PUNCHING SHEAR RESISTANCE OF FLAT SLABS STRENGTHENED WITH NEAR SURFACE-MOUNTED CFRP BARS

Hikmatullah Akhundzada (Ph.D)\textsuperscript{1}, Ted Donchev (Ph.D)\textsuperscript{2}, Diana Petkova (Ph.D)\textsuperscript{3}

\textsuperscript{1} PhD Student, at Kingston University, Penrhyn Rd, Kingston upon Thames, KT1 2EE, UK. Email: hikmat09@hotmail.com
\textsuperscript{2} Associate Professor, at Kingston University, Penrhyn Rd, Kingston upon Thames, KT1 2EE, UK. Email: t.donchev@kingston.ac.uk
\textsuperscript{3} Senior Lecturer, at Kingston University, Penrhyn Rd, Kingston upon Thames, KT1 2EE, UK. Email: d.petkova@kingston.ac.uk

ABSTRACT

This paper presents the effectiveness of strengthening slab-column connections against punching shear failure with near-surface mounted (NSM) carbon fibre-reinforced polymer (CFRP) bars. The experimental program consists of preparing and testing eight samples, two control and six strengthened samples. The main variables of the experiment are the strengthening layout and the cross-section area of CFRP bars. The results show that NSM strengthening increases the ultimate load by up to 44%. And the strengthening delays formation of the first crack in concrete thus maintaining a linear behaviour for load-displacement and load-strain curves for higher level of load. The NSM strengthening increases the flexural stiffness by over 100% and maintains a strong bond with concrete throughout the loading. The flexural strength of the slab increases, which subsequently improves the punching shear capacity. The experimental results are compared with several design codes by modification and implementation of Chen & Li’s method. There is a good agreement between the calculated ultimate capacity of the strengthened samples and the obtained experimental results.

Keywords: Flat Slabs; Reinforced Concrete; Punching Shear; Strengthening; NSM; CFRP.
INTRODUCTION

With the development of novel materials, the strengthening methods against punching shear failure for flat slabs has evolved over the decades. The traditional methods for increasing the punching shear resistance are: (i) Increasing the depth of the slab; (ii) Post-installation of shear reinforcement; (iii) Enlargement of the column head with concrete; (iv) Enlargement of the column head with a steel structure; and, (v) Increasing the cross-section of the column (Elbakry et.al 2015; Ruiz 2011). Although these techniques are shown to be effective in practice, there are certain limitations such as susceptibility to corrosion, high self-weight, and difficulties in installation. The use of fibre-reinforced polymers (FRP) overcomes these shortcomings and can potentially become a feasible alternative for the current methods.

The existing literature mainly focuses on two methods for strengthening slab against punching shear failure: direct shear strengthening and indirect flexural strengthening. In the flexural strengthening method, FRP materials (sheets, laminates, or bars) are bonded to the tension surface of the slab to act as flexural reinforcement. In the direct shear strengthening method, vertical holes are drilled through the slab. The FRP is placed inside the holes and the cavity is filled with epoxy adhesive to bond the FRP with concrete. These methods enhance the load-carrying capacity of flat slabs by either delaying the punching shear failure or changing the failure mode to flexural or flexural punching.

Externally bonded reinforcement (EBR) is the most commonly used method for strengthening concrete structures with FRP. However, the major disadvantage of FRP EBR strengthening is the premature debonding of the FRP from the surface of the slab (Bilotta et.al 2015). Recently, the researchers are focusing on the use of near-surface mounted (NSM) strengthening of beams and slabs as an alternative to the EBR approach. NSM strengthening method is reported to have many advantages over EBR strengthening such as stronger bond with concrete, protection against accidental impact and higher fire resistance due to embedment of FRP reinforcement in the concrete (De Lorenzis 2007). Bilotta et.al (2015) investigated the efficiency of NSM and EBR
flexural strengthening of RC elements. They concluded that the NSM method is less sensitive to
debonding and is more effective in increasing the peak-load. Seo et.al (2013) found that the NSM
 technique exhibits 1.5 times higher bond strength and shows higher magnitude of strain compared
to the EBR technique for flexural strengthening of RC beams. An experimental investigation was
conducted by Hassan and Rizkalla (2004) for flexural strengthening of RC beams with NSM
CFRP bars. The strengthened samples displayed significantly higher ultimate load, yield load,
and post-cracking stiffness. The authors proposed a minimum anchorage length of 800 mm and a
maximum usable strain of 0.7-0.8% of the CFRP bars.

