Strengthening technique for RC slab-column connections with large openings using FRP woven wraps

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Strengthening of RC slab-column connections in flat slabs may be essential in many cases such as introducing new openings near columns in an existing solid slab to form ducts. Thirteen half-scale RC flat plate connections are prepared. Each test specimen represents a portion of a slab bounded by the lines of contra-flexure around the column. Two different sizes of openings were chosen, and the models were designed to fail by punching shear. Three different control models with and without openings were loaded until failure. The control specimens were then repaired and strengthened by using steel plates. The other models were strengthened, without pre-loading to enhance the punching shear resistance. Two different techniques of strengthening were applied. The first was executed by applying steel plates or Glass Fiber Reinforced Polymer (GFRP) woven wraps. The second was carried out by using Near Surface Mounted (NSM) steel bars, intertwined GFRP or CFRP stirrups manufactured manually form FRP woven wraps and stitched through the thickness as a new technique. The results of tested slabs are reported and compared. 3-D Nonlinear finite elements with embedded reinforcement were developed and applied to simulate the strengthening methods. The experimental and the analytical results are compared. The results showed significant improvements in the overall behavior of slab-column connections with openings by proper application of FRP woven wraps. Using GFRP and CFRP wraps by the suggested technique increased the ultimate punching shear strength by about 36% and 45%, respectively.

في البلاطات اللاكمرية الخرسانية المسلحة قد يتطلب الأمر عمل فتحات مستجدة في البلاطات المصمتة بجوار الأعمدة مما يؤدي إلى زيادة إجهادات القص الثاقب بشكل كبير ولذلك يلزم إيجاد طرق مناسبة لتقوية هذه البلاطات. في هذا البحث تم اختبار عدد ثلاثة عشر نموذجاً. اثني عشر منها ذات فتحة تم تدعيمها لمقاومة إجهادات القص بواسطة طرق تعتمد على استخدام نسيج الألياف البوليمرية الزجاجية ثنائية الاتجاه والكربونية أحادية الاتجاه. وقد تم تقسيم البلاطات المختبرة إلي أربعة مجموعات. الأولى بها ثلاثة بلاطات مرجعية تم تحميلها وانهيارها نتيجة القص الثاقب ثم معالجتها وتقويتها باستخدام ألواح من الصلب ومسامير قص والمجموعة الثانية بها أربعة بلاطات ذات فتحات مدعمة بطريقتين الأولى لصق ألواح من الصلب مثبتة بمواد إيبوكسية ومسامير قص والثانية باستخدام كانات حديد بالقرب من السطح تمر خلال سمك البلاطة. والمجموعة الثالثة بها أربعة بلاطات تم تقويتها باستخدام نسيج الألياف الزجاجية والمجموعة الرابعة بها نموذجين تم تدعيمهما باستخدام نسيج الألياف الكربونية. وقد تم تحميل نماذج البلاطات بأحمال متزايدة وتسجيل قيم حمل التشرخ وحمل الانهيار والترخيم وشكل الشروخ للنماذج المختلفة أثناء التحميل ومقارنتها قبل وبعد التدعيم. وتم اقتراح نموذج لا خطي ثلاثي الأبعاد باستخدام العناصر المحددة لتمثيل منطقة اتصال البلاطات بالأعمدة وطريقة تدعيمها. وأخيراً تم مقارنة النتائج المعملية والتحليلية. وقد اتضح من النتائج مدى كفاءة الطرق المقترحة باستخدام نسيج الألياف البوليمرية الزجاجية والكربونية في زيادة مقاومة القص الثاقب بنسب ٣٦% و٤٥% على الترتيب وتوافق نتائج التحليل العددي مع النتائج المعملية.

Keywords: Fiber reinforced polymers, Finite element, Openings, Punching shear, Strengthening

1. Introduction

In Reinforced Concrete (RC) flat slabs, punching shear may cause brittle failures due to high transverse shear stresses at slab-column connections. This may be attributed to insufficient punching strength due to essential works such as changing of building use, the need to install new services as sanitary pipes, ventilation, heating, air conditioning, and electrical ducts that require openings in the slabs, corrosion of reinforcement and subsequent rehabilitation techniques where part of the concrete must be replaced, and finally due to construction or design errors. Nowadays, a large number of flat slabs may be considered structurally deficient by today's standards [1] as a result of introducing new methods and factors in the design procedure or need of structural upgrading to meet new seismic design requirements [2-5]. In order to preserve flat slab buildings, strengthening is often considered essential to increase their capability to maintain the public safety [6]. Vicinity of a column is a zone where transverse shear stresses are the largest [7-9]. Openings in this region decrease the shear strength of the slab system and the reduction depends on the location and the dimensions of the openings [10, 11]. Transverse shear stresses in these cases are caused not only by concentrated loads but also by moments that must be transferred between the slab and the column [12]. National building codes [2-5] usually define the allowable dimensions of openings in the design of flat slabs. Wherever such openings are created, strengthening to preserve the capacity is often considered essential to maintain or increase the flexure [13] and shear capability [14]. There are mainly two methods of strengthening to increase the punching shear strength of solid concrete slabs; the first by increasing the slab thickness in the vicinity of the column by providing a drop panel or a column capital, and the second by providing shear reinforcement [15, 16]. In many cases drop panels and column capitals may be unacceptable for esthetic reasons. A practical method to increase the punching resistance of the slab may be the use of shear reinforcement [17].

Different forms of steel may be efficient for strengthening concrete member, such as steel built up sections, plates and bars. Some researches investigated the effect of steel plates with steel bolts to strengthen the slab-column connections in both flexure and shear [18, 19], and some others used only steel studs [20, 21] or stirrups for increasing the shear strength [17].

The rehabilitation and strengthening of structural members with composite materials, such as carbon, glass, kevlar, and aramid Fiber-Reinforced Polymers (FRP), have recently received great attention [22]. Labor savings inherent with its lightweight and comparatively simple installation, its high tensile strength, and immunity to corrosion have made FRP an attractive alternative to traditional retrofitting techniques. Field applications over the last years have shown excellent performance of FRP retrofitted structures. Nowadays, carbon and glass fiber strips, rods and wraps woven in one or multi-directions are widely used as strengthening materials.

Many researchers have used FRP for strengthening the flexure strength of slabs [23-26]. Several studies have investigated the use of externally bonded FRP composites or Near Surface Mounted (NSM) reinforcement to improve the strength and stiffness of RC slabs, but most have addressed flexural strength, not punching shear. A very limited amount of experimental data exists on strengthening of slab-column connections with openings. Some researches have increased the punching strength by applying FRP strips as additional layers on the surfaces of the slabs only [27, 28] or both the surfaces and the column [17, 29]. Most previous researches increased the punching strength by applying FRP strips or rods on the surfaces of the slab-column connections [30-33]. The literature showed that only a very few studies used FRP as punching shear reinforcement by acting as studs [34].

No available literature that addressed the strengthening of flat slab-column connections with large openings adjacent the columns that do not meet the national building codes requirements and limitations [2-5].

This paper reports on a series of tests conducted to assess and compare the ability and efficiency of traditional materials represented by steel bars or plates, as well as Advanced Composite Materials (ACM) represented by glass and carbon reinforced polymers (GFRP and CFRP) woven wraps for strengthening punching shear of slab-column connections with large openings.

2. Research significance

This research investigates new innovative techniques for increasing the punching shear strength of concrete slab-column connections with and without openings. The first method is executed by using FRP stirrups manufactured from intertwined FRP wraps and epoxy resin. For each stirrup two parallel sides are executed as NSM FRP reinforcement embedded in the cover of the slab and the other two parallel sides are stitched through thickness in holes perpendicular to the plane of the slab to enhance the performance of the retrofitted slab by restraining the discontinuity of the slab at the shear cracks. The other method is by adding additional bonded layers from FRP woven wraps to the slab surfaces and the sides of the opening to cover the intertwined dowels as well as to resist the additional moments due to unsymmetrical conditions. The suggested methods for strengthening are applied for slab-column connections with large opening using GFRP and CFRP roving wraps as a test for their efficiency. The results are compared with traditional methods of using steel plates, bolts and NSM steel stirrups.

3-D nonlinear isoparametric finite elements program with embedded reinforcement is developed by Meleka [35] and applied to simulate the nonlinear behavior of concrete, steel and strengthening method. The numerical results are compared with the experimental results to verify the method of the analysis.

3. Codes provisions

Punching shear failure is characterized by the slab fracturing along planes that extend from the column-slab interface on the compressed face of the slab through the depth of the slab in an inclined direction away from the column. Most researches on the punching shear strength of slabs have been concerned with developing empirical formulas for ultimate shear stress resistance [2-5]. Ultimate shear stress is obtained by dividing the shearing force by the area of an assumed critical section at a certain distance from the column perimeter. Egyptian Code of Practice, ECP [2] assumes the shear failure plane to have an angle of inclination of 45^o from the slab surface and proposes the use of a critical section perimeter half the effective slab thickness from the column periphery. In the absence of an unbalanced moment, the ultimate punching shear stress q_{up} due to factored loads Q_{up} can be calculated as:

$q_{up} = \frac{Q_{up}}{(b_o d)}$, where b_o is the perimeter of the shear critical section d/2 from the column periphery.

