Tests on reinforced concrete slabs with cut-out openings strengthened with fibre-reinforced polymers

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This paper presents the results of experimental investigations on reinforced concrete slabs strengthened using fibre-reinforced polymers (FRP). Eight tests were carried out on four two-way slabs, with and without cut-out openings. Investigations on slabs with cut-outs revealed that the FRP can be placed only around the edges of the cut-out when retrofitting the slabs whereas, in the situation of inserting cut-outs combined with increased demands of capacity, it is necessary to apply FRP components on most of the soffit of the slab. The proposed strengthening system enabled the load and deflection capacities of the FRP-strengthened slabs, in relation to their un-strengthened reference slabs, to be enhanced by up to 121% and 57% for slabs with and without cut-outs respectively.

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1. Introduction

The load carrying capacity of reinforced concrete (RC) slabs may be compromised for a number of reasons, including design errors, building code changes, structural damage and changes of functional use by creating new openings.

The experimental research presented in this paper deals with the structural rehabilitation of RC two-way slabs, with and without cut-out openings. One method that can be used to increase their load capacity is to apply fibre-reinforced polymers (FRP) as externally bonded (EB) or near surface mounted (NSM) reinforcement. Several guidelines for designing and applying FRPs as strengthening systems for RC structures have been published [1,2]. However, how to use FRPs to strengthen structural elements with cut-out openings is only addressed to a small extent in these guidelines.

Many researchers [3–10] tested the feasibility of restoring or improving the load capacity of solid slabs by means of EB FRPs. Despite the efficiency of the method, the majority of the retrofitted elements experienced debonding as a failure mode. To solve this challenge, several researchers [11–13] successfully tested different anchorage systems for FRPs applied as EB reinforcement on slabs.

Furthermore, plane elements (i.e. RC walls) could also be strengthened using mechanical anchored FRPs thus being efficient in preventing debonding [14]. The NSM technique, which is relatively new compared to EB, has been proven to produce better anchoring behaviour than EB [15]. This technique introduced a new debonding mode, the slip of the reinforcement in the concrete groove. However, this failure mode is preferred to the sudden debonding of EB strips [16].

In the literature, there are several studies of slabs with cut-out openings strengthened with FRP materials [17–25]. Casadei et al. [17] tested one-way slabs with both centrally located openings and openings near the supports, strengthened by carbon FRP (CFRP) laminates. This method has been proved to be effective only for the case with openings in the sagging region. The presence of the openings in the hogging region increased the shear stress in the concrete slab, leading to premature failure [17].

Lower tensile forces in the steel reinforcement accompanied by a more favourable crack distribution were important improvements when using FRP strips for strengthening one-way slabs with a rectangular cut-out in the centre of each slab [18]. Although the method produced an ultimate bearing capacity similar to the one recorded for the control element, the elements failed due to debonding.

In another series of tests, Tan and Zhao [19] found that all the strengthened slabs with symmetric and asymmetric openings that they investigated exhibited the same load capacity as un-strengthened slabs with openings, with some cases being even higher.

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Flexural failure mode was associated with small-sized openings whereas a new failure pattern with non-orthogonal yield lines initiating from the corners of the cut-out was reported for large-sized openings. The same researchers also proved that CFRP sheets are more effective compared with CFRP plates because of the premature debonding of the latter. In relation to the position of the opening, it was found that specimens with openings placed in the maximum moment zone failed in flexural mode whereas a new failure pattern with non-orthogonal yield lines initiated from the corners of the cut-out was reported for large-sized openings located in the shear zone failed in shear mode [20].

The location of load application and the type of the loading surface was believed to play an important role in determining the failure behaviour [21]. Using a line load configuration induces stress concentrations which can have a negative influence on the location where debonding starts [21].

According to [22], the NSM CFRP strips performed better than the EB CFRP plates when used for strengthening slabs with centred openings due to the greater resistance to debonding. When EB CFRP plates were used together with FRP anchors, the flexural capacity of the slab was fully restored.

Compared to one-way slabs, less research has been carried out on FRP-strengthened two-way RC slabs with cut-out openings. Casadei et al. [23] claimed to be the first to report tests on RC slabs with openings and strengthened with CFRP laminates around the cut-out. The anchorage system prevented the premature debonding of the laminates which yielded into full utilisation of the FRPs. Enochsson et al. [24] tested two-way slabs strengthened with FRP composite materials. The tests revealed that specimens with larger openings have a higher load capacity and stiffness than the ones with smaller openings. Although this contradicted their design method, Enochsson et al. [24] have justified this as “the slabs with the large openings behave closer to a system of four beams than a slab”.

