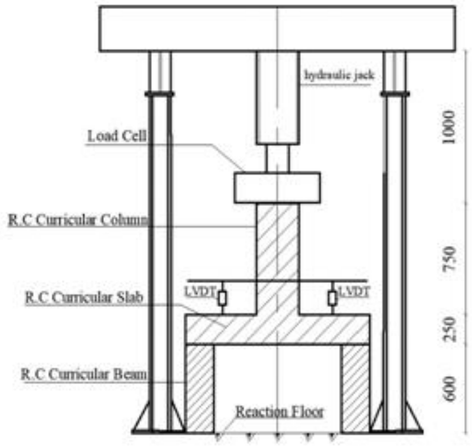
Steel shear studs arranged on rings.



(e) Specimen S6
Single leg steel stirrups arranged on rings.



(f) Specimen S7
Orthogonal closed steel stirrups.



(g) Sketch of test set-up and instrumentations.

Fig. 2. Details of the used internal strengthening, arrangement of shear reinforcement and of test set-up and instrumentations.

The yield and the ultimate strength of the used steel reinforcement bars of diameter 8,10,12 mm is 337,540,565 and 458,695,705 MPa and the Young's modulus, E_s is 198,216,217 GPa,respectively.Table 1 shows the experimental results for all tested specimens compared to the control specimen (S2).

Table 1Experimental results compared to the control specimen S2.

SpecimenNo	$P_{cr}/P_{cr(S2)}$%	$\Delta_{cr}/\Delta_{cr(S2)}$%	$P_u/P_{u(S2)}$%	$\Delta_u/\Delta_{u(S2)}$%	$S.S/S.S_{(S2)}$%	$D.D/D.D_{(S2)}$%	$T/T_{(S2)}$%
S1	208	217	125	166	76	97	190
S2	100	100	100	100	100	100	100
S3	188	121	143	209	68	121	433
S4	145	177	152	214	71	108	334
S5	162	105	109	111	98	101	131
S6	179	110	114	117	97	180	184
S7	194	247	157	223	70	103	300

For specimen S_2 P_{cr}=196.6 kN, Δ_{cr} = 5.50 mm, P_u = 468.3 kN, Δ_u=9.6 mm, S.S= 48.78 kN/mm, D.D=1.22, T=2066 kN.mm

All specimens have similar cracking at terns in the tangential and radial directions. Tangential and radial cracks increased by increasing the applied loads. The test results showed that the first crack load (P_{cr1}) increased by using internal shear reinforcement. The first cracking load $P_{cr1}$is increased by 108%, 88%, 45%, 62%, 79% and 94%, respectively for specimens S1, S3, S4, S5, S6 and S7 compared to the control specimen S2. Specimen S1 has the higher first cracking load while specimen S4 has the smallest cracking load compared to the control specimen S2. Specimen S7 has the best value of the first cracking load compared to all specimens which reinforced with radial and tangential main and secondary reinforcement. Specimen S3 which strengthened with radial studs has first cracking load more than specimens S5 and S6 which strengthened with radial and ring shear stirrups.

Specimens S1 and S2 are failed in punching shear failure while specimens S3, S4, S5, S6 and S7 are failed in punching-flexural failure which took more time to occur. The test results show that the ultimate load (P_u) is increased by using internal strengthening techniques. The ultimate load is increased by 25%, 43%, 52%, 9%, 14% and 57%, respectively for specimens S1, S3, S4, S5, S6 and S7 compared to the control specimen S2. Specimen S7 failed at 57% of the ultimate failure load compared to the control specimen S2.

The use of radial and ring steel shear studs (bolts) (specimens S3 and S4) is better than using radial and ring steel shear stirrups (specimens S5 and S6), where the increase in the ultimate load for radial and ring steel shear studs is 43% and 52%, respectively compared to the control specimen S2, while these values are 9% and 14%, respectively for radial and ring steel shear stirrups. The use of orthogonal arrangement of closed steel stirrups (specimen S7) improved the ultimate load by 9%, 3%, 30% and 28%, respectively more than radial, ring steel shear studs and radial steel shear stirrups.

