NUMERICAL ANALYSIS OF TWO-WAY RC SLABS FLEXURAL STRENGTHENED WITH NSM CFRP LAMINATES

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1 INTRODUCTION

Gluing fibre reinforced polymer (FRP) laminates by a structural adhesive into thin slits cut on the concrete cover of reinforced concrete (RC) elements is a strengthening technique, designated Near Surface Mounted (NSM), which is gaining increasing attention of practitioners, engineers and scientists interested in structural rehabilitation. The efficacy of the NSM technique for the flexural strengthening of RC structures has been proven by research and applications (Nanni et al. 2004). This efficacy has been explored, mainly to increase the positive bending moments of statically determinate RC beams (Blaschko and Zilch 1999; El-Hacha and Rizkalla 2004; Barros and Fortes 2005, Kotynia 2006) and slabs (Barros et al. 2008; Bonaldo et al. 2008). However, due to the characteristics of the application of this strengthening technique, it is particularly appropriate to increase the negative bending moments (developed at the interior supports) of continuous (two or more spans) RC slabs. In fact, the opening process of the slits can be easily executed by the equipment used to open crack control joints in concrete slabs.

When a continuous RC structure is strengthened with FRP materials, its ductility and plastic rotation capacity may be, however, restricted or even extinct, due to, principally, the linear-elastic tensile behaviour of the FRP up to its brittle failure (Arduini et al. 1997, Casadei et al. 2003). As flexural members retrofitted with externally bonded reinforcing (EBR) technique tend to fail by brittle premature plate debonding well before the FRP tensile strength capacity is reached, the ductility, particularly the plastic rotation capacity, can be severely reduced, decreasing the available degree of moment redistribution (Oehlerls et al. 2004a). The tests of El-Refaie et al. (2003a, 2003b), Ashour et al. 2004 and Oehlerls et al. (2004a) show that, in general, premature debonding of the external strengthening system is the dominant failure mechanism. However, according to the approach used by these authors to quantify the moment redistribution, significant moment redistribution was obtained in the tests (El-Refaie et al. 2003a, Oehlerls et al. 2004a/2004b, Oehlerls et al. 2006, Liu et al. 2006a), which contradicts the existing design guidelines (Concrete Society 2000, fib 2001, ACI 440 2007) that suggest that moment redistribution should not be allowed for RC members strengthened with EBR technique.

On the other hand, tests with simply supported RC members strengthened with NSM strips (Hassan and Rizkalla 2003, Täljsten et al. 2003, Barros et al. 2007) have shown that NSM strengthening elements debond or fail at much higher strain than EBR strengthening systems, therefore, in general, NSM strengthened members are expected to have a much more ductile behaviour than EBR strengthened members. Therefore, NSM technique seems to have high potential for the strengthening of negative bending moment regions, since relatively easy and fast strengthening procedures are required.

The first preliminary studies on moment redistribution of statically indeterminate RC members strengthened with NSM technique were conducted at the Adelaide University, in Australia (Liu et al. 2006b). A significant amount of moment redistribution was attained using NSM technique, when compared with EBR technique. Park and Oehlerls (2000) performed tests on a series of continuous beams strengthened with externally
bonded steel or FRP reinforcement over the positive (sagging) and negative (hogging) bending moments regions. The plates were applied on either the tension face or the side faces of the beam. For both steel and FRP plated beams, plate debonding was observed. This indicates that, although steel is a ductile material, the externally bonded steel plates can still reduce the ductility of the retrofitted beam, depending on the plating dimensions and positions, and almost zero moment redistribution was obtained in all the tests. Ashour et al. (2004) performed tests on sixteen RC continuous beams with different arrangements of internal and external reinforcement. All beams were strengthened with CFRP sheets or plates over the hogging and/or sagging regions. All strengthened beams exhibited a higher load capacity but lower ductility compared with their respective unstrengthened control beams.

Recently, Bonaldo (2008) carried out an experimental program to assess the moment redistribution capacity of two-way RC slabs flexural strengthened with NSM CFRP laminates. In spite of the increase of the flexural resistance of the sections at the hogging region has exceeded the target values (25% and 50%), the moment redistribution was relatively low, and the increase of the load carrying capacity of the strengthened slabs was limited to 21%. In the present work, this experimental program is analyzed in depth in order to assess the possibilities and challenges of the NSM technique in terms of flexural strengthening effectiveness, moment redistribution and ductility performance of continuous RC slabs. Using the results obtained in the experimental program, the predictive performance of a constitutive model implemented into a FEM-based computer program was appraised. With the help of this computer program, a high effective NSM flexural strengthening strategy is proposed, capable of enhancing the slab’s load carrying capacity and assuring high levels of ductility (Bonaldo et al. 2008).
2 EXPERIMENTAL PROGRAM

This section deals with the experimental program carried out to analyse the moment redistribution capability of two-way span RC slabs, strengthened according to the NSM technique for the increase of the resistant negative bending moment of the cross sections placed at the slab intermediate support (IS) and under load lines (LS). The tests are described and the obtained results are presented and analyzed.

2.1 SLAB STRIPS SPECIMENS

In the present study nine RC continuous slab specimens were used, divided in three specimens per group. The specimens were divided in two groups. The first group consisted of unstrengthened RC slab specimens that formed the control set (SL15, SL30 and SL45). The second group consisted of six slab specimens strengthened with CFRP laminates according to NSM technique (SL15s25, SL15s50, SL30s25, SL30s50, SL45s25 and SL45s50). The resume of the tested specimens is presented, as follows:

i) SL15 series: designed to redistribute 15% of the elastic negative moment of the cross section placed at the intermediate support.
   - SL15: reference slab (without any FRP strengthening system);
   - SL15S25: NSM CFRP laminate strengthened slab, designed to increase 25% the negative moment;
   - SL15S50: NSM CFRP laminate strengthened slab, designed to increase 50% the negative moment.

ii) SL30 series: designed to redistribute 30% of the elastic negative moment of the cross section placed at the intermediate support.
   - SL30: reference slab (without any FRP strengthening system);
   - SL30S25: NSM CFRP laminate strengthened slab, designed to increase 25% the negative moment;
   - SL30S50: NSM CFRP laminate strengthened slab, designed to increase 50% the negative moment.

iii) SL45 series: designed to redistribute 45% of the elastic negative moment of the cross section placed at the intermediate support.
   - SL45: reference slab (without any FRP strengthening system);
   - SL45S25: NSM CFRP laminate strengthened slab, designed to increase 25% the negative moment;
   - SL45S50: NSM CFRP laminate strengthened slab, designed to increase 50% the negative moment.
2.2 Specimen AND Test Configuration

According to ACI 318 (2004), the maximum deflection ($\delta_{\text{max}}$) that can be considered on the design of reinforced concrete members is equal to $L/480$. For $L = 2800 \text{mm}$ and considering $E = 30 \text{GPa}$, the maximum deflection of this element is 5.8 mm, which corresponds to an applied load of about 46.20 kN. It should be mentioned that the slab specimens were designed considering a load increment of 10%, which corresponds to an applied load of 50.82 kN. The test configuration represented in Figure 1 was used in the experimental program in order to evaluate the moment redistribution capability of statically indeterminate RC slab strips, strengthened according to the NSM technique for increasing the flexural load carrying capacity. The cross-section details of the unstrengthened and strengthened slab strips are shown in Figures 2 and 3, respectively.

![Figure 1 – Test setup configuration for analysing the moment redistribution capability of indeterminate RC slab strips (a), specimens dimensions (b), elastic curve (c) and moment redistribution concept (d). All dimensions in mm.](image)
<table>
<thead>
<tr>
<th>Slab Strip ID</th>
<th>Cross Section ID</th>
<th>Cross Section (dimensions in mm)</th>
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<tbody>
<tr>
<td>SL15</td>
<td>$S_1 - S_1'$</td>
<td>$A_1 = 2\phi 12\text{mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$A_2 = 4\phi 12\text{mm} + 3\phi 8\text{mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$A_3 = 5\phi 12\text{mm}$</td>
</tr>
<tr>
<td>SL30</td>
<td>$S_1 - S_1'$</td>
<td>$A_1 = 2\phi 12\text{mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$A_2 = 3\phi 12\text{mm} + 4\phi 10\text{mm}$</td>
</tr>
<tr>
<td></td>
<td>$S_2 - S_2'$</td>
<td>$A_1' = 2\phi 12\text{mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$A_2' = 2\phi 10\text{mm} + 1\phi 12\text{mm}$</td>
</tr>
<tr>
<td>SL45</td>
<td>$S_1 - S_1'$</td>
<td>$A_1 = 6\phi 12\text{mm}$</td>
</tr>
<tr>
<td></td>
<td>$S_2 - S_2'$</td>
<td>$A_1' = 2\phi 12\text{mm} + 1\phi 8\text{mm}$</td>
</tr>
</tbody>
</table>

Figure 2 – Specimens cross-section dimensions for the unstrengthened slab strips. All dimensions in mm.
<table>
<thead>
<tr>
<th>Slab Strip ID</th>
<th>Cross Section ID</th>
<th>Cross Section (dimensions in mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL15s25</td>
<td>$S_1 - S_1'$</td>
<td><img src="" alt="Image" /></td>
</tr>
<tr>
<td>SL15s50</td>
<td>$S_1 - S_1'$</td>
<td><img src="" alt="Image" /></td>
</tr>
<tr>
<td>SL30s25</td>
<td>$S_1 - S_1'$</td>
<td><img src="" alt="Image" /></td>
</tr>
</tbody>
</table>

Figure 3 – Specimens cross-section dimensions for the strengthened slab strips. All dimensions in mm.
Note that all the specimens, according to each series, have the same reinforcement along the length of the slab strip, such that the tensile reinforcement in the hogging region was less than the tensile reinforcement in the sagging region, to ensure that the hogging region reached its moment capacity first. In this way, the hogging region was allowed to redistribute moment to the sagging region as the static moment was being increased. It needs to be emphasised that the test program was specifically designed to study moment redistribution, so shear reinforcement was not considered.

2.3 Prediction of the elastic moments

The negative and positive elastic bending moments, calculated according to the theory of elasticity, for each slab strips series are present in Figures 4 to 6 and a resume can be found in Tables 1 to 3.

<table>
<thead>
<tr>
<th>SL15 series</th>
<th>Table 1 – Elastic bending moments of SL15 series.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_{\text{static}}^+$ (kN·m)</td>
</tr>
<tr>
<td>SL</td>
<td>20.83</td>
</tr>
<tr>
<td>SL15</td>
<td>22.70</td>
</tr>
<tr>
<td>SL15s25</td>
<td>28.37</td>
</tr>
<tr>
<td>SL15s50</td>
<td>34.05</td>
</tr>
</tbody>
</table>

\[ M_{\text{static}}^+ (\text{SL}) = 20.83 \text{ kN.m} \]
\[ M_{\text{static}}^- (\text{SL}) = 24.99 \text{ kN.m} \]
\[ R_L (\text{SL}) = 14.88 \text{ kN} \]
\[ R_C (\text{SL}) = 31.39 \text{ kN} \]
\[ F_{(\text{SL})} = 47.60 \text{ kN} \]

\[ M_{\text{static}}^+ (\text{SL15}) = 22.70 \text{ kN.m} \]
\[ M_{\text{static}}^- (\text{SL15}) = 21.24 \text{ kN.m} \]
\[ R_L (\text{SL15}) = 16.21 \text{ kN} \]
\[ R_C (\text{SL15}) = 32.73 \text{ kN} \]
\[ F_{(\text{SL15})} = 47.60 \text{ kN} \]

\[ M_{\text{static}}^- (\text{SL15s25}) = 26.55 \text{ kN.m} \]
\[ R_L (\text{SL15s25}) = 20.27 \text{ kN} \]
\[ R_C (\text{SL15s25}) = 39.23 \text{ kN} \]
\[ F_{(\text{SL15s25})} = 59.50 \text{ kN} \]

\[ M_{\text{static}}^- (\text{SL15s50}) = 31.86 \text{ kN.m} \]
\[ R_L (\text{SL15s50}) = 22.70 \text{ kN} \]
\[ R_C (\text{SL15s50}) = 39.37 \text{ kN} \]
\[ F_{(\text{SL15s50})} = 69.80 \text{ kN} \]
Figure 4 – Elastic bending moments of SL15 Series.

Table 2 – Elastic bending moments of SL30 series.

<table>
<thead>
<tr>
<th></th>
<th>$M_{\text{static}}^+$ (kN·m)</th>
<th>$M_{\text{static}}^-$ (kN·m)</th>
<th>$\Delta M^-$ (kN·m)</th>
<th>$\Delta F^-$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL</td>
<td>20.93</td>
<td>25.12</td>
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<td>SL30</td>
<td>24.70</td>
<td>17.58</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>SL30s25</td>
<td>30.87</td>
<td>21.98</td>
<td>4.40</td>
<td>11.96</td>
</tr>
<tr>
<td>SL30s50</td>
<td>37.05</td>
<td>26.37</td>
<td>8.79</td>
<td>23.92</td>
</tr>
</tbody>
</table>
### SL45 series

#### Table 3 – Elastic bending moments of SL30 series.

<table>
<thead>
<tr>
<th></th>
<th>M(_{\text{static}}^+) (kN·m)</th>
<th>M(_{\text{static}}^-) (kN·m)</th>
<th>ΔM(^-) (kN·m)</th>
<th>ΔF(^-) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL</td>
<td>23.19</td>
<td>27.83</td>
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<td>SL45</td>
<td>29.45</td>
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<td>SL45s25</td>
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<td>SL45s50</td>
<td>44.17</td>
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<td>7.66</td>
<td>26.50</td>
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</table>

Figure 5 – Elastic bending moments of SL30 Series.
2.4 Prediction of the Slab Strip Capacity

2.4.1 Flexural Capacity of Slabs Strips

The following assumptions were made in calculating the flexural resistance of a section strengthened with an externally applied FRP system:

- Design calculations are based on the dimensions, internal reinforcing steel arrangement, and material properties of the existing member being strengthened;
- The strains in the steel reinforcement and concrete are directly proportional to the distance from the neutral axis. That is, a plane section before loading remains plane after loading;
- There is no relative slip between external FRP reinforcement and the concrete;
- The shear deformation within the adhesive layer is neglected because the adhesive layer is very thin with slight variations in its thickness;
- The maximum usable compressive strain in the concrete is 0.0035;
- The tensile strength of concrete is neglected; and
- The FRP reinforcement has a linear elastic stress-strain relationship to failure.

