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APPLICATION OF HIGH PERFORMANCE CONCRETE FOR RECONSTRUCTION

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Abstract.

One of the sustainability aspects of the construction is a sufficient durability. In objectively justified cases, the durability is enhanced by reconstruction. To ensure economic design of the reconstruction, non-liner approaches are often applied. This paper deals with aănonlinear numerical simulation of disassembled reinforced concrete (RC) panels strengthened by an ultra-high performance concrete (UHPC) layer. Based on the prior experimental programme, simulations of four-point bending tests of the original and strengthened panels are created using software ATENA Science. Calibration of the material parameters is based on the destructive and non-destructive investigations performed in the experimental programme. The comparison of the experimental and numerical model loading curves indicates that additional mechanical testing will be needed in order to achieve an accurate numerical simulation. Although the bending test of the RC panel and the crack mouth opening displacement (CMOD) test performed on the applied UHPC beam were calibrated, the final model of the UHPC strengthened panel does not correspond completely with the experimental measurements. In this paper the possible reasons for this result are discussed.

Keywords: Analysis, high-performance concrete, reconstruction.

1. State of the art

In recent decades, environmental aspects have been attracting more and more attention in the field of building structures. One of the possible approaches to an environmentally friendlier structure is prolongation of its service life. By improving the no-longer complying parts of structures or further utilization of removed building components, the negative impacts of construction can be reduced greatly.

However, an increased demand for the load-bearing capacity or a different loading type of the recycled component very often results in necessary strengthening of the structure. In the case of concrete structures, an application of a new additional concrete layer, usually from a type of high-performance fibre concrete, has been proposed and investigated as a potential strengthening method [1–13].

In the previous studies, the layer from high-performance (HPC) or ultra-high performance (UHPC) concrete has been generally applied in four strengthening configurations in thickness from 20 to 100ămm (most frequently 30 mm).

A strengthening layer located in the compression zone was most frequently used in the case of panels [2, 5, 8, 12]. The main advantage of this configuration lies in its easier performing as the strengthening layer is applied on the upper surface, thus no complex formwork is needed. Concrete beams have been, in the conducted experiments, strengthened in the compressive zone [1, 4, 10, 11], the tension zone [3, 7, 11], two-sided (left and right surfaces) [7, 13], and three-sided (aăcombination of all of the above) [3, 7, 9, 11, 13].

In order to determine the most effective strengthening configuration, a number of studies compared the impact of the layer location on the load-bearing capacity of the element [7, 11]. The results indicated that the three-sided configuration reaches the highest values (an increase of up to 89% in the loadbearing capacity) following by the two-sided (around 47%) and the tensile configuration with as low as 16%. Not surprisingly, the conducted experiments also showed that the magnitude of the tensile strength of the strengthening layer has a decisive influence especially in the three-sided configuration, whereas it has only a little effect in the case of compressive zone [7].

Another important aspect of the HPC or UHPC strengthening method is the adhesion of the new layer with the original concrete component. In [7, 13] which have dealt with this topic, a transverse tensile strength test was performed on cylindrical or cubic specimens which were prepared from both the original basic concrete and UHPC. The results of these experiments were also further used in numerical finite element (FE) modelling in order to describe the behaviour of the contact layer.

To further improve the adhesion of the layers, researchers have been also investigating different surface treatments, such as certain adhesion agents

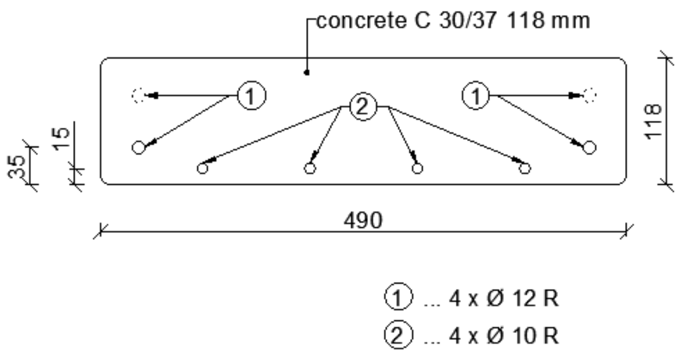

Figure 1. A typical geometry of the non-strengthened panels based on destructive investigations.