Agbossou et.al (2008) investigated the effectiveness of strengthening slab-column connections
with externally bonded CFRP sheets. They reported increasing the ultimate capacity of the
strengthened samples by 15 – 30% which was directly proportional to the number of layers of
CFRP strips. Esfahani et.al (2009) investigated the strengthening of interior slab-column
connections with CFRP sheets. The strengthened samples displayed higher ultimate capacities
compared to the control samples. The improvement due to punching shear strengthening with
CFRP sheets was more prominent for slabs made with low amount of steel reinforcement and
high strength concrete. In a similar study, Harajli & Soudki (2003) suggested that the punching
shear capacity of slab-column connections can be enhanced by up to 45%. The efficiency of the
strengthened specimen could further improved by increasing the width of CFRP strips. Similarly,
Faraghaly & Ueda (2011) concluded that the punching shear capacity of slabs could be increased
by up to 40% with EBR CFRP sheets. Increasing the width of CFRP sheets directly enhanced the
ultimate capacities of the strengthened specimens. Akhundzada et.al (2019) found that the use of
non-bolted transverse anchorages delayed the debonding of CFRP laminates which subsequently
increased the punching shear capacity. The anchorages retain over 90% of the residual strength
after reaching their peak load. In a similar study, Akhundzada et.al (2018) proposed that the
orthogonal positioning of the CFRP laminates is more efficient compared to diagonal positioning
of the laminates for enhancing the punching shear capacity of flat slabs. Abdullah et.al (2013)
investigated the use of prestressed CFRP plates for punching shear strengthening of slabs. The slabs strengthened with prestressed CFRP plates displayed significantly lower ultimate load over non-prestressed slabs. In a similar study, Kim et.al (2010) found that the use of prestressed CFRP plates can improve the punching shear capacity by up to 20%. The difference in their findings could be attributed to different anchorage length, steel reinforcement ratio and positioning of the CFRP plates. In the studies above, the rupture of FRP was not reported and the magnitude of the strain in FRP was low. El-Salakawy et. Al (2004) studied the punching shear behaviour of edge column retrofitted with CFRP and GFRP sheets. Some of the slabs were additionally retrofitted with steel bolts acting as shear reinforcement. The samples without shear bolts failed in punching shear while the samples with shear bolts failed in flexure. The samples with flexural CFRP and GFRP sheets increased the ultimate load by up to 23% while the samples with additional shear bolts increased the ultimate load by up to 30%. Chen & Li (2004) investigated the influence of flat slabs with bi-directional GFRP sheets. They found that the ultimate load can be improved by up to 45% and 95% when using single-layer and double-layer GFRP sheets, respectively.

Abdul-Kareem (2019) studied the punching shear strengthening of slab-column connections with the EBR and NSM method. The NSM CFRP reinforcement had a square shape and was positioned in the tension surface of the slab, around the vicinity of the column in orthogonal and skewed configuration. The authors concluded the NSM strengthening method is twice more efficient in increasing the punching shear capacity compared to the EBR method. Azizi & Talaeeitaba (2019) conducted a numerical analysis of strengthening flat slabs with CFRP rebars in grooves (EBRIG) and on grooves (EBROG) method. The punching capacity of the numerical strengthened samples improved by up to 62%. George & Mohan found that the EBROG method with FRP could improve the punching shear capacity of flat slabs by up to 58%.

A significant number of studies have concluded that the direct shear strengthening of flat-slabs against punching shear failure with FRP materials (sheets, strands, rods and bolts) is highly efficient and can change the failure mode from punching to flexural or flexural punching (Binici
This research aims to experimentally investigate the efficiency of the NSM method to strengthen slab-column connection against punching shear failure. The two main variables of this study are: the strengthening layout and the cross-sectional area of CFRP bars. The experimental results are analysed and compared with the predictions of the design codes based on the development of analytical modelling. The presented research is a continuation of previously published work about EB strengthening of slabs against punching shear failure by the authors (Akhundzada et.al 2018; Akhundzada et.al 2019).

EXPERIMENTAL PROGRAMME

Test specimens

The proposed research consists of preparing and testing eight slabs with a central column to present an internal two-way spanning slab-column connection. The slabs have dimensions of 1500×1500×120 mm and column head of 150×150×150 mm as shown in Fig 1. All slabs are reinforced with top (tension) reinforcement of 15H8 @100 c/c and bottom (compression) reinforcement of 8H6 @200 c/c. The column head is reinforced with 4H10 L-bars and 3 No. 6 mm links. The slabs with the above parameters were chosen to ensure that punching shear failure occurs within the test slabs. Two slabs serve as control samples and the remaining six slabs are strengthened with CFRP bars.

The tested specimens are prototypes of an actual flat slab, scaled down by a factor of 0.5. The actual slab is 240 mm thick and is supported by a grid of columns at 6x6 m. The slab has hogging reinforcement of H16 @200 c/c. The spacing of the hogging reinforcement is adjusted to ensure the maximum spacing of rebars is within the allowable limits. The tested slabs only represent the junction between the column and the slab where punching shear failure is supposed to occur.
Material properties

Ready-mix concrete was used for the experiment to imitate real-life construction. The concrete was produced by mixing natural (Thames Valley) aggregates and sand in Portland cement with water to cement ratio of 0.6. One batch of concrete was used to cast the slabs. The concrete was then cured for two weeks by covering it with wet hessian sheets and at a temperature of 26°C. The characteristic cylindrical compressive strength and the characteristic tensile strength of the concrete was determined during the testing day (28 days) according to BS EN 12390-13 (BSI, 2013). The compressive cylinder strength $f_c$ was 30 MPa and the tensile strength $f_{ctm}$ was 2.8 MPa with a standard deviation of 0.65 and 0.71 respectively.

The steel reinforcement has a characteristic yield strength of 500 MPa and is designated as grade 500 (BS, 2005).

The CFRP bars used in this experiment had a spirally wound surface to ensure improved bond with the concrete. Fig 2 shows the cross-sectional area of the CFRP bars used in this experiment. The CFRP bars had a tensile strength of 1800-2200 MPa and elastic modulus of 140-150 GPa with a minimum rupture strain of 0.0129. The CFRP bars had a fibre content of 63%. The surfaces of the CFRP bars were treated with epoxy and were additionally threaded to create a spirally wound surface. The values for the mechanical properties of the CFRP bars were provided by the manufacturer and were based on the nominal cross-sectional area. A commercially available two-component (resin and polyamine hardener) structural adhesive was used to bond the CFRP bars to concrete. The structural adhesive (WEBER, 2021) had a compressive strength of 85 MPa and tensile strength of 17 MPa. The elastic modulus for the epoxy adhesive was 9.8 GPa.