According to the ECP, the total ultimate shear stress when considering the moment transferred to column, can be calculated from a simplified equation:

$q_{up} = \frac{Q_{up}\beta}{(b_o d)}$, where β =1.15, 1.3 and 1.5 for interior, edge and corner columns, respectively.

For the purposes of design, flat plate systems may be divided into column and middle strips in two perpendicular directions. The column strip width on each side of the column centerline is equal to 1/4 of the length of the shorter span l in the two perpendicular directions. As an alternative to detailed analysis for slabs with openings, ECP defines that the maximum permitted opening size in the area common to intersecting column strips as 1/20 of the corresponding span. The part of the critical perimeter contained between two tangents drawn to the outline of the opening from the center of the column is considered to be ineffective [2]. The American Concrete Institute (ACI) [3] defines it as 1/16 of the shorter span. British Standard (BS) [4] defines that maximum permitted opening size in the area common to intersecting column strips is 1/20 the shorter span. Eurocode 2 (EC2) [5]defines that when shortest distance between the perimeter of the column and the edge of the opening does not exceed 6 times the effective depth d, that part of the critical perimeter contained between two tangents drawn to the outline of the opening from the center of the column is considered to be ineffective . In this research the two opening sizes are considered in the area common to intersection column strips adjacent to the column and equal to 1/8 and 1/4 of the corresponding span.

4. Experimental program

An experimental test program was carried out to study the potentiality of using different methods and materials in the repair and strengthening of RC flat slab-column connections with large openings failed in punching shear. The models of slab-column connections tested in this study were half-scale models of a typical prototype flat-plate structure with equal columns spacing of 4 m center to center in both directions. The test specimen represents a portion of a slab bounded by the lines of contra-flexure around the interior column. The chosen dimensions for the tested slabs were prepared so that the slab is located in the negative moment region around the interior column and inside the line of contra-flexure. The test specimens were designed to be simply supported along the four edges with clear spans 100 cm in both directions. This test arrangement is convenient to simulate the actual boundary conditions in the prototype. The slabs were designed according to the ECP-203 [2] to fail in punching shear prior to flexure, so that the shear strength contribution of the strengthening laminates could be measured.

Thirteen models of flat slab-column connections were cast and tested for this purpose. One slab has no opening, six slabs have a square opening 25×25 cm, and the other six slabs each has a rectangular opening 25×50 cm. Those specimens were labeled in accordance with numerical values denoting the length of the opening. Three specimens served as control specimens; slab without opening (S0), slab with an opening 25×25 cm (S25), and slab with an opening 25×50 cm (S50). The control slabs were loaded until failure, then repaired and strengthened by steel plates and bolts forming Group (A) and labeled SRS0, SRS25, and SRS50, respectively. The letter R indicates for the repairing process and the second S for using steel. The other ten slabs have an opening and were strengthened before loading by three different materials and classified into three groups: B, C, and D. Group (B) contains four specimens; two specimens were strengthened by steel plates, SS25 and SS50, and the other two specimens were strengthened by NSM steel bars, SNS25 and SNS50, where N indicates for NSM technique. Group (C) contains four specimens; two were strengthened by GFRP woven wraps, SG25 and SG50, and the other two were strengthened by NSM GFRP manufactured rods, SNG25 and SNG50, where G denotes for GFRP. Group (D) comprises two specimens SNC25 and SNC50, where C refers to the strengthening material CFRP.

4.1. Test specimens

Thirteen square slab specimens 115×115×10 cm were cast with stub column 15×15×40 cm at the centre of the slab. Specimens were cast with normal density concrete of approximately 25 MPa cube strength. High tensile steel bars of 10 mm diameters were used as top and bottom reinforcement. The tension reinforcement was a mesh; 11 Φ 10 mm in both directions, while the compression reinforcement was a mesh; 7 Φ 10 mm in both directions as shown. The average effective depth was 8 cm. The concrete columns were reinforced with 4 Φ 10 longitudinal high tensile steel bars and with normal mild steel 6 mm as stirrups. Full details of the control slabs S25, dimensions and the details of reinforcement are shown in fig. 1.

4.2. Repair and strengthening schemes

Control specimens were loaded until failure and then repaired with steel plates. The other models were strengthened before loading to enhance the punching shear strength of slabs with large opening. After strengthening, specimens were subjected to a concentric monotonically increasing load until failure. Specimens were classified into the following groups:

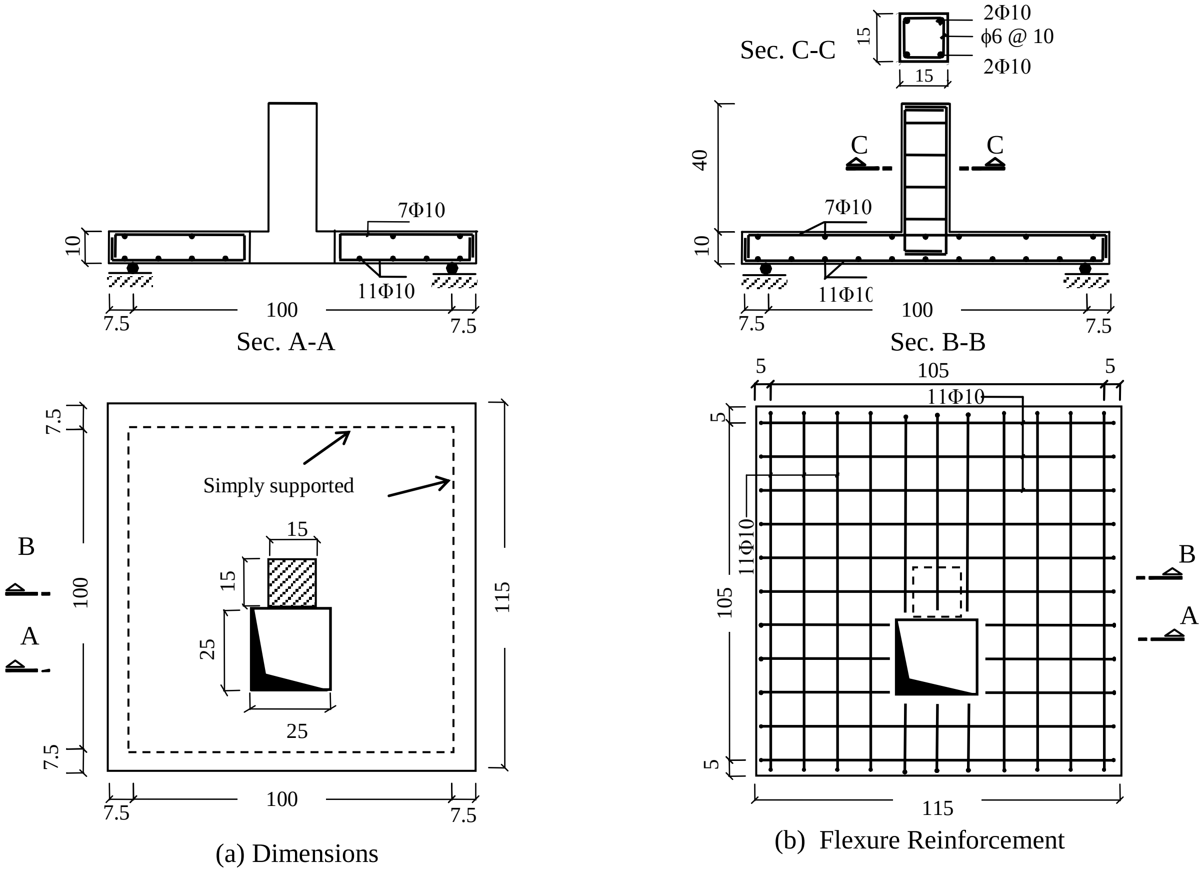


Fig. 1. Dimensions and details of reinforcement of the control slab S25.

Group A: Repair and strengthening using steel plates

The three control specimens were loaded until failure and then repaired and strengthened with steel plates, SRS0, SRS25 and SRS50. The procedure of repair and strengthening can be summarized as follows:

1. Loose particles and dust were removed and cracks were then injected by Sikadur 52 [37]. Slab surfaces were coated by bonding material [38]. New concrete mix was then added to cover the damaged areas. The specimens were cured for seven days before strengthening.
2. Steel plates, ASTM-A36 Mild, 5mm thickness were then fixed at top and bottom surfaces, internal sides of the opening and 10 cm of the stub column. All steel strips were welded together.
3. Bolts of diameters 8 mm were installed in holes drilled through the thickness of the specimens and the steel plates to fasten the top and bottom surfaces together. The shear bolts were designed from steel bars (regular strength) with two ends threaded. Sikadur 31 [37] was used to guarantee bonding between steel plates, bolts and concrete. The bolts were then tightened against the concrete surface through the bearing plates by steel nuts. Fig. 2 shows the dimensions of steel plates and the arrangements of bolts.

