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Punching Shear Behavior of Reinforced Concrete Flat Slabs with Steel Shear heads

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ABSTRACT

Punching shear failure is considered one of the main problems that must be considered during the design of flat slabs especially at the critical slab column connection zone. This paper is part of an ongoing research program conducted at the concrete laboratory of the Faulty of Engineering, Cairo University to assess the effect of different types of shear reinforcement on the punching behavior of reinforced concrete flat slabs. In this paper, shear heads in the form of embedded steel beams were used to improve the performance of slabs against punching failure. An experimental program was carried out using six half scale specimens of flat slabs with dimensions of 1100 mm x 1100 mm and 150 mm thickness and each was provided with a central square concrete column of 150 mm x 150 mm with 400 mm height. The main investigated parameters were the arm length, configuration and dimensions of the steel beams by using two different types: hot rolled and built-up sections. The experimental results showed that using embedded steel beams at the column-slab connection significantly improved the punching shear capacity and ductility of flat slabs. The test results were compared against values obtained using the provisions of the ACI international design code.

1. Introduction

Punching shear failure is one of the governing modes of failure to be considered in reinforced concrete flat slabs. This type of failure occurs suddenly causing a brittle failure. Once the punching stresses exceed the capacity of concrete around the column slab connection, the slab physically separates from the column which in turn disturbs the equilibrium of the structural system. Different ways can be used to improve the punching capacity of RC slabs for example increasing the slab thickness through drop panels or using column heads. A more effective method can be by using shear reinforcement which comes in different forms such as closed stirrups or headed shear studs. Extensive researches were done on the effect of the different types of shear reinforcement on the behavior of RC flat slabs [1-11] and the results showed that using different types of shear reinforcement is effective in resisting the punching stresses.

Another form of shear reinforcement can be found in the use of shear-heads. Shear reinforcement in the form of structural I or channel shaped sections can be placed in between the top and bottom layers of longitudinal reinforcement presenting a viable option in resisting punching failure in flat slabs. Provisions regulating the usage of shear heads as punching shear reinforcement were first introduced in the ACI 318-1971 [12] based on experimental data conducted by Corley and Hawkins 1968. [13] The provisions were slightly revised in 1977 [14] where they were not changed up to ACI 318-14. [15] In addition, shear heads were introduced in the slab-column connection report ACI 352.1R-11 [16] as a possible alternative. Unfortunately, provisions for the design using shear heads cannot be found in the ACI 318-19 [17] as well as other design codes whether the Eurocode 2 [18], the BS8110 [19] or the Egyptian code of practice ECP 203-2018 [20] where the latter only allows the use of closed stirrups to resist punching shear in flat slabs.

Few research is available in the literature that deal with the application of shear heads as punching reinforcement in RC slab-column connections such as Corley and Hawkins [13] and Godycki and Kozicki. [21] This may be mainly due to the concerns of the congestion due to the column's main reinforcing bars and shear heads crossing the column in both the vertical and horizontal directions. However, recently the use of concrete filled tube (CFT) or hollow section columns in tall buildings is becoming increasingly popular. In this case, the congestion

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problem can be overcome and the combined advantages of speedy construction using flat slabs and small column cross section in case of steel columns can be utilized. Kim et al. [22] reported that their proposed CFT column to RC slab connections showed improved punching shear capacity. Later, Lee et al. [23] studied the performance of CFT column- RC slab connections with shear head keys under seismic loads. Piel and Hanswille [24] proposed a simplified design formula for a new type of shear head reinforcement based on test results and finite element analysis. Majeed and Abbas [25] tested eight reinforced concrete flat plate slabs with shear head collars where different parameters were studied. Bompa and Elghazouli [26-30] performed a group of studies on the ultimate behavior of the hybrid RC flat slabs using embedded shear head connected to steel column connections. An experimental program was performed on specimens with central steel column, welded to an embedded steel beam. They took into consideration several parameters such as the length of the shear head, slab effective depth, size of the steel beam used and concrete compressive strength. A numerical program was also performed using the ABAQUS program. The finding of their studies allowed for the development of an analytical model for hybrid slabs.

Other shear head systems were also reported including composite cruciform systems consisting of vertical plates acting as shear-heads and provided with welded studs and fan-shaped systems made of wide tee pieces [26]. Cantone et al. [31] proposed a novel punching shear reinforcing system where the embedded shear heads were replaced by large diameter shear studs arranged horizontally in the compression side of the slab which could result in a more economical solution.

Based on the above, using shear heads as shear reinforcement provides promising results and can effectively improve the punching capacity and ductility of flat slabs. It was reported that they can increase the punching capacity by 70% under static loads [13] and 40-70% under cyclic loading [21]. In addition, using shear heads increases the effective perimeter of the critical section for shear. Thus, the main aim of the current research is to investigate the behavior of RC flat slabs under monotonic loading using fully integrated ACI type shear head as a shear reinforcement system resisting the punching stresses. This shear reinforcement system is represented by using two embedded steel I beam sections crossed and welded to each other where different parameters are investigated through the experimental program such as the arm length and the configuration position of the steel beams, as well as the web thickness using built-up sections. The test results are compared to analytical results obtained from the ACI 318-14 [15] design code.

2. Experimental Program

The experimental program conducted in this research comprised six half scale specimens representing interior slab – column connections. All the specimens were loaded using incremental vertical load up to failure at the Concrete Research Laboratory, Cairo University. The slab dimensions were 1100 mm x 1100 mm with 150 mm thickness. For all specimens a clear cover was provided of 20 mm and the clear span was 1000 mm. The slabs were cast with a central square concrete column of 150 mm x 150 mm with 400 mm height for applying the vertical load as shown in Figure 1.

