NUMERICAL ANALYSIS OF CONCRETE BEAMS STRENGTHENED WITH CFRP – A STUDY OF ANCHORAGE LENGTHS

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ABSTRACT

The advantages of Fibre Reinforced Polymer (FRP) strengthening have been shown time and again during the last decade. All over the world several thousand structures have been retrofitted using FRP. Buildings and civil structures usually have a very long life and it is not uncommon that the demands on the structure change with time. The structures may have to carry larger loads at a later date or fulfil new standards. In extreme cases, a structure may need repair due to an accident, or due to errors made during the design or construction phase. To guarantee the function of the strengthening properties, anchorage of the FRP is essential. Without sufficient anchorage lengths, full utilization of the strengthening material cannot be achieved, leading to possible premature failure. In this paper, experimental work and numerical analyses of three different Carbon Fibre Reinforced Polymer (CFRP) strengthening techniques have been carried out. The techniques are externally bonded plates, sheets and the use of Near Surface Mounted Reinforcement (NSMR). The aim is to find a critical anchorage length, where a longer anchorage length does not contribute to the load bearing capacity. Three different anchorage lengths have been investigated; 100, 200 and 500 mm. The finite element program ABAQUS has been used for the numerical study. The results show that a critical anchorage length exists for plates and sheets as well as for NSMR. However, the present study also shows that an exact critical anchorage length may be difficult to estimate, at least with the present test set-up. Further tests and investigations of the constitutive model for the concrete are needed.

KEYWORDS

FRP, concrete, strengthening, plates, sheets, NSMR, anchorage length, finite element.

INTRODUCTION

All over the world there are structures intended for living and transportation. The structures are of varying quality and function, but they are all ageing and deteriorating over time. Some of these structures are in such a bad condition that they need to be replaced. There is also a need for upgrading existing structures. Errors can have been made during the design or construction phase so that the structure needs to be strengthened before it can be fully utilized. New and increased demands from the transportation sector or change of use can also be reasons for upgrading existing structures. A strengthening method that has been increasingly used during the last decade is Plate Bonding with FRP materials. A composite plate or sheet of relatively small thickness is bonded with an epoxy adhesive to, in most cases, a concrete structure to improve its structural behaviour and strength. In this paper, bonding of NSMR is also included in the plate bonding method. Extensive research and laboratory testing all over the world shows strengthening with FRP is very effective and that a considerable effect can be achieved if the strengthening process is carried out accurately. There is no doubt that of the potential and considerably economical advantages in FRP strengthening. However, if the technique is to be used effectively, it requires a sound understanding of both the short- and long-term behaviour of the bonding system used. It also requires reliable information concerning the adhesion between concrete and composite. Consequently, proper understanding of the anchorage behaviour is essential.

The anchorage lengths of the strengthening material bonded to the concrete play an important role for the full utilization of the strengthening inset and avoidance of premature failure due to bond failure. Several investigations of anchorage lengths to efficiently transfer the force between the strengthening material and concrete have been carried out. The most extensive research has been done with plates, Täljsten (1994, 1997),
Bizindavyi and Neale (1999) and Chen and Teng (2001), but research with sheets, Aiello and Pecce (2001), and NSMR rods, De Lorenzis and Nanni (2002) and De Lorenzis et al. (2002), have also been carried out. There are many factors that can affect the anchorage length of a bonded strengthening material. It is important to investigate and understand what anchorage length is needed to create a secure bond. It is also important to gain knowledge and understanding of the factors that affect the anchorage length, such as surface treatment prior to bonding, surface irregularities, temperature, humidity and workmanship. The most common methods for pretreating are sand blasting and grinding of the concrete surface. Both methods have been used and both can provide a surface for a sufficient bond. Surface irregularities that cannot be sand blasted or grinded away is another factor. With the irregularities the bond strength will change, but how much? In some cases there can be cavities in the concrete that will be filled with different kinds of material to achieve full bond. This area might be weaker than the rest of the concrete affecting the bond strength. Temperature and humidity, those parameters can easily be controlled. If skilled workers are carrying out the strengthening work, then no major workmanship problems should be expected. The quality of the concrete is also an important aspect, a high quality concrete will probably bond better to the strengthening material compared to a low quality concrete – but is there a major difference? Depending on strengthening method used, plates, sheets or NSMR, is there different critical anchorage lengths? In Figure 1, typical strengthening schemes with plates, sheets and NSMR are shown.