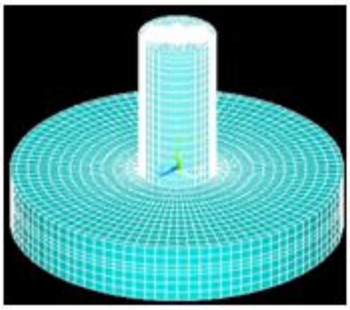
The test results show that the secant stiffness (S.S) for all tested specimens is less than that of the control specimen S2 which means that more ductile behavior can be achieved due to the use shear reinforcement as internal strengthening as shown in Table 1. The displacement-ductility (D.D) increased by 21%, 8% and 80%, respectively for specimens S3, S4 and S6 compared to the control specimen S2 while for specimens S1, S5, and S7 the effect of use shear reinforcement is insignificant. The results show that the toughness (T) is increased due to use of shear reinforcement act as internal strengthening. The toughness is increased by 90%, 333%, 234%, 31%, 84% and 200%, respectively for specimens S1, S3, S4, S5, S6 and S7 compared to the control specimen S2 as shown in Table 1.

3.Numerical Analysis using ANSYS Program

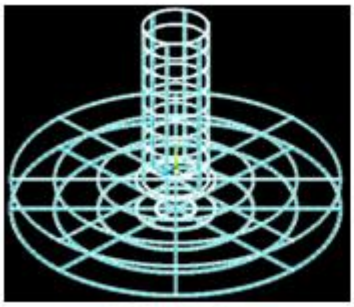
The nonlinear finite elements method is one of the most numerical simulation techniques of structural analysis, which predicts all the results as forces, stresses and deformations with the highest possible accuracy. ANSYSV.15 program [24] can be used to model both concrete and reinforcement through the structural elements and the materials properties. All tested circular slabs were 12odeled using the nonlinear finite element analysis program ANSYS to demonstrate the capability of ANSYS model to represent the circular slabs behavior in punching shear.

3.1Elements, loads and boundary conditions

Solid 65 and link 180 elements were used to present concrete and steel reinforcement. Solid 65 element has the ability of cracking in tension and crushing in compression. Solid65 element was defined by eight nodal points which has three translational degrees of freedom x, y, and z without rotations. Gaussian integration scheme of 2 x 2 x 2 was used for the computation of the element stiffness matrix. Figure 3 shows the 3-D model for concrete and steel reinforcement. All slabs were assumed to be supported on hinged supports along its perimeter. The load is applied as one-point loading at the center of the slab.



(a) Concrete element (Solid 65)



(b) Reinforced bars (Link180)

Fig. 3. ANSYS idealization of all tested slabs.

The bond between steel reinforcement and concrete is assumed to be perfect bond.Open and closed shear confections are taken as 0.2 and 0.80. Through the nonlinear behavior, concrete is assumed to be plastic and cracking is occurred in three perpendicular directions. Displacement increments were used until reached the ultimate failure load.

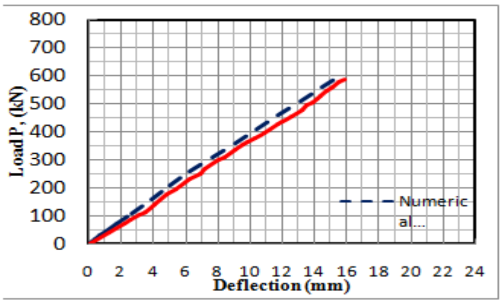
The numerical results were compared with the experimental results to verify the accuracy of the numerical models and check the compatibility of the results with ACI code 318-19 [2] and ECP 203-2020 code [6].