To illustrate the concepts presented in this chapter, this section describes the application of the singly-reinforced rectangular section concepts on the design of the slab strips.

Figure 7 illustrates the internal strain and stress distribution for a rectangular section under flexure at the ultimate limit state.
The parameters presented in Figure 7 are as follows: \( \lambda \) and \( \eta \) define the rectangular stress block of concrete in compression, \( b \) is the cross section width and \( f_c \) (considered herein as \( f_{cm} \)) is the average concrete strength. \( \lambda \) is the ratio of the depth of the idealized rectangular stress block to the neutral axis depth and \( \eta \) is the ratio of the average compressive stress to the concrete compressive strength.

According to Eurocode 2 (2004), \( 0.80 \lambda = 0 \) and \( 1.00 \eta = 1 \). Considering the failure mode controlled by crushing of the concrete in compression and the yielding of the steel reinforcement, the internal forces are calculated as follows:

- **Compressive force** \( (F_c) \):
  \[
  F_c = \lambda \cdot x \cdot b \cdot \eta \cdot f_{cm} 
  \]
  \[
  F_c = (0.80) \cdot x \cdot (375) \cdot (1.00) \cdot f_{cm} = 9000x \quad \text{[MPa]} 
  \]

- **Tensile Force** \( (F_s) \):
  \[
  F_s = A_s \cdot f_y 
  \]

Due to the determination of the strain distribution, the neutral axis depth, \( x \), is initially assumed and the correct value is iteratively determined when the equilibrium of internal forces is satisfied. Strains and stresses in different materials can be calculated.

\[
\sum F_i = 0 \rightarrow F_c = F_s
\]

Consequently, the flexural strength, \( M_{af} \), is calculated by taking moments of internal forces about the level of tensile steel as follows:

\[
M_{af} = F_c \cdot z = F_s \cdot z
\]
where:

\[ z = d_s - \frac{\lambda}{2} \]  

All the slabs strips have a concrete cover of 25 mm thickness. The longitudinal reinforcement ratio is defined by:

\[ \rho_{sl} = \frac{A_{sl}}{b_d d_s} \times 1100 \]  

where:

\[ d_s = h - a \]  

When introducing the CFRP laminate a similar approach is used to calculate the flexural resistance of a strengthened section. In this way, a term concerning to force developed by the FRP material is added (Equation (8)) and the internal forces equilibrium is given by:

- Tensile force at CFRP laminate strips (\( F_j \)):
  \[ F_j = A_j \cdot \varepsilon_j \cdot E_j \]  

\[ \sum F_i = 0 \rightarrow F_c = F_i + F_j \]  

The maximum strain level that can be achieved in the FRP reinforcement is governed by either of the following conditions: (i) the strain level developed in the FRP at the point at which concrete crushes, (ii) the strain level when the FRP ruptures, or (iii) the strain level when the FRP debonds from the substrate (ACI 440, 2007). For each internal arrangement, based on the theory of elasticity, the bending capacities of sagging and hogging regions were calculated, as well as the strains at the laminates when reaching a compressive strain of \( \varepsilon_c = 3.5 \% \). Complementary to these calculations, a Moment-Curvature cross-sectional analysis of the hogging and sagging regions of the slabs was carried out with “Docros” computer program and results are also shown in Tables 4 to 9.
### Table 4 – Elastic bending moments and strains at sagging region – SL15 series.

<table>
<thead>
<tr>
<th>Slab ID</th>
<th>$A_s$ (mm$^2$)</th>
<th>$F_t$ (kN)</th>
<th>$F_r$ (kN)</th>
<th>$F_r/F_c$ x (N/mm)</th>
<th>x (mm)</th>
<th>$M_{rd}$ (kN·m)</th>
<th>$n_f$ (%)</th>
<th>$\varepsilon_f$ (%)</th>
<th>Calculated Docros</th>
<th>Docros</th>
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### Table 5 – Elastic bending moments and strains at hogging region – SL15 series.

<table>
<thead>
<tr>
<th>Slab ID</th>
<th>$A_s$ (mm$^2$)</th>
<th>$F_t$ (kN)</th>
<th>$F_r$ (kN)</th>
<th>$F_r/F_c$ x (N/mm)</th>
<th>x (mm)</th>
<th>$M_{rd}$ (kN·m)</th>
<th>$n_f$ (%)</th>
<th>$\varepsilon_f$ (%)</th>
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### Table 6 – Elastic bending moments and strains at sagging region – SL30 series.

<table>
<thead>
<tr>
<th>Slab ID</th>
<th>$A_s$ (mm$^2$)</th>
<th>$F_t$ (kN)</th>
<th>$F_r$ (kN)</th>
<th>$F_r/F_c$ x (N/mm)</th>
<th>x (mm)</th>
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<th>$n_f$ (%)</th>
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</table>

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Table 7 – Elastic bending moments and strains at hogging region – SL30 series.

<table>
<thead>
<tr>
<th>Slab ID</th>
<th>$A_s$ (mm$^2$)</th>
<th>$F_s$ (kN)</th>
<th>$F_f$ (kN)</th>
<th>$F_c$ (N)</th>
<th>$F_c/x$ (N/mm)</th>
<th>$x$ (mm)</th>
<th>$M_{rd}$ (kN·m)</th>
<th>$n_f$</th>
<th>$\varepsilon_f$ (‰)</th>
<th>$M_{rd}$ (kN·m)</th>
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Table 8 – Elastic bending moments and strains at sagging region – SL45 series.

<table>
<thead>
<tr>
<th>Slab ID</th>
<th>$A_s$ (mm$^2$)</th>
<th>$F_s$ (kN)</th>
<th>$F_f$ (kN)</th>
<th>$F_c$ (N)</th>
<th>$F_c/x$ (N/mm)</th>
<th>$x$ (mm)</th>
<th>$M_{rd}$ (kN·m)</th>
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<th>$\varepsilon_f$ (‰)</th>
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Table 9 – Elastic bending moments and strains at hogging region – SL45 series.

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<th>Slab ID</th>
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<th>$F_s$ (kN)</th>
<th>$F_f$ (kN)</th>
<th>$F_c$ (N)</th>
<th>$F_c/x$ (N/mm)</th>
<th>$x$ (mm)</th>
<th>$M_{rd}$ (kN·m)</th>
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2.4.2 SHEAR CAPACITY OF SLABS STRIPS

ACI 318R (2008)

According to ACI-318R (2008), $V_n$ is nominal shear strength given by

$$V_n = V_c + V_s \quad (10)$$

where $V_c$ is nominal shear strength provided by concrete calculated and $V_s$ is nominal shear strength provided by shear reinforcement.

The shear strength is based on an average shear stress on the effective cross section $b_u d$. In a member without shear reinforcement, shear is assumed to be carried by the concrete web. In this case, the shear strength provided by concrete $V_c$ is assumed to be the same for RC elements with and without shear reinforcement and is taken as the shear causing significant inclined cracking. According to the standard, the shear strength for elements subject to shear and flexure only is given by

$$V_c = 2 \lambda \sqrt{f_y b_u d} \quad (11)$$

Where $\lambda$ is a modification factor which reflects the reduced mechanical properties of lightweight concrete, all relative to normal weight concrete of the same compressive strength, and is not considered in this work.

Where shear reinforcement perpendicular to axis of member is used,

$$V_s = \frac{A_s f_y d}{s} \quad (12)$$

where $A_s$ is the area of shear reinforcement within spacing $s$. In this work stirrups were used to simplify the construction of the longitudinal steel reinforcement, without shear reinforcement purposes, within spacing of 50mm.

EUROCODE 2 (2004)

According to EC2 (2004), the shear resistance of a member with shear reinforcement is equal to

$$V_{cd} = V_{cd,c} + V_{cd,r} + V_a \quad (13)$$

Where the design value for the shear resistance, $V_{cd,c}$, is given by

$$V_{cd,c} = \left[ C_{cd,c} k (100 \rho f_{ck})^0 + k_s \sigma_{cp} \right] b_u d \quad (14)$$

with a minimum of

$$V_{cd,c} = (V_{cd,c} + k_s \sigma_{cp}) b_u d \quad (15)$$

where:

$f_{ck}$ is in MPa

$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ with $d$ in mm

$\rho = \frac{A_d}{b_u d} \leq 0.02$

$A_d$ is the area of the tensile reinforcement,

$b_u$ is the smallest width of the cross-section in the tensile area [mm]

$\sigma_{cp} = N_{ed} / A_c < 0.2 f_{cd}$ [MPa]

$N_{ed}$ is the axial force in the cross-section due to loading or prestressing [in N]

$A_c$ is the area of concrete cross section [mm$^2$]
$V_{rd,c}$ is [N]

The recommended value for $C_{rd,c}$ is $0.18/\gamma_c$ and $k_1$ is 0.15. For members with vertical shear reinforcement, the shear resistance, $V_{rd}$ is the smaller value of:

$$V_{rd,c} = \frac{A_w}{s} f_{yd} \cot \theta$$

and

$$V_{rd,max} = \alpha_{vw} b_w c v_{rd} / (\cot \theta + \tan \theta)$$

where:

- $A_w$ is the cross-sectional area of the shear reinforcement,
- $s$ is the spacing of the stirrups,
- $f_{yd}$ is the design yield strength of the shear reinforcement,
- $v_1$ is a strength reduction factor for concrete cracked in shear, and
- $\alpha_{vw}$ is a coefficient taking account of the state of the stress in the compression chord.

The recommended value of $v_1$ is 0.6 for $f_{ck} \leq 60$ MPa and $0.9 - f_{ck} / 200 > 0.5$ for $f_{ck} \geq 60$ MPa.

### 2.4.3 Crack Spacing Analysis

**REBAP, 1983**

According to the Portuguese design code for reinforced and pre-stressing concrete structures (REBAP, 1983), an average crack spacing of the stabilized crack propagation phase can be calculated for the elements under tensile or bending loads using the following formula

$$s_{w,c} = 2 \left( c + \frac{s}{10} \right) + \eta_1 \cdot \eta_2 \cdot \frac{\phi}{\rho_c}$$

where:

- $c$ is the concrete cover of the longitudinal reinforcement,
- $s$ is the rebar spacing ($s \leq 15\phi$),
- $\eta_i$ is the steel reinforcement bond condition parameter, given by 0.4 and 0.8 for high or normal bond rebars, respectively.
- $\eta_1$ is a coefficient that depends on the distribution of the tensile stress developed at the cross-section, given by

$$\eta_2 = 0.25 \cdot \frac{\varepsilon_1 + \varepsilon_2}{2 \cdot \varepsilon_1}$$

where $\varepsilon_1$ and $\varepsilon_2$ are the strain in the bottom and top level of the concrete surrounding the tension reinforcement, respectively.
- $\phi$ is the steel reinforcement diameter,
- $\rho_c$ is the effective reinforcement ratio given by $A_s / A_{ce}$, where $A_s$ is the steel reinforcement area and $A_{ce}$ is the effective area of concrete in tension.

In order to consider the contribution of the CFRP laminates, each CFRP laminate is transformed in an equivalent steel bar of cross-section $A_{eq}^s$ and diameter $\phi_{eq}^s$, by applying the following expressions:

$$A_{eq}^s = \frac{E_s}{E_t} \cdot A_{CFRP}$$
in which

\( E_{\text{CFRP}} \) is the modulus of elasticity of the CFRP laminate,

\( E_s \) is the modulus of elasticity of the steel reinforcement, and

\( A_{\text{CFRP}} \) is the cross-sectional area of the CFRP laminate.

For the slabs including CFRP laminates, Equation (1) can be transformed into the following expression:

\[
\frac{s_{\text{m}}}{\ell_{s,\text{m}}} = 2\left( \frac{c_m + s_m}{10} \right) + \eta_1 \cdot \frac{\phi_m \cdot \rho_s}{\rho_c} \tag{21}
\]

where:

- \( c_m \) is the average concrete cover of the longitudinal reinforcement, given by the average of steel and equivalent bars \( \left( = \frac{c_s + c_s^{\text{eq}}}{2} \right) \)
- \( s_m \) is the average rebar spacing \( (s \leq 15\phi) \),
- \( \eta_1 \) is the steel reinforcement bond condition parameter, given by 0.4 and 0.8 for high or normal bond rebars, respectively.
- \( \eta_2 \) is a coefficient that depends on the distribution of the tensile stress developed at the cross-section, given by
  \[
  \eta_2 = 0.25 \cdot \varepsilon_1 + \varepsilon_2 \cdot 2 \cdot \varepsilon_1
  \]
  where \( \varepsilon_1 \) and \( \varepsilon_2 \) are the strain in the bottom and top level of the concrete surrounding the tension reinforcement, respectively.
- \( \phi \) is the steel reinforcement diameter,
- \( \rho_s \) is the effective reinforcement ratio given by \( (A_s + A_s^{\text{eq}}) / A_{s,\text{eff}} \), where \( A_s \) is the steel reinforcement area and \( A_{s,\text{eff}} \) is the effective area of concrete in tension.