![Figure 1](image.png)

Figure 1 Schematic sketch of a) plate, b) NSMR and c) sheets.

Several numerical analyses of reinforced concrete strengthened with CFRP have been carried out in recent years and the preferable tool for these analyses is the finite element (FE) method. These concern primarily studies of plate bonding but there are also studies of bonding of NSMR. There are two strategies when modelling the behaviour of CFRP strengthened concrete structures; with or without the adhesive layer. Since the predominant failure of the bonding occurs in the concrete, most FE calculations are performed without considering the adhesive layer. Consequently, this implies that the constitutive model for the reinforced concrete is an important part of the numerical analysis. 2D FE analyses are performed in e.g. Rahimi and Hutchinson (2001) where concrete beams were strengthened with externally bonded FRP plates and a damage model handled the concrete, Teng et al. (2002) made an elastic parametric study of the interfacial stresses in plated RC beams, Yang et al. (2003) studied the concrete cover separation failure in FRP plated RC beams and used a discrete crack model for the concrete, Wu and Yin (2003) have investigated the fracture energy in both the concrete (mode I) and the adhesive (mode II) and the interactions between those through a parametric study (a smeared crack model was used for the concrete), Neto et al. (2004) studied fracture energy mode II for the adhesive and used a discrete crack model for the adhesive, Perera et al. (2004) made an adherence analysis of CFRP strengthened RC beams with a damage model for the concrete, Camata et al. (2004) investigated end peeling and mid-span debonding on RC beams and used a combined smeared and discrete crack model for the concrete, Lu et al. (2005) studied the debonding process in pull tests using a meso-mechanical model for the concrete. 3D FE analyses have been conducted by e.g. Hassan and Rizkalla (2003) who studied NSMR strips in RC beams.

The main aim of this paper is to determine the critical anchorage length for three types of strengthening methods; sheets, plates and NSMR. This is done both experimentally and numerically. Three anchorage lengths have been studied; 100, 200 and 500 mm. In this paper, consideration to different pre-treatment or repair materials, such as use of putties, before strengthening have not been taken into consideration. Nor have different concrete qualities been investigated. However, this has partly been investigated by Andersson and Spett (2002). In the numerical analysis, the nonlinear finite element method is used and the suitability of the constitutive model of the reinforced concrete, damaged plasticity, is evaluated.

**EXPERIMENTAL WORK**

A specially designed four-point bending test equipment was developed to test the bond strength, see figure 2. This test beam consisted of three main parts; part A of steel, part B of concrete, and one part C, that was bolted to part A. The part C was tailor-made to the strengthening method investigated, i.e. it had different configuration.
for plates, sheets and NSMR respectively. The purpose with this set-up was to make the testing more efficient.

The concrete parts (part B) were prefabricated and delivered to the laboratory. In the laboratory testing, two different surface treatments were investigated, grinding and sandblasting, at the same time irregularities and different substrates to repair the concrete surface before strengthening were investigated, see Andersson and Spett (2002). The CFRP was bonded to the steel plate, part C, which was then bolted to part A. The concrete and steel parts were during handling fixed together to avoid bending of the beam. The adhesive and the CFRP in the tests have been delivered by Sto Scandinavia AB. The material and geometrical data for the CFRP is listed in Table 1.

![Experimental test set-up.](image)

Table 1 Material parameters for the carbon fibre materials.

<table>
<thead>
<tr>
<th>Type</th>
<th>Modulus of elasticity [GPa]</th>
<th>Tensile strength [MPa]</th>
<th>Dimension [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>StoFRP Bar E 10 C Rod</td>
<td>140</td>
<td>3200</td>
<td>10x10</td>
</tr>
<tr>
<td>StoFRP Plate S 50 C Plate</td>
<td>160</td>
<td>2800</td>
<td>1.5x49</td>
</tr>
<tr>
<td>StoFRP Sheet S300 C200 Sheet</td>
<td>234</td>
<td>4500</td>
<td>0.11x150</td>
</tr>
</tbody>
</table>