3.2Analysis of the numerical results

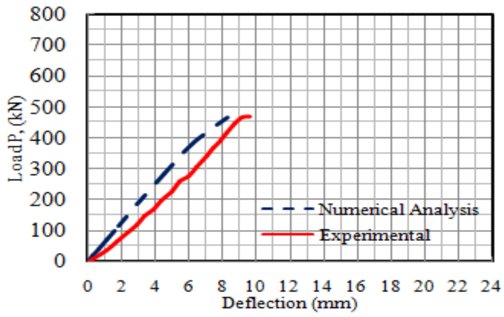
Table 2showsthe numerical results from the finite element and the comparison with the experimental results. The comparison shows that the average and the standard deviation for the ratio ($P_{u\ Nm}/P_{uexp.}$) and($D_{u\ Num}/D_{uexp.}$) are 1.02,0.93 and 0.01, 0.12, respectively. Figure4 shows the experimental and numerical load deflection curves and Fig. 5 shows sample of experimental and numerical crack pattern.

Table2:Comparison of the numerical and the experimental results.

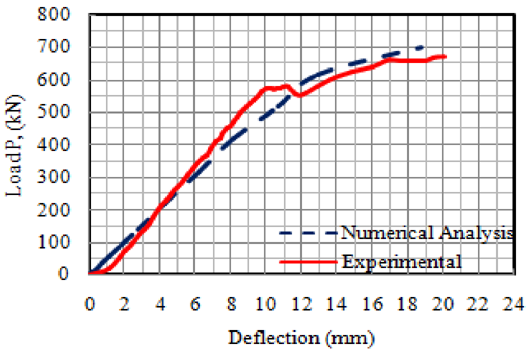
Specimen No.	Numerical		Comparison	
	$P_{u\ Num}$ (kN)	$\Delta_{u\ Num}$ (mm)	$P_{u\ Num}/P_{u\ exp.}$	$\Delta_{u\ Num}/\Delta_{u\ exp}$
S1	600	15.66	1.02	0.98
S2 (control)	480	8.66	1.02	0.90
S3	701	18.75	1.05	0.93
S4	720	22.35	1.01	1.09
S5	520	8.07	1.01	0.75
S6	540	8.46	1.01	0.76
S7	740	22.71	1.01	1.06
Average	-	-	1.02	0.93
Standard Deviation	-	-	0.01	0.12