El-Sayed et al. [25] proved the benefits of using mechanically-fastened EB FRPs over the conventionally applied EB FRPs. The latter provided a lower performance in serviceability compared with the mechanically-fastened technique.

De Lorenzis and Teng [26] concluded that the NSM technique is less prone to debonding, can be pre-stressed more easily and is better protected against fire, chemical and mechanical damage. However, in some cases, it could be more beneficial to use both NSM and EB techniques especially when the concrete cover is limited.

In most of the above-mentioned research programs, the cut-outs were created in the centre of the tested slabs and the applied strengthening techniques were either EB or NSM types. In this research, mixed retrofitting solutions (NSM + EB) are tested on two-way RC slabs with cut-out openings located on the sides of the element.

The first objective of the research program was to verify how the cut-out openings influence the loading behaviour of the slabs. This study also provides relevant information about the influence of the surface and position of the openings on un-strengthened slabs loaded with distributed loads on small areas.

The second objective was to investigate whether the FRP strengthening solutions can restore and increase the load capacity of slabs with cut-outs in comparison to that of the full slab and their corresponding unstrengthened slabs with openings, respectively.

2. Experimental tests

2.1. The test specimens

Four RC two-way concrete slabs were cast. The specimens were designed with a ratio between the clear length and clear width of about 1.55 (see Fig. 1) with dimensions of 2650 × 3850 × 120 mm. The clear span-to-thickness ratio was 20 for the short edge of the slabs. The elements replicate two-way single span simply-supported slabs, designed according to EN1992-1-1 [27]. The top of the slab was reinforced along its contour for constructional reasons only. Reinforcement at the bottom consisted of welded wire meshes made of bars with a diameter of 4 mm, arranged at a spacing of 100 mm in both directions parallel to the edges of the elements. The concrete cover provided for the outermost steel reinforcement bars (i.e. rebar placed parallel to the short edge of the slabs) had a thickness of 15 mm. The steel reinforcement ratios, based on the effective depth on the short and long edges, were 0.117% and 0.127%, respectively. The steel reinforcement ratio, based on total thickness, was 0.105%. Elements with openings were detailed in such a way as to replicate cut-outs sawn into a full element i.e. no additional reinforcement was placed around the edges of the openings.

The first specimen, denoted FS-01, was a full slab and served as the reference. The second slab, RSC-01, had a small opening. Two identical specimens with large openings were cast, designated RLC-01 and RLC-02. Details of their geometries are shown in Figs. 2 and 3.

2.2. Material properties

The average cubic compressive strength of concrete ($f_{cm}$) was determined based on 12 cube tests [28] at the time of testing of each slab. Three cubes were tested for each slab. All tests were carried out after 28 days. Tensile tests of the steel reinforcement were carried out on 20 samples based on specifications described in [29]. Five samples were tested for each cast slab, 4 batches in total. The properties determined were the yield stress ($f_{y}$), tensile strength ($f_{t}$) and ultimate strain ($\epsilon_{uk}$). Commercial CFRP products were used for strengthening the slabs. These products consisted of high strength NSM strips, plates and sheets. All the mechanical properties of these materials are shown in Table 1.

2.3. Design and detailing of the CFRP strengthening

The CFRP components were bonded to the soffit of the slabs in two directions. The CFRP components parallel to the short edge of the specimens were installed using the NSM technique and those on the direction parallel to the long edge of the slabs were installed using the EB technique.

The required amount of CFRP was determined using the following procedure. For specimen FS-01-FRP (the full slab), the capable tensile force of steel reinforcement was matched to that of the CFRP components to be installed. The strengthening for specimen RLC-02-FRP was similarly designed, with the only difference being that the NSM bars intercepted by the cut-out opening were placed in the immediate vicinity of the opening. This design procedure aimed to cover the scenario when a slab is damaged and strengthened to give a higher load capacity. For slabs RSC-01-FRP and RLC-01-FRP, the FRP system was designed so that its tensile capacity equalled that of the steel reinforcement that was removed when the slabs were sawn. This second procedure aimed to test whether the capacity of the slab can be restored to its un-strengthened, undamaged state using FRP. See Fig. 2 for the details and geometrical properties of the applied strengthening.