Two different bonding agents were used, Sto BPE Lim 417 (Part A+B) for the sheets and Sto BPE Lim 465 (Part A+B) for the plates and NSMR. For the tests presented in this paper, sandblasting was used both on concrete and steel surfaces. The steel surfaces were immediately protected with a steel primer to avoid corrosion on the steel. The sandblasted surfaces were treated according to the material supplier’s recommendation, i.e. vacuum cleaning and surface treatment with a primer, Sto BPE Primer 50 Super. 24 hours after the primer was applied the concrete specimens were strengthened. For the sheets, the adhesive was applied on the steel and concrete parts, B and C at the same time, three layers of sheets were applied and the fabric was wetted according to the manufactures recommendations. The procedure for the plates was somewhat different, here the adhesive was applied on the plates and the substrates were the mounted together, a constant adhesive thickness of 2 mm was obtained by distance holders. For the square NSMR rods, the adhesive was applied in the pre-sawed groove in the concrete part, part B, and in the slot in the steel, part C. The thickness of the adhesive was estimated to 2.5 mm for the NSMR bar around its perimeter. The thickness for the sheets was estimated to 0.5 mm. This was also checked after testing and the deviation from the estimation was minor. All bonding was carried out indoors in a controlled environment at 20 ± 2 °C and 55 % RH. Electrical strain gauges were used to measure the tensile strain in several points along the CFRP. Figure 3 shows the positions of gauges for the plate with an anchorage length of 500 mm.

Table 2 Material parameters for the bonding materials.

<table>
<thead>
<tr>
<th>Type</th>
<th>Modulus of elasticity [GPa]</th>
<th>Tensile strength [MPa]</th>
<th>Viscosity [Pa s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sto BPE Primer 50 Super</td>
<td>Epoxy</td>
<td>---</td>
<td>0.8</td>
</tr>
<tr>
<td>Sto BPE Lim 417</td>
<td>Epoxy</td>
<td>2</td>
<td>50 Tixotropic</td>
</tr>
<tr>
<td>Sto BPE Lim 465</td>
<td>Epoxy</td>
<td>7</td>
<td>31 Tixotropic</td>
</tr>
</tbody>
</table>
In this study 8 specimens were tested and compared with the result from FE analysis; plates and NSMR with an anchorage length of 100, 200 and 500 mm and sheets with 100, 200 and 500 mm. However, due to failure in the measurement equipment the experimental results for the 100 mm sheets could not be extracted.

**NUMERICAL ANALYSIS**

**General**

The numerical analysis is a full 3D nonlinear finite element analysis of the experimental set-up using the commercial FE program ABAQUS/Standard. Two symmetry lines in the test set-up have been utilized when building the FE models. The symmetry lines are the apparent symmetry line along the middle of the concrete beam and the "symmetry" line at the hinge. In the latter case, the symmetry is certainly present during elastic deformations but can be questionable in the nonlinear region. It has been assumed that this irregularity does not have a significant influence on the results. The concrete and the steel in the beam are modelled by eight-node linear brick elements with reduced integration. The reinforcement is represented by linear truss elements. The NSMR and the adhesive are modelled by eight-node linear brick elements with reduced integration and the sheet/plate by four-node shell elements with reduced integration. Typical FE models for the three strengthening techniques are shown in Figure 4. Full bond is assumed in the NSMR case, i.e. there is full transfer of load between CFRP and adhesive and between adhesive and concrete. The bonding of the plate/sheet to the concrete is also modelled by assuming full bond to concrete without considering the adhesive layer. Full bond between the steel reinforcement and the concrete is assumed. The boundary conditions for the model are set for the support, along the middle of the beam, in the hinge and at the CFRP.
Material models

Many concepts for describing the quasi-brittle behaviour of reinforced concrete are present today, e.g. discrete crack, smeared crack, inner softening band (Tano 2001). In this paper, a damaged plasticity model has been used for the concrete. It is a constitutive model included in ABAQUS (Hibbitt et al. 2005, Lubliner et al. 1989, Lee and Fenves 1998) and has been successfully used by the authors in numerical analyses of CFRP strengthened concrete slabs with openings (Enochsson et al. 2005). The evolution of the yield surface is controlled by two hardening variables, one in tension and one in compression. Non-associated flow is assumed where the flow potential $g$ is the Drucker-Prager hyperbolic function. The crack propagation is modelled by using continuum damage mechanics, i.e. stiffness degradation. This means that the modulus of elasticity is reduced in the concrete where cracking occur. A prominent advantage of the damaged plasticity model is that the calculations actually converge. This in comparison to the smeared crack model where often stress locking or other numerical problems appear causing the solution to degrade or not to converge.