Strengthening with CFRP

The grooves at the tension surface of the slab for this research were created by placing timber strips into fresh concrete and were removed after hardening. The use of wooden strips or foam for creating the grooves in concrete is widely used in NSM related research (Novidis et.al 2007;
Wahab et al. 2011; Gopinath et al. 2016). The cross-sectional geometry of the grooves was square, and its dimensions were $(1.5d_b \times 1.5d_b)$ where $d_b$ is the diameter of the CFRP bars. Fig 3 shows the typical groove detail of the 8mm CFRP bar. The groove dimensions fell within the optimum values indicated by Lee et al. (2013). The CFRP bars were cleaned with white spirit and the dust was removed from the grooves to ensure a stronger bond between the CFRP bars and the concrete. The epoxy adhesive was mixed with a paddle mixer and put onto a mortar gun. The grooves were then partially filled with the mortar gun and the CFRP bars were placed and pressed into the grooves. The remaining cavity was filled with epoxy and the surface was flattened with a trowel. The process of preparation and strengthening the slabs is shown in Fig 4.

**Strengthening layout**

The details of the strengthened samples with NSM CFRP bars as shown in table 1. The CFRP bars are positioned in orthogonal directions by using two strengthening layouts as shown in Fig 5. Four bars are used for strengthening layout one and eight bars are used for strengthening layout two. The bars are positioned at a distance of 60 mm from the perimeter of the column at the tension surface of the slab in the first layout. In the second layout, the bars are positioned around the perimeter of the column and also at a distance of 120 mm from the perimeter to the column. The two strengthening layouts chosen for this research effectively intercepts the punching shear crack and is expected to utilize the maximum capacity of the strengthening material. The chosen layouts will allow for the development of dowel forces in the CFRP reinforcement at the intersection point with the inclined shear crack as indicated in Fig 5. Three different bar diameters are used for each of the strengthening layouts i.e., 6 mm, 8 mm and 10 mm.

**Instrumentation and test set up**

The response of the slab under monotonic loading was monitored by the instrumentation shown in Fig 6. Five linear variable displacement transducers (LVDTs) were used to measure the vertical displacement of the slab. The LVDTs were positioned at the middle, quarter-span and close to
support of the slab. Three strain gauges (SGs) were attached at the mid-point of steel
reinforcement to monitor the development of strain in steel reinforcement. Furthermore, four SGs
were attached to the mid-point of the CFRP bars and additional six strain gauges were attached
to the tension surface of the concrete. Four dial gauges (DGs) were positioned over the supporting
frame to measure the movement of the testing rig.

The test set up is shown in Fig 7 and 8. The load was applied through the column head in an
upward direction. Eight rectangular hollow sections (RHS) columns were bolted to a strong
concrete floor to provide support for the slab. The slab was supported on top and bottom by steel
angles bolted to RHS. Smooth surface steel bars were placed between the slab and the angles to
allow free rotation of the slab at the edges. The slab was simply supported on four sides. A stress
distributor plate was placed under the column to prevent localized crushing of the concrete. A
load cell was positioned over the hydraulic jack to monitor the load. The minor deformation of
the testing rig was monitored throughout the loading and was taken into consideration during the
analysis. The displacement measuring instruments were supported by a light steel frame built
above the testing rig and was not connected with the rig. The load application was force-controlled
and was applied at a rate of 1 kN/min and the readings were captured at a rate of 0.1 sec.

EXPERIMENTAL RESULTS

Failure modes

All of the tested samples failed under a classical punching shear failure at the point of ultimate
load. With the increasing level of load, the initial cracking developed in the radial direction. The
punching shear circular crack started to develop away from the perimeter of the column towards
the later stages of loading. A sudden drop in the load was observed after reaching the maximum
capacity, which is considered as the failure point. The column head with a truncated slab section
was physically separated from the slab. In all cases, the failure was abrupt and happened without
initial warning signs. The CFRP bars kept a strong bond with the concrete throughout the loading process. The rupture and bond failure of CFRP bars were neither observed nor recorded during the test. A typical failure of one of the strengthened samples is presented in Fig 9.

**Load-displacement response**

The mid-span deflection of the slab is taken as the difference between the deflection of the slab and the average vertical displacement of the supporting frame. The load-displacement relationship at the centre of the slab for strengthening layouts 1 and 2 are shown in Fig 10. This relationship was linear for all the slabs before the formation of the first crack in the concrete. In radial direction the first crack appeared at a loading level of between 40-50 kN for all samples. At this stage, the slabs displayed a stiff response which could be attributed to the un-cracked concrete section. After the first crack, the load-displacement relationship was majorly dependent on the cross-sectional area of CFRP bars and the strengthening layout.

The first circular crack around the perimeter of the column for control samples CS1 and CS2 started to shape at load of 110-120 kN which caused higher deformability in the load-displacement graph Fig 10. The deformability of the control samples kept increasing as the rate and number of cracks started to increased. The average deflection at the centre of the slab for the samples strengthened with layout one (L1-6, L1-8, L1-10) and layout two (L2-6, L2-8, L2-10) was correspondingly 38% and 41% lower compared to control samples at the point of maximum load. The deformability of the samples strengthened with CFRP bars was lower compared to control samples. Slabs L1-6 and L2-6 exhibited higher deformability amongst the strengthened cases after reaching a load of 135 kN. The larger deformability of these samples could be attributed to small bar diameter allowing for relatively larger deflection throughout the loading.