Strengthened reference specimens have had the suffix FRP added to their nomenclature. For example, RSC-01-FRP refers to a reinforced concrete slab with a Rectangular Small Cut-out which has been strengthened.

2.4. Test setup, loading protocol and instrumentation

It was planned to load each slab beyond the point where the tensile reinforcement yielded, then unload, apply the FRP
strengthening and test again until collapse. In total, this testing regime yielded eight tests performed on four slabs.

The test setup consisted of a 1 m high discontinuous peripheral wall made out of brick masonry and reinforced concrete beams, a rigid loading frame and a hydraulic jack. The load was distributed over a central patch of $600 \times 1200$ mm through a spatial steel assembly, and was applied in controlled increments of 5 kN. A vertical cross-section through the test setup is shown in Fig. 1.

![Fig. 1. Test setup, top view of the test setup with dimensions of the slab and general overview of the test setup.](image)

The CFRP strengthening systems applied are shown in Fig. 2.

![Fig. 2. Detailing of the CFRP strengthening systems applied.](image)
The positioning of the displacement transducers (D) and strain gauges installed on the reinforcement (R) and the cracking pattern resulting after loading the reference specimens.

Table 1
Mechanical properties of the materials.

| Element       | Concrete $f_{cm}$ (MPa) | Reinforcement $f_y$ (MPa) | $\varepsilon_{uk}$ (%) | $E_{FRP}$ (GPa) | $E_{NSM}$ (GPa) | Plates $E_{NSM}$ (GPa) | $\varepsilon_{FRP}$ (%) | Sheets $E_{NSM}$ (GPa) | $\varepsilon_{FRP}$ (%) |
|---------------|-------------------------|--------------------------|------------------------|-----------------|----------------|------------------------|------------------------|------------------------|------------------------|
| FS-01(-FRP)   | 65                      | 596.7                    | 2.7                    | 165             | 165           | 165                    | 231                    | 1.7                    |
| RSC-01(-FRP)  | 62                      | 537.2                    | 2.4                    | 165             | 165           | 165                    | 231                    | 1.7                    |
| RLC-01(-FRP)  | 66                      | 546.1                    | 3.4                    | 165             | 165           | 165                    | 231                    | 1.7                    |
| RLC-02(-FRP)  | 62                      | 548.3                    | 3.1                    | 165             | 165           | 165                    | 231                    | 1.7                    |

The slabs were laid on a layer of fresh mortar, which permitted horizontal settling under their own weight. The supporting area had a width of 125 mm. This type of support prevented the maximum recorded vertical deflection reached the allowable deflection (i.e. $L/250 = 9.6$ mm) according to EN1992-1-1 [27].

position of the load patch (i.e. the centre of the full slab) was maintained throughout all 8 tests, regardless of the geometry of slabs with cut-outs; even if asymmetrical, it provided an un-favourable type of loading for all specimens. All slabs were pre-cracked and the loading was stopped to avoid total collapse when the maximum recorded vertical deflection reached the allowable deflection (i.e. $L/250 = 9.6$ mm) according to EN1992-1-1 [27].
gravitational displacements but allowed the uplift of the corners and edges of the slabs. Ten displacement transducers were installed to measure the deflection of the slabs, as shown in Fig. 3. The location of the transducers was fixed for all 8 tests. However, between tests, some were removed as they would be located inside the area of a cut-out opening. For each specimen, 4–6 strain gauges were installed on the bottom steel reinforcement, located as shown in Fig. 3. Strain gauges were also installed on the FRPs to monitor the strain at debonding; their positions are highlighted in Fig. 4. The locations of the displacement transducers are similar to the ones used for the reference tests. Due to space limitations, only some selected deflections and strain measurements are presented here, thus for further details, see [30].

3. Results and discussions

3.1. Tests on reference slabs

The results of the tests are shown in Table 2. Fig. 5 shows graphs of the load displacement response recorded for all tests. The full slab (FS-01) developed the highest load capacity while the slabs with the largest openings (RLC-01 & 02) had the lowest capacity. The crack pattern of the full slab (FS-01) developed along the yield lines and under the loading area (Fig. 3). The cracks under the loading area indicate punching failure of the slab under the loading surface, see Fig. 6. The crack pattern of the slab with small-sized opening (RSC-01) initiated at the re-entrant corner of the cut-out.
and developed in a direction quasi-parallel to the long edge of the specimen (Fig. 3).