Group B: Strengthening using steel plates and bars

The second Group B: contains four slabs that strengthened before loading by two techniques; the first by adding external steel plates, SS25 and SS50, the procedure of strengthening as in Group A. The other two specimens SNS25 and SNS50 were strengthened by adding steel bars. Each steel bar is embedded in the cover at the top and bottom surfaces as NSM reinforcement and passing through the thickness of the slab to act as dowels. The final form of the strengthening bars was as stirrups fig. 3. In the second technique, grooves 12×12 mm were cut in the concrete surface. Holes of diameter 10 mm were drilled at the ends of the grooves transversely through the thickness of the slab. Steel bars ϕ 8 mm were then placed in the grooves and passed through holes to act as stirrups. The grooves were then filled with epoxy to flow around the bars. Fig. 3 shows the arrangement and shapes of the NSM reinforcement.

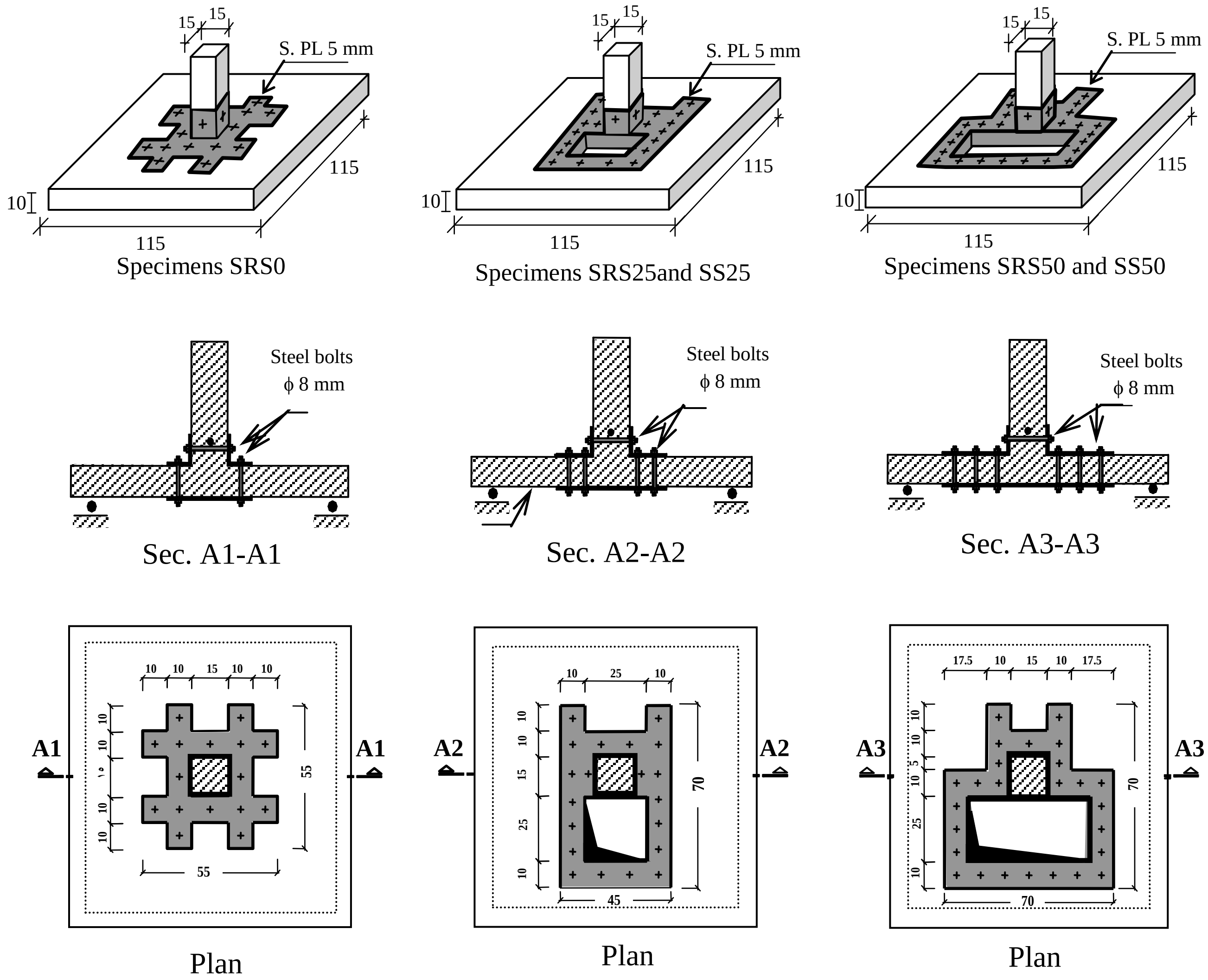
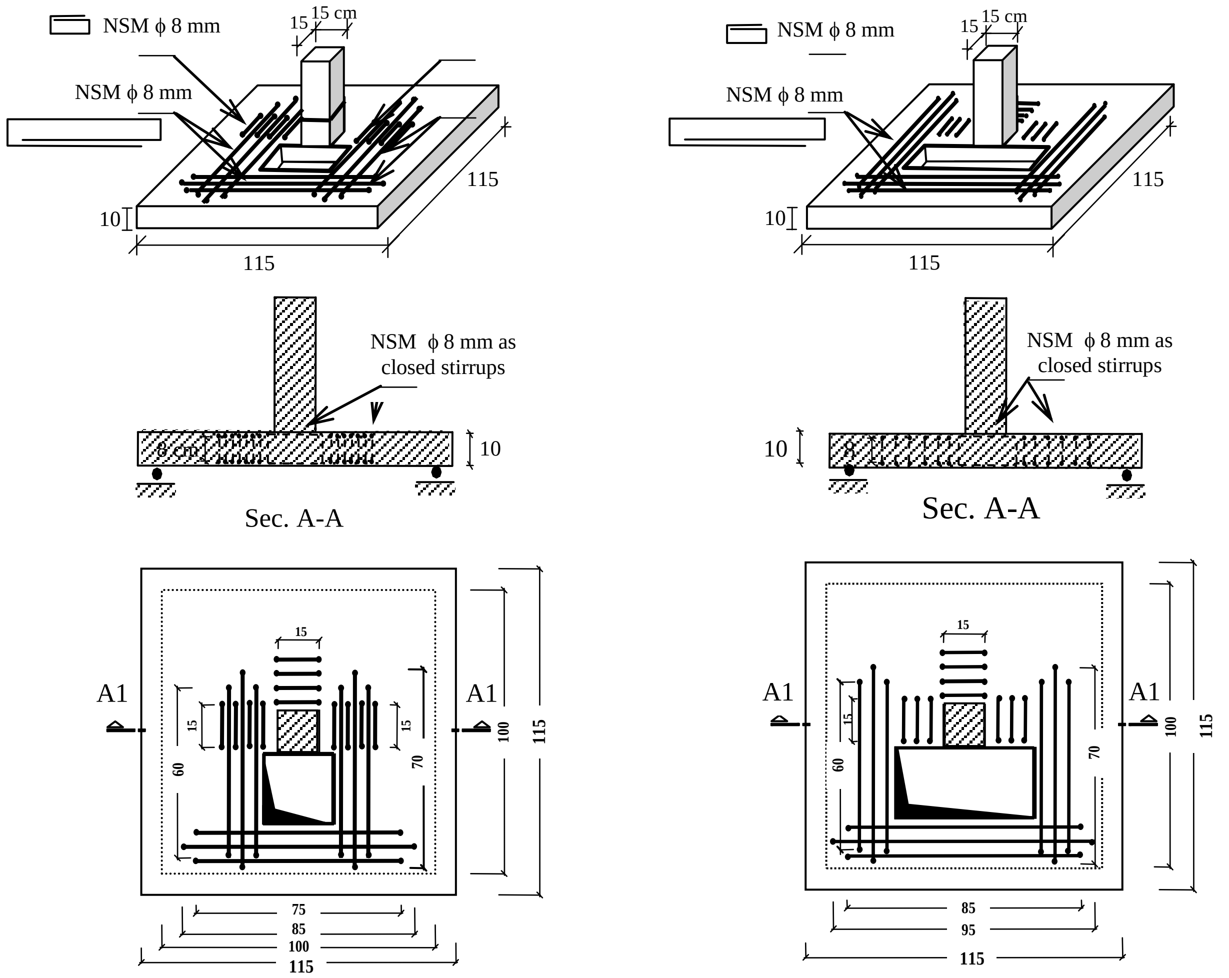


Fig. 2. Repair and strengthening using steel plates.



Specimens SNS25 SNG25 and SNC25 Specimens SNS50 SNG50 and SNC50

Fig. 3. Strengthening using NSM steel bars, GFRP or CFRP intertwined stirrups.

Group C: Strengthening using GFRP

Group C: comprises four slabs strengthened by GFRP Woven Roving (WR). Fig. 4 shows the common FRP rods used in other researches. Fig. 5 shows the GFRP WR used for strengthening in the research.