**CEB-FIB Model Code (1990)**

According to the CEB-FIP Model Code (1990), the average crack spacing, at stabilized cracking phase, may be estimated using the length over which slip between steel and concrete occurs, according to the following Equation:

\[
\frac{s_{\text{m}}}{\ell_{s,\text{m}}} \approx \frac{2}{3} \cdot \frac{\phi_s}{3.6 \cdot \rho_{s,\text{eff}}} \tag{22}
\]

where

- \( \ell_{s,\text{m}} = \frac{\phi_s}{3.6 \cdot \rho_{s,\text{eff}}} \) for stabilized cracking phase
- \( \ell_{s,\text{m}} \) is the length of slipping between steel and concrete,
- \( \phi_s \) is the steel bar diameter,
- \( \rho_{s,\text{eff}} \) is the effective reinforcement ratio \( (= A_s / A_{s,\text{eff}}) \),
- \( A_{s,\text{eff}} \) is the effective area of concrete in tension, the area of concrete surrounding the tension reinforcement.

For the slabs strengthened with CFRP material, each laminate was converted in an equivalent steel bar (of area \( A_s^{\text{eq}} \) and diameter \( \phi_s^{\text{eq}} \)) as described in previous sections.

The parameters, therefore, can be adjusted for strengthened members:
\[ \ell_{s,\text{max}} = \frac{\phi_m}{3.6 \cdot \rho_{s,\text{ef}}} \]  \hspace{1cm} (23)

where

- \( \phi_m \) is the mean bar diameter, which is the average of steel and equivalent steel bars,
- \( \rho_{s,\text{ef}} \) is the effective reinforcement ratio \( (= A_s + A_{s,\text{eq}} / A_{s,c}) \),
- \( A_{s,c} \) is the area of concrete surrounding the tension reinforcement, steel and equivalent steel bars, and was computed using the mean concrete cover and mean bar diameter.


EC 2 (2000) provides Equation 24 for RC members subjected mainly to flexure and direct tension. The final average crack spacing can be estimated from:

\[ s_m = 50 + 0.25 \cdot k_1 \cdot k_2 \cdot \frac{\phi}{\rho_s} \]  \hspace{1cm} (24)

where

- \( k_1 \) accounts for the bond properties of the bar and - for flexural cracking 0.8 for high bond bars and 1.6 for plain bars);
- \( k_2 \) depends on the strain distribution (0.5 for bending and 1.0 for pure tension);
- \( \phi \) is the steel bar diameter;
- \( \rho_s \) is the effective reinforcement ratio \( (= A_s / A_{s,\text{ef}} \) , where \( A_s \) is the area of reinforcement contained within the effective tension area and \( A_{s,\text{ef}} \) is the effective area of concrete in tension, the area of concrete surrounding the tension reinforcement.

For the slabs including CFRP laminates, the cross-section area of each CFRP laminate \( A_{\text{CFRP}} \) is transformed in an equivalent steel area \( A_{s,\text{eq}} \), as shown before.

**EUROCODE 2 (2004)**

EUROCODE 2 (2004) proposes Equation 25 for the evaluation of the average crack spacing of RC members subjected mainly, to flexure and direct tension,

\[ s_m = k_3 \cdot c + k_4 \cdot k_2 \cdot k_4 \cdot \frac{\phi}{\rho_{s,\text{ef}}} \]  \hspace{1cm} (25)

where

- \( \phi \) is the bar diameter;
- \( c \) is the cover of the longitudinal reinforcement;
- \( k_3 \) accounts for the bond properties of the bar - for flexural cracking (0.8 for high bond bars and 1.6 for plain bars);
- \( k_4 \) is a coefficient which takes into account of the distribution of strain (0.5 for bending and 1.00 for pure tension);

The recommended values of \( k_3 \) and \( k_4 \) are 3.4 and 0.425, respectively;
\( \rho_{\text{eff}} \) is the effective reinforcement ratio \( (= A_f/A_{\text{c,ef}}) \), where \( A_f \) is the cross-section area of the reinforcement contained within the effective tension area, \( A_{\text{c,ef}} \) is the effective area of concrete in tension, the area of concrete surrounding the tension reinforcement. For the slabs including CFRP laminates, the cross-section area of each CFRP laminate \( (A_{\text{CFRP}}) \) is transformed in an equivalent steel area \( (A^*) \).

### 2.4.4 Bond of NSM System

For NSM systems, when using a rectangular bar with large aspect ratio, a minimum groove size of \( 3.0a_b \times 1.5b_b \), as depicted in Figure 8, is suggested, where \( a_b \) is the smallest bar dimension.

![Minimum groove size](image)

Figure 8 – Minimum groove size.

The minimum clear groove spacing for NSM FRP bars should be greater than twice the depth of the NSM groove to avoid overlapping of the tensile stresses around the NSM bars. Furthermore, a clear edge distance of four times the depth of the NSM groove should be provided to minimize the edge effects that could accelerate debonding failure (Hassan and Rizkalla 2003). Bond properties of the NSM FRP bars depend on many factors such as cross-sectional shape and dimensions and surface properties of the FRP bar (Hassan and Rizkalla 2003; De Lorenzis et al. 2004). Figure 9 shows the equilibrium condition of an FRP bar with an embedded length equal to its development length \( l_{db} \) having a bond strength of \( \tau_{\text{max}} \).

![Equilibrium condition of an FRP bar](image)

Figure 9 – Equilibrium condition of an FRP bar.

Using a triangular stress distribution, the average bond strength can be expressed as \( \tau_{\text{max}} = 0.5\tau_{\text{max}} \). Average bond strength \( \tau_{\text{b}} \) for NSM FRP bars in the range of 3.5 to 20.7 MPa has been reported (Hassan and Rizkalla 2003; De Lorenzis et al. 2004); therefore, \( \tau_{\text{max}} = 6.9\text{MPa} \) is recommended for calculating the bar development.
length. Via force equilibrium, the following equations for development length can be derived for rectangular bars:

\[
 l_{db} = \frac{a_f b_f}{2(a_h + b_h)(\tau_e)} f_{\mu} \quad (26)
\]

Considering \( f_{\mu} = (0.7)(2500) = 1750\text{MPa} \)
and \( \varepsilon_{\mu} = (0.7)(16) = 11.2\% \)

\[
 l_{db} = \frac{(1.4\text{mm})(9.4\text{mm})}{2(1.4\text{mm} + 9.4\text{mm})(6.9\text{MPa})} (1750\text{MPa}) = 154.52\text{mm}
\]

\[
 l_{db} = \frac{(1.4\text{mm})(20\text{mm})}{2(1.4\text{mm} + 20\text{mm})(6.9\text{MPa})} (1750\text{MPa}) = 165.92\text{mm}
\]

Considering \( f_{\mu} = 2500\text{MPa} \)
and

\[
 l_{db} = \frac{(1.4\text{mm})(9.4\text{mm})}{2(1.4\text{mm} + 9.4\text{mm})(6.9\text{MPa})} (2500\text{MPa}) = 220.75\text{mm}
\]

\[
 l_{db} = \frac{(1.4\text{mm})(20\text{mm})}{2(1.4\text{mm} + 20\text{mm})(6.9\text{MPa})} (2500\text{MPa}) = 237.03\text{mm}
\]

To achieve the debonding design strain of NSM FRP bars \( \varepsilon_{\mu} \), the bonded length should be greater than the development length given in Equation (26).

### 2.5 Measuring Devices

Figure 10 shows the arrangement of the test setup used in the experimental program, formed by continuous RC slab strips simply supported with two equal spans and two concentrated loads applied at the middle of each span. It should be mentioned that a servo-controlled test equipment with two independent actuators was used in the experimental program.

To measure the vertical deflection along each span of the slab strip, six displacement transducers (LVDT 82803, LVDT 60541, LVDT 82804, LVDT 19906, LVDT 18897, and LVDT 3468) were applied, as indicated in Figure 10. The LVDTs 60541 and 18897 were used to control the actuators used to apply the load at the middle of the spans of the slab strips. These LVDTs controlled the test at a displacement rate of 10µm/s up to the deflection of 50 mm. After this deflection, the actuators internal LVDTs were used to control the test at a displacement rate of 20µm/s.
The force \( F_{522} \) applied at the left span was measured using a load cell of ±200 kN and accuracy of ±0.03% (designated Ctrl_1), placed between the loading steel frame and the actuator of 150 kN load capacity and 200 mm range. In the right span, the load \( F_{123} \) was applied with an actuator of 100 kN and 200 mm range, and the corresponding force was measured using a load cell of ±250 kN and accuracy of ±0.05% (designated Ctrl_2).

To monitor the reaction forces, load cells were installed under two supports. One load cell (CELL AEP_200) was positioned at the central support (nonadjustable support), placed between the reaction steel frame (HEB 300 profile) and the support device. The other load cell (CELL MIC_200) was positioned in-between the reaction steel frame and the apparatus of the adjustable right support. These cells have a load capacity of 200 kN and accuracy of ±0.05%.

The translational movements in the vertical and horizontal (longitudinal axis of the slab strip) directions, and the rotations along these axes were restrained in the central support. In the other two supports only the translational movements and rotations along the vertical axis were restrained. To evaluate the rotation of the supports in which the load cells were installed two linear displacement transducer were used, LVDT 47789 and LVDT 61531 at the central support, and LVDT 31923 and LVDT 50855, at the right support, respectively (see Figure 10).

Figures 11 to 13 show the arrangements of strain gauges for the slab strips of series SL15, SL30 and SL45, respectively. Parts (a) and (b) of these figures present the lay-out of the strain gauges at the steel bars, in the negative and positive longitudinal reinforcements, respectively. Part (c) shows the lay-out of the strain gauges at the concrete under compression, in the negative and positive moment regions. The last parts of the figures, parts (d) and (e), show the positions of the strain gauges at one CFRP laminate, in the negative region of the strengthened specimens.
To monitor the strain variation in the internal steel bar reinforcement, concrete and CFRP laminate strengthening, a total of 17 strain gauges were installed for the unstrengthened slab strips, while 26 strain gauges were applied in the strengthened ones (Figures 11 to 13). Eleven SGs were installed in the steel bars, seven of them in steel bars at top surface in the region of the intermediate support (SG1 to SG7) and the other four in steel bars at bottom surface in the regions of the applied loads (SG8 to SG11, Figures 11(a), 11(b), 12(a), 12(b), and 13(a) and 13(b). Six SGs were (SG12 to SG17) applied at the external concrete surface in the compression regions (Figures 11(c), 12(c) and 13(c). Finally, three SGs (SG18 to SG20) were installed along one CFRP laminate in the region of maximum negative moment (at the central support section) and three SGs (SG21 to SG26) were bonded along two CFRP laminates in the region of maximum positive moment (loaded section, Figures 11(d), 11(e), 12(d) and 12(e), 13(d) and 13(e). In this experimental program FLA-3-11, FLA-3-11-3L and PL-60-11 strain gauges from TML were used in steel bars, CFRP laminates and concrete, respectively.

The main technical characteristics of the used displacement transducers are included in Table 10.

<table>
<thead>
<tr>
<th>LVDT device</th>
<th>Linear range (mm)</th>
<th>Linearity deviation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>82803</td>
<td>±25</td>
<td>±0.09</td>
</tr>
<tr>
<td>60541a</td>
<td>±50</td>
<td>±0.31</td>
</tr>
<tr>
<td>80804</td>
<td>±25</td>
<td>±0.10</td>
</tr>
<tr>
<td>19906</td>
<td>±25</td>
<td>±0.07</td>
</tr>
<tr>
<td>18897b</td>
<td>±50</td>
<td>±0.08</td>
</tr>
<tr>
<td>3468</td>
<td>±25</td>
<td>±0.08</td>
</tr>
<tr>
<td>47789</td>
<td>±2.5</td>
<td>±0.06</td>
</tr>
<tr>
<td>61531</td>
<td>±2.5</td>
<td>±0.09</td>
</tr>
<tr>
<td>50855</td>
<td>±2.5</td>
<td>±0.09</td>
</tr>
<tr>
<td>31923</td>
<td>±2.5</td>
<td>±0.16</td>
</tr>
</tbody>
</table>

* Control the actuator placed at the left span (F<sub>22</sub>)
* Control the actuator placed at the right span (F<sub>123</sub>)

The experimental work was divided into two parts. In phase I, the slabs were pre-loaded to mimic the behaviour in-service rehabilitated slab strips. Therefore, according to the load-displacement response obtained at the reference slab, the damage was inflicted by loading the slab up to 50% of the displacement required to achieve the yield of the steel reinforcement at the hogging region (IS). Then, the mid-span displacement level registered in the end of the first phase of the test was sustained during the strengthening
process up to the test date by using a steel profile IPE 100 anchored to the steel frame by a 20 mm diameter steel tie-rod. To transfer uniformly each applied vertical load (F) to the entire width of the slab strip, a rigid steel plate (375 mm × 70 mm × 40 mm) was placed in between the steel profile and the slab specimen. Finally, four dials gauges were placed between the loading steel frame and the slab in order to maintain the same mid-span displacement registered by the actuators at LS in the first phase of the test.

Phase II of the program consisted on applying the CFRP laminate strips according to the NSM technique into the concrete cover of the specimens with initial damage via pre-cut slits. Afterwards, the strengthened specimens were loaded up to failure following the cure of the adhesive. The apparatus used to sustain and control the mid-span displacement is shown in Figure 14.
Figure 11 – Arrangements of strain gauges for the slab strips of series SL15.

(a) top view – lay-out of the strain gauges at the steel bars in the negative longitudinal reinforcement. (b) lay-out of the strain gauges at the steel bars in the positive longitudinal reinforcement – bottom view. (c) strain gauges at the concrete compressive surfaces – top and bottom view.
Figure 12 – Arrangements of strain gauges for the slab strips of series SL30.