The slabs with large openings, RLC-01 & 02, had identical geometries; however, the ultimate capacity was different. A 10% higher capacity in the favour of the former was recorded. The origin of this difference was identified after testing. Due to a test procedure error with slab RLC-02, the outermost steel reinforcement was placed along its length, while for all other slabs, the outermost steel reinforcement was placed along their short side. This misplacement decreased the internal lever arm of the structural reinforcement, thus reducing its capacity. By creating the two types of cut-outs in the three slabs, the slabs’ area decreased to 86.71% (by creating the small cut-out) and to 74.74% (by creating large cut-outs) of the total area of the full slab. Both the size and location of the cut-out opening influenced the load capacity. Although the area of RLC-01 is 10% smaller than RSC-01, the ultimate loads are relatively similar. The elastic limit was reached when at least one strain gauge indicated a value of the strain (\( \varepsilon_y \)) presented in Table 2. No tension stiffening effect was accounted for in the evaluation of these strains.

3.3. Test predictions by yield line theory

3.3.1. The yield line theory

The yield line theory was presented by Ingerslev [31] and further developed by Johansen [32] and Wood and Jones [33]. This method predicts the load at which the flexural capacity of slabs is reached using the rigid plastic theory in accordance to the upper bound theorem. The procedure employs the use of predefined crack patterns (yield lines) [33]. Different layout patterns of the yield lines can be assumed resulting in several upper bound solutions. For design purposes the minor value is chosen. The failure load can be calculated using two different techniques: (1) the virtual-work method and (2) the equilibrium method. The virtual-work method assumes that at collapse the work done due to a virtual imposed displacement is equal to the internal work dissipated along the yield lines [33]. As an alternative, the equilibrium method differs from the work method “in that the equilibrium of each of the rigid regions is considered” [33]. The two techniques yield the same results; therefore here the capacity of the slabs was computed using the virtual work method only (Eq. (1)).

\[
\sum \left[ \int q \, dx \, dy \right]_{\text{each region}} = \sum \left[ \int \varepsilon_y \, m_b \, ds \right]_{\text{each yield line}}
\]

The left-hand side of Eq. (1) represents the external work, with \( q \) denoting the load on unit area and \( \varepsilon_y \) the virtual displacement, while the right-hand side represent the internal work with \( \varepsilon_y \) denoting the normal rotation of the yield line and \( m_b \) the capable moment along the yield line.

The ultimate bending moment along the yield line can be found considering the equilibrium condition shown in Fig. 10:

\[
m_b L = (m_e - L \sin \alpha) \sin \alpha + (m_v - L \cos \alpha) \cos \alpha
\]

\[
m_b = m_e \sin^2 \alpha + m_v \cos^2 \alpha
\]

Table 2

| Element       | Load (kN) | Cracking load (kN) | S (%) | R (%) | F (kN/m²) | MD (mm) | \( \varepsilon_y \) (%) | FY (kN) |
|---------------|-----------|--------------------|-------|-------|-----------|---------|-------------------------|---------|
| FS-01         | 118       | NA                 | 100   | 100   | 13.4      | 10.3 (D1) | 0.269                   | 46.75 (R1) |
| RSC-01        | 87        | 65                 | 87    | 73    | 11.3      | 11.4 (D4) | 0.273                   | NA      |
| RLC-01        | 75        | 60                 | 75    | 63    | 11.3      | 9.6 (D7)  | 0.274                   | 61.75 (R3) |
| RLC-02        | 67        | 55                 | 75    | 57    | 10.1      | 9 (D7)   | 0.298                   | 61.5 (R1) |
| FS-01-FRP     | 186       | 100                | 100   | 100   | 20.9      | 45 (D1)  | –                       | –       |
| RSC-01-FRP    | 86        | 46                 | 87    | 46    | 11.1      | 33 (D1, D2, D4) | –                       | –       |
| RLC-01-FRP    | 75        | 40                 | 75    | 40    | 11.2      | 8.5 (D7)  | –                       | –       |
| RLC-02-FRP    | 147       | 79                 | 75    | 79    | 22.2      | 62.2 (D1) | –                       | –       |

\( S = \) ratio, expressed in %, between the surface of one specimen with an opening and that of the full slab. \( R = \) ratio, expressed in %, between the load of one specimen with an opening and that of the full slab. \( F = \) normalised load at the surface of one specimen. \( D = \) displacement at maximum load. \( MD = \) maximum displacement at maximum load, with the transducers that recorded the values given in parentheses. \( FY = \) First yielding load with the gauges that recorded the values given in parentheses.
where $m_x$, $m_y$ are the moment capacities per unit width in the $x$- and $y$-directions, respectively, calculated according to [27]:

$$m_{x,y} = A_{x,y} \cdot f_{yk} \cdot 0.9d$$

(4)

where $A_{x,y}$ are the areas of the reinforcement per unit width and $d$ is the effective depth.