Figure 1: Loading setup

Ten equally spaced deformed bars each of 10 mm diameter and five 12 mm bars were used as the main flexural reinforcement for the six slabs to avoid premature flexural failure while ten 10 mm diameter bars were used for the compression reinforcement. The yield stress of the longitudinal reinforcement was 360 N/mm^2 according to the manufacturer. Slab S1 was considered as the control specimen which was not reinforced in shear. Shear heads were used in the other specimens as punching shear reinforcement. The shear heads were installed between the top and bottom layers of the longitudinal reinforcement meshes. The steel grade of the shear heads was ST (37) with yield stress fy of 240 N/mm^2 and ultimate strength fu = 360 N/mm^2 which follows the provisions of the Egyptian code of practice for steel construction and bridges ECP 205-2001 [32]. The shear heads were divided into two types; the first one is a hot rolled steel beam type IPE No 80, and the second was a built-up section that consisted of two flanges with plate dimensions (46 x 5.2) mm and web plate with dimensions (69.5 x 7.6) mm. The built-up section was designed to have the same flange dimension as the IPE No 80 section but with twice the web thickness. Both sections; the hot rolled and the built ups; had the same web height of 80 mm. Figure 2 shows the dimensions of the steel beam type IPE No 80 while Tables 1 shows the details of the two types of shear heads used and their physical properties.

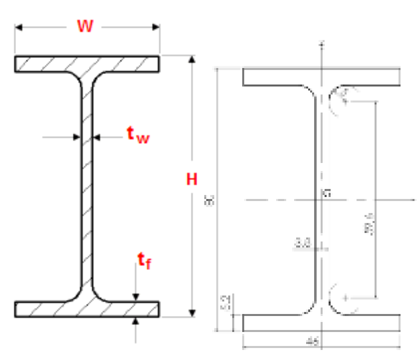


Figure 2: Details and dimensions of steel beam type IPE No 80

Table 1: Properties of the steel sections used in specimens

Beam Data		Hot Rolled (IPE80)	Built-up section
W	**Width of flange (mm)**	**46**	**46**
t_f	**Thickness of flange (mm)**	**5.2**	**5.2**
h_v	**Height of the steel section(mm)**	**80**	**80**
t_w	**Thickness of the web (mm)**	**3.8**	**7.6**
A	**Cross sectional area (mm^2)**	**764**	**1007**
Y bar	**Vertical eccentricity of C.G from the soffit (mm)**	**40**	**40**
I_x	**Moment of Inertia about (x-x) axis (mm^4)**	**801000**	**883775**
I_y	**Moment of Inertia about (y-y) axis (mm^4)**	**84900**	**86904**
S_x	**Section modulus about (x-x) axis (mm^3)**	**20000**	**22094**
S_y	**Section modulus about (y-y) axis (mm^3)**	**3690**	**3778**
r_x	**Radius of gyration about (x-x) axis (mm)**	**32.40**	**26.92**
r_y	**Radius of gyration about (y-y) axis (mm)**	**10.5**	**9.3**

According to the ACI 318-14 [15] provisions, the punching shear strength for slabs without shear reinforcement is taken as the lowest value computed from the three following equations:

$$V_c = 0.17\left(1+\frac{2}{\beta}\right)\sqrt{fc'}b_o d \quad (1)$$

$$V_c = 0.083\left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{fc'}b_o d \quad (2)$$

$$V_c = 0.33\sqrt{fc'}b_o d \quad (3)$$

Where:

β: ratio of the long side to the short side of the load area, α_s: equals 40 for interior columns, 30 for edge columns, 20 for corner columns, f_c': specified compressive strength of concrete, d: effective depth of the slab, b_o: critical shear perimeter

As for slabs reinforced with shear heads, ACI 318-14 [15] states that "the critical section crosses each shear head arm at a distance equal to three-quarters of the arm length measured from the face of the column" and is defined so that the perimeter is a minimum. However, it does not need to be closer than d/2 to the edge of the supporting column. Factored shear head strength against vertical loads cannot be greater than $0.33\phi\sqrt{fc'}$ for the critical perimeter defined in Figures 3b and 3c or $0.58\phi\sqrt{fc'}$ for the critical perimeter defined in Figure 3a, where ϕ is the shear strength reduction factor.

For each arm of the shear head, the plastic moment strength M_p shall satisfy equation 4.

$$M_P = \frac{V_U}{2\phi n}\left[h_V + \alpha_V\left(L_V - \frac{C_1}{2}\right)\right] \quad (4)$$

Where:

V_u: factored shear force at section, h_v: height of the shear head cross section, l_v: minimum required length of shear head arm, n: number of the shear head arms, ϕ: corresponds to tension reduction factors, α_V: is the relative flexural stiffness of the shear head to the concrete, c_1: column dimension in the direction under study.