The strengthening was executed by two new techniques. The first two slabs SG25, SG50 were strengthened by both intertwined GFRP WR rods acting as stirrups and additional layers of GFRP warps. The intertwined rods were manual manufactured by using GFRP wraps to form rods with diameter about 8mm. The dry woven wraps were saturated by Sikadur-330 before intertwining. Fig. 6 shows manufacturing the intertwined rods from wraps. Fig. 7 shows the testing machine for measuring the ultimate tensile strength of the intertwined rods. After stitching the intertwined rods, four additional layers of GFRP WR wraps were bonded at the top and bottom surfaces to cover the strengthened area, 10 cm of the column and the internal sides of the opening. Fig. 8 shows the method of strengthening.

The other two specimens SNG25 and SNG50 were strengthened by the NSM reinforcement intertwined GFRP WR rods ϕ 8 mm acting as stirrups with the same procedure as in Group B. Fig. 9 shows the method of stitching. Fig. 10 shows the arrangement and shape of the NSM GFRP intertwined reinforcement.

Group D: Strengthening using CFRP

Group D contains two slabs SNC25, SNC50 that were strengthened by using NSM CFRP intertwined stirrups manufactured from SikaWrap Hex-230C [37] with the same procedure as in Group C. The intertwined rods were manual manufactured by using CFRP woven wraps with breadth about 15 cm and with the required length. The dry woven wraps were saturated by Sikadur-330 before intertwining. Fig. 10 illustrates the arrangement of the intertwined stirrups. Table 1 summarizes the experimental program for different specimens of slab-column connections tested in this research.

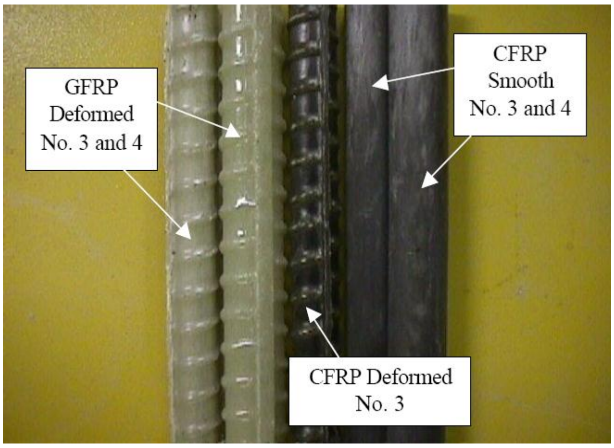


Fig. 4. Common FRP rods.

Fig. 5. FRP wraps used for strengthening.

Fig. 6. Manufacturing intertwined rods from FRP wraps.

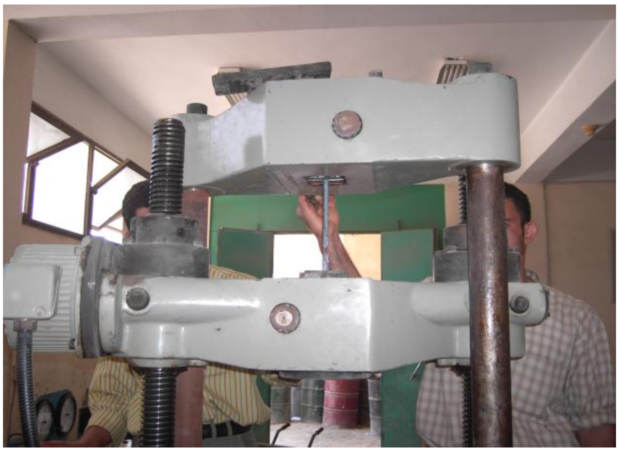


Fig. 7. Testing intertwined GFRP rods.

4.3. Test set-up and instrumentation

The slabs specimens were subjected to a concentric monotonically increasing load until failure. The loading rig is shown in fig. 11-a. Loads were applied in increments using a hydraulic jack of 500 kN maximum capacity. Deflections, first cracking loads and ultimate failure loads were recorded. Propagation of cracks was marked after each load increment up to failure. Dial gauges of 0.01 mm accuracy and total capacity of 11 mm were used for deflection measurements. Special arrangement was designed for each dial gauge to fix it in its exact position and to ensure proper readings. Fig. 11-b.

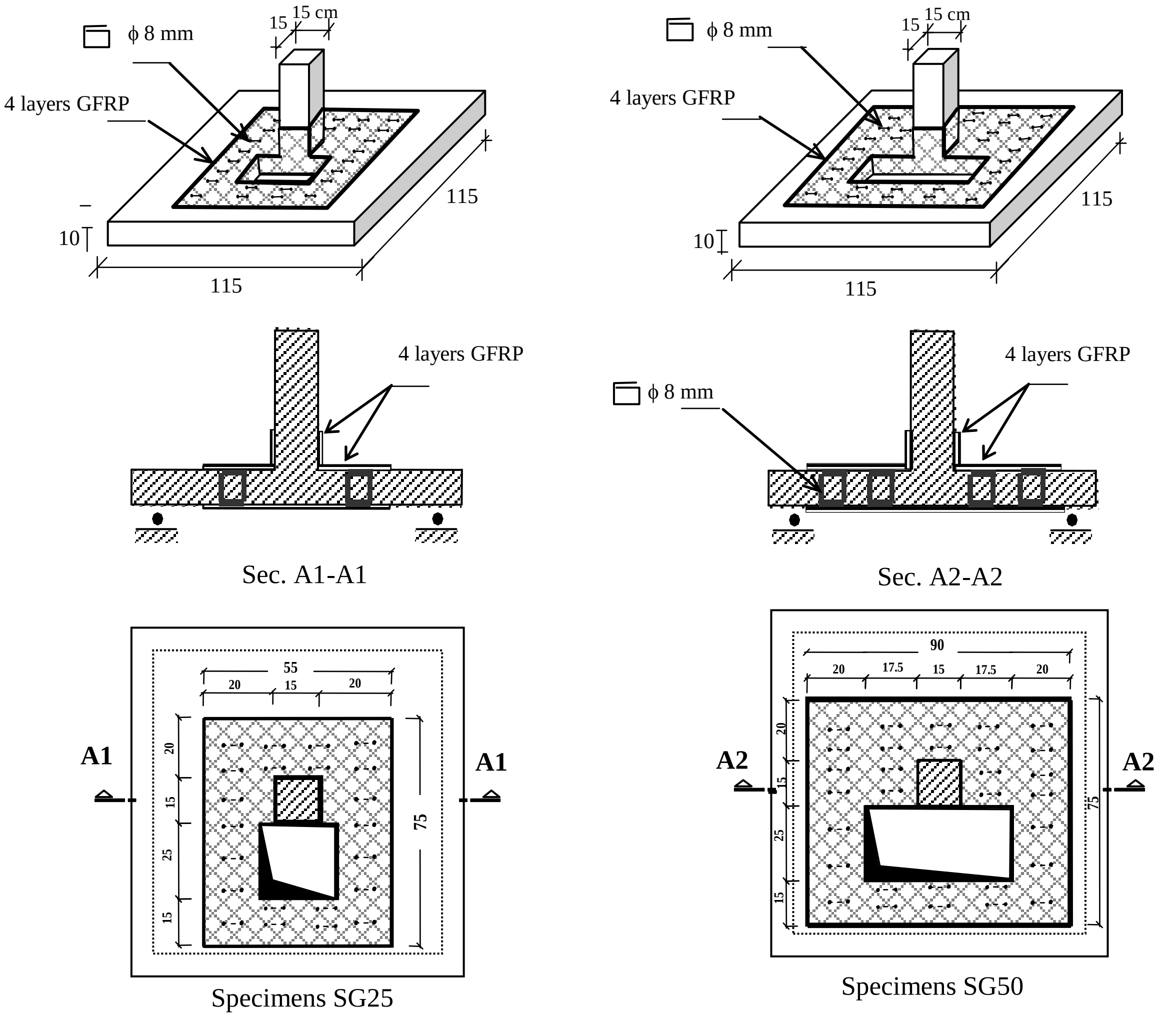


Fig. 8. Strengthening of slab SG25, SG50 by GFRP woven wraps.

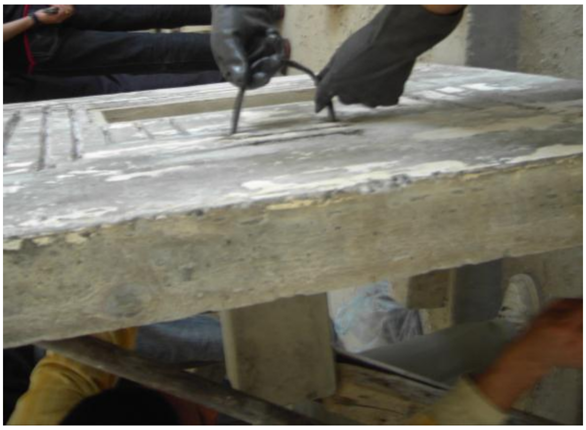


Fig. 9. Stitching FRP of intertwined rods through the slab thickness.