If the slab is isotropically reinforced (i.e. $m_x = m_y$), Eq. (3) reduces to $m_b = m_x = m_y$. In the present study, due to differences in effective depths along the two axes, the $m_x$ and $m_y$ moments are slightly different. For simplicity these differences were disregarded in these calculations.

The angle between axis of rotation of each region and yield line determines the slope of the yield line. For a full slab these yield lines intersect the corners at 45°. Due to symmetry along the longitudinal axis, the same assumptions can be made for the slab with large cut outs. It was shown by Kennedy and Goodchild [34] that assuming 45° will produce only a 3% error compared with theoretically determined angle.

For slabs with asymmetric openings, different yield line patterns can be assumed, depending on the opening size and position,
as noted by Park and Gamble [35]. For slabs with openings at corners, Park and Gamble [35] proposed different possible yield line patterns that are most likely to occur function of the opening size. The equation for finding the ultimate load of each pattern type includes several unknown terms (i.e. $a$, $b$, $x$) which define the theoretical positions of the yield lines. The exact values for these terms are determined by differentiating the constitutive equations and finding the maximum value for $\frac{\partial q}{\partial x} = 0$, $\frac{\partial q}{\partial y} = 0$, $\frac{\partial q}{\partial \lambda} = 0$. This mathematical procedure is laborious due to nonlinearity of the equations. Moreover, Wood and Jones [33] suggested that such a technique may not always be used due to discontinuity in the slab boundaries. In this study the layout of the yield lines are assumed to start from the re-entrant corner of the cut-out opening. The failure load per unit area was derived from Eq.(1) for each slab. Due to space limitations only the final solution will be stated as follows: Eq.(5) – ultimate uniformly distributed load/unit area for a full slab; Eq.(6) – ultimate uniformly distributed load/unit area for slab with small cut-out opening; Eq.(7) – ultimate uniformly distributed load/unit area for slab with large cut-out opening.

In this study two different approaches were used: (1) pre-tests predictions: the yield line theory was applied assuming the theoretical distribution of the yield lines (“pre-test yield lines” in Fig. 11) and (2) post-tests predictions: the yield line theory was applied to the real crack pattern observed on the tested slabs (“post-test yield lines” in Fig. 11). All slabs were simply supported along their contour, therefore only positive yield lines have developed.

3.3.2. Pre-test predictions

The following assumptions were made for pre-test predictions:

(a) the resisting moment of the un-strengthened slabs was evaluated assuming that the steel reinforcement intersected by the yield lines is yielding, the value of the yield stress, $f_{yk}$, was assumed as in Table 1

(b) the FRP reinforcement around openings was uniformly distributed over the entire surface of the slab

(c) the resisting moment of the FRP-strengthened slabs was evaluated assuming only the strength contribution of the FRP strengthening. The steel reinforcement, considered already yielded, is neglected. The yield stress, $f_{yk}$, was replaced in Eq.(4) with the strength of FRP corresponding to its rupture strain (i.e. 1.7% for all FRP components)

Fig. 11. Pre and post-test yield line patterns for both strengthened and unstrengthened slabs.
Using this approach the capacities of the slabs predicted by yield line theory are overestimated compared to those from tests, see Table 3. These predictions are not accurate because the crack pattern observed in tests did not developed according to the one assumed in the yield line theory. The reason of these deviations lies in the load strategy adopted. The loading system was not able to simulate a uniformly distributed load over the entire surface. Under high loads the slabs partially lifted from the supports, therefore changing the stress distribution in the slab towards the supports, and consequently the cracking pattern.