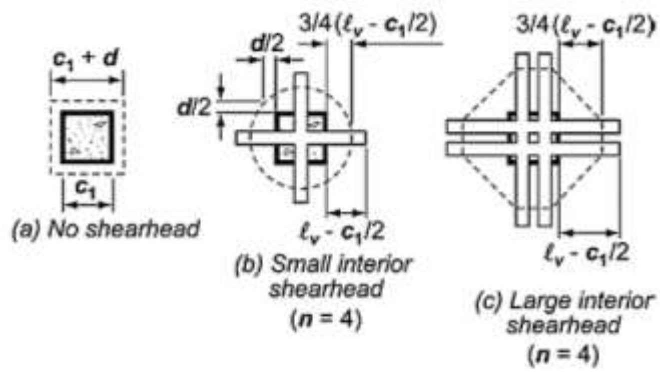


Figure 3: Location of critical section as defined by ACI 318-14 [15]

As a guide for the value of l_v needed, the punching capacity of the tested specimen without shear reinforcement was computed using equations 1 to 3 taking strength reduction factors as 1. This yielded a value of 271.8 kN. Any shear load higher than that value would require adding shear reinforcement. A preliminary design for shear reinforcement was conducted where the required minimum perimeter b_o, of the critical section is calculated using V_u = 271.8 kN thus giving b_o = 1120 mm. At the same time, it is required that the critical section be taken at three-quarters of the distance from the face of the column to the end of the shear head. The critical perimeter can be approximately defined as $b_0 = 4\sqrt{2}\left[\frac{c1}{2}+\frac{3}{4}\left(lv-\frac{c1}{2}\right)\right]$. Solving this equation, the value for l_v can be calculated and this yields the required length of each arm of the shear head needed in this case as approximately 240 mm. In addition, it was reported by Bompa and Elghazouly [26] that the arm length (l_v) to slab radius (R_s) ratios should be taken in the range of 0.2-0.4 as apart from that they appear less effective. Hence, the value of l_v was chosen between 200 to 500 mm thus covering values lower and higher than the above recommendations.

Table 2 and Figures 4 to 6 show the details of the six specimens used in experimental program. The main parameters considered in this research were as follows:

1. Embedded steel beam length which varied from 400 to 1000 mm giving an arm length (l_v) with values of 200 to 500 mm.

2. Embedded steel beam configuration where both orthogonal configurations in the XY plan directions as well as diagonal configuration were used.
3. Embedded steel beam web thickness which can be studied through the comparison between the hot rolled section IPE80 (3.8 mm) and the built-up section (7.6 mm).

The six specimens were cast using ready mix concrete supplied by a local company. The concrete mix was designed with a target cube compressive strength of 40 N/mm^2 after 28 days. Concrete components consist of fine aggregate, coarse aggregate, cement, silica fume, super-plasticizer, and potable water. The coarse aggregate used was crushed dolomite with nominal maximum size of 20 mm with relatively rough surface texture and the fine aggregate used was dune sand. The cement used was ordinary Portland cement. The cement was tested to satisfy the physical properties of the Egyptian specification limits according to 373-1991 requirements. ADDICRET additive was used as super-plasticizer. Six standard cubes with dimensions (150x150x150) mm were cast at the same time as the specimens. Three cubes were tested after 7 days giving an average of 34.4 N/mm^2 while the other three cubes were tested after 28 days giving a compressive strength of 40 N/mm^2. Figure 7 shows the forms and steel arrangements of the specimens tested in this research

Table 2: Properties of the tested specimens

Slab	Mesh Reinforcement		Shear head Details					
	Tension side	Compression side	Type	Total Length (mm)	l_v (mm)	l_v-c_1/2 (x) (mm)	L_v/R_s	Configuration
S1	10Y10/m' +5Y12/m'	5Y10/m'	------------	---	---	---	---	----
S2			IPE No 80	400	200	125	0.2	Orthogonal
S3			IPE No 80	700	350	275	0.35	Orthogonal
S4			IPE No 80	1000	500	425	0.5	Orthogonal
S5			IPE No 80	700	350	244	0.35	Diagonal
S6			Built Up Section	700	350	275	0.35	Orthogonal

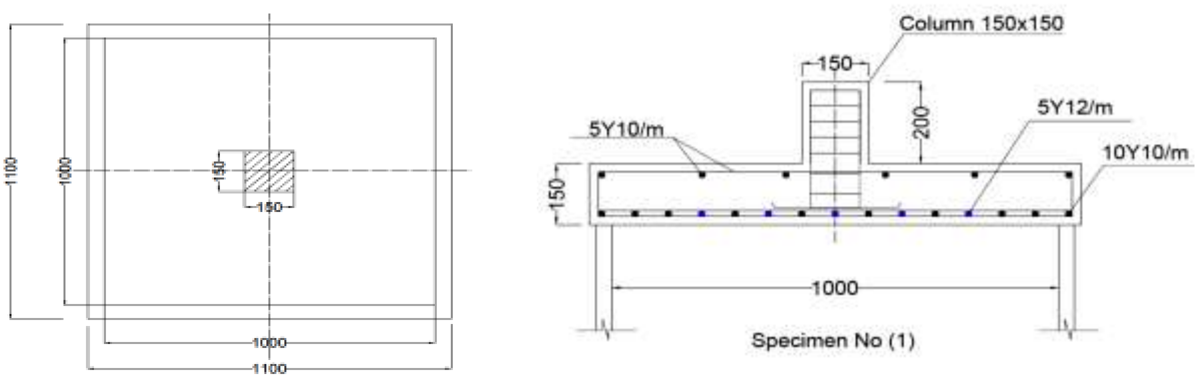


Figure 4: Details and dimensions of the control specimen (S1)