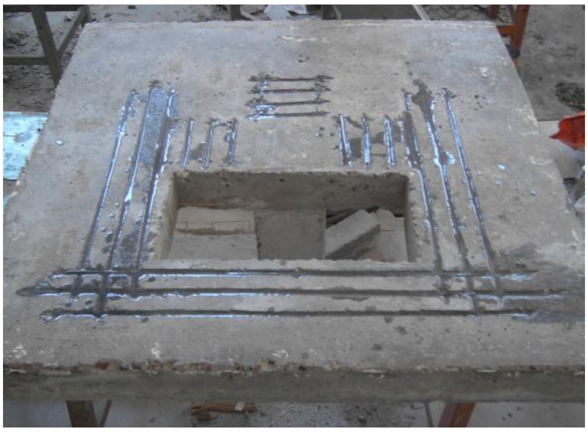
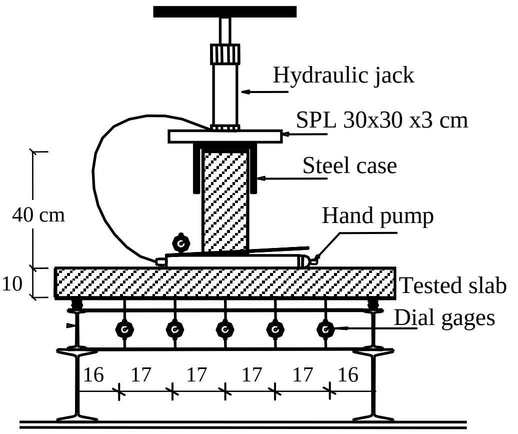
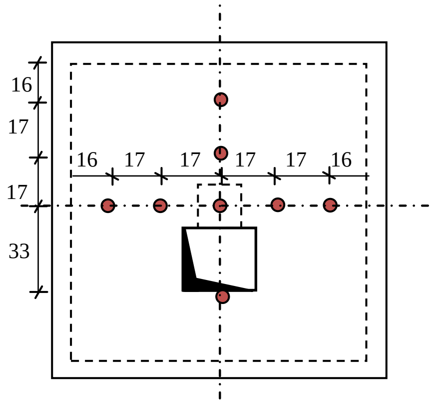


Fig. 10. Installing NSM FRP reinforcement in the slab.



(a) Test rig



(b) Arrangement of dial gauges

Fig. 11. Test setup and instrumentation.

Table 1
The experimental test program

Group	*Specimen code*	*Opening Size, cm*	*Repair and strengthening system*
Control slabs	S0	---	Loaded up to failure, P_f.
	S25	25×25	Loaded up to failure, P_f.
	S50	25×50	Loaded up to failure, P_f.
Group (A) repaired	SRS0	---	Slab S0 was repaired and strengthened with steel plates around the column at top and bottom surfaces and fixed by epoxy and bolts, fig. 2.
	SRS25	25×25	Slab S25 was repaired and then strengthened with steel plates around the column and the opening at top and bottom surfaces, fig. 2.
	SRS50	25×50	Slab S50 was repaired and then strengthened with steel plates around the column and the opening at top and bottom surfaces, fig. 2.
Group (B) strengthened with SP	SS25	25×25	Steel plates were fixed around the column at top and bottom surfaces and the sides of the opening with epoxy and bolts, fig. 2.
	SS50	25×50	Steel plates were fixed around the column and the opening at the top and bottom surfaces with epoxy and bolts, fig. 2.
	SNS25	25×25	NSM steel bars ϕ 8 mm were placed around the column and the opening, fig. 3.
	SNS50	25×50	NSM steel bars ϕ 8 mm were placed around the column and the opening, fig. 3.
Group (C) strengthened with GFRP	SG25	25×25	GFRP intertwined stirrups ϕ 8 mm around the column and the opening. Four layers of GFRP WR wraps were then bonded at top and bottom surfaces, fig. 8.
	SG50	25×50	GFRP intertwined stirrups ϕ 8 mm around the column and the opening. Four layers of GFRP WR wraps were then bonded at top and bottom surfaces, fig. 8.
	SNG25	25×25	NSM GFRP intertwined reinforcement ϕ 8 mm around the column and the opening, fig. 3.
	SNG50	25×50	NSM GFRP intertwined stirrups ϕ 8 mm around the column and the opening, fig. 3.
Group (D) strengthened with CFRP	SNC25	25×25	NSM CFRP intertwined stirrups ϕ 8 mm around the column and the opening, fig. 3.
	SNC50	25×50	NSM CFRP intertwined stirrups ϕ 8 mm around the column and the opening, fig. 3.

4.4. Properties of materials

The specimens were constructed using a normal density concrete. The concrete was produced in the Menoufiya laboratory using Ordinary Portland cement, clean sand, graded gravel, potable water. Testes were carried out according to ECP-203-07 [2] to define the properties for both concrete and steel. Table 2 summarizes the concrete mix properties used for casting the models while the properties of the used steel are given in table 3. Table 4 gives the mechanical properties of the GFRP WR and CFRP wraps used in this work. Table 5 gives the mechanical properties of the used epoxies.

5. Experimental results

The behavior of the tested slabs were investigated through recording deflections, cracking and ultimate loads, crack propagation at different stages of loading and failure modes. The results were compared to evaluate the used methods of repair and strengthening.

Table 2
Mix proportions for one cubic meter of concrete

Mix proportions, kg / m^3				Unit weight, kg/m^3	W/C	Compressive strength, MPa	
Cement	Water	F.A.	C.A.			7 days	28 days
350	175	602	1204	2331	0.5	19.5	25.0

Where F.A.: sand as fine aggregate, and C.A.: gravel as coarse aggregate

Table 3
Properties of steel

Steel type	Yield strength, MPa	Tensile strength, MPa	Elongation, %	Young's modulus, MPa
High tensile	379	545	15.11	$2.1x10^5$
Mild steel	260	387	23.21	$2.0\ x10^5$
Steel plates	250	400	20	$2.1x10^5$

Table 4
Mechanical properties of GFRP and CFRP wraps [36, 37]

Property	Glass fiber woven roving wraps (WR)	CFRP (SikaWrap hex-230C)
Fabric thickness, cm	0.17	0.12
Tensile strength, MPa	284	4020
Modulus of elasticity, MPa	13000	22500
Elongation %	2.0	1.7

Table 5
Mechanical properties of the used epoxies in MPa [37, 38]

Property	Sikadur-52 (7 days)	Sikadur-31 (7 days)	Sikadur-330 (7 days)	EXUIT -50 (14 days)
Compressive strength	53	65	80	70
Flexural strength	50	40	60	34
Tensile strength	25	20	34	23
Modulus of elasticity	1060	4600	3500	4000

5.1. Deflections

Fig. 12 shows comparison of the deflection lines at a section along the center of the tested slabs. Deflections lines were drawn at the ultimate failure P=100 kN, which represent the failure load of the control slab S50. Reference slabs S25and S50 showed an increase of 33.3% and 105%, respectively in mid-span deflection compared to S0 as shown in fig. 12-a. Fig. 12-b shows the effect of using steel plates for retrofitting the failed slabs in punching shear. Deflections decreased by about 36%, 38% and 54% for slabs SRS0, SRS25 and SRS50, respectively in comparison to their corresponding reference slabs. Figs. 12-c and 12-d show the comparison of the used strengthening techniques in cases of opening 25×25 and 25×50, respectively. Utilizing FRP wraps by the new techniques in case of slabs SG25, SNG25 and SNC25 decreased the maximum deflection by about 17%, 21%, and 33%, respectively compared to the control slab S25. The deflections of the strengthened slabs SG50, SNG50 and SNC50 decreased by about 16%, 27%, and 35%, respectively compared to the control slab S50. It is noted that the overall deflection behavior was improved and curves were nearly smooth and symmetrical around the slab centerline. This indicates that the applied strengthening techniques were capable to compensate the reduction of the slab stiffness due to opening. Fig. 13 shows comparison of the load deflection curves for the repaired slabs SRS0, SRS25 and SRS50 at the center. It is observed that the utilized method of repair improved the deflection behavior for slabs with or without openings. At failure load of control slabs, the deflections of slabs SRS0, SR25, SR50 decreased by about 10%, 50% and 53% less than the corresponding control slabs deflection. Fig. 14 shows comparison between the load deflection curves for the strengthened slabs SS25, SS50, SNS25 and SNS50. At failure load of slab S25, the deflections of SS25 and SNS25 decreased by about 64% and 25%, respectively in comparison to the deflection of S25. The deflections of slabs SS50 and SNS50 decreased by about 65% and 8%, respectively in comparison to the deflection of S50. It is noted that the method of external bonded layers with bolts increased the overall stiffness more than the NSM reinforcement. Figs. 15 and 16 compare the methods of strengthening by NSM reinforcements of slabs with openings 25×25 and 25×50. At failure load of the control slab S25, the deflection of slabs SNS25, SNG25 and SNC25 decreased by about 25%, 32% and 42% in comparison to S25, while at the failure load of slab S50, the maximum deflections of SNS50, SNG50 and SNC50 decreased by about 16%, 27% and 35% in comparison to the deflection of S50.