(a) top view – lay-out of the strain gauges at the steel bars in the negative longitudinal reinforcement, (b) lay-out of the strain gauges at the steel bars in the positive longitudinal reinforcement – bottom view, (c) strain gauges at the concrete compressive surfaces – top and bottom view.
Figure 13 – Arrangements of strain gauges for the slab strips of series SL45.
(a) top view – lay-out of the strain gauges at the steel bars in the negative longitudinal reinforcement, (b) lay-out of the strain gauges at the steel bars in the positive longitudinal reinforcement – bottom view, (c) strain gauges at the concrete compressive surfaces – top and bottom view.

Figure 14 – Apparatus to sustain and control the mid-span displacement level applied on the specimen during the strengthening process.

2.6 MATERIALS CHARACTERIZATION
The following sections detail the various materials used in the present experimental program.

2.6.1 CONCRETE READY-MIXES
The detailed concrete mix proportions and the main properties of the ordinary ready-mix concretes used in the construction of the slab strips are shown in Table 11. Cylinder specimens with a diameter of 150 mm and a
height of 300 mm were used to obtain the compressive strength and the Young Modulus of the ready-mix concretes, determined according to E397 (1993).

Table 11 – Mixture proportions and main properties of the ready-mix concretes.

<table>
<thead>
<tr>
<th>Components</th>
<th>Mixture designations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SL15</td>
</tr>
<tr>
<td>Cement II 42.5 R (kg/m³)</td>
<td>-</td>
</tr>
<tr>
<td>Fly ash (kg/m³)</td>
<td>-</td>
</tr>
<tr>
<td>Fine river sand (kg/m³)</td>
<td>-</td>
</tr>
<tr>
<td>Coarse river sand (kg/m³)</td>
<td>-</td>
</tr>
<tr>
<td>Brita 1 – 4 a 10mm (kg/m³)</td>
<td>-</td>
</tr>
<tr>
<td>Course aggregate (kg/m³)</td>
<td>-</td>
</tr>
<tr>
<td>W/B Ratio</td>
<td>-</td>
</tr>
<tr>
<td>Plasticizer (kg)</td>
<td>-</td>
</tr>
<tr>
<td>CHRYSOPLAST Superplasticizer (kg)</td>
<td>-</td>
</tr>
<tr>
<td>Slump (mm)</td>
<td>-</td>
</tr>
<tr>
<td>$f_{c,28d}$ (MPa)</td>
<td>26.37</td>
</tr>
</tbody>
</table>
Table 12 shows detailed information regarding the compressive properties of the three concrete mixes at the age of 28 days and at the testing age of the slab strips. The concrete compressive strength, at the slab testing age, was measured from at least four cylinders with diameter equal to 150 mm and height equal to 300 mm, immediately after testing the slabs.

<table>
<thead>
<tr>
<th>Slab ID</th>
<th>At 28 days</th>
<th>At the slabs testing age</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{cm}$ (N/mm$^2$)</td>
<td>$E_{cm}$ (kN/mm$^2$)</td>
</tr>
<tr>
<td></td>
<td>$E_{cm}$ (kN/mm$^2$)</td>
<td></td>
</tr>
<tr>
<td>SL15</td>
<td>26.37 (1.06)</td>
<td>24.43 (1.07)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SL30</td>
<td>28.40 (1.61)</td>
<td>29.83 (0.29)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SL45</td>
<td>42.14 (0.42)</td>
<td>28.32 (1.54)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(value) Standard deviation.


2.6.2 Reinforcing Steel

The mechanical properties of the steel reinforcement bars were determined experimentally indirect tension tests by Bonaldo (2008) and are indicated in Table 13.

Table 13 – Mechanical properties of the reinforcing steel.

<table>
<thead>
<tr>
<th>Steel bar diameter (ϕs)</th>
<th>Sample ID</th>
<th>Modulus of Elasticity (kN/mm²)</th>
<th>Yield stress (0.2%)a (N/mm²)</th>
<th>Strain at yield stressb</th>
<th>Tensile strength (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 mm</td>
<td>1</td>
<td>187.216</td>
<td>449.28</td>
<td>0.0026</td>
<td>568.28</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>206.373</td>
<td>449.28</td>
<td>0.0024</td>
<td>569.09</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>187.819</td>
<td>444.42</td>
<td>0.0026</td>
<td>562.61</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>193.803</td>
<td>447.66</td>
<td>0.0025</td>
<td>566.66</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>10.890 (5.62%)</td>
<td>2.81 (0.63%)</td>
<td>0.0001 (4.68%)</td>
<td>3.53 (0.62%)</td>
</tr>
<tr>
<td>8 mm</td>
<td>1</td>
<td>195.402</td>
<td>423.93</td>
<td>0.0024</td>
<td>578.30</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>203.159</td>
<td>420.29</td>
<td>0.0023</td>
<td>576.93</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>203.838</td>
<td>419.83</td>
<td>0.0023</td>
<td>581.03</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>200.80</td>
<td>421.35</td>
<td>0.0023</td>
<td>578.75</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>4.687 (2.33%)</td>
<td>2.25 (0.53%)</td>
<td>0.0001 (2.65%)</td>
<td>2.09 (0.36%)</td>
</tr>
<tr>
<td>10 mm</td>
<td>1</td>
<td>183.329</td>
<td>463.37</td>
<td>0.0027</td>
<td>576.44</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>175.858</td>
<td>441.80</td>
<td>0.0027</td>
<td>573.82</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>175.518</td>
<td>435.68</td>
<td>0.0027</td>
<td>577.61</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>178.235</td>
<td>446.95</td>
<td>0.0027</td>
<td>575.95</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>4.415 (2.48%)</td>
<td>14.55 (3.25%)</td>
<td>0.0000 (0.45%)</td>
<td>1.94 (0.34%)</td>
</tr>
<tr>
<td>12 mm</td>
<td>1</td>
<td>192.199</td>
<td>427.83</td>
<td>0.0024</td>
<td>528.41</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>200.108</td>
<td>449.69</td>
<td>0.0024</td>
<td>545.41</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>202.759</td>
<td>449.89</td>
<td>0.0024</td>
<td>545.82</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>198.356</td>
<td>442.47</td>
<td>0.0024</td>
<td>539.88</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>5.494 (2.77%)</td>
<td>12.68 (2.87%)</td>
<td>0.0000 (0.19%)</td>
<td>9.93 (1.84%)</td>
</tr>
</tbody>
</table>

a Yield stress determined by the "Offset Method", according to ASTM 370 (2002)

b Strain at yield point, for the 0.2 % offset stress

(value) Coefficient of Variation (COV) = (Standard deviation/Average) × 100
2.6.3 CFRP LAMINATES

The results for the failure tensile stress, failure tensile strain and modulus of elasticity of the CFRP samples tested are included in Table 14. To determine the tensile mechanical properties of the CFRP laminate, five tensile tests in coupon specimens were carried out according to ISO 527-5 (1993) and ASTM 3039 (1993).

Table 14 – Mechanical properties of the reinforcing steel.

<table>
<thead>
<tr>
<th>CFRP laminate height</th>
<th>Sample ID</th>
<th>Ultimate tensile stress (N/mm²)</th>
<th>Ultimate tensile strain (%)</th>
<th>Modulus of Elasticity a (kN/mm²)</th>
<th>Modulus of Elasticity b (kN/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 mm</td>
<td>1</td>
<td>2879.13</td>
<td>18.45</td>
<td>156.100</td>
<td>165.500</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2739.50</td>
<td>17.00</td>
<td>158.800</td>
<td>161.400</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>2952.00</td>
<td>17.70</td>
<td>166.600</td>
<td>166.600</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>2942.32</td>
<td>17.81</td>
<td>153.620</td>
<td>163.970</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>2825.20</td>
<td>17.40</td>
<td>161.400</td>
<td>167.030</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>2867.63</td>
<td>17.67</td>
<td>159.304</td>
<td>164.900</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td></td>
<td>88.10 (3.07%)</td>
<td>0.54 (3.04%)</td>
<td>5.01 (3.15%)</td>
<td>2.29 (1.39%)</td>
</tr>
</tbody>
</table>

| 20 mm                | 1         | 2858.799                       | 18.37303                   | 155.5976                        | -                               |
|                      | 2         | 2782.862                      | 17.6256                    | 157.8875                        | -                               |
|                      | 3         | 2706.926                      | 17.28808                   | 156.5775                        | -                               |
| Average              |           | 2782.86                       | 17.76                      | 156.69                          | -                               |
| Std. Dev.            |           | 75.94 (2.73%)                 | 0.56 (3.13%)               | 1.15 (0.73%)                    | -                               |

a According to ISO 527-1 and ISO 527-5 (1993)
b Tensile Chord Modulus of Elasticity, according to ACI (2002) and ASTM 3039 (1993)

(value) Coefficient of Variation (COV) = (Standard deviation/Average) × 100

2.7 SPECIMENS PREPARATION AND STRENGTHENING

The procedures of the specimens' preparation and the NSM strengthening technique are described in the followings sections.

2.7.1 PREPARATION OF THE SLAB STRIP SPECIMENS

Nine reinforced concrete slabs strips of about 375 mm × 5850 mm, with 120 mm thickness, were grouped in three series of three slab strips and cast at distinct periods from different concrete mixes. For each concrete batch, sixteen cylindrical concrete specimens, of 150 mm diameter and 300 mm depth were cast, four for compressive strength control at 28 days of age and twelve for compressive strength control at the slabs
testing age. The slabs and the cylinders moulds used are shown in Figure 15(a). For each concrete batch, the slabs were cast in two layers, each one vibrated using an electrical concrete poker vibrator with a 25 mm tip and 50 Hz frequency, see Figure 15.

![Figure 15](image-url.png)

**Figure 15 – Slab strips specimens: formwork setup (a), concrete casting (b), concrete vibration (c) and final aspect (d).**

After casting the slabs and the cylinders concrete specimens, their top surfaces were immediately finished manually using a smooth plastic float and were covered with wet burlap sacks, which were monitored and kept wet for two days. After this curing period, the slabs and the cylinders specimens were removed from the moulds and maintained in natural laboratory environmental conditions up to 28 days.

### 2.7.2 NSM STRENGTHENING TECHNIQUE

After the pre-cracking phase, the specimens designed to increase 25% or 50% the negative moment were strengthened.
Each NSM specimen was prepared by first saw cutting pre-grooves perpendicular to the concrete surface. They were made with a Black&Decker diamond saw cut machine (model CD-115) and were used as a guide for a straight line. Afterwards, the grooves were made using a Hilti diamond saw cutter machine, model DC 230-S (HILTI, 2004). The slits had about 4.5 or 4.6 mm width and 15 or 27 mm depth on the concrete cover of the slab’s surface that will be in tension for the laminate strips of 10 or 20 mm height, respectively.

In order to eliminate the dust resultant from the sawing process, the slits were cleaned using compressed air before bonding the laminate strips to the concrete into the slits. The CFRP laminates were cleaned with acetone to remove any possible dirt.

Figure 16 shows an overview before installing the CFRP laminates into the slits. The laminate strips were fixed to the concrete slits using an epoxy adhesive. The slits were filled with the epoxy adhesive using a spatula, and the CFRP laminates were then gradually introduced into the slits.
2.8 RESULTS AND DISCUSSION

The results and discussion of the six tested slab strips are presented in this chapter, focusing on the load-displacement response, load carrying capacity, failure mode and slab’s ductility, as well as bond stress along the laminate strips.

2.8.1 UNSTRENGTHENED SLAB STRIPS

2.8.1.1 SL15 slab strip

The unstrengthened SL15 slab strip is shown in Figure 17, before and after having been tested. The span deflections, steel reinforcement strain, concrete strain, support reaction, and rotation of the reaction devices are shown in Figures 18 to 23, respectively.
Figure 17 – SL15 specimen before (a), and after having been tested (b).

Figure 18 – Relationship between applied load and deflections at spans of the SL15.
Figure 19 – Relationship between average load and tensile strain of the negative longitudinal steel reinforcement for the SL15 slab strip.

Figure 20 – Relationship between average load and tensile strain of the positive longitudinal steel reinforcement for the SL15 slab strip.
Figure 21 – Relationship between average load and compressive strain of the concrete at sagging region for the SL15 slab strip.

Figure 22 – Relationship between average load and compressive strain of the concrete at hogging region for the SL15 slab strip.
Figure 23 – Relationship between average load and support devices rotation for the SL15 slab strip.

Figure 24 shows the distribution of the cracks at the slab strip at hogging and sagging regions at the end of the test. Furthermore, the cracks formed at the side face of the specimen at IS are shown in Figure 24(c). At failure, an average crack spacing of 79.27 mm was measured, where approximately 10 flexural cracks were formed over the hogging region of each span, with the last flexural cracks in the region at approximately 580 mm from the intermediate support.
2.8.1.2 SL30 slab strip

The unstrengthened SL30 slab strip is shown in Figure 25, before and after having been tested. The span deflections, steel reinforcement strain, concrete strain, support reaction, and rotation of the reaction devices are shown in Figures 26 to 32, respectively.
Figure 25 – SL30 specimen before (a), and after having been tested (b).

Figure 26 – Relationship between applied load and deflections at spans of the SL30.
Figure 27 – Relationship between average load and tensile strain of the negative longitudinal steel reinforcement for the SL30 slab strip.

Figure 28 – Relationship between average load and tensile strain of the positive longitudinal steel reinforcement for the SL30 slab strip.
Figure 29 – Relationship between average load and compressive strain of the concrete at sagging region for the SL30 slab strip.

Figure 30 – Relationship between average load and compressive strain of the concrete at hogging region for the SL30 slab strip.
Figure 31 – Relationship between average load and support devices rotation for the SL30 slab strip.