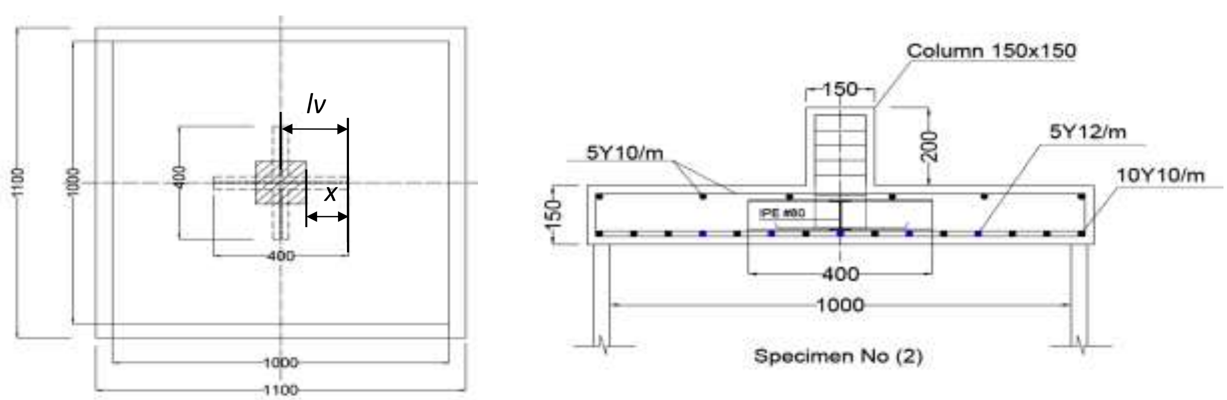
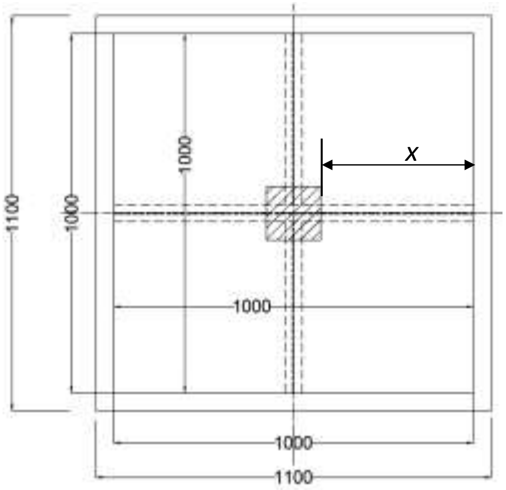
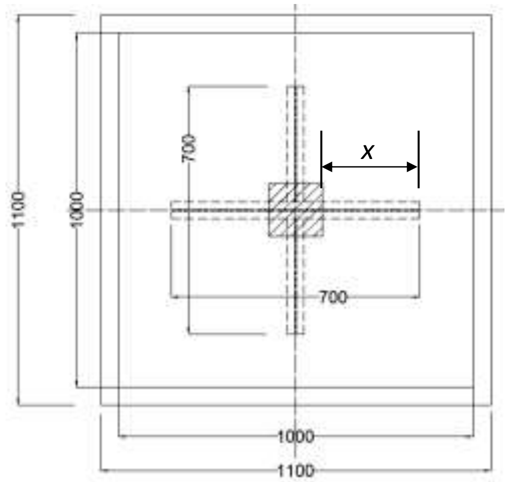


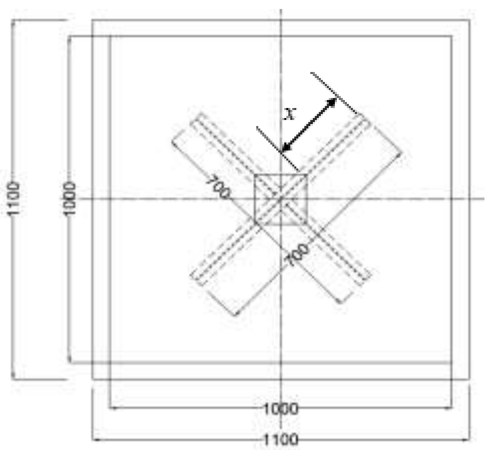
Figure 5: Details and dimensions of specimen (S2)



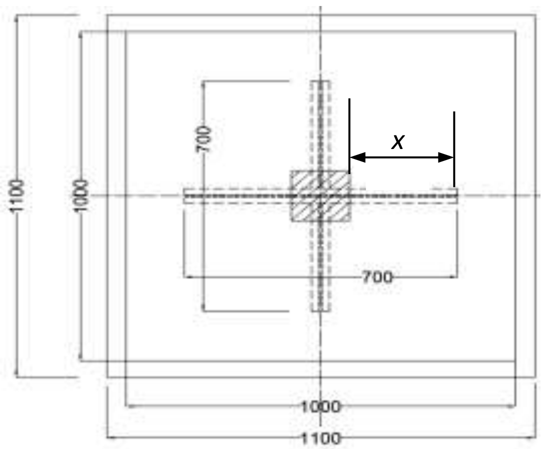
(a) Specimen (S3)



(b) Specimen (S4)



(c) Specimen (S5)



(d) Specimen (S6)

Figure 6: Shear head layout for slabs S3 to S6

(a) Specimen (S1)

(b) Specimen (S2)

(c) Specimen (S3)

(d) Specimen (S4)

(e) Specimen (S5)

(f) Specimen (S6)

Figure 7: Reinforcement details for all specimens

The specimens were loaded upside down as shown in Figure 8. Load was applied through a central jack applying vertical downward loading at the column. The specimens were restrained vertically around the perimeter simulating the inflection point of a flat slab. Deflections were measured using three linear variable differential transducers (LVDTs) located under the load and at the support as well as at mid distance between the loading point and the support as shown in Figure 9. Strain gauges were attached to measure strain in the shear heads at d/2 from the column edge (where d is the depth of the slab = 130 mm) as well as at the maximum loaded point in the flexural reinforcement. To avoid failure of the columns, its top part was confined externally using a steel cap attached at the testing time.