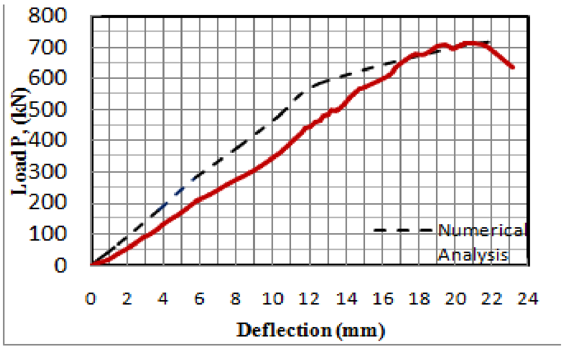
(a) Specimen S1



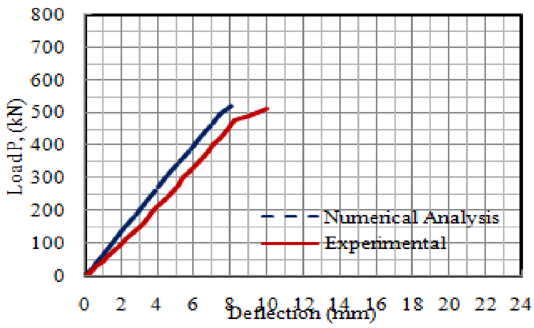
(b) Specimen S2



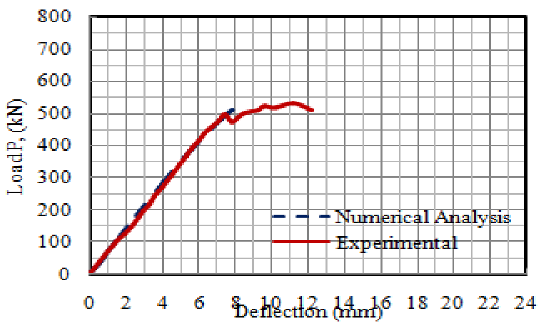
(c) Specimen S3



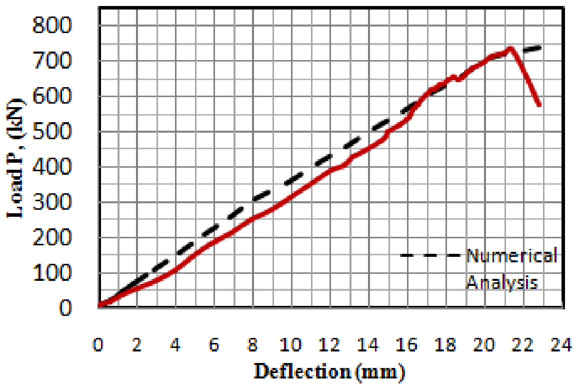
(d) Specimen S4



(e) Specimen S5



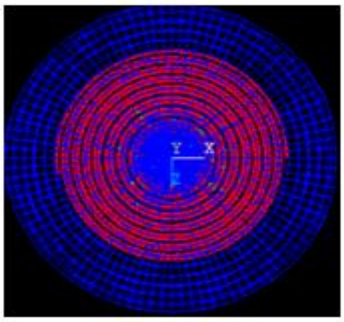
(f) Specimen S6



(g) Specimen S7

Fig. 4.Experimental and numerical load deflection curves.

(a) Experimental crack pattern for S5



(b) Numerical crack pattern for S5

Fig. 5. Sample of experimental and numerical crack pattern.

4.Analytical Results using ECP 203-2020[6] and ACI 318-19 codes [2] and Proposed Correction Factors

Table 3shows the analytical results from ECP 203-2020 code [6] and ACI 318-19 code [2] compared with the experimental results by the authors[23].The comparison shows that the average, standard deviationand variance for the ratio ($P_{uACI}/P_{uexp.}$), ($P_{uECP}/P_{uexp.}$), and (P_{uECP}/P_{uACI}) are 1.62 ,1.18 ,0.76 ,0.37 ,0.21,0.17, and 0.12,0.04,0.03 respectively. The results show that both ECP 203-2020 code [6] and ACI 318-19[2] are overestimated in calculating the ultimate failure load and ECP 203-2020 code [6]has less overestimated values compared to ACI-318-2019 code [2] for the range of the studied variables in this research. Codes comparison indicates a significant variation in the punching shear predictions from code to another. This result has been concluded by the third author Ph.D. [25].

Correction factors were proposed to correct the results from ACI 318-19 code [2] for punching. For orthogonal reinforcement details, the correction factor = 1.0 while for radial and tangential reinforcement details = 0.80. To consider the shear reinforcement type, the correction factors are taken 0.0015, 0.0033 and 0.0050 multiplied by (($F*N*f_y$)/S) for studs, single leg stirrups and closed stirrups respectively where F: is diameter; N: the number of branches; f_y is the yield strength and S: is the spacing. For Egyptian code ECP 203-2020 code [6], the correction factors were used 0.0023, 0.0033 and 0.0050 multiplied by (($F*N*f_y$)/S) for studs, single leg stirrups and closed stirrups respectively. For both codes, the total resistance of concrete and shear reinforcement was taken not exceed 1.35 the concrete resistance for punching without shear reinforcement. Table 4 shows comparison of the experimental and analytical results for ACI 318-2019 [2] and ECP 203-2020 code [6] after correction.