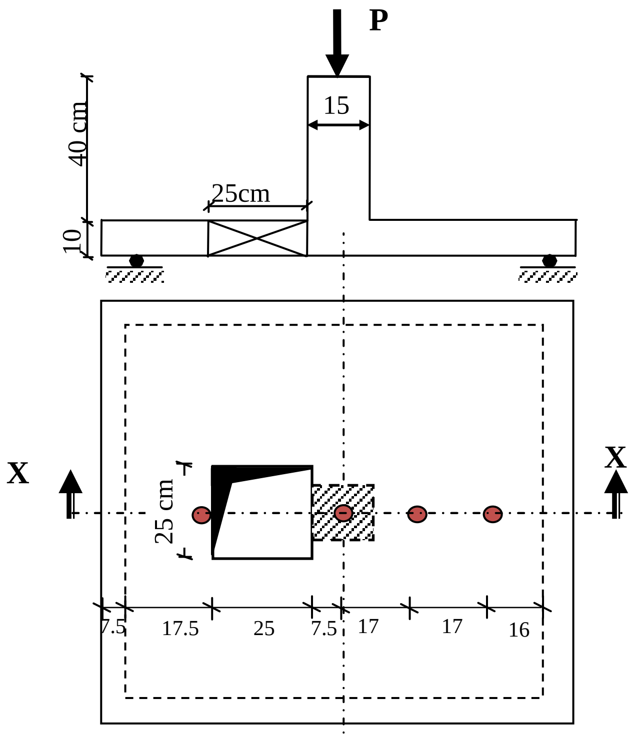
5.2. Cracking and ultimate loads

Fig. 17 shows the cracking and ultimate failure loads for all tested slabs. It is noted that the slabs strengthened by steel plates around the column and the opening at top and bottom surfaces showed the highest ultimate failure load.

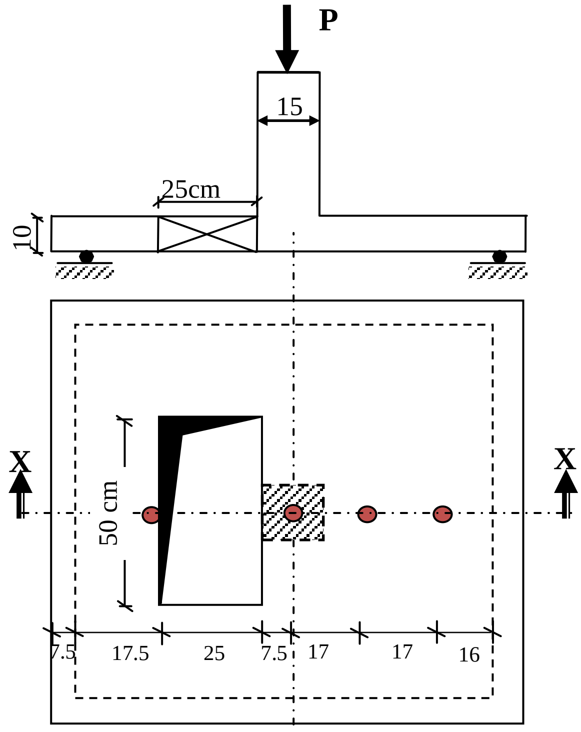
The failure load of slab strengthened with NSM reinforcement SNS25, SNG25 and SNC25 increased by about 43%, 50% and 64%, in comparison to slab S25, while for slabs SNS50, SNG50 and SNC50 the failure loads increased by about 27%, 36% and 45%, respectively in comparison to slab S50. Slabs SG25 and SG50 showed an increase of 57% and 37%, respectively in the ultimate load carrying capacity compared to that of the un-strengthened slab. It is observed that the strengthened slabs with FRP wraps by the new techniques, showed a distinct gain in the load carrying capacity compared to the corresponding reference slabs. This confirms the efficiency of utilizing the new techniques for strengthening slab-column connections in punching shear.

5.3. Ductility

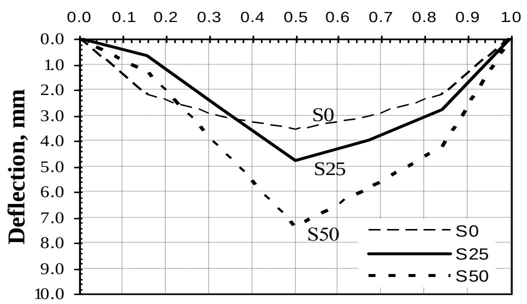
Ductility of slab is defined as the ratio between the maximum deflection due to the ultimate load and the maximum deflection at the first cracking load. Fig. 18 compares the ductility for all tested slabs. The Ductility of slabs SNS25, SNG25 and SNC25 increased by about 42%, 42% and 39% in comparison to slab S25, while slabs SNS50, SNG50 and SNC50 increased by 38%, 33% and 31%, respectively in comparison to slab S50. It is



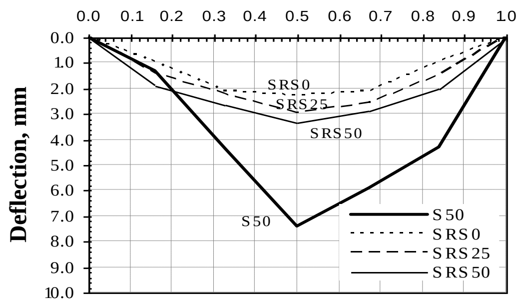
Opening 25×25 cm



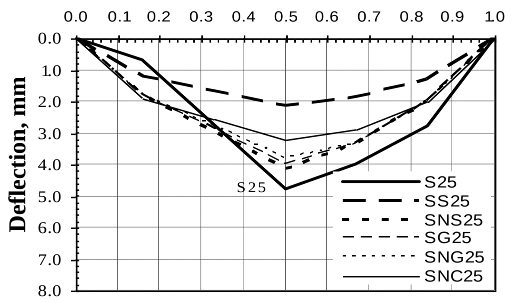
Opening 50×50 cm



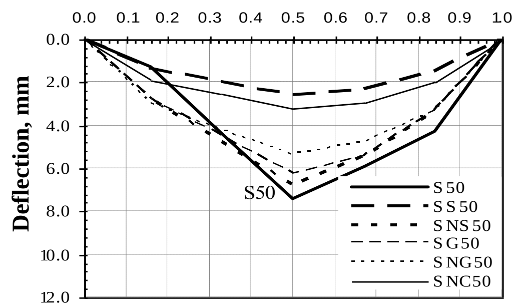
(a) Deflection lines for control slabs



(b) Deflection lines for repaired slabs



(c) Deflection lines for strengthened slabs with an opening 25×25 cm



(d) Deflection lines for strengthened slabs with an opening 50×50 cm

Fig. 12. Deflection lines for tested slabs along sec X-X at load P=10t.

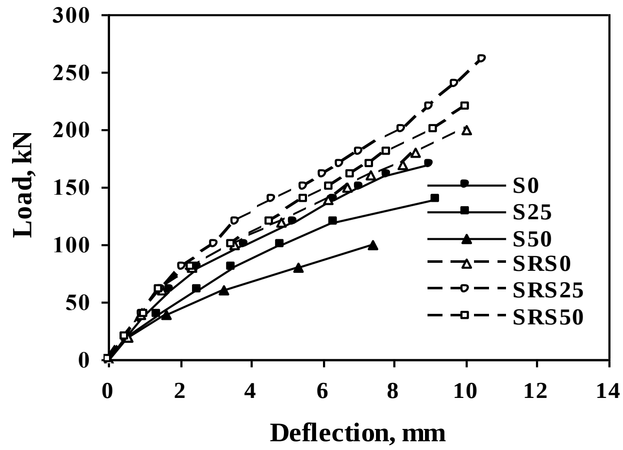


Fig. 13. Comparison between load-deflection curves at the center of the repaired slabs (Group A).

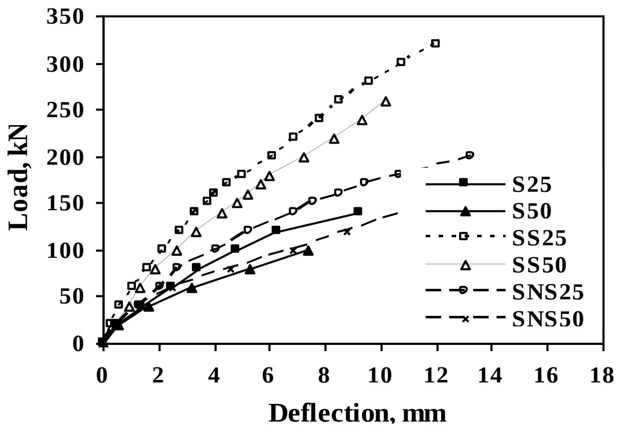


Fig. 14. Comparison between load-deflection curves at the center of strengthened slabs (Group B)

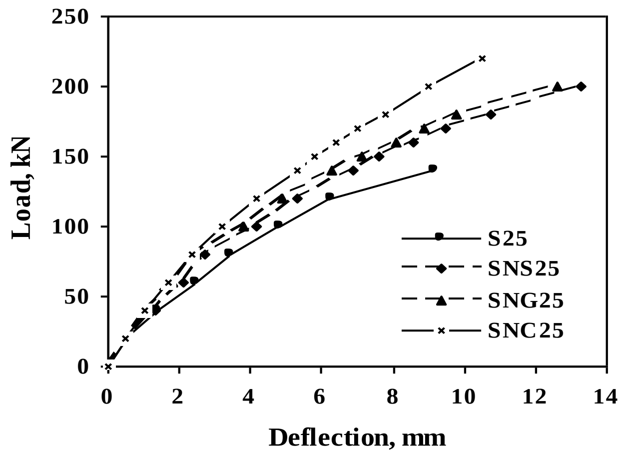


Fig. 15. Comparison between Load-deflection curves for slabs with opening 25×25 strengthened by NSM reinforcement.