The concrete behaviour in tension is linear elastic until cracking is initiated and a strain softening response is assumed in the post failure region. The post failure behaviour is specified in terms of the stress-displacement response in order to minimize mesh sensitivity. It defines the tension softening behaviour and is described here by a bi-linear curve, see Figure 5. The fracture energy for mode I, $G_f$, is the area under the softening curve and is estimated to 100 N/m. The tensile damage is specified by an assumed linear relationship between the tension damage variable $d_t$ and the crack opening $\delta$. The maximum value of the damage variable $d_t$ is set to 0.9, and the maximum crack opening $\delta_0$ is calculated from the fracture energy, see Figure 5. The tensile strengths of the concrete, $f_{ct}$, are estimated from the splitting tests as 80 % of the splitting strengths, see Table 3. In compression, the concrete behaviour is linear elastic until initial yield stress $\sigma_{c0}$ is reached. The material enters the plastic regime with a strain hardening before the ultimate compressive stress $\sigma_{cu}$, followed by strain softening. The ultimate compressive stress is obtained experimentally from compressive tests, see Table 3. The initial yield stress for the concrete is assumed to be 60 % of the ultimate stress and the typical strain at ultimate stress is 0.2 %. In this analysis, the nonlinear part of the constitutive model in compression is somewhat unnecessary since the initial compressive yield stress will not be exceeded. The damage evaluation in compression is omitted since crushing does not occur. By following the Swedish concrete code BBK 04 (2004), the modulus of elasticity is estimated from the compressive strength and Poisson’s ratio is assumed to be 0.2.

![Figure 5 The concrete tension softening curve and the relationship between the tension damage variable, $d_t$, and the crack opening, $\delta$.](image)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet 100, 200</td>
<td>57.3</td>
<td>35</td>
<td>4.73</td>
<td>3.78</td>
</tr>
<tr>
<td>Sheet 500</td>
<td>48.7</td>
<td>33</td>
<td>4.18</td>
<td>3.34</td>
</tr>
<tr>
<td>Plate 100, 200, 500</td>
<td>44.6</td>
<td>33</td>
<td>3.81</td>
<td>3.05</td>
</tr>
<tr>
<td>NSMR 100, 200, 500</td>
<td>50.8</td>
<td>34</td>
<td>3.95</td>
<td>3.16</td>
</tr>
</tbody>
</table>

The stress levels in the steel plate, the steel reinforcement and the CFRP for these tests are always in the elastic region, i.e. it is sufficient to model the steel and CFRP as linear elastic materials. The modulus of elasticity for steel is assumed to be 210 GPa. The CFRP is modelled as an isotropic material due to the loading conditions in the test set-up. The adhesive is linear elastic until failure and the post-failure relationship is assumed to be ideal- elastoplastic for simplicity. Poisson’s ratio for the steel, CFRP and adhesive is set to 0.3.
RESULTS

Failure load

Table 4 shows the failure loads of the experimental work and the numerical analysis. The failure load for the numerical analysis is defined as the load level at which the simulation fails to converge. A large difference is noted when changing the anchorage length from 100 to 200 mm in both the plate and NSMR cases. The failure load is approximately doubled in the experimental results. In the experimental results for the plate, there is no significant difference in the failure load for the anchorage lengths of 200 and 500 mm. For the NSMR, the failure load is increasing when going from an anchorage length of 200 to 500 mm but not nearly as much as between 100 and 200 mm. For the sheets, it can be noted that though there seems to be a small difference between 200 and 500 mm it could be greater since the concrete strength for the anchorage lengths are not the same. The numerical analyses are predicting lower failure loads than the experimental results except in the case of NSMR for an anchorage length of 100 mm. The difference in results can be a consequence of underestimating the concrete’s failure energy. Also the isotropic damaged plasticity model is weakening the concrete too much when extensive cracking is occurring. For the sheets, the numerical failure loads are not significantly different from each other. When studying the stress states at failure for the different anchorage lengths it seems that the calculations cannot continue with the cracking past 100 mm. The calculations have convergence problems. This is also noted for NSMR but then at an anchorage length of approximately 250 mm. There is a similar pattern for the plate but then at approximately 300 mm.

Table 4 Failure loads of the test set-up.