Figure 8: The loading test setup

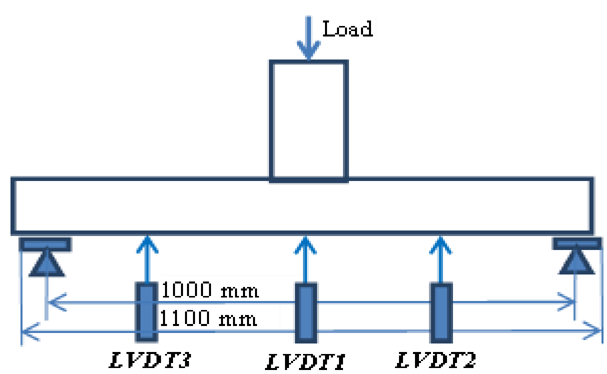
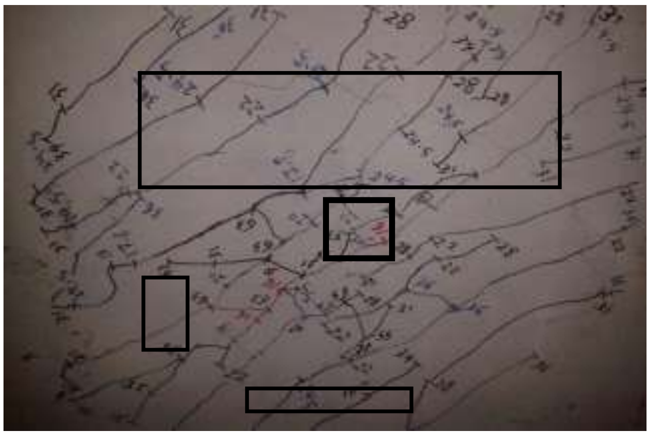


Figure 9: Schematic layout of the test setup

3. Test Results and Discussion

3.1. Cracking patterns and failure modes

Generally, the final failure mode for the test specimens was punching shear failure. This was supported by the crack propagation in the tension side of the slabs at the vicinity of the slab column connections which started around the columns and propagated in a radial manner. Figure 10 shows the cracking pattern for the control specimen S1 at failure. The tension side showed cracks forming in a diagonal pattern while no visible damage was noticed on the compression side except for the separation of the punching cone which started in the tension side and then penetrated the slab to be also seen on the compression side.