Figure 32 – Relationship between the applied load and support reaction for the SL30 slab strip.

Figure 33 shows the distribution of the cracks at the slab strip at hogging and sagging regions at the end of the test. Furthermore, the cracks formed at the side face of the specimen at IS are shown in Figure 33(c). At failure, an average crack spacing of 76 mm was measured, where approximately 11 flexural cracks were formed over the hogging region of each span, with the last flexural cracks in the region at approximately 40 mm from the intermediate support.
2.8.1.3 SL45 slab strip

The unstrengthened SL45 slab strip is shown in Figure 34, before and after having been tested. The span deflections, steel reinforcement strain, concrete strain, support reaction, and rotation of the reaction devices are shown in Figures 35 to 41, respectively.
Figure 34 – SL45 specimen before (a), and after having been tested (b).

Figure 35 – Relationship between applied load and deflections at spans of the SL45.
Figure 36 – Relationship between average load and tensile strain of the negative longitudinal steel reinforcement for the SL45 slab strip.

Figure 37 – Relationship between average load and tensile strain of the positive longitudinal steel reinforcement for the SL45 slab strip.
Figure 38 – Relationship between average load and compressive strain of the concrete at sagging region for the SL45 slab strip.

Figure 39 – Relationship between average load and compressive strain of the concrete at hogging region for the SL45 slab strip.
Figure 40 – Relationship between average load and support devices rotation for the SL45 slab strip.

Figure 41 – Relationship between the applied load and support reaction for the SL45 slab strip.

Figure 42 shows the distribution of the cracks at the slab strip at hogging and sagging regions at the end of the test. Furthermore, the cracks formed at the side face of the specimen at IS are shown in Figure 42(c). At failure, an average crack spacing of 79.41 mm was measured, where approximately 9 flexural cracks were formed over the hogging region of each span, with the last flexural cracks in the region at approximately 420 mm from the intermediate support.
Figure 42 – Distribution of the cracks at hogging (a) and sagging (b) region and (c) cracks formed at the side face of the specimen.

2.8.2 STRENGTHENED SLAB STRIPS

2.8.2.1 SL15s25 slab strip

The strengthened SL15s25 slab strip is shown in Figure 43, before and after having been tested. The span deflections, steel reinforcement strain, CFRP laminate strips strain, concrete strain, support reaction, and rotation of the reaction devices are shown in Figures 44 to 52, respectively.
Figure 43 – SL15s25 specimen before (a), and after having been tested (b).

Figure 44 – Relationship between applied load and deflections at spans of the SL15s25.
Figure 45 – Relationship between average load and tensile strain of the negative longitudinal steel reinforcement for the SL15s25 slab strip.

Figure 46 – Relationship between average load and tensile strain of the positive longitudinal steel reinforcement for the SL15s25 slab strip.
Figure 47 – Relationship between average load and compressive strain of the concrete at sagging region for the SL15s25 slab strip.

Figure 48 – Relationship between average load and compressive strain of the concrete at hogging region for the SL15s25 slab strip.
Figure 49 – Relationship between average load and compressive strain of the laminate strips at sagging regions for the SL15s25 slab strip.

Figure 50 – Relationship between average load and compressive strain of the laminate strips at hogging regions for the SL15s25 slab strip.
The slab strip had two and four CFRP laminate strips of 1.4mm x 20 mm mounted to the tension face of the specimen over the sagging and hogging regions, respectively. At the first phase of the test, the slab strip was loaded up to a displacement of about 5.40 mm, which corresponds to an applied load of about 14 kN. Flexural cracking was first observed at an applied load of 6 kN, which correspond to a reaction force of 2.17 kN at the external support. This caused a maximum moment at the sagging and hogging regions of 3.04 and 2.39 kN.m,
respectively. Upon further loading, lots of flexural cracks formed over the hogging region, as shown in Figure 53(a). The increase of the load increases the number of flexural cracks and formed herringbone cracks along the CFRP laminates at hogging regions. Figures 53(b) to 53(d) show the distribution of the herringbone cracks at the slab strip failure where it can be noticed the CFRP laminate cover separation.

Concerning to the sagging region, flexural cracking was first observed at an applied load of 6 kN and increased up to the failure of the slab strip, as shown in Figure 54.
2.8.2.2 SL15s50 slab strip

The strengthened SL15s50 slab strip is shown in Figure 55, before and after having been tested. The span deflections, steel reinforcement strain, concrete strain, support reaction, and rotation of the reaction devices are shown in Figures 56 to 64, respectively.
Figure 55 – SL15s50 specimen before (a), and after having been tested (b).

Figure 56 – Relationship between applied load and deflections at spans of the SL15s50.
Figure 57 – Relationship between average load and tensile strain of the negative longitudinal steel reinforcement for the SL15s50 slab strip.

Figure 58 – Relationship between average load and tensile strain of the positive longitudinal steel reinforcement for the SL15s50 slab strip.
Figure 59 – Relationship between average load and compressive strain of the concrete at sagging region for the SL15s50 slab strip.

Figure 60 – Relationship between average load and compressive strain of the concrete at hogging region for the SL15s50 slab strip.
Figure 61 – Relationship between average load and compressive strain of the laminate strips at sagging regions for the SL15s50 slab strip.

Figure 62 – Relationship between average load and compressive strain of the laminate strips at hogging regions for the SL15s50 slab strip.
The slab strip had three CFRP laminate strips mounted to the tension face of the specimen over the sagging region: one of 1.4 mm x 10 mm and two of 1.4 mm x 20 mm. Furthermore, four CFRP laminate strips of 1.4 mm x 20 mm were applied at hogging region. At the first phase of the test, the slab strip was loaded up to a displacement of about 5.40 mm, which corresponds to an applied load of about 14 kN. Flexural cracking was
first observed at an applied load of 5 kN, which correspond to a reaction force of 1.81 kN at the external support. This caused a maximum moment at the sagging and hogging regions of 2.53 and 2.03 kN.m, respectively. Upon further loading, lots of flexural cracks formed over the hogging region, as shown in Figure 65(a). The increase of the load increases the number of flexural cracks and formed herringbone cracks along the CFRP laminates at hogging regions. Figures 65(b) to 65(d) show the distribution of the herringbone cracks at the slab strip failure where it can be noticed the CFRP laminate cover separation.

Concerning to the sagging region, flexural cracking was first observed at an applied load of 4 kN and increased up to the failure of the slab strip, as shown in Figure 66.

![Image](a)

![Image](b)

![Image](c)
2.8.2.3 SL30s25 slab strip

The strengthened SL30s25 slab strip is shown in Figure 67, before and after having been tested. The span deflections, steel reinforcement strain, concrete strain, support reaction, and rotation of the reaction devices are shown in Figures 68 to 76, respectively.

Figure 65 – Distribution of the cracks at hogging region along the test: (a) 14 kN, (b) before and (c-d) after failure.

Figure 66 – Distribution of the cracks at sagging region along the test: (a) end of the first phase and (b) after failure of the slab.
Figure 67 – SL30s25 specimen before (a), and after having been tested (b).

Figure 68 – Relationship between applied load and deflections at spans of the SL30s25.
Figure 69 – Relationship between average load and tensile strain of the negative longitudinal steel reinforcement for
the SL30s25 slab strip.

Figure 70 – Relationship between average load and tensile strain of the positive longitudinal steel reinforcement for the
SL30s25 slab strip.
Figure 71 – Relationship between average load and compressive strain of the concrete at sagging region for the SL30s25 slab strip.

Figure 72 – Relationship between average load and compressive strain of the concrete at hogging region for the SL30s25 slab strip.
Figure 73 – Relationship between average load and compressive strain of the laminate strips at sagging regions for the SL30s25 slab strip.

Figure 74 – Relationship between average load and compressive strain of the laminate strips at hogging regions for the SL30s25 slab strip.
The slab strip had four CFRP laminate strips mounted to the tension face of the specimen over the sagging regions (two of 10 mm x 1.4 mm and two of 20 mm x 1.4 mm). Additionally, two CFRP laminate strips of 20 mm x 1.4 mm were installed at the hogging region. At the first phase of the test, the slab strip was loaded up to a displacement of about 5.80 mm, which corresponds to an applied load of about 17 kN. Flexural cracking was first observed at an applied load of 6 kN, which corresponds to a reaction force of 1.99 kN at the external support devices rotation.
support. This caused a maximum moment at the sagging and hogging regions of 2.79 and 2.82 kN.m, respectively. Upon further loading, lots of flexural cracks formed over the hogging region, as shown in Figure 77(a). The increase of the load increases the number of flexural cracks and formed herringbone cracks along the CFRP laminates at hogging regions. Figures 77(b) to 77(d) shows the distribution of the herringbone cracks at the slab strip failure where it can be noticed the CFRP laminate cover separation.

Concerning to the sagging region, flexural cracking was first observed at an applied load of 10 kN and increased up to the failure of the slab strip, as shown in Figure 78.

Figure 77 – Distribution of the cracks at hogging region along the test: (a) 17 kN, (b) before and (c-d) after failure.
In Table 15 are included the maximum values registered experimentally for applied load, lateral and central support reaction. In this table the negative moment values were derived from structural analysis. In Table 15 are also included the percentage of increase in negative moment and applied load, from which it can be noticed that the increases in applied load are entirely in agreement with the increases in bending capacity and that target increases in negative moment were satisfactorily attained.

Table 15 – Maximum values registered at experimental test.

<table>
<thead>
<tr>
<th>Slab strip specimen ID</th>
<th>Target increase in negative moment</th>
<th>Negative moment (kN.m)</th>
<th>Positive moment (kN.m)</th>
<th>Maximum load at F 123 (kN)</th>
<th>Central support reaction AEP200 (kN)</th>
<th>Lateral support reaction MIC200 (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL15</td>
<td>-</td>
<td></td>
<td></td>
<td>47.87</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SL15s25</td>
<td>25%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SL15s50</td>
<td>50%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SL30</td>
<td>-</td>
<td>19.87</td>
<td>23.02</td>
<td>47.07</td>
<td>63.94</td>
<td>16.44</td>
</tr>
<tr>
<td>SL30s25</td>
<td>25%</td>
<td>27.76</td>
<td>36.72</td>
<td>72.29</td>
<td>96.95</td>
<td>26.23</td>
</tr>
<tr>
<td>SL30s50</td>
<td>50%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 78 – Distribution of the cracks at sagging region along the test: (a) 17 kN and (b) after failure of the slab.

2.8.3 Deformational Behavior

In Table 15 are included the maximum values registered experimentally for applied load, lateral and central support reaction. In this table the negative moment values were derived from structural analysis. In Table 15 are also included the percentage of increase in negative moment and applied load, from which it can be noticed that the increases in applied load are entirely in agreement with the increases in bending capacity and that target increases in negative moment were satisfactorily attained.
The load-mid span deflection curves of the tested slab strips are presented in Figures 79 to 81. As it can clearly be noticed, the experimental load displacement responses of the unstrengthened and strengthened slabs have a typical quadrilinear diagram, which corresponds to the following behavioural phases: (a) the uncracked elastic response; (b) crack propagation in the negative and positive moment regions (central support and middle spans) with steel bars in elastic stage; (c) steel reinforcement post-yielding stage at the central support and crack propagation in the positive moment regions (spans) with steel bars in elastic stage; (d) steel reinforcement post-yielding stage at the central support and middle spans.

<table>
<thead>
<tr>
<th></th>
<th>-</th>
<th>16.88</th>
<th>28.32</th>
<th>52.52</th>
<th>67.66</th>
<th>20.23</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL45s25</td>
<td>25%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SL45s50</td>
<td>50%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(value) percentage of increase in relation to the reference specimen

Figure 79 – Slab mid-span deflection – SL15 series.
The unstrengthened control slabs behaved in a perfectly plastic manner after the formation of the plastic hinges at intermediate support and loaded sections. It can be noticed an increase of the stiffness of the element when using the NSM CFRP strengthening technique.
2.8.4 Failure Modes

Table 16 presents the deflection at the end of test and the corresponding load, and the failure mechanism registered, for the slab strips of series SL15, SL30 and SL45, respectively. The failure mechanism of the reference slabs (SL15, SL30 and SL45) was governed by flexure failure mode, i.e. by yielding of internal reinforcements, with extensive cracking in the tension flange, followed by concrete crushing in compression parts. The slab strip strengthened to increase 25% (SL15S25) failed in shear at the intermediate support, by intermediate shear crack mechanism with extensive cracking in the tension surface. The failure mechanism was also governed by flexural failure: yielding of the internal steel reinforcements followed by the concrete compression failure and CFRP laminate cover separation. The slab strip strengthened to increase 50% the flexural capacity of central support section, (SL15s50) failed by CFRP laminate cover separation or debonding, followed by yielding of internal reinforcements and by concrete crushing in compression parts. Concerning to the slab strip SL30s25, designed to increase 25% the flexural capacity of central support section, the failure mechanism was governed by laminate cover separation or debonding.