**Flexural stiffness**

The flexural stiffness is defined as the ratio between the ultimate load and the maximum deflection at the mid-point of the slab. This ratio explains the deformability of the samples in relation to
their ultimate load as indicated in Fig 11. The flexural stiffness is calculated in two stages, before
and after the concrete cracking.

In general, the strengthened samples displayed significantly higher stiffness compared to the
control samples during the two stages. The difference in the stiffness of the control and
strengthened samples are relatively low before the cracking of the concrete. As the concrete starts
to crack, the difference increases accordingly. On average, the increase in stiffness for the samples
using strengthening layout one and two were 1.76 and 2.75 respectively before the cracking of
the concrete. However, this ratio increased to 2.11 and 3.6 for the two strengthening layouts after
cracking of the concrete. It could be extrapolated that the degree of dowel action from the CFRP
bars contributing towards higher stiffness of the slab is higher after cracking of the concrete.

The increase in stiffness is directly proportional to the increase in cross-sectional area and the
number of CFRP bars. The samples with strengthening layout 2 displayed higher stiffness
compared to the samples with strengthening layout 1 due to higher number of CFRP bars. The
samples with larger diameter of CFRP bars exhibited higher stiffness within their corresponding
strengthening layout. Sample L2-10 displayed the highest and sample L1-6 displayed the lowest
increase in stiffness compared to the average stiffness of the control samples.

**CFRP strains**

The load-strain curve at the mid-point of the CFRP bars is shown in Fig 12. The linear behaviour
of the load-strain curve at the initial stages of loading shows that the concrete is not cracked. This
behaviour changes after initiation of micro-cracking and development of substantial cracks in the
tension surface of the slab.

The CFRP and the concrete maintained their bond which did not fail under the increasing
monotonic loading during the whole process of testing. This behaviour is well illustrated by the
increasing level of strain in relation to the increase in load. The rupture of the CFRP bars was not
observed during the test and the slabs failed by the formation of the circular punching shear crack
in the concrete. The bars were exposed via removing the epoxy cover after the failure to check for their integrity and it was cross-checked with the data from strain gauges.

The CFRP bars reached up to 45% of its rupture strain before failure of the slab. The strain utilization was relatively higher for samples L1-6 and L2-6. On the other hand, samples L2-8 and L2-10 exhibited the lowest level of strain at any given point of loading amongst all other strengthened samples.

Concrete strain

The concrete strain in the radial direction at the centre, quarter-span, and end of the slabs is shown in Fig 13. The strain readings presented are taken at the peak-load before failure of the slabs. The strain profile along the loading span is similar to a natural distribution curve i.e., it is highest at the centre of the slab and exponentially decreases with the distance along the span. The slabs developed severe cracking at the point of maximum load reaching strains of 0.02.

Eurocode 2 (EC2 2004) requires checking the shear resistance at the face of the column and at the basic control perimeter of $2d$ (where $d$ is the effective depth of the slab) from the face of the column. The critical section of $2d$ is shown in dotted line in Fig 15. The capacity of the slab should exceed the applied shear forces at these critical perimeters. The stress concentration is significantly lower outside these perimeters. This shows that the maximum concentration of stresses are within basic the control perimeters and the punching shear failure plane is likely to form inside this region, for slabs without shear reinforcement.

Cracking

When the slabs were subjected to vertical load, the first cracks were formed in the radial direction at the tension surface of the slabs. A circular crack around the perimeter of the column started to develop at a load level of 70-100 kN as shown in Fig 14. The radial cracks kept increasing in the circumferential direction. The punching shear crack started to develop after a significant increase in load away from the face of the column. The failure occurred by full physical separation of a
truncated conical surface from the remaining parts of the slab. The cracking was detected by visual observation and recorded throughout the testing.

The first crack for both control samples CS1 and CS2 appeared at a load of around 41 kN and formed at random locations on the tension surface of the slabs. The formation of the first crack for samples with strengthening layout one L1-6, L1-8 and L1-10 occurred on average at a load of 45 kN. The samples with strengthening layout two L2-6, L2-8 and L2-10 delayed the appearance of the first crack and it was formed at a load of around 48 kN. The position and length of the first cracks for the CFRP strengthened samples were in orthogonal direction, parallel to the CFRP strips. The crack formed as a straight line in the middle of the slab, going from one end to the other end and crossing over the column-head. However, the first crack formed at random locations in the radial direction for control samples CS1 and CS2.

The punching shear crack was roughly circular and appeared at some distance away from the vicinity of the column. Strengthening the slabs with NSM CFRP bars did not change the shape of punching shear crack. The shear failure plane developed partially at random locations at the tension surface of the slab and kept growing until failure. The formation of cracks in the epoxy adhesive (used to attach NSM bars to concrete) occurred at later stages of loading compared to concrete. This could be attributed to the higher flexural capacity of the epoxy.