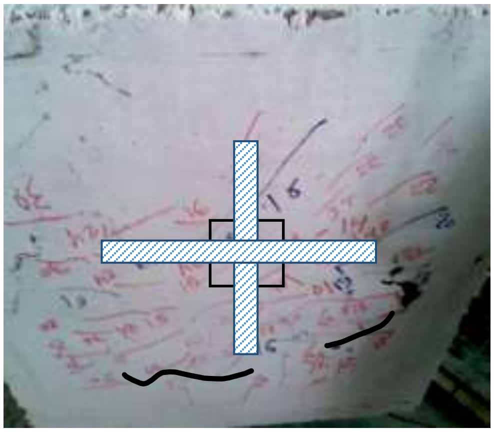


a) Slab S1 - Tension side

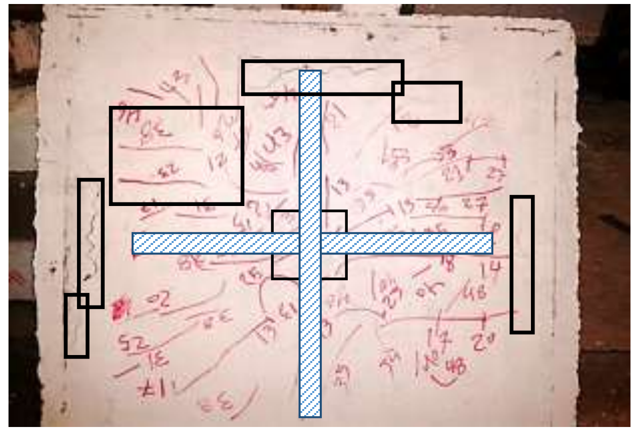
b) Slab S1 - Compression side

Figure 10: Cracking patterns for slab S1

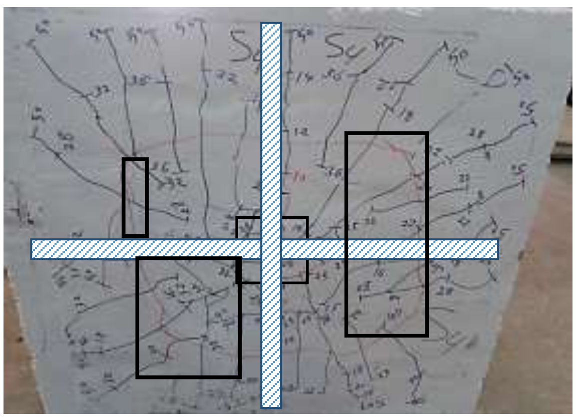
For the rest of the specimens S2 to S6 reinforced using shear heads, the cracking patterns at failure were slightly different for each specimen. The cracks propagated in a radial manner starting from the column towards the edge of the specimens and a circumferential crack indicative of punching failure was observed. However, the extent of the cracking zone and the location of the tangential punching crack were different. Figure 11 shows the crack patterns for slabs S2 to S6. Comparing S2, S3 and S4 with different arm lengths, the radius of the cracking zone increased as the arm length increased. For specimens S2 and S3, the critical punching section was observed at a distance zero to d/2 from the tip of the shear head thus forming a larger outer critical section for the longer arm length. However, for S4 the critical section was seen at 260 mm from the face of the column. This means that the shear head arm length affects the location of the critical section for punching. This was also supported by Bompa and Elghazouly [26] where they noticed that, for short shear heads, the critical section is located further outside the shear head concrete interface relative to the case of long shear heads. The same finding can be seen in the results of specimens S2 to S4. Slab S5 followed a pattern similar to that of S3 while S6 showed a definite larger critical section than the previous specimens with the cracking pattern extending up to the edge of the specimen and the cracking surface occurring almost at the supports.



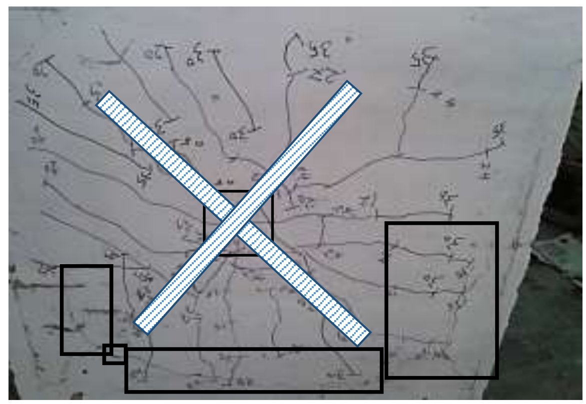
a) Slab S2



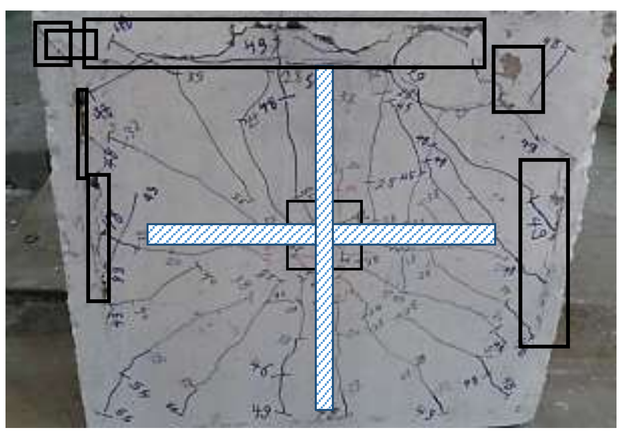
b) Slab S3



c) Slab S4



d) Slab S5



e) Slab S6

Figure 11: Crack patterns for slabs S3 to S6

3.2. *Load deflection curves and punching shear capacity*

The measured experimental data of all the specimens are listed in Table 3. The energy absorption shown is based on the displacement and was calculated using the area under the load deflection curves.

Table 3: Experimental results for the tested specimens

	First Crack Load (kN)	Deflection at First Crack Load (mm)	Ultimate Load (kN)	Deflection at Ultimate Load (mm)	Energy Absorption (mm^2)	Max RFT Strain (μm/m)	Strain in the embedded beam (μm /m)
S1	**80**	**1.02**	**313**	**6.55**	**394037**	**1736**	**--**
S2	**100**	**1.52**	**425**	**7.86**	**677744**	**2344**	**2575**
S3	**60**	**1.95**	**485**	**8.21**	**689565**	**2990**	**1290**
S4	**100**	**2.60**	**521**	**9.89**	**744730**	**2960**	**592**
S5	**90**	**2.25**	**445**	**8.38**	**594996**	**2061**	**866**
S6	**90**	**2.48**	**490**	**8.47**	**512248**	**3038**	**592**

Figures 12 to 14 show the load–deflection curves for all the specimens. The load deflection curves initially followed a linear pattern with a slope decreasing as cracks started to form until the peak load is reached.

Comparing specimens S1, S2, S3 and S4, increasing the value of the arm length l_v causes a significant improvement in the ultimate punching strength of the tested specimens. The ultimate load for specimens S2, S3 and S4 increased by 36%, 55% and 66%, respectively relative to the control specimen S1 and the maximum measured deflection of specimens S2, S3 and S4 increased by 20%, 25% and 51%, respectively. Energy absorption also significantly increased where the increase for specimens S2, S3 and S4 was 72%, 75% and 89% respectively compared with specimen S1 at its failure load.

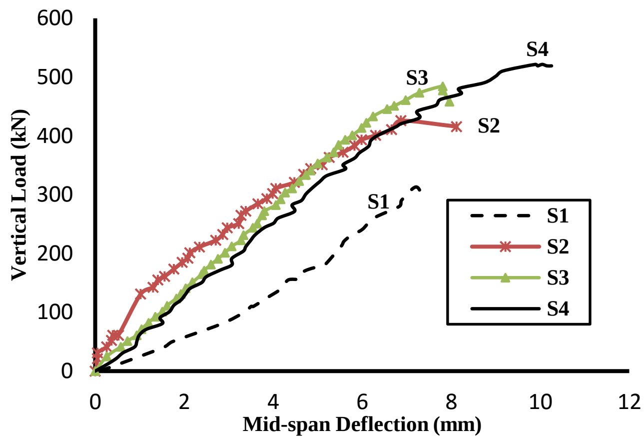


Figure 12: Load deflection curves for slabs S1 to S4

This enhancement in the punching behavior is mainly due to the higher contact surface between the shear-heads and surrounding concrete which improves the concrete integrity with the shear heads as has been reported by previous researchers [22] and [27]. However, the rate of increase in the ultimate load is not constant with different arm lengths as shown in Figure 15. The rate slightly declines with the increase of the ratio of l_v/R_s.