<table>
<thead>
<tr>
<th>Slab strip specimen ID</th>
<th>Ultimate deflection $\delta$ (mm)</th>
<th>Ultimate load $F$ (kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL15</td>
<td>68.92 69.68 69.30</td>
<td>47.43 47.29 47.36</td>
<td>Yielding of internal reinforcement at central support - yielding of internal reinforcement at spans – concrete crushing at central support – concrete crushing at loaded sections.</td>
</tr>
<tr>
<td></td>
<td>69.95 71.51 70.73</td>
<td></td>
<td>Yielding of internal reinforcement at central support / yielding of internal reinforcement at spans – concrete crushing at central support / concrete crushing at loaded sections – shear failure by formation of the intermediate shear crack and CFRP laminate cover separation.</td>
</tr>
<tr>
<td>SL15s25</td>
<td>51.03 52.18 51.61</td>
<td>69.5 71.5 70.73</td>
<td>Yielding of internal reinforcement at central support / yielding of internal reinforcement at spans – concrete crushing at central support / concrete crushing at loaded sections – CFRP laminate cover separation and formation of the intermediate shear crack.</td>
</tr>
<tr>
<td></td>
<td>69.95 69.34 69.24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SL30</td>
<td>55.55 53.97 54.76</td>
<td>48.09 47.01 47.55</td>
<td>Yielding of internal reinforcement at central support - yielding of internal reinforcement at spans – concrete crushing at central support – concrete crushing at loaded sections.</td>
</tr>
<tr>
<td></td>
<td>66.68 66.04 66.36</td>
<td></td>
<td>Yielding of internal reinforcement at central support / yielding of internal reinforcement at spans – concrete crushing at central support / concrete crushing at loaded sections – CFRP laminate cover separation.</td>
</tr>
</tbody>
</table>

Table 16 – Failure Modes.
cover separation and formation of the intermediate shear crack.

Yielding of internal reinforcement at central support - yielding of internal reinforcement at spans – concrete crushing at central support – concrete crushing at loaded sections

2.8.5 Bond stress between CFRP laminate strips and concrete

The average CFRP-concrete bond stresses developed along the CFRP laminate strips were evaluated using the strains recorded in the strain gauges installed on the laminate strips. The bond stress in-between the consecutive strain gauges was assumed as:

\[
\tau_{bm}^{RL} = \frac{\Delta F_{RL}}{Ab_{RL}^{L}}
\]  

(27)

where

- \( \Delta F_{RL} \) is the difference of axial force in the laminate, between the two strain gauges \( E_{CFRP} \cdot \Delta \varepsilon \cdot A \);
- \( Ab_{RL}^{L} \) is the area of the lateral CFRP faces in contact with the adhesive, between strain gauges position( \( p \cdot L_{RL}^{L} \));
- \( E_{CFRP} \) is the Young’s modulus;
- \( \Delta \varepsilon_{RL} \) is the difference in axial strain between the strain gauges positioned at right and left sections ( \( \varepsilon_{SG}^{R} - \varepsilon_{SG}^{L} \));
- \( A_{CFRP} \) is the laminate cross sectional area;
- \( p \) is the perimeter of the contact surface between CFRP laminate and epoxy adhesive (considered here equal to \( 2 \cdot w_{j} \));
- \( L_{RL}^{L} \) is the distance between two consecutive strain gauges;
- \( \varepsilon_{SG}^{R} \) is the axial strain registered experimentally in the right strain gauge;
- \( \varepsilon_{SG}^{L} \) is the axial strain registered experimentally in the left strain gauge.

Equation (27) can be recast in the form:

\[
\tau_{bm}^{RL} = \frac{E_{CFRP} \cdot A_{CFRP} \cdot \Delta \varepsilon_{RL}}{2w_{j} \cdot L_{RL}^{L}}
\]  

(28)
The variation of the stress during the applied load is shown in Figures 82 to 87, for the slab strip specimens strengthened with NSM technique.

![Figure 82 - Bond stress variation for the slab strips SL15s25 – hogging region.](image1)

![Figure 83 - Bond stress variation for the slab strips SL15s25 – sagging region.](image2)
Figure 84 – Bond stress variation for the slab strips SL15s50 – hogging region.

Figure 85 – Bond stress variation for the slab strips SL15s50 – sagging region.
The observation of the results prompts the following main remarks:

(i) A very low bond stress was developed at the interfaces CFRP laminate-epoxy adhesive-concrete up to yield of the steel reinforcement at the hogging region.

(ii) The significantly increase in the average bond stress found in all figures is associated with the plastic hinge formed at the central support. The average bond stress variation in the stage from the formation of plastic hinge at hogging region to the formation of plastic hinge at sagging regions was roughly linear. As the load continues to increase, more cracks were formed, including herringbone cracks, which signified slip propagation. Thus, the propagation of the concrete cracking led to the decrease of the bond stress.
(iii) As the slab strip displaced continued to increase up to the failure, the CFRP laminate cover separation was caused due to the large curvature developed by the strengthened specimen (as a result of the high stiffness offered by the CFRP laminates) and because of the damaged concrete surface.

(iv) In all cases the maximum bond stress did not exceed 3.00 MPa, which can be considered a low bond stress for the NSM CFRP laminate technique.

2.8.6 **INDICATORS OF MOMENT REDISTRIBUTION**

According to CEB-FIB Model Code (1993), the coefficient of moment redistribution, \( \delta = \frac{M_{rd}}{M_{elas}} \), is defined as the relationship between the moment in the critical section after redistribution \( (M_{rd}) \) and the elastic moment \( (M_{elas}) \) in the same section calculated according to the theory of elasticity, while \( \eta = (1 - \delta) \cdot 100 \) is the moment redistribution percentage. The percentages of moment redistribution for the six slab strips tested are shown in Figure 88.
Figures 89 to 91 show the variation of the hogging (M) and sagging (M+) moments as the applied load $F_{123}$ increased. The divergences from the elastic moments and the obtained results mean that the moment redistribution mechanism was formed.

It can be noticed that the slab strips behaved elastically up to the cracking formation and then diverged, indicating that the moment was redistributing from the hogging to the sagging regions. In this way, the increase of the applied load led to a decrease of the negative moment and an increase of the positive moment, especially after the yield of the steel reinforcement.
In the SL15, when the steel reinforcement yields at the hogging region, a moment redistribution of \( xx \% \) was obtained. Afterwards, a moment redistribution of \( xx \% \) was obtained at the yielding of steel reinforcement at the sagging region.

With respect to the NSM strengthened slab strips of SL15 series, the NSM strengthened slab strips exhibited a moment redistribution rate of about 9.27 % at the yielding of steel reinforcement at the central support section for the SL15s25 slab strips. At the yielding of reinforcement at the sagging region, the moment redistribution decreased to about 11.49 % for the SL15s25 slab strip.

Concerning to SL15s50 slab strip, a moment redistribution of 1.77 % was obtained when the steel reinforcement yields at the hogging region. Afterwards, a moment redistribution of 6.45 % was obtained at the yielding of steel reinforcement at the sagging region.

In the SL30, when the steel reinforcement yields at the hogging region, a moment redistribution of 3.36 % was obtained. Afterwards, a moment redistribution of 20.21 % was obtained at the yielding of steel reinforcement at the sagging region. With respect to the NSM strengthened slab strips of SL30 series, the NSM strengthened slab strips exhibited a moment redistribution rate of about 7.89 % at the yielding of steel reinforcement at the central support section for the SL30s25 slab strips. At the yielding of reinforcement at the sagging region, the moment redistribution decreased to about 21.45 % for the SL3025 slab strip.

With respect to the NSM strengthened slab strips of SL45 series, the NSM strengthened slab strips exhibited a moment redistribution rate of about 21.06 % and 36.38 % at the yielding of steel reinforcement at the central and loaded cross-sections, respectively.
Figure 89 – Moment variation for the slab strips of SL15 series: (a) negative and (b) positive bending moments.
Figure 90 – Moment variation for the slab strips of SL30 series: (a) negative and (b) positive bending moments.
2.8.7 LOADING CARRYING CAPACITY

Table 17 resumes the results obtained experimentally for two scenarios: when a plastic hinge formed at the hogging region (at intermediate support zone, IS); when a plastic hinge formed at the sagging region (at loaded section, LS).

In this Table, $F^I_{y}$ and $F^L_{y}$, are the average loads at the formation of the plastic hinge at IS and LS, respectively, $\bar{u}^I_y$ and $\bar{u}^L_y$, are the average deflection for $F^I_y$ and $F^L_y$, respectively, $\varepsilon^I_{c,max}$ and $\varepsilon^L_{c,max}$ are the maximum concrete strains at IS and LS, $\varepsilon^I_{s,max}$ and $\varepsilon^L_{s,max}$ are the maximum strains in steel bars at IS and LS, respectively, $\varepsilon_{f,max}$ is the maximum strain in the CFRP laminates at hogging or sagging regions, $\overline{F}$ is the average load when a concrete compressive strain of 3.5 ‰ was recorded at the IS ($\varepsilon^I_{c,max} = 3.5 \%$) and $\varepsilon^I_{f,max}$ and $\varepsilon^L_{f,max}$ are the maximum strains in the CFRP laminates and in steel bars at $\overline{F}$. It was assumed that a plastic hinge has formed when yield strain was attained at the steel bars of this region. The following remarks can be pointed out:

(i) After concrete crack initiation, the slab stiffness decreased significantly, but the elasto-cracked stiffness was almost maintained up to the formation of the plastic hinge at the hogging region;

(ii) Up to the formation of the plastic hinge at the hogging region the tensile strains in the laminates are far below their ultimate tensile strain. At concrete crushing (assumed as -3.5%), for the slab strips of SL15 series, the maximum tensile strain in the laminates did not exceed the 5 % and 6 % at hogging and sagging regions, respectively. Concerning to the slab strips of SL30 series, the laminates reached the maximum tensile strain of 8.46 % and 5.60 % at hogging and sagging regions, respectively.

Figure 91 – Moment variation for the slab strips of SL45 series: (a) negative and (b) positive bending moments.
(iii) The force-deflection relationship evidences that, up to the formation of the plastic hinge at the hogging region, the laminates had a marginal contribute for the slabs load carrying capacity.
(iv) The deflection at \( \bar{F}_{y}^{IS}, \ bar{u}_{y}^{IS} \) was not significantly affected by the presence of the laminates.
(v) The increment of load between the formation of the plastic hinge at hogging and at sagging regions decreased with the decrease of moment redistribution and, for each series, in general, this increment decreased with the increase of the percentage of laminates;
(vi) It is visible that the strengthening arrangements applied in the hogging and sagging regions are very effective in terms of increasing the load carrying capacity of the three series of slabs.

Tables 17 and 18 resume the results obtained experimentally when a plastic hinge formed hogging and sagging regions.

### Table 17 – Main results – Experimental program.

<table>
<thead>
<tr>
<th>Series</th>
<th>( F_{y}^{IS} ) (kN)</th>
<th>( u_{y}^{IS} ) (mm)</th>
<th>( \varepsilon_{y_{c_{max}}}^{IS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{IS} ) (%)</th>
<th>( \varepsilon_{y_{c_{max}}}^{IS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{IS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{LS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{LS} ) (%)</th>
<th>( \varepsilon_{y_{c_{max}}}^{LS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{LS} ) (%)</th>
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<tbody>
<tr>
<td>SL15</td>
<td>Reference 38.46 15.18 -1.95 -1.52 2.49 3.03 -----</td>
<td>44.97 22.52 -4.55 -2.78 m.d. 3.37 -----</td>
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<tr>
<td>SL15s25</td>
<td>44.50 17.15 -1.93 1.97 2.45 2.65 2.51 54.72 22.17 -2.83 -2.43 2.74 3.12 4.04</td>
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<tr>
<td>SL15s50</td>
<td>46.32 18.06 -2.10 -1.63 m.d m.d 1.45 56.09 23.60 -3.40 -2.19 m.d. m.d. 4.34</td>
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<tr>
<td>SL30</td>
<td>Reference 35.58 14.53 -1.70 1.86 2.62 -----</td>
<td>45.70 23.92 -4.80 -2.28 2.86 2.90 -----</td>
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<tr>
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<td>39.62 13.97 -1.60 1.87 2.88 1.97 54.82 23.01 -3.78 -2.53 2.86 4.75 2.65</td>
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<tr>
<td>SL30s50</td>
<td>Reference 31.99 11.35 -1.38 1.10 1.52 3.31 -----</td>
<td>49.02 23.51 -4.67 -2.10 2.46 2.76 -----</td>
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</table>

### Table 18 – Main results at concrete crashing.

<table>
<thead>
<tr>
<th>Series</th>
<th>( F_{y}^{LS} ) (kN)</th>
<th>( u_{y}^{LS} ) (mm)</th>
<th>( \varepsilon_{y_{c_{max}}}^{IS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{IS} ) (%)</th>
<th>( \varepsilon_{y_{c_{max}}}^{IS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{IS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{LS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{LS} ) (%)</th>
<th>( \varepsilon_{y_{c_{max}}}^{LS} ) (%)</th>
<th>( \varepsilon_{x_{c_{max}}}^{LS} ) (%)</th>
<th>MR (%)</th>
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<td>SL15s25</td>
<td>59.82 27.30 -3.75 -3.50 2.88 2.93 4.86 5.67 6.20</td>
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</tr>
<tr>
<td>SL15s50</td>
<td>62.00 30.35 -4.50 -3.50 m.d m.d 5.07 6.10 6.20</td>
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</tr>
<tr>
<td>SL30</td>
<td>Reference 46.14 30.52 -6.25 -3.50 2.83 4.64 ----- -----</td>
<td>19.93</td>
<td></td>
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<td></td>
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<tr>
<td>SL30s25</td>
<td>59.91 28.54 -5.09 -3.50 4.41 4.43 8.46 5.60 21.45</td>
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</tr>
<tr>
<td>SL30s50</td>
<td>Reference 50.24 29.87 -5.61 -3.50 2.29 2.85 ----- ----- 39.21</td>
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Azurém, 4800-085 Guimarães
At the yield initiation of the steel bars of the sagging regions the increase percentage of load carrying capacity provided by the used flexural strengthening arrangements are: 21.68 % and 24.73 % for SL15s25 and SL15s50; 19.95 % and xx % for SL30s25 and SL30s50; xx % and xx % for SL45s25 and SL45s50. At a concrete compressive strain of 3.5‰ in the sagging regions, the increase percentage of load carrying capacity provided by the used flexural strengthening arrangements was: 31.32 % and 36.11 % for SL15s25 and SL15s50; 29.84 % and xx % for SL30s25 and SL30s50; xx % and xx % for SL45s25 and SL45s50.