Ultimate punching shear capacity

The maximum capacity of the tested samples is presented in Table 2. The control samples CS1 and CS2 failed under classical punching failure after reaching a maximum load of 141 kN and 146 kN respectively. The retrofitted samples with CFRP bars displayed significantly higher punching capacity compared to the average failure load of the two control samples. The capacity of the strengthened samples within each of the strengthening layout was very similar. Increasing the cross-sectional area of the CFRP bars did not have any noticeable influence on the ultimate capacity in this case. The maximum strain recorded for CFRP bars was around 45% of its rupture
strain (refer to Fig 12). The maximum allowable capacity of the CFRP bars was not utilized and the samples failed under concrete shear failure.

Increasing the number of CFRP bars considerably improved the ultimate load. The samples with strengthening layout one (L1-6, L1-8 and L1-10) increased the ultimate load by about 18%. The average increase for strengthening layout two (L2-6, L2-8 and L2-10) was around 41% compared to control samples. Sample L2-10 exhibited the highest increase amongst other strengthened cases and increased the ultimate load by 44%. Positioning the CFRP bars over a larger area intersected the punching shear failure plane at several locations and delayed the punching shear failure, which subsequently translated into enhanced load-carrying capacity.

ANALYTICAL PREDICTIONS

Design codes expressions

The existing design codes predict the punching shear capacity of conventional steel-reinforced concrete only. The ultimate load is obtained by considering several factors such as steel reinforcement ratio, compressive strength of concrete, slab depth and size of the column. The design guidance requires checking the punching capacity of slabs at the face of the column and at critical perimeters. The FIB Model Code (FIB MC, 2010) defines the critical perimeter at a distance of $0.5d$ (where $d$ is the depth of slab) from the face of the column. The Eurocode 2 (EC2, 2004) identifies the critical perimeter at $2d$. Fig 15 shows the location of critical/control perimeter according to the design codes. The following expressions, without considering capacity reduction factors, are adopted for estimating the punching capacity of slabs without shear reinforcement. The following expressions are used for the purpose of comparison with the design codes.
Eurocode 2

Eurocode 2 (EC2, 2004) proposes the following expression to estimate the punching shear capacity of RC slabs.

\[ V_c = 0.18k(100\rho_1 \cdot f_{ck})^{1/3} \cdot d \cdot u + k_1\sigma_{cp} \geq (V_{min} + k_1\sigma_{cp}) \]  

(1)

\[ k = 1 + \sqrt{\frac{200}{d}} \leq 2 \]  

(2)

\[ \rho = \sqrt{(\rho_{1z} \cdot \rho_{1y})} \leq 0.02 \]  

(3)

\[ V_{min} = 0.035k^{3/2}f_{ck}^{1/2} \]  

(4)

In the above expressions, \( d \) represents the effective depth of the slab, \( u \) represents the critical control perimeter, term \( k \) is a size factor, \( \rho_1 \) is the flexural reinforcement ratio, \( f_{ck} \) is the characteristic compressive strength of concrete, \( \sigma_{cp} \) is a factor related to prestressing and \( V_{min} \) shows the minimum shear capacity.

**FIB MC 2010**

FIB MC (2010) provides four levels of approximation denoted by term \( \psi \) for calculating the rotation of the slab. The level one approximation is used in this instance due to negligible redistribution of moments.

\[ V_{rc} = k_\psi \times f_{ck} \times u \times d \]  

(5)

\[ k_\psi = \frac{1}{1.5 + 0.9k_{dg}\psi d} \leq 0.6 \]  

(6)

\[ k_{dg} = \frac{32}{16 + d_g} \]  

(7)

\[ \psi = 1.5 \times \frac{r_s f_{yd}}{d \cdot E_s} \]  

(8)
The term $k_y$ is related to the rotation of the slab, $d_g$ is the maximum size of aggregate used in concrete, $r_s$ is the radius of the separated slab element, $f_{yd}$ is the yield strength of steel and $E_s$ is the elastic modulus of flexural reinforcement.

**Adoption of Chen & Li method for NSM**

The design codes previously discussed estimate the punching capacity of slabs at critical perimeters by considering effective depth, reinforcement ratio and compressive strength of the concrete. However, there are no known design codes for calculating the punching capacities of slabs strengthened with NSM CFRP bars. The design approach adopted in this study is based on Chen & Li (2005) method. This design approach considers FRP as flexural reinforcement and introduces two terms to be replaced in the design codes. The term $\rho_{eq}$ and $d_{eq}$ are introduced to replace $\rho$ and $d$ to take the influence of reinforcement ratio and effective depth into consideration.

This method assumes a perfect bond between the concrete and the CFRP bars. This assumption is true for this experiment and it was confirmed by visual inspection and strain data. The distribution of forces, stresses, and strains within the cross-section of the slab is presented in Fig 16. It should be noted that the diagram is modified to change the EBR FRP to NSM FRP strengthening. According to this approach, the maximum flexural capacity is achieved, when the concrete reaches strain of 0.003 or the CFRP reaches its rupture strain. The strain in CFRP bars and steel reinforcement is determined by linear strain distribution.

\[
\varepsilon_s = \frac{d - c}{c} \varepsilon_{cu} \tag{9}
\]

\[
\varepsilon_f = \frac{h_1 - c}{c} \varepsilon_{cu} \tag{10}
\]
In the expressions above, \( \varepsilon_s \) is the strain in steel reinforcement; \( \varepsilon_{cu} \) is the strain in concrete which is taken as 0.003 and \( \varepsilon_f \) is the strain in CFRP bars. The stresses in steel and CFRP bars can be found using the following expressions:

\[
 f_s = E_s \varepsilon_s \quad \text{for} \quad \varepsilon_s < \varepsilon_y \quad (11)
\]

\[
 f_s = f_y \quad \text{for} \quad \varepsilon_s \geq \varepsilon_y \quad (12)
\]

\[
 f_f = E_f \varepsilon_f \quad \text{for} \quad \varepsilon_f < \varepsilon_{fu} \quad (13)
\]

Where \( \varepsilon_y \) and \( \varepsilon_{fu} \) shows the yield and ultimate strain in CFRP bars; \( f_y \) is the yield stress of flexural steel reinforcement; \( E_f \) is the CFRP elastic modulus and \( E_s \) is the steel elastic modulus.