Table 3: Comparison of the experimental and analytical results for ACI 318-2019[2] and ECP 203-2020 code [6] before correction

Specimen No.	Analytical Results		Comparison			Comments
	$P_{u\ ACI}$ (kN)	$P_{u\ ECP}$ (kN)	$P_{u\ ACI}/P_{u\ exp.}$%	$P_{u\ ECP}/P_{u\ exp.}$%	$P_{u\ ECP}/P_{u\ ACI}$%	
S1	668.99	651.3	1.14	1.11	0.97	Both codes are overestimated
S2 (Control)	668.99	651.3	1.43	1.39	0.97	Both codes are overestimated
S3	1383.00	720.7	2.06	1.08	0.52	Both codes are overestimated
S4	1379.72	719.0	1.93	1.01	0.52	Both codes are overestimated

S5	940.44	720.7	1.84	1.41	0.77	Both codes are overestimated
S6	938.21	719.0	1.76	1.35	0.77	ACI code is overestimated
S7	855.53	655.6	1.16	0.89	0.77	ACI code is overestimated
Average	-	-	1.62	1.18	0.76	Both codes are overestimated
Standard Deviation	-	-	0.37	0.21	0.17	
C.O.V.	-	-	0.12	0.04	0.03	

Table 4: Comparison of the experimental and analytical results for ACI 318-2019[2] and ECP 203-2020 code [6] after correction

Specimen No.	Analytical Results		Comparison		
	$P_{u\ ACI}$ (kN)	$P_{u\ ECP}$ (kN)	$P_{u\ ACI}/P_{u\ exp.}$%	$P_{u\ ECP}/P_{u\ exp.}$%	$P_{u\ ECP}/P_{u\ ACI}$%
S1	602.09	651.3	1.03	1.11	1.08
S2 (control)	481.67	521	1.03	1.11	1.08
S3	728.15	720.7	1.09	1.08	0.99
S4	726.42	719	1.02	1.01	0.99
S5	533.23	504.5	1.04	0.98	0.95
S6	531.96	503.3	1.00	0.95	0.95
S7	762.28	721.2	1.04	0.98	0.95
Average	-	-	1..03	1.03	1.00
Standard Deviation	-	-	0.03	0.07	0.06
C.O.V.	-	-	0.03	0.03	0.03

5.Conclusions

For the range of the studied variables in this research the following conclusions can be drawn:

1. The use of closed steel stirrups in orthogonal arrangement is the best method to increase the first crack load, deflection at cracking load, ultimate load, deflection at ultimate load ductility, and toughness by 94%, 147%, 57%, 223%, 3% and 200 %, respectively compared to the specimen without shear reinforcement.
2. With the addition of single leg stirrups as shear reinforcement particularly in case of ring arrangement, the ultimate failure loads were increased. An increase in the ultimate load 52.3% was recorded for specimens with shear reinforcement

compared to slab without shear reinforcement.

3. Nonlinear analysis using ANSYS program can represent the behavior of circular R.C slabs in punching to a good extent, where the average and standard deviation of the predicted to the measured cracking load, ultimate failure load and deflection at ultimate load are 1.07,1.02,0.93 and 0.28,0.01,0.12, respectively.
4. To the range of the investigated parameters, the application of nonlinear finite element analysis using ANSYS 15.0 package yielded satisfactory load-deflection responses.
5. Both ECP 203-2020 code [6] ACI 318-2019 code [2] overestimate the ultimate failure load where ECP 203-2020 code [6] gives less overestimate results compared to ACI 318-2019 code [2] for the range of the studied variables in this research and not in general.
6. The average analytical-to-experimental ultimate failure loads, standard deviation and variance are 1.18, 0.21 and 0.04, respectively from ECP 203-2020 code [6].The average analytical-to-experimental ultimate failure loads, standard deviation and variance are 1.62, 0.37 and 0.12, respectively from ACI 318-2019 code [2].
7. The proposed correction factors for punching shear capacity of flat slab resulted in more accurate results compared with ECP 203-2020 code [6] and ACI 318-2019 code [2] predictions. The average modified predicted-to-experimental ultimate loadis1.03 with standard deviation and C.O.V of 0.07 and 0.03,respectively.

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