### 2.8.8 SHEAR CAPACITY OF THE SLAB STRIPS

The slab shear prediction, calculated according ACI 318R (2008) and EC2 (2004), is resumed in Table 19. Complementary to these calculations, the experimental data registered at the maximum applied load \( F_{123} \) and the support reactions are presented, as well as the shear capacity.

<table>
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<tr>
<td></td>
<td>( f_{cm} ) (MPa)</td>
<td>( V_c ) (kN)</td>
<td>( V_r ) (kN)</td>
</tr>
<tr>
<td>SL15</td>
<td>30.36</td>
<td>38.85</td>
<td>4.76</td>
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<tr>
<td>SL15s25</td>
<td>32.64</td>
<td>40.28</td>
<td>4.76</td>
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<tr>
<td>SL15s50</td>
<td>34.51</td>
<td>41.42</td>
<td>4.76</td>
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<tr>
<td>SL30</td>
<td>30.10</td>
<td>38.68</td>
<td>4.76</td>
</tr>
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<td>SL30s25</td>
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<td>40.25</td>
<td>4.76</td>
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<td>SL30s50</td>
<td>34.25</td>
<td>45.83</td>
<td>4.76</td>
</tr>
<tr>
<td>SL45</td>
<td>42.25</td>
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<td></td>
</tr>
<tr>
<td>SL45s25</td>
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<td></td>
<td></td>
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<tr>
<td>SL45s50</td>
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</tbody>
</table>

These values reveal that the aimed increase in terms of slab’s load carrying capacity was attained. It can be noticed that the reference slabs (SL15, SL30 and SL45) did not reach the maximum shear capacity. However, the prediction of the maximum shear capacity of the strengthened slab strips are close to ones experimentally obtained, which justifies the failure mode of some specimens. Thus, there is an agreement between calculated and experimental measured values. In this work, since the slabs have not specific reinforcements for the shear resistance, the maximum load of all simulated slabs might be limited by their out-of-plane shear.
2.8.9 Ductility Analysis

Ductility is defined as the capacity of a material, cross-section, or structure to sustain considerable plastic deformation without loss of strength capacity. When applied to RC elements, the term ductility implies the ability to sustain significant inelastic deformation prior to collapse. As the evolving technology of using CFRP laminates for strengthening RC structures has attracted much attention in recent years, understanding the effects of such materials on the ductility of RC members is an important aspect on the structural performance of the FRP-strengthened structure. A method, based on the ductility index commonly used, is herein considered to analyze the ductility of the RC elements strengthened in both the hogging and sagging regions. Ductility of RC members has generally been measured by parameters designated as bending ductility indexes. In this work, the displacement and curvature ductility of the numerically analyzed slabs strips are compared. Displacement ductility index, \( \mu_{\Delta} \), is defined as the ratio between the deflection at LS at the ultimate condition (\( \Delta_{uLS} \)) and the deflection at the yielding of the tension reinforcement at the loaded section (\( \Delta_{y}\)):

\[
\mu_{\Delta} = \frac{\Delta_{uLS}}{\Delta_{y}}
\]  

The curvature ductility index, \( \mu_{\chi} \), is defined as the ratio between the curvature at LS at the ultimate condition (\( \chi_{uLS} \)) and the curvature at the yielding of the tension reinforcement at loaded section (\( \chi_{yLS} \)):

\[
\mu_{\chi} = \frac{\chi_{uLS}}{\chi_{yLS}}
\]  

The deflection and the curvature at ultimate condition (\( \Delta_{uLS} \) and \( \chi_{uLS} \)) were obtained when a compressive strain of 3.5 \( \% \) was attained in the concrete surface at loaded section. Table xx lists the values of the ductility indexes obtained for the three series of slabs. It is worth noting that:

(i) When compared to the reference slabs of the tested series, the increase of \( \Delta_{uLS} \) in the strengthened slabs was larger than the increase of \( \Delta_{y} \), resulting \( \mu_{\Delta} \) values that are higher in these later slabs than in the former ones.

(ii) For a compressive strain of 3.5 \( \% \) in the concrete surface at loaded sections, the following \( \eta \) values were obtained: xx \%, 10.41 \% and 6.20 \% for SL15, SL15s25 and SL15s50; 20.21 \%, 19.52 \% and xx \% for SL30, SL30s25 and SL30s50; 37.84 \%, xx \% and xx \% for SL45, SL45s25 and SL45s50.

Table 20 – Main results – Ductility analysis.

<table>
<thead>
<tr>
<th>Series</th>
<th>( F_{yLS} ) (kN)</th>
<th>( \Delta_{uLS} ) (mm)</th>
<th>( \Delta_{yLS} ) (mm)</th>
<th>( \mu_{\Delta} )</th>
<th>( \chi_{uLS} ) (x10^-6)</th>
<th>( \chi_{yLS} ) (x10^-6)</th>
<th>( \mu_{\chi} )</th>
<th>Decrease over reference slab (%)</th>
<th>( M_{Ry} ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
<td>44.97</td>
<td>24.69</td>
<td>22.52</td>
<td>1.09</td>
<td>m.d.</td>
<td>m.d.</td>
<td>m.d.</td>
<td>m.d.</td>
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<tr>
<td>SL15s25</td>
<td>54.72</td>
<td>27.30</td>
<td>22.17</td>
<td>1.23</td>
<td>0.0680856170</td>
<td>0.0550839362</td>
<td>1.23</td>
<td>10.41</td>
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</tr>
<tr>
<td>SL15s50</td>
<td>56.03</td>
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<td>23.60</td>
<td>1.28</td>
<td>m.d.</td>
<td>m.d.</td>
<td>m.d.</td>
<td>m.d.</td>
<td>6.20</td>
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<tr>
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<td>1.27</td>
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<td>0.0542291064</td>
<td>1.24</td>
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<td>20.21</td>
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</table>
2.8.10 Crack Spacing Analysis

Table 21 resumes the theoretical and experimentally measured cracks for the tested slab strips. The average crack spacing, experimentally measured after the slabs having been tested, was determined as schematically described in Figure 92. The expressions from the Portuguese design code of reinforced and pre-stressing concrete structures, CEB-FIP Model Code (1993) and EUROCODE 2 (2000/2004) were herein considered to calculate the theoretical mean spacing between cracks for the tested slab strips for two scenarios: at the maximum applied load $F_{123}$ and at the yielding of the steel reinforcement at IS and LS, respectively.

![Figure 92 – Determination of the average crack spacing in the tested slab strips.](image)

<table>
<thead>
<tr>
<th></th>
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<tbody>
<tr>
<td>SL15</td>
<td>79.27</td>
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Figure 92 – Determination of the average crack spacing in the tested slab strips.
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<th>SL15</th>
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<td>35.05</td>
<td>27.69</td>
<td>28.15</td>
<td>25.04</td>
<td>29.31</td>
<td>29.31</td>
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<td>64.95</td>
<td>65.20</td>
<td>63.51</td>
<td>63.51</td>
<td>65.94</td>
<td>65.94</td>
<td>65.94</td>
</tr>
</tbody>
</table>

a Average of measurements taken in the mid-base of the top surfaces of the slab strip, at IS
b Average of measurements taken in the mid-base on bottom surfaces of the slab strip, at LS

The neutral axis depth was derived from the strain gauges data, for a elongation close to the first yield of the conventional reinforcement
The cross section strain distribution was derived from the strain gauges data, for an elongation close to the yield of the conventional reinforcement
(value) average value when is the case

### 3 NUMERICAL SIMULATION

Compared to the expressions of REBAP (1983) and EUROCODE 2 (2000), the expressions of CEB-FIP Model Code (1992), EUROCODE 2 (2004) lead to better results in terms of predicting crack spacing for the unstrengthened. However, it is shown that the use of various code equations to estimate crack spacing in the same member can result in significantly different values. Therefore, there is no agreement on the most appropriate model to predict the behavior of strengthened slabs in both EBR and NSM strengthened RC elements.

#### 3.1 Predictive performance of the model

To simulate the behavior of these slabs, which are NSM flexural strengthened in both the hogging and sagging regions, the values of the properties of the constitutive models adopted in the simulations of the slabs tested experimentally are also used. The finite element mesh, support and load conditions are also assumed equal to those adopted in the simulations of the experimentally tested slabs.

**Concrete constitutive model**

According to the present model, a concrete slab is considered a plane shell formulated under the Reissner-Mindlin theory (Barros, 1995). In order to simulate the progressive damage induced by cracking and...
plasticity, the shell element is discretized in layers. Each layer is considered in a state of plane stress. The incremental strain vector derived from the incremental nodal displacements obtained under the framework of a nonlinear FEM analysis is decomposed in an incremental crack strain vector, $\Delta \varepsilon^c$, and an incremental strain vector of the concrete between cracks, $\Delta \varepsilon^{co}$. This last vector is decomposed in an elastic reversible part, $\Delta \varepsilon^e$, and an irreversible or plastic part, $\Delta \varepsilon^p$, resulting

$$\Delta \varepsilon = \Delta \varepsilon^c + \Delta \varepsilon^{co} = \Delta \varepsilon^c + \Delta \varepsilon^e + \Delta \varepsilon^p$$  \hspace{1cm} (31)

The incremental stress vector can be computed from the incremental elastic strain vector,

$$\Delta \sigma = D^{co} \Delta \varepsilon^{co}$$  \hspace{1cm} (32)

where $D^{co}$ is the concrete tangent constitutive matrix,

$$D^{co} = \begin{bmatrix} D^{co}_{in} & \phi \\ \phi & D^{co}_{out} \end{bmatrix}$$  \hspace{1cm} (33)

with $D^{co}_{in}$ being the in-plane stiffness matrix and $D^{co}_{out}$ the out-of-plane shear stiffness matrix. In the present model concrete behavior is assumed linear elastic in terms of out-of-plane shear. Therefore, the concrete nonlinear behavior is only considered in the $D^{co}_{in}$ constitutive matrix.

For linear elastic uncracked concrete, $D^{co}_{in}$ is designated by $D^{eco}_{in}$ being defined elsewhere (Barros and Figueiras, 2001).

In cracked concrete, with the concrete between cracks in linear elastic state, $D^{co}_{in}$ is replaced in Equation (33) by $D^{eco}_{in}$.

$$D^{co}_{in} \Rightarrow D^{eco}_{in} = D^{eco}_{in} - D^{eco}_{in} \left[ T^{cr} \right]^T \left( \hat{D}^{cr} + T^{cr} D^{eco}_{in} \left[ T^{cr} \right]^T \right)^{-1} T^{cr} D^{eco}_{in}$$  \hspace{1cm} (34)

where $T^{cr}$ is a transformation matrix that depends on the direction of the cracks formed at a sampling point and $\hat{D}^{cr}$ is the constitutive matrix of the set of cracks. Each crack is governed by the following constitutive relationship

$$\Delta \sigma^{cr} = D^{cr} \Delta \varepsilon^{cr}$$  \hspace{1cm} (35)

where $\Delta \sigma^{cr}$ is the incremental local crack stress vector. This vector has the following components,
In this equation, \( \Delta \varepsilon_i^\sigma \) is the incremental crack strain vector, which has the following components

\[
\Delta \varepsilon_i^\sigma = \begin{bmatrix} \Delta \varepsilon_i^\sigma \end{bmatrix}^T
\]  

(36)

And

\[
D_i^\sigma = \begin{bmatrix} D_i^\sigma & 0 \\ 0 & D_{ii}^\sigma \end{bmatrix}
\]  

(38)

is the crack stiffness matrix, where \( D_i^\sigma \) and \( D_{ii}^\sigma \) are the fracture mode I and the fracture mode II stiffness modulus of the smeared cracks, respectively. In Equation (38) \( D_i^\sigma \) is characterized by the fracture parameters, namely the stress at crack initiation, \( \sigma_i^\sigma \) (see Figure 93), the fracture energy, \( G_i \), the shape of the softening law and the crack band width, \( l_b \). In smeared crack models the fracture zone is distributed over \( l_b \), which must depend on the finite element geometric characteristics in order to assure that the results of the FEM analysis are not dependent on the finite element mesh refinement (Bazant and Oh, 1993). Therefore, \( \Delta \varepsilon_i^\sigma = \Delta w / l_b \), where \( \Delta w \) is the total increment of width of the cracks smeared in the crack band width. In the present numerical simulation \( l_b \) is assumed to be equal to the square root of the area of the corresponding integration point. Research on the fracture mechanics of cement based materials has shown that the trilinear \( \sigma^\sigma - \varepsilon^\sigma \) diagram represented in Figure 93 is suitable for the simulation of the fracture mode I of this type of materials.

The fracture mode II modulus, \( D_{ii}^\sigma \), is obtained with the following expression (Barros, 1995)

\[
D_{ii}^\sigma = \frac{\beta}{1 - \beta} G_i
\]  

(39)
where $G_c$ is the concrete elastic shear modulus and $\beta$ is the shear retention factor, which is defined by

$$\beta = \left(1 - \frac{E_{cu}}{E_{cu,n}}\right)^n$$

(40)

In this equation $n$ is an integer parameter that can assume distinct values in order to simulate different levels of concrete shear stiffness degradation (Barros 1995).