The compression force in concrete, tension force in steel reinforcement and tension force in CFRP bars is obtained from the following expressions:

\[
 C_c = 0.85 f'_c a b \quad (14)
\]

\[
 T_s = A_s f_s \quad (15)
\]

\[
 T_f = A_f f_f \quad (16)
\]

In the expressions above, \( a \) is the depth of rectangular stress block; \( b \) is the unit width of the slab and the cross-sectional area of steel reinforcement and the CFRP bars is denoted by \( A_s \) and \( A_f \).

The depth of the neutral axis is obtained by conducting iterations of the equilibrium of internal forces until the following equation is satisfied.

\[
 C_c = T_s + T_f \quad (17)
\]

After taking moment about the steel reinforcement axis, the following expression is obtained:

\[
 M_{nf} = C_c \left( d - \frac{a}{2} \right) + T_f (h_1 - d) \quad (18)
\]
The influence of the CFRP bars and their positioning with respect to the depth of the slab could be calculated backwardly.

\[
d_{eqv} = \frac{M_{nf}}{T_s + T_f} + \frac{a}{2} \quad (19)
\]

\[
\rho_{eqv} = \frac{T_s + T_f}{b d_{eqv} f_s} \quad (20)
\]

The terms \(d_{eqv}\) and \(\rho_{eqv}\) are then substituted in the design codes to obtain the ultimate capacity of strengthened slabs.

**Comparison of results**

The ultimate capacities obtained from the experimental work and the capacities from the design codes are presented in table 2. The predictions of both Eurocode 2 (2014) and FIB MC (2010) are somewhat similar for estimating the punching capacities of the two control slabs CS1 and CS2. However, the predictions of the modified design codes were relatively conservative for the strengthened slabs using Chen & Li’s (2005) method. In general, these predictions provided more accurate values for the samples strengthened with layout one as compared to samples strengthened with layout two.

Chen and Li’s method restricts the concrete strain to 0.003 but during the experiment, a significantly higher level of strain was recorded at the centre of slabs (refer to Fig 12). The model also assumed full bond of CFRP with concrete which was observed during the experiment for all slabs. This method could be used for estimating the punching capacities of flat slabs strengthened with NSM FRP bars.

**DISCUSSION**

According to Moe (1961), the punching shear strength of flat slabs can be established from its flexural capacity. Increasing the flexural reinforcement of flat slabs directly improves its flexural capacity, but it also indirectly contributes to the punching shear capacity. Therefore, the provision
of flexural NSM CFRP reinforcement enhances the punching shear capacity of flat slabs. This effect is more pronounced for slabs with lower reinforcement ratio. The NSM reinforcement around the perimeter of the column intersects the punching shear crack and delays its growth. Introducing greater numbers of CFRP bars around the punching area is more effective as it increases the number of intersection points with the shear crack.

Changing the cross-sectional area did not noticeably influence the ultimate load because the CFRP bars did not reach their rupture strain. The slabs were over-strengthened in this specific case. A relationship between the cross-sectional area of the CFRP bars and the ultimate capacity could be established if the failure occurs via rupture of the CFRP bars. This relationship can be achieved by using smaller CFRP bar sizes.

The increase in the ultimate load for the strengthened samples is due to the development of dowel forces in the CFRP bars when they cut across the inclined shear crack. When the conically shaped crack is developed over the column head, it creates a shear failure plane with the remaining parts of the slab. These shear forces are resisted by the aggregate interlock and dowel action of the steel and CFRP reinforcement. The CFRP reinforcement restricts the crack widening by the development of dowel forces. The concrete cover is the main parameter upon which the dowel mechanism is dependent (CEB-FIP 1993; CEB 1996). Deeper concrete cover and higher tensile splitting strength of concrete allow for the development of higher level of dowel forces. The samples strengthened with layout two developed twice the amount of dowel forces compared to samples strengthened with layout one, due to the amount of CFRP bars. Fig 17 shows the variation in the dowel forces between the two strengthening schemes.

The development of vertical forces due to the membrane effect in the CFRP reinforcement is also contributing towards increasing the ultimate load. Kinnunen and Nylander (1960) examined the contributions from the dowel forces and the membrane effects, for the punching shear capacity of flat slabs. According to their conclusion, slab punching shear capacity improves if the ratio and strength of flexural reinforcement increases.
Sample L1-8 and L2-6 were strengthened with roughly the same amount of CFRP reinforcement but the increase in the ultimate load for sample L2-6 was two times greater than sample L1-8. Sample L2-6 satisfied the maximum bar spacing in the area affected by punching shear whilst sample L1-8 had large unreinforced regions, which allowed for the development of punching shear crack at a relatively lower level of loading.