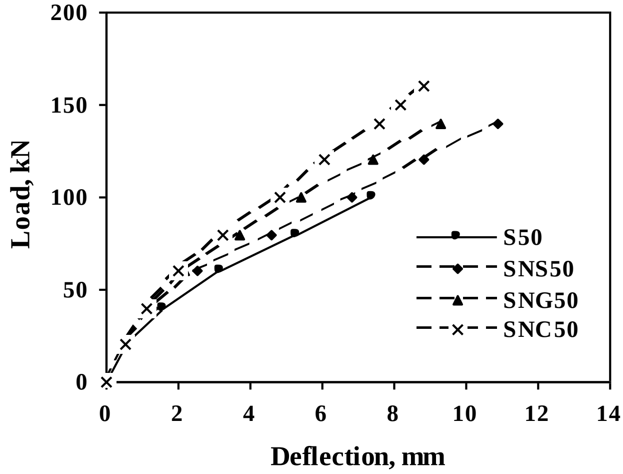


Fig. 16. Comparison between load-deflection curves for slabs with opening 25×50 strengthened by NSM reinforcement.

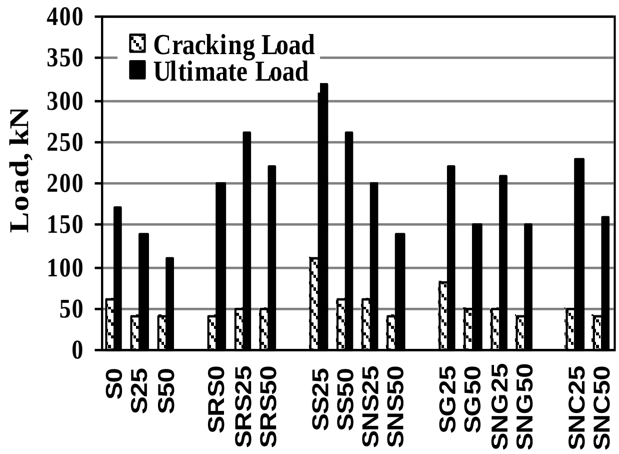


Fig. 17. Cracking and ultimate loads of tested slabs.

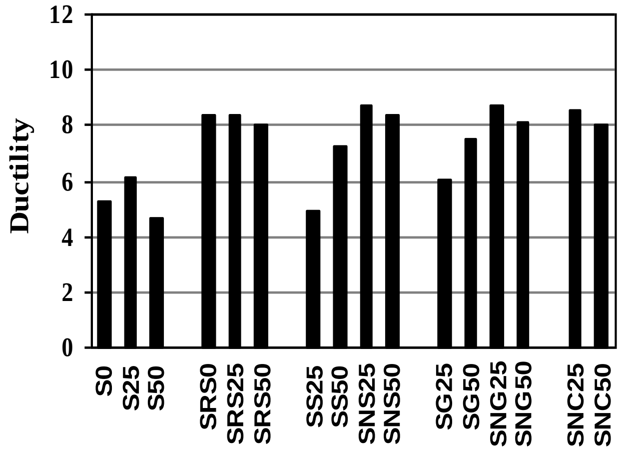


Fig. 18. Ductility of tested slabs.

Fig. 19. Crack pattern of slab S25
(P_{cr} = 40 kN , P_{ult} = 140 kN).

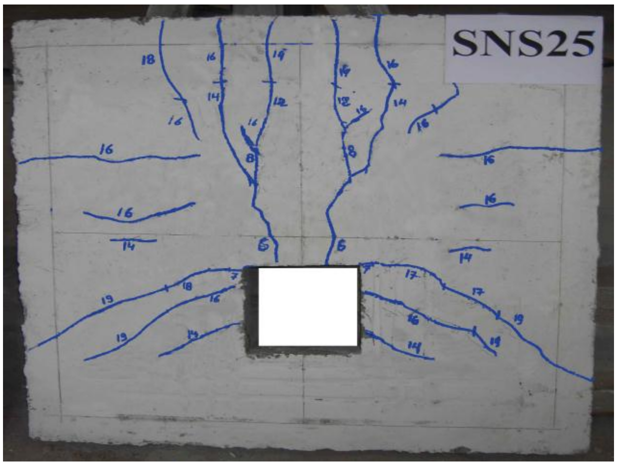


Fig. 20. Crack pattern of slab SNS25
(P_{cr} = 60 kN , P_{ult} = 200 kN).

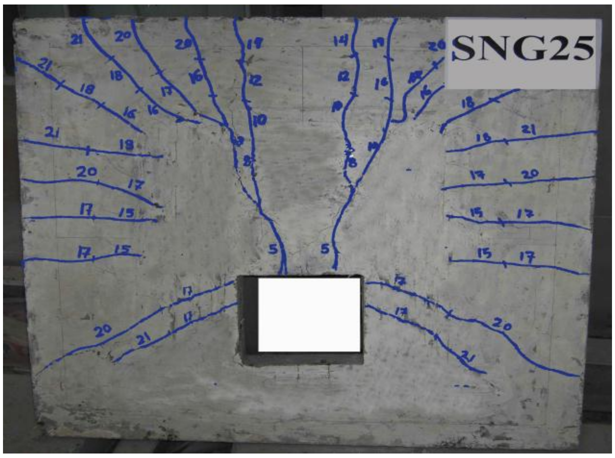


Fig. 21. Crack pattern of slab SNG25
(P_{cr} = 50 kN , P_{ult} = 210 kN).

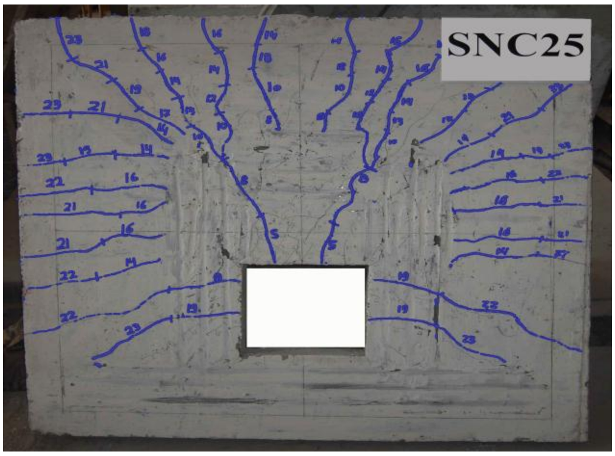


Fig. 22. Crack pattern of slab SNC25
(P_{cr} = 50 kN , P_{ult} = 230 kN).

noted that strengthened slabs by steel plates around the column and the opening at top and bottom surfaces showed the lowest ductility in comparison to NSM technique. This supports the reliability of utilizing the new techniques for strengthening slab-column connections in order to avoid the brittle failure.

5.4. Cracking patterns

The control slabs failed in punching shear failure mode. Fig. 19 shows the crack patterns of the control slab S25. The punching failure area was shifted toward the solid area away from the opening. Figs. 20, 21 and 22 show the crack patterns of the strengthened slabs SNS25, SNG25 and SNC25. It is observed that all utilizing methods of strengthening changed the mode of failure to flexure. Cracks initiated at the perimeter of the column and propagated outwards in the week directions between the NSM reinforcement. It is noted that the new strengthening technique by NSM FRP wraps improved the stiffness of the opening and reduced the propagation of cracks.

6. Finite element analysis

RC slabs were modeled in many researches as two dimensional structures by 2-D finite element analysis with steel reinforcement [39] as smeared layers. Strengthening of the existing RC structure may be essential in many cases. Some researchers modeled the strengthening layers as additional layers [40]. Meleka [41] suggested a new model to simulate the three dimensional RC structures by 2-D compound finite elements to represent the strengthening layers in any face at any different direction. Other researches modeled concrete and fiber reinforced concrete by 3-D finite element analysis with steel reinforcement as embedded bars [35, 42]. Meleka [43] developed a computer program which was applied in this research utilizing nonlinear three dimensional isoparametric brick elements to represent concrete, reinforcement and NSM strengthening bars as follows:

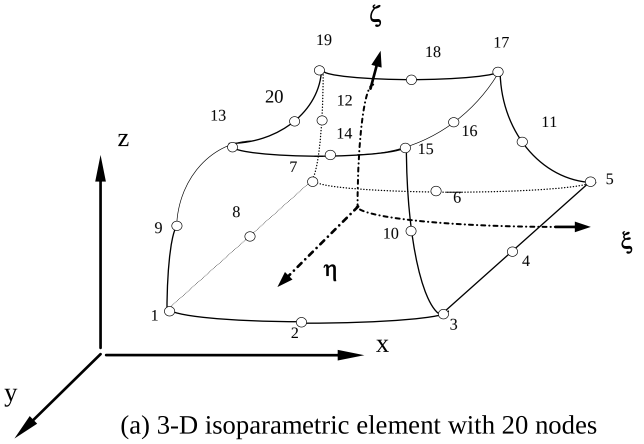
6.1. Constitutive models

In this study, the finite element analysis was performed by using isoparametric brick element with 20 nodes. Each node has three degrees of freedom to represent the concrete. Each element has its own local coordinate system ξ, η, ζ as shown in fig 23-a. Failure criterion of Ottosen's model [44] was chosen in the analysis for concrete. The proposed model is based on the stress-strain curve for uniaxial compression. The actual secant value, E_s, of Young's modulus which represents the stress-strain curve under triaxial loading is considered. The expression of the secant values, of Poisson's ratio for uniaxial compression loading is generalized to triaxial compressive loading by using of the nonlinearity index, β [42]. In three dimensional stress space; σ_1, σ_2, σ_3, cracks are assumed to occur normal to any of the principal stress as shown in fig. 23-b. At any Gauss point up to three cracks may occur in three different directions. A crack is assumed to be occurred if either the failure criterion defined by Ottosen is violated, or the maximum principal stress σ_1 exceeds the tensile strength of concrete f_t. Both reinforcement and NSM strengthening bars were represented by embedded bars within the concrete element fig. 23-C. A gradual release of the concrete stress component normal to the cracked plane was considered in the present study. The modulus of elasticity was assumed to decrease due to cracking when the strain increases as shown in fig. 23-d. The values of the moduli of elasticity and the characteristic strength of the strengthening materials define their constitutive models. In case of the steel reinforcement, the complete stress strain relationship is defined; that is, linear elastic, yield, hardening in both compression and tension.

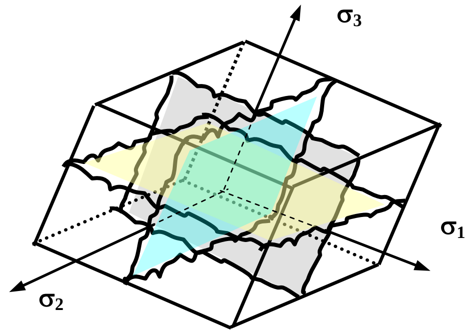
The tested models were represented by suitable finite element meshes to simulate the real boundary conditions. For slabs with opening, elements within the perimeter of the opening were omitted. Total numbers of 124, 118 and 114 elements were considered for slabs without opening, with opening 25x25 and 25×50, respectively. The stub column was represented by three elements. Fig. 24 shows the mesh used for slabs with an opening 25×50. The steel reinforcement and FRP strengthening rods were modeled as embedded fully bonded reinforcement in the concrete element in their exact position with unidirectional mechanical properties [46].

6.2. Finite element analysis versus the experimental results

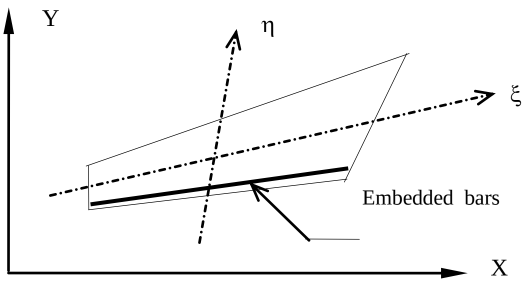
The finite element analysis is implemented for the cases of slabs strengthened with NSM reinforcement. A comparison between the numerical and the experimental results in terms of the cracking and ultimate load carrying capacity are shown in figs. 25 and 26, respectively. It is noted that the cracking loads of numerical results showed slight decreases than the experimental results. This may be attributed to the difficulties to specify the hair cracks during the test. The numerical results showed small increases than the experimental results. Specimens SNS25, SNG25 and SNC25 showed differences of 3%, 5% and 8 %, respectively, whereas specimens SNS50, SNG50 and SNC50 showed differences of 6%, 7% and 9 %, respectively. It can be observed that the finite element model can represent the tested slabs appropriately. The finite element program can be used to study different cases of strengthened specimens that are not included in the experimental program.



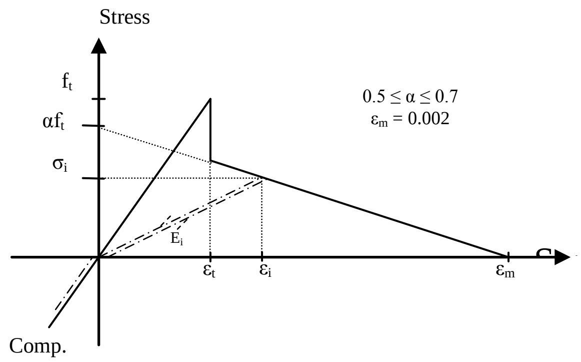
(a) 3-D isoparametric element with 20 nodes



(b) Cracks at Gauss Point



(c) Modeling of reinforcement



(d) Tension stiffening

Fig. 23. Modeling of RC test slabs and reinforcement.

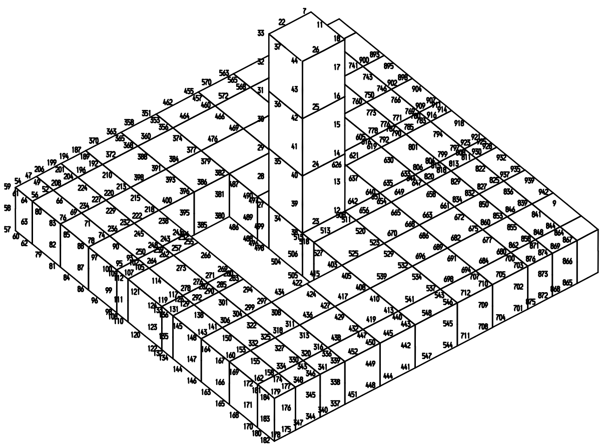


Fig. 24. FEM mesh for slab with opening 25×50.

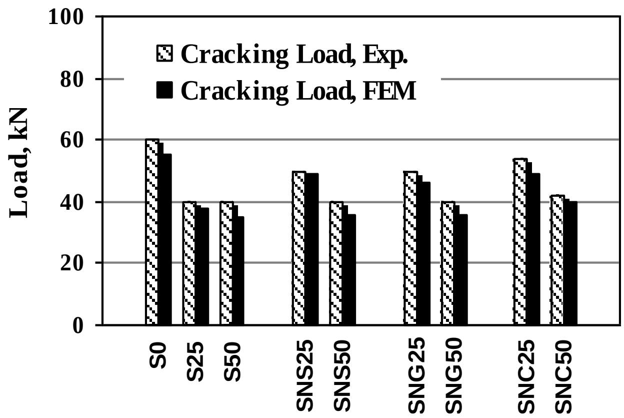


Fig. 25. Comparison between the numerical and the experimental cracking loads.

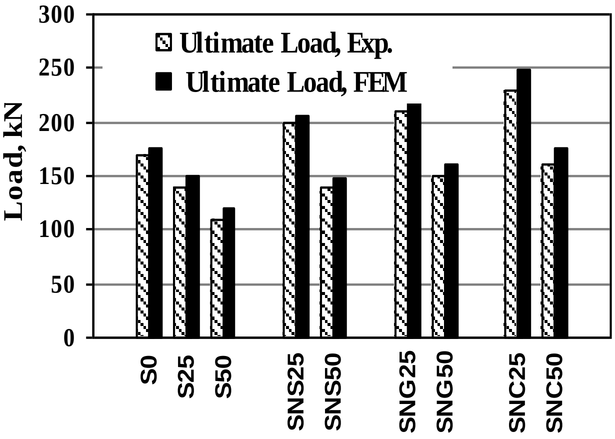


Fig. 26. Comparison between the numerical and the experimental ultimate failure loads.

7. Conclusions

Two new techniques for shear strengthening of flat slab-column connections with large openings were described, and tests were presented to evaluate these techniques. The first method depends on installing external bonded layers of FRP woven wraps over intertwined rods manufactured from warps submerged in epoxy resin and stitched through the thickness of the slab to act as stirrups. In the second technique, NSM intertwined FRP rods were utilized. The new suggested techniques are compared with the traditional method using bolts and steel plates and also with NSM reinforcement. Based on the presented experimental and analytical results, it can be concluded that:

1. Applying the new techniques of punching shear strengthening with the suggested procedure was sufficient to achieve positive results. Strengthening slabs with an opening having length equal to 1/8 of the corresponding span using one layer of CFRP and four layers of GFRP wraps showed an average gain in the ultimate load carrying capacity of about 64%, 50%, respectively. In addition, strengthening slabs having an opening 1/4 of the corresponding span using one layer of CFRP and four layers of GFRP wraps showed an average gain of about 45%, 36%, respectively.
2. The new techniques increased the ductility of the tested slab-column connections with openings as well as improved the load-deflection behavior.
3. The utilized techniques provide means for changing the failure mode from punching to flexural.
4. The new methods may be promising and attractive alternative to traditional techniques as a practical solution for strengthening existing structures with and without openings. It is comparatively simple to install, does not change the appearance of slabs, and have immunity to corrosion.
5. The finite element model with embedded reinforcement can represent the problem and can be used to study different cases of strengthened slabs that are not included in the experimental program.

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