3.3.3. Post-test predictions

The assumptions presented above are valid for this approach also. However for specimen RSC-01-FRP the value of the yield stress was computed using the debonding strain; recall that this strengthening system failed by debonding, see Fig. 8. For this specimen debonding occurred at the far end of the slab where strain gauges were not installed (see Figs. 4 and 7). Simple to more complex models can be used for the evaluation of the bond strength [36–38]. D’Antino and Pellegrino [36] reviewed the performance of several bond models. That work did not indicate any model to predict accurately the strains at debonding. Therefore here the Fib Bulletin 14 [1] formulation was used to estimate the strain at debonding; the bond strain resulted is 0.8%.

The ultimate capacity of both strengthened and un-strengthened slabs were calculated using the yield lines observed from tests (Fig. 11). Table 3 shows that the average ratio between experimental values and those predicted based on the real crack pattern are more accurate than those predicted by theoretically assumed ones. For all elements, the predicted values were within the acceptable 10% limit [35] except for slab RLC-01 being however, on the conservative side.

### Table 3
Comparison of the ultimate load with analytical predictions.

| Test     | Slab area (m²) | Test results | Yield line predictions |
|----------|----------------|--------------|------------------------|
|          |                | q (kN/m²)    | Pₜₜ (kN) | Pre-tests predictions | Accuracy | Post-tests predictions | Accuracy |
| FS-01    | 8.88           | 13.29        | 118      | 20.38  | 181 | 0.65 | 13.74 | 122 | 0.97 |
| RSC-01   | 7.70           | 11.30        | 87       | 23.50  | 181 | 0.48 | 11.69 | 90  | 0.97 |
| RLC-01   | 6.64           | 11.29        | 75       | 14.91  | 99  | 0.76 | 9.34  | 62  | 1.2  |
| RLC-02   | 6.64           | 10.09        | 67       | 14.31  | 95  | 0.71 | 9.04  | 60  | 1.11 |
| RSC-01-FRP | 7.70         | 11.17        | 86       | 14.55  | 112 | 0.77 | 10.65 | 82  | 1.05 |
| RLC-01-FRP | 6.64         | 11.29        | 75       | 11.75  | 78  | 0.96 | 12.95 | 86  | 0.87 |
| RLC-02-FRP | 6.64         | 22.14        | 147      | 23.49  | 156 | 0.94 | 20.63 | 137 | 1.07 |
| Average  |                |              |          |        |    | 0.77 | 20.63 | 137 | 1.03 |
| Standard deviation |            | 0.16         |          |        |    | 0.10 |

- The results for slabs RSC-01-FRP and RLC-01-FRP showed that using a quantity of FRP equivalent to the steel reinforcement removed by sawing the cut-out, the capacity of the slab can be restored fully, even when damaged prior to strengthening.
- In order to restore and increase the capacity beyond the design value of the un-strengthened slab, the strengthening system was designed to replace the reinforcement in the slab. Tests on slabs FS-01-FRP and RLC-02-FRP showed an increase in ultimate capacity of up to 57% and 121% respectively, compared to the values recorded during the tests on the un-strengthened specimens FS-01 and RLC-02 respectively. Slab RLC-02-FRP had the highest ultimate capacity relative to its effective area.
- The tests have also shown that debonding problems can be avoided by using the NSM technique. In all tests, the strengthening systems primarily failed due to rupture. The debonding of the FRP plate used in the test RSC-01-FRP was due to a secondary failure mode, since it occurred at the far end of the slab. It is unclear to the authors whether the loading system used produced a favourable effect on the bonding properties; this research subject needs further investigation.
- Because of their superior mechanical properties compared to steel reinforcement, the FRPs enabled better stress redistribution and, consequently, a more uniform cracking distribution. The new formed major cracks show that this behaviour was due to the un-damaged part of the steel reinforcement that had yielded during the control testing.
- One practical problem in applying the yield-line method is that designers must consider a large number of failure mechanisms to ensure that the lowest collapse load is found. This procedure implies lengthy calculations and skilled engineering to ensure that the right collapse mechanism is chosen, especially in cases like slabs with openings, were the general assumptions are not fully applicable. In this respect, analytical pre-tests predictions lead to un-conservative values whereas for post-tests prediction indicated a good approximation. However, for design purposes it is not common to carry out laboratory investigations. Perhaps numerical analysis could be a tool to overcome this challenge.
- This study tested four types of strengthening configurations using high strength FRP. How different strengthening configurations and different FRP material properties might influence the capacity of slabs with cut-out openings is a subject for future work.

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