Changing the configuration position of the embedded steel beam into diagonal layout in S5 caused a reduction in the ultimate shear capacity by 11% compared to S3 with orthogonal layout. While the ductility was slightly improved, and the energy absorption increased compared to the control specimen S1. The decrease in the ultimate shear strength can be due to the shorter clear embedded arm length measured from the corner of the column which is indicated using the symbol x in Figure 5. The value of x was 244 mm in case of S5 compared to 275 mm in case of S3.

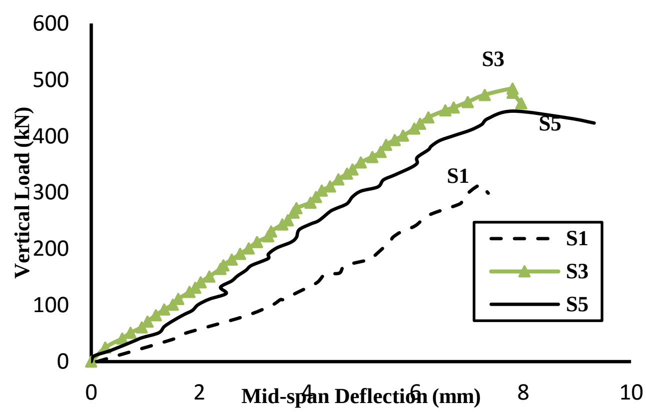


Figure 13: Load deflection curves for slabs S1, S3 and S5

Increasing the shear head web thickness in slab S6 compared to slab S3 slightly increased the ultimate load as well as the maximum deflection as shown in Figure 14. In addition, the increase in the energy absorption was reduced from S3 to S6 by about 25.7%. Hence, increasing the steel beam web thickness slightly improved the specimen ductility. However, more research efforts need to be performed to study the effect of the web thickness on the punching capacity of flat slabs.

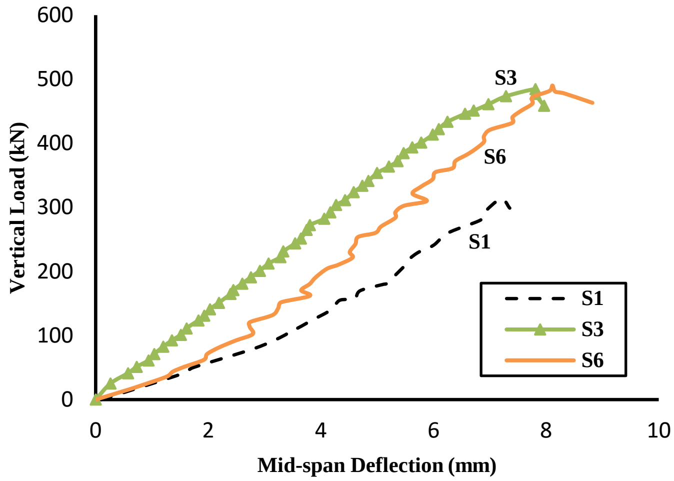


Figure 14: Load deflection curves for tested slabs S1, S3 and S6

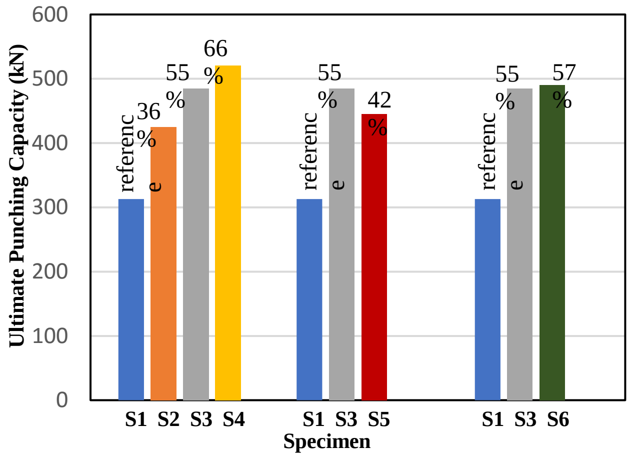


Figure 15: Ultimate punching capacity for the tested specimens

3.3. Steel strains in flexural reinforcement and embedded beams

Figure 16 and Table 3 show the steel strain in the longitudinal flexural reinforcement. For specimens S1, S2, and S3 the flexure reinforcement did not reach the yield strain thus confirming the punching failure as mentioned before. Specimen S4 and S5 showed yielding in the longitudinal reinforcement but the final failure for the two specimens occurred by punching. This is confirmed by the major circumferential crack which occurred before the flexural longitudinal reinforcement reached full flexural failur. Specimen S6 exhibited a ductile behavior for the flexural reinforcement as shown by the plateau in Figure 16. However, failure eventually occurred due to punching cracks which propagated near the edge of the slab and can thus be considered as a combined ‘flexural punching’ mode.