Alexander and Simmonds (1990) concluded that the concentration of reinforcement over the column strip is less effective compared to equal distribution of reinforcement over a wider area. The equal distribution of reinforcement allowed for further development of dowel forces, which subsequently delayed the punching shear failure.

Strengthening slab-column connections with NSM CFRP bars significantly increases the cracking load, stiffness, and ultimate capacity. The bonded length provided for the CFRP bars is sufficient for this specific size of the slab. The CFRP bars forms a strong bond with concrete and the system does not suffer from debonding. This results in utilizing the maximum allowable capacity of CFRP bars which subsequently enhances the ultimate load. The NSM strengthening of slab-column connection is significantly more efficient than EBR strengthening mainly in terms of bond performance and increasing the ultimate load.

The negative moment (hogging) region in flat slabs specifically in car parks is exposed to heavy vehicular impact. External strengthening with FRP EBR causes durability issues and poses a major fire risk. The use of NSM as an alternative to EBR strengthening overcomes such issues. It should be noted, that the NSM method requires sufficient concrete cover for creating grooves in concrete.
CONCLUSIONS

In this study, the punching shear strength of interior slab-column connections retrofitted with NSM CFRP bars is experimentally investigated. The study concentrates on the influence of the cross-sectional area of CFRP bars and the strengthening layout. Eight slab-column connections were tested under monotonic load and the following conclusions are drawn:

1. The use of NSM CFRP bars improves the shear capacity of slab-column connections. Sample L2-10 increased the ultimate load by up to 44% compared to control samples.

2. Increasing the number of CFRP bars considerably enhances their ultimate load. The average strength gain for strengthening layout one and two is 18% and 41% respectively.

3. Strengthening delays formation of the first crack in concrete which subsequently results in maintaining a linear relationship for load-displacement and load-strain curves.

4. CFRP NSM strengthening significantly increases the flexural stiffness. The increase in stiffness is directly related to the strengthening layout and the cross-sectional area of CFRP bars. The maximum flexural stiffness was recorded for sample L2-10 which shows an increase of 100% compared to control samples.

5. The ultimate capacities of strengthened slabs with NSM CFRP bars can accurately be calculated by the adoption of Chen & Li’s method in the design codes. The proposed method could be incorporated into design codes.
<table>
<thead>
<tr>
<th>NOTATION</th>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_f$</td>
<td>Cross-sectional area of FRP reinforcement</td>
<td></td>
</tr>
<tr>
<td>$A_s$</td>
<td>Cross-sectional area of steel reinforcement</td>
<td></td>
</tr>
<tr>
<td>$a$</td>
<td>Depth of the rectangular stress block</td>
<td></td>
</tr>
<tr>
<td>$b$</td>
<td>Breadth of the slab</td>
<td></td>
</tr>
<tr>
<td>$c$</td>
<td>Column side length (dimension)</td>
<td></td>
</tr>
<tr>
<td>$d$</td>
<td>Effective depth of the slab</td>
<td></td>
</tr>
<tr>
<td>$d'$</td>
<td>Height of concrete cover</td>
<td></td>
</tr>
<tr>
<td>$d_b$</td>
<td>Diameter of the bar</td>
<td></td>
</tr>
<tr>
<td>$d_{eqv}$</td>
<td>Equivalent effective depth for the slab</td>
<td></td>
</tr>
<tr>
<td>$d_g$</td>
<td>Maximum aggregate size in concrete mix</td>
<td></td>
</tr>
<tr>
<td>$d_g$</td>
<td>Concrete cover on the side of the slab</td>
<td></td>
</tr>
<tr>
<td>$E_f$</td>
<td>Modulus of elasticity for FRP</td>
<td></td>
</tr>
<tr>
<td>$E_s$</td>
<td>The modulus of elasticity for steel</td>
<td></td>
</tr>
<tr>
<td>$f_{ck}$</td>
<td>Compressive strength of concrete</td>
<td></td>
</tr>
<tr>
<td>$f_c'$</td>
<td>Compressive strength of concrete</td>
<td></td>
</tr>
<tr>
<td>$f_y$</td>
<td>Yield strength of steel</td>
<td></td>
</tr>
<tr>
<td>$h$</td>
<td>Depth of the slab section</td>
<td></td>
</tr>
</tbody>
</table>
$h_1$ Height between the compression surface of the concrete to the centre of the FRP reinforcement in the slab

$k$ Size factor for the effective depth of the slab

$k_1$ Empirical factor representing the nominal stresses

$k_{dg}$ Parameter related to the maximum aggregate size

$k_\psi$ Parameter related to the rotation of the slab

$T_f$ Tensile force in the FRP reinforcement

$T_s$ Tensile force in the steel reinforcement

$u$ Length of the control perimeter in the slab

$u_0$ First perimeter of the column

$V_{min}$ Minimum shear capacity of the slab

$V_{u,predicted}$ Maximum punching shear capacity predicted by the design codes

$V_{u,test}$ Maximum punching shear capacity of the tested samples

$\gamma_s$ Radius of the separated slab element

$\varepsilon_{cu}$ Strain in the concrete

$\varepsilon_f$ Strain in the FRP reinforcement

$\varepsilon_{fu}$ Ultimate strain in the FRP

$\varepsilon_s$ Strain in the steel reinforcement

$\varepsilon_y$ Yield strain in the FRP

$\pi$ Ratio of circle circumference to its diameter (constant)
Flexural steel reinforcement ratio