<table>
<thead>
<tr>
<th>Strengthening</th>
<th>Anchorage length [mm]</th>
<th>Experimental Failure loads [kN]</th>
<th>Numerical Failure loads [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet</td>
<td>100</td>
<td>28.5</td>
<td>28.5</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>52.5</td>
<td>28.5</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>56.1</td>
<td>30.2</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>21.6</td>
<td>16.9</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>41.2</td>
<td>25.4</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>40.4</td>
<td>30.3</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>27.0</td>
<td>28.5</td>
</tr>
<tr>
<td>Plate</td>
<td>200</td>
<td>60.9</td>
<td>42.2</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>85.6</td>
<td>42.2</td>
</tr>
</tbody>
</table>

Strain distributions

Figures 6 – 8 show the strain distribution along the middle of the CFRP. The experimental strains are represented by discrete points (Exp.) and the numerical results by continuous lines (FE). The strain distribution of the sheet and the plate with an anchorage length of 500 mm shows that after 250 mm the strains in the CFRP are negligible. The strains in the NSMR show a similar behaviour; the strains are a little larger at 250 mm. For all strengthening cases, the strain distribution for an anchorage length of 500 mm shows an increase at 50 mm for higher load levels. This is believed to be the result of a localized crack. A sharp corner is formed at this location and the CFRP is forced to bend over it hence an increase in strain.

The strain distributions in the plate and the NSMR in the numerical results correspond well with the experimental results. This is not the case for the sheet where there is a noticeable difference in strain distribution. This could be explained by uncertainties in using the correct cross sectional area of the sheet in the simulations.

Figure 6 The strain distribution along the middle of the sheet.
DISCUSSION

The results for the plate investigated in the test show that at an anchorage length of approximately 200 mm the bonded CFRP cannot sustain any more load even if the anchorage length is increased to 500 mm, i.e. the critical anchorage length for the plate is approximately 200 mm. This is supported by the strain distribution in the plate. At approximately 250 mm, the strains in the CFRP are negligible. In the NSMR case, there is not an apparent critical anchorage length. The failure load is increasing with the anchorage length between 200 and 500 mm though not as much as between 100 and 200 mm. The strain distribution show that the strains have decreased at 250 mm but not as much as in the plate case. A suggestion is here to carry out additional laboratory tests. Since the sheet only have experimental results for 200 and 500 mm, a critical anchorage length could not be established but based on these two lengths it can be assumed the critical length is less than 200 mm. The strain distribution for the sheet is similar to the plate. As can be seen in the comparison between the numerical and the experimental results for the failure loads, the numerical analyses underestimates the failure load. Firstly, the FE analysis has convergence problems. The correct response would be a continued cracking for larger anchorage lengths. Secondly, the progressing cracking zone is weakening the concrete too much. This is believed to be the result of the isotropic damaged plasticity model. The damage affects the stiffness in all directions yielding a too soft response. A cracked zone is formed with equal damage in all direction due to the constitutive model, not as in reality where discontinuities appears and have an anisotropic behaviour. The fracture energy, the tensile strength and the shape of the softening response for concrete are quite important parameters for the constitutive model in finite element calculations. These are well known facts and a slight variance in these parameters can significantly reduce the accuracy of the solution.

CONCLUSIONS

Three anchorage lengths for three strengthening techniques, sheets, plates and NSMR, have been investigated both experimentally and numerically. The purpose of the study is to possibly establish critical anchorage lengths for the three strengthening methods. The critical anchorage length for the sheet can not clearly be established, however, there is a tendency that the critical anchorage length is less than 200 mm. At an anchorage length of 200 mm for the plate, the results show that a critical anchorage length is attained. Increasing the anchorage length adds safety to the structure but does not increase the load carrying capacity. The NSMR strengthening method does not show a clear result regarding the critical anchorage length. However, it is obvious that a critical anchorage length must exist at a length exceeding the tested 200 mm. The numerical results are sensitive to the values of the fracture energy, the tensile strength and the shape of the softening response for the concrete. Accurate methods of determining these material parameters are essential for the finite element analysis. Also, since the crack paths in the analysed test was complex including both mode I and II behaviour the damaged plasticity model used here may not be suitable. It can also be mentioned that, when the material properties of the constituents or geometry are changed, most likely the critical anchorage lengths will also change.
ACKNOWLEDGMENTS

This study has been funded by SKANSKA, Sto Scandinavia AB and SBUF (The Development Fund of the Swedish Construction Industry). Björn Andersson and Magnus Spett are greatly acknowledged for their experimental work while performing their Master’s thesis. The help and support by Håkan Johansson, Lars Åström and Georg Danielsson at Testlab, Luleå University of Technology, in discussion and preparation of the test setup is also appreciated.

REFERENCES