Strain in the embedded shear heads was measured at a distance d/2 from the column face and the results are shown in Table 3 and Figure 17. The beam in slab S2 exceeded the yield strain while in S3 the shear head almost reached the yield strain. For slab S4 the yield strain was not reached. This is due to the short arm length in slab S2 which caused higher stresses in the shear head followed by S3 and finally S4. For slab S5 the strain values did not reach yield strain. Beams in slabs S4 with the longer arm length and slab S6 with the thicker web showed the lowest strain value at failure.

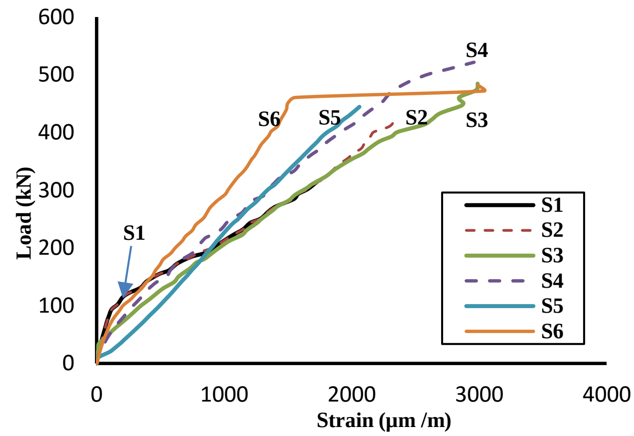


Figure 16: Strain in longitudinal reinforcement for slabs S1 to S6

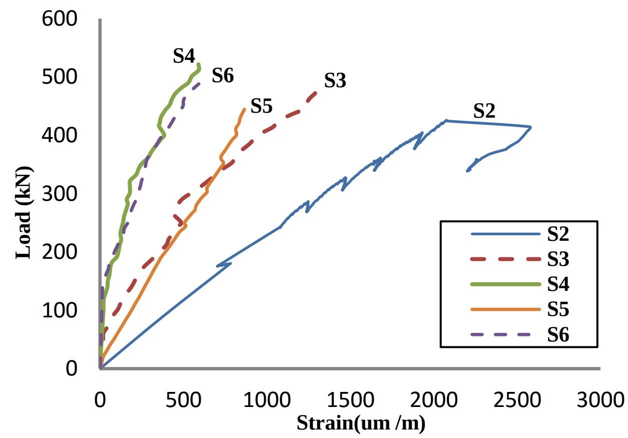


Figure 17: Strain in shear heads for slabs S1 to S6

4. Comparison between the test results and the ACI

This section contains a comparison between the ultimate punching capacities obtained from the test results and those calculated according to the ACI 318-14 [15]. The equations used were presented earlier in section 2. Table 4 shows the comparison between the analytical and experimental results. In Table 4, V_{p1} is the value calculated at the outer critical section located at 3/4 (l_v-c_1/2) and V_{p2} is calculated at the inner critical section at d/2 from the face of the column where the values used are the least of these two values. Compared to the test results, the values obtained using the ACI 318-14 [15] provisions are generally conservative. It should be noted that for S2 the value calculated was lower than that of S1 without shear reinforcement so the value of 271.8 kN which is the capacity of S1 was adopted. For slab S5, the governing value for the shear capacity was the maximum value at the critical section closer to the column.

The value b_o shown in Table 4 represents the critical perimeter needed according to the code provisions for the punching capacity obtained from the test results to satisfy the stress requirements at the critical perimeter. The length l_v is the arm length needed to satisfy this perimeter. The arm length used was generally smaller than these values except for Slab S4. However, the use of these short shear heads showed improvement in the punching shear capacity of the slab column connections compared to the control slab S1.

Table 4: Comparison between the experimental results and ACI 318-14 [15]

Slab	L_v (mm)	V_{exp}* (kN)	ACI 318-14 [15]				
			V_{p1} (kN)	V_{p2} (kN)	V_{exp}/V_p	b_o (mm)	L_v (required) (mm)
S1	--	313	271.8	477.7	1.15	--	--
S2	200	425	231.7 (271.8)	477.7	1.56	1751	388
S3	350	485	386.1	477.7	1.26	1999	446
S4	500	521	540.5	477.7	1.09	2147	481
S5	350	445	396.8	477.7	1.12	1834	407
S6	350	490	386.1	477.7	1.27	2019	451

*V_{exp}: Experimental test values

5. Conclusions

Six reinforced concrete flat slab specimens were tested under punching. One slab was used as a control specimen with no shear reinforcement while the other five specimens were reinforced with embedded shear heads. The specimens had variable parameters and according to the experimental results; the following conclusions can be made:

1. Shear head system is considered an effective method to resist punching shear in flat slabs where an increase of the punching capacity of 66% can be obtained. This system is simple, easy to handle, and does not require increasing the slab thickness.
2. Increasing the arm length of the shear heads effectively increases the punching strength of the flat slabs.
3. The configuration of the shear heads did not have a significant effect on the punching shear capacity of the slabs.
4. Increasing the web thickness of the shear heads by using a built-up section had a slight effect on the punching capacity.
5. Experimental results obtained through this research were compared with those calculated using the ACI 318-14 design code. The code provisions are conservative but provide a simple procedure that can be used safely by design engineers to estimate the punching shear capacity of flat slabs provided with embedded steel beams.
6. Based on the current research output, it can be suggested that using shear heads by the structural engineers with the given cross section of IPE No. 80 extending an arm length of half the slab radius can significantly improve the punching shear capacity of the flat slabs.

Conflict of Interest

The authors declare no conflict of interest.

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