In the elasto-plastic uncracked concrete, the in-plane material stiffness matrix $D_{nb}$ of Equation (34) is replaced with $D_{nb}^{ep}$ (Sena-Cruz et al., 2004).

$$D_{nb}^{ep} = H - \frac{H \frac{\partial f}{\partial \sigma} \left( \frac{\partial f}{\partial \sigma} \right)^T H}{h + \left( \frac{\partial f}{\partial \sigma} \right)^T H \frac{\partial f}{\partial \sigma}}$$

(41)

where,

$$H = \left[ D_{nb}^{ep} \right]^{-1} + h \Delta \frac{\partial f}{\partial \sigma} \left( \frac{\partial f}{\partial \sigma} \right)^{-1}$$

(42)

being $\frac{\partial f}{\partial \sigma}$ the flow vector and $hc$ a scalar function that depends on the hydrostatic pressure. The aim of $hc$ is the amplification of the contribution of $\Delta \lambda \frac{\partial f}{\partial \sigma}$ to the plastic strain increment vector, $\Delta \varepsilon^p$

$$\Delta \varepsilon^p = \Delta \lambda h \frac{\partial f}{\partial \sigma}$$

(43)

In (5.12) $\Delta \lambda$ is the variation of the plastic multiplier, which was assumed to be equal to the variation of the hardening parameter, $\Delta \kappa$, since a strain-hardening hypothesis was assumed. The yield surface proposed by Owen and Figueiras was adopted,

$$f(\sigma, \kappa) = \left( \sigma^T P \sigma \right)^{1/2} + q' \sigma - \bar{\sigma}(\kappa) = 0$$

(44)

being $P$ the projection matrix and $q'$ the projection vector (Sena-Cruz et al., 2004). Fig. 74 represents the relationship between the yield stress, $\bar{\sigma}$, and the hardening parameter, $\kappa$, used to simulate the hardening and softening phases of plain concrete behavior. The equations of $\bar{\sigma}(\kappa)$ are published in Sena-Cruz et al (2004).
For the case of cracked concrete with concrete between cracks exhibiting elasto-plastic behavior, $D_{mb}^{\text{co}}$ of (5.3) is replaced by $D_{mb}^{\text{crco}}$ (Sena-Cruz et al., 2004).

$$D_{mb}^{\text{co}} \Rightarrow D_{mb}^{\text{crco}} = D_{mb}^{\text{co}} - D_{mb}^{\text{crco}} \left[ T_{\text{co}}^{\text{T}} \right]^T \left( \tilde{D}_{\text{co}}^{\text{T}} + T_{\text{co}}^{\text{T}} D_{mb}^{\text{crco}} T_{\text{co}} \right)^{-1} T_{\text{co}}^{\text{T}} D_{mb}^{\text{crco}}$$ (45)

where $D_{mb}^{\text{crco}}$ is defined by Equation (42).

**Constitutive model for steel**

For modeling the behavior of the steel bars, the stress-strain relationship represented in Figure 75 was adopted (Sena-Cruz, 2004). The curve (under compressive or tensile loading) is composed of four branches (see Equation (47)). To define the four branches, three points $PT1 = (\sigma_{sy}, \epsilon_{sy})$, $PT2 = (\sigma_{sh}, \epsilon_{sh})$ and $PT3 = (\sigma_{su}, \epsilon_{su})$ and the parameter $p$ are required. Typically, the value of the parameter $p$ varies between 1.0 and 4.0. Unloading and reloading linear branches with slope $E_s$ are assumed in the present approach.

$$E_s = \sigma_{sy}/\epsilon_{sy}$$ (46)

The curve $\sigma_s - \epsilon_s$ is given by

$$\sigma(s) = \begin{cases} E_s \epsilon_s & \text{if } \epsilon_s \leq \epsilon_{sy} \\ E_s \epsilon_s + \sigma_{sh} & \text{if } \epsilon_{sy} < \epsilon_s \leq \epsilon_{sh} \\ \sigma_{su} + (\sigma_{sh} - \sigma_{su}) \left( \frac{\epsilon_{sy} - \epsilon_s}{\epsilon_{su} - \epsilon_{sh}} \right)^p & \text{if } \epsilon_{sh} < \epsilon_s \leq \epsilon_{su} \\ 0 & \text{if } \epsilon_s > \epsilon_{su} \end{cases}$$ (47)

Where

$$E_{sy} = (\sigma_{sh} - \sigma_{su})/(\epsilon_{sh} - \epsilon_{su})$$ (48)
Constitutive model for CFRP strips

A linear elastic stress-strain relationship was adopted to simulate the behavior of NSM CFRP laminates applied in the RC slabs.

3.2 SIMULATION OF THE TESTS

Materials properties and finite element mesh

Tables 22 and 23 include the values of the parameters adopted for the characterization of the constitutive models for the concrete and steel, respectively. The CFRP laminates were assumed as an isotropic material of an elasticity modulus of 156 GPa and null Poisson's coefficient, since the consideration of their real anisotropic properties has marginal influence in terms of their contribution for the behavior of NSM strengthened RC slabs.

<table>
<thead>
<tr>
<th>Parameter definition</th>
<th>SL15</th>
<th>SL30</th>
<th>SL45</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poisson’s ratio ($\nu_c$)</td>
<td>0.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial Young’s modulus ($E_c$)</td>
<td>24.54 GPa</td>
<td>20 GPa</td>
<td></td>
</tr>
<tr>
<td>Compressive strength ($f_{c1}$)</td>
<td>30.36 MPa</td>
<td>28.40 MPa</td>
<td></td>
</tr>
<tr>
<td>Strain at peak compression stress</td>
<td>$\varepsilon_{c1} = 2.2 \times 10^{-3}$</td>
<td>$\varepsilon_{c1} = 2.2 \times 10^{-3}$</td>
<td>$\varepsilon_{c1} = 2.2 \times 10^{-3}$</td>
</tr>
<tr>
<td>Parameter defining the initial yield surface (1)</td>
<td>$\alpha_c = 0.4$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tri-linear tension softening/stiffening diagram (2)</td>
<td>$f_t = 1.79$ MPa; $f_y = 1.49$ MPa;</td>
<td>$G_f = 0.065$ N/mm; $G_f = 0.062$ N/mm;</td>
<td></td>
</tr>
</tbody>
</table>
Parameter defining the mode I fracture energy available to the new crack (Barros 1995)
\[ n = 2 \]

Shear retention factor (p1 factor of Equation (10))
\[ p_1 = 2 \]

Crack band-width
Square root of the area of Gauss integration point
\[ \alpha_{th} = 30^\circ \]

Threshold angle (see Figure 1, Barros 1995)

Maximum number of cracks per integration point
2

Due to the structural symmetry, only half of the slab was considered in the numerical simulations. Figure 96 shows the eight nodded finite element mesh adopted to discretize the half part of the slab. The support conditions are also represented in this figure. The slab thickness was discretized in 20 layers.

### Table 23 – Values of the parameters of the steel constitutive model (see Figure 95).

<table>
<thead>
<tr>
<th>Steel bar diameter</th>
<th>PT1({\varepsilon_{n,1}[-];\sigma_{n}/[MPa]})</th>
<th>PT2({\varepsilon_{n,2}[-];\sigma_{n}/[MPa]})</th>
<th>PT3({\varepsilon_{n,3}[-];\sigma_{n}/[MPa]})</th>
<th>(E_s) (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ø 8mm</td>
<td>(2.50×10^{-3}; 421.00)</td>
<td>(4.42×10^{-2}; 526.25)</td>
<td>(8.85×10^{-2}; 555.72)</td>
<td>200.80</td>
</tr>
<tr>
<td>Ø 10mm</td>
<td>(2.50×10^{-3}; 446.00)</td>
<td>(3.07×10^{-2}; 446.00)</td>
<td>(1.31×10^{-1}; 557.50)</td>
<td>178.24</td>
</tr>
<tr>
<td>Ø 12mm</td>
<td>(2.50×10^{-3}; 445.00)</td>
<td>(3.05×10^{-2}; 445.00)</td>
<td>(1.02×10^{-1}; 547.35)</td>
<td>198.36</td>
</tr>
</tbody>
</table>

3.3 RESULTS

Figures 97 to 121 compare the load-deflection curves obtained numerically and recorded experimentally for the slabs of SL15, SL30 and SL45 series, respectively.

3.3.1 UNSTRENGTHENED SLAB STRIPS
3.3.1.1 SL15

Figure 97 – Relationship between applied load and deflections at spans of the SL15.

Figure 98 – Relationship between load and tensile strain of the negative longitudinal steel reinforcement for the SL15 slab strip.
Figure 99 – Relationship between load and tensile strain of the positive longitudinal steel reinforcement for the SL15 slab strip.

Figure 100 – Relationship between load and compressive strain of the concrete at sagging region for the SL15 slab strip.
3.3.1.2 SL30

Figure 101 – Relationship between load and compressive strain of the concrete at hogging region for the SL15 slab strip.

Figure 102 – Relationship between applied load and deflections at spans of the SL30.
Figure 103 – Relationship between load and tensile strain of the negative longitudinal steel reinforcement for the SL30 slab strip.

Figure 104 – Relationship between load and tensile strain of the positive longitudinal steel reinforcement for the SL30 slab strip.
3.3.2 STRENGTHENED SLAB STRIPS

3.3.2.1 SL15s25

Figure 105 – Relationship between load and compressive strain of the concrete at sagging region for the SL30 slab strip.

Figure 106 – Relationship between load and compressive strain of the concrete at hogging region for the SL30 slab strip.
Figure 107 – Relationship between applied load and deflections at spans of the SL15s25.

Figure 108 – Relationship between load and tensile strain of the negative longitudinal steel reinforcement for the SL15s25 slab strip.
Figure 109 – Relationship between load and tensile strain of the positive longitudinal steel reinforcement for the SL15s25 slab strip.

Figure 110 – Relationship between load and compressive strain of the concrete at sagging region for the SL15s25 slab strip.
3.3.2.2 SL15s50

Figure 112 – Relationship between applied load and deflections at spans of the SL15s50.
Figure 113 – Relationship between load and tensile strain of the negative longitudinal steel reinforcement for the SL15s50 slab strip.

Figure 114 – Relationship between load and tensile strain of the positive longitudinal steel reinforcement for the SL15s50 slab strip.
3.3.2.3 SL30s25

Figure 115 – Relationship between load and compressive strain of the concrete at sagging region for the SL15s50 slab strip.

Figure 116 – Relationship between load and compressive strain of the concrete at hogging region for the SL15s50 slab strip.
Figure 117 – Relationship between applied load and deflections at spans of the SL30s25.

Figure 118 – Relationship between load and tensile strain of the negative longitudinal steel reinforcement for the SL30s25 slab strip.
Figure 119 – Relationship between load and tensile strain of the positive longitudinal steel reinforcement for the SL30s25 slab strip.

Figure 120 – Relationship between load and compressive strain of the concrete at sagging region for the SL30s25 slab strip.
3.4 Discussion

The Figures previously presented compare the load-deflection curves obtained numerically and recorded experimentally for the slabs of SL15, SL30 and SL45 series, respectively. The quite good predictive performance of the model is also visible in the strains of the steel bars, concrete and CFRP strips.

The relationship between the load and the deflection at the loaded sections for the six tested slab strips were presented. It is visible that the strengthening arrangements applied in the hogging and sagging regions are very effective in terms of increasing the load carrying capacity of the three series of slabs.

Tables 24 to 25 resume the results obtained numerically when a plastic hinge formed at the sagging region (at loaded section, LS). At the yield initiation of the steel bars of the sagging regions the increase percentage of load carrying capacity provided by the used flexural strengthening arrangements are: 10.90 %, 1.00 % and 1.73 % for SL15, SL15s25 and SL15s50; 26.91 %, 16.49 % and xx % for SL30, SL30s25 and SL30s50; xx %, xx % and xx % for SL45, SL45s25 and SL45s50. At a concrete compressive strain of 3.5‰ in the sagging regions, the increase percentage of load carrying capacity provided by the used flexural strengthening arrangements was: 12.53 %, 0.00 % and 0.32 % for SL15, SL15s25 and SL15s50; xx %, xx % and xx % for SL30, SL30s25 and SL30s50; xx %, xx % and xx % for SL45, SL45s25 and SL45s50. These values reveal that the aimed increase in terms of slab’s load carrying capacity was attained. Since the slabs have not specific reinforcements for the shear resistance, the maximum load of all simulated slabs might be limited by their out-of-plane shear.
The percentages of moment redistribution for the six slab strips tested are shown in Figure 122. Figures 123 to 125 show the variation of the hogging ($M^-\gamma$) and sagging ($M^+\gamma$) moments as the applied load $F_{123}$ increased. The divergences from the elastic moments and the obtained results mean that the moment redistribution mechanism was formed.
Figure 122 – Percentages of moment redistribution for (a) SL15, (b) SL30 and (c) SL45 series.
Figure 123 – Moment variation for the slab strips of SL15 series: (a) negative and (b) positive bending moments.
Figure 124 – Moment variation for the slab strips of SL30 series: (a) negative and (b) positive bending moments.

Figure 125 – Moment variation for the slab strips of SL45 series: (a) negative and (b) positive bending moments.
4  CONCLUSIONS

This work deals with the NSM flexural strengthening of continuous RC slab strips using NSM technique in order to increase the load carrying and the moment redistribution capacities. This experimental program was analysed and it was concluded the use of CFRP laminates at both hogging and sagging regions are essential to the effectiveness of the NSM technique for this type of structures.

Using the obtained experimental results, the capability of a FEM-based computer program to predict with high accuracy the behaviour of this type of structures up to its collapse was highlighted. Using this program, the high effectiveness of this technique for the increase of the load carrying capacity was attested when correct NSM flexural strengthening arrangements are used. Additionally, if the NSM strengthening system is designed properly and precautions are taken to prevent shear or debonding failure, relevant moment redistribution levels can occur along the strengthened elements up to their final failure.

5  REFERENCE

ACI Committee 318. (2008). ACI 318-08 - Building code requirements for structural concrete and Commentary, American Concrete Institute, Detroit.

ACI Committee 440. (2007). ACI 440 - Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures, American Concrete Institute.