Average reinforcement ratio

Reinforcement ratio in Y-Y direction

Reinforcement ratio in Z-Z direction

Equivalent reinforcement ratio for the slab

Concrete stresses due to prestressing of reinforcement

Angle between the horizontal axis and the deformed slab
DATA AVAILABILITY STATEMENT

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request. The data includes:

1. Pictures showing the failure mode of all samples
2. Deflection of the samples at quarter-span
3. Strain development in steel reinforcement
4. Excel sheets showing the detailed calculation for obtaining the analytical results
REFERENCES


FIB (Federation internationale du beton. Fib model code for concrete structures) 2010. *FIB model code.* Lausanne, Switzerland.


http://dx.doi.org/10.1016/j.compstruct.2012.08.038


Table 1. Sample description

<table>
<thead>
<tr>
<th>Slab ID</th>
<th>Strengthening Layout</th>
<th>Number of bars $N_b$</th>
<th>Bar diameter $d_b$ (mm)</th>
<th>Test variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Control slab</td>
</tr>
<tr>
<td>CS2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Control slab</td>
</tr>
<tr>
<td>L1-6</td>
<td>Layout 1</td>
<td>4</td>
<td>6</td>
<td>Strengthened</td>
</tr>
<tr>
<td>L1-8</td>
<td>Layout 1</td>
<td>4</td>
<td>8</td>
<td>Strengthened</td>
</tr>
<tr>
<td>L1-10</td>
<td>Layout 1</td>
<td>4</td>
<td>10</td>
<td>Strengthened</td>
</tr>
<tr>
<td>L2-6</td>
<td>Layout 2</td>
<td>8</td>
<td>6</td>
<td>Strengthened</td>
</tr>
<tr>
<td>L2-8</td>
<td>Layout 2</td>
<td>8</td>
<td>8</td>
<td>Strengthened</td>
</tr>
<tr>
<td>L2-10</td>
<td>Layout 2</td>
<td>8</td>
<td>10</td>
<td>Strengthened</td>
</tr>
</tbody>
</table>
Table 2. Comparison of ultimate loads with design codes

<table>
<thead>
<tr>
<th>Specimen designation</th>
<th>$V_{u, test}$ (kN)</th>
<th>$V_{u, predicted}$ (kN)</th>
<th>$V_{u, test} / V_{u, predicted}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EC 2</td>
<td>FIB MC</td>
<td>EC 2</td>
</tr>
<tr>
<td>CS1</td>
<td>141</td>
<td>135</td>
<td>128</td>
</tr>
<tr>
<td>CS2</td>
<td>146</td>
<td>135</td>
<td>128</td>
</tr>
<tr>
<td>L1-6</td>
<td>168</td>
<td>153</td>
<td>144</td>
</tr>
<tr>
<td>L1-8</td>
<td>172</td>
<td>161</td>
<td>148</td>
</tr>
<tr>
<td>L1-10</td>
<td>167</td>
<td>168</td>
<td>150</td>
</tr>
<tr>
<td>L2-6</td>
<td>202</td>
<td>163</td>
<td>149</td>
</tr>
<tr>
<td>L2-8</td>
<td>197</td>
<td>174</td>
<td>153</td>
</tr>
<tr>
<td>L2-10</td>
<td>206</td>
<td>182</td>
<td>155</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td>1.1</td>
</tr>
</tbody>
</table>
LIST OF FIGURES

Fig. 1. Typical cross-section of the slab (dimensions in mm)

Fig. 2. Cross-sectional area of the CFRP bars (mm)

Fig. 3. Typical groove detail for 8mm CFRP bar (dimensions in mm)

Fig. 4. (a) Steel reinforcement in moulds (b) Finished grooves for strengthening (c) CFRP bars (d) Placing CFRP bars in grooves

Fig. 5. (a) Cross-section layout 1 (b) plan layout 1 (c) cross-section layout 2 (d) plan layout (dimensions in mm)

Fig. 6. Slab instrumentation (dimensions in mm)

Fig. 7. Test set up and instrumentation side view

Fig. 8. Test set up

Fig. 9. Punching failure of slab L2-8

Fig. 10. Load-deflection response at mid-span

Fig. 11. Flexural stiffness of all samples

Fig. 12. Load-strain relationship in CFRP bars

Fig. 13. Strain profile in the tension surface of slabs at ultimate load

Fig. 14. Cracking pattern for control and CFRP strengthened samples

Fig. 15. Critical/control punching shear perimeter for design codes

Fig. 16. Strain, stress and force distribution in slab section

Fig. 17. Development of dowel forces in the strengthened slabs
3 No. H6 links @ 90 c/c

H8 @ 100 mm c/c

H6 @ 200mm c/c

4 No. H10 L bars

150

675

1500

675

Square

98

150

120

270

150

270
8mm CFRP rebar

12mm x 12mm groove filled with epoxy adhesive

8mm CFRP rebar
Theoretical punching shear crack

Note: All units are in mm.
Figure 16

- **Section**
  - $h$, $h_1$, $d$
  - CFRP bar
  - Steel bar

- **Strain Distribution**
  - $\varepsilon_{cu}$
  - $\varepsilon_s$
  - $\varepsilon_f$

- **Stress Distribution**
  - $0.85f_c$
  - $a/2$

- **Force**
  - $C_c$
  - $f_s$
  - $f_f$
  - $T_s$
  - $T_f$
Areas where dowel forces are developed
Areas where dowel forces are not developed