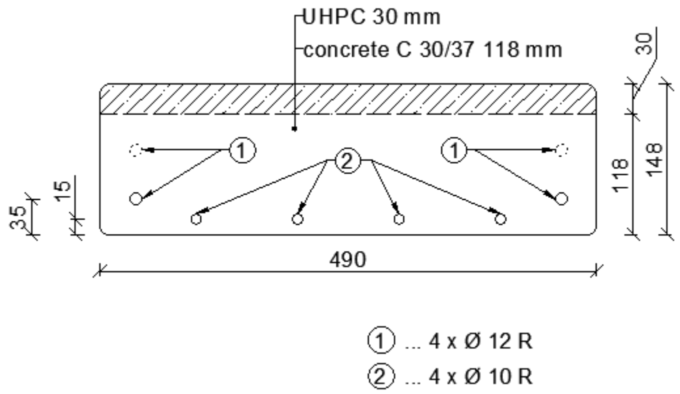

Figure 2. Typical geometry of the strengthened panels after the UHPC layer application [14].

(e.g. epoxy resin) or roughening (e.g. sandblasting, hydrodemolition, or wire-brushing). A study [7] showed, that specimens with an epoxy resin surface treatment reach higher values in the transverse tensile strength test but lower values in the shear strength test compared to specimens with a surface treated with sandblasting. Importantly, the further bending test showed that the sandblasted surface leads to higher load-bearing capacity compared to the epoxy treatment. The finding indicates that the strength of the new-to-old concrete bonding depends primarily on its shear strength.

This paper focuses on numerical modelling of the UHPC strengthening method. As high-performance concrete is rather economically and ecologically demanding material, careful optimization of its placement and characteristics is crucial for the sake of maintaining the positive effects of the application of the method. In order to achieve an effectively strengthened concrete structure by an UHPC layer, it is necessary to describe characteristics of the materials and their interaction as accurately as possible. For this reason, a large number of studies have been conducting numerical non-linear analysis alongside mechanical testing [7, 11–13]. By using an appropriate finite element software (e.g., ATENA, ABAQUS) and material models, the studies have been generally able to successfully describe the structure behaviour with only little (no more than 13.4%) deviations from the mechanical test results.

However, several issues in the topic of FE modelling needs further clarification as they might influence the model accuracy. Firstly, the compressive and tensile stress-strain relationship for the UHPC is a rather complex problem, as it is significantly different from the normal strength concrete. Further, the characteristics of the bonding between the original substrate and new UHPC layer may distort the results, although the majority of previous studies considered perfect bonding.

In our paper, we focus on FE numerical analysis of four-point bending tests performed on prefabricated reinforced concrete panels strengthened by a UHPC layer. In this study, several calibration procedures of material parameters are presented, and the outcomes are compared with experimental measurements. Further, the specifics of designing recycled structures are discussed and further development is outlined.

2. Experimental program

2.1. Specimens

Bending tests were performed on several prefabricated reinforced concrete (RC) panels, which have been located in outside conditions for several years. The panels were around 120 mm high, 490 mm wide, and 2750 mm long; however, the dimensions varied slightly in every specimen. Destructive tests showed that the panels were reinforced with steel reinforcement - two ⌀12 mm bars at the bottom surface and two ⌀12 mm bars at the upper surface. However, the performed destructive investigations revealed that the location of the bars was slightly different in every specimen. Further, due to the inadequate reinforcement at the bottom surface, four ⌀10mm B500B bars were glued into formed grooves to increase the load-bearing capacity of the panels.

Before casting of the UHPC layer, the upper surfaces of the panels were treated by hand hydro-jetting to ensure adequate bonding of the original substrate with the new layer. After the treatment, the UHPC layers were cast in the compressive zone in thickness of 30 mm. This study focuses on two experimental series - reference panels with no strengthening layer (Figure 1) and panels with 30 mm UHPC layer (Figure 2).

2.2. Material properties

In order to determine the strength class of the original RC panels, a number of destructive and nondestructive tests were performed. According to the measurements, the material was categorized as C30/37.

The material characteristics of the prepared ultra-high strength fibre concrete (UHPC) were investigated by destructive mechanical testing. The compressive test conducted on cubic specimens determined its mean compressive strength as 133 MPa. Further, a three-point bending test, with support span 500 mm, was conducted on pre-notched beam specimens (150 × 150 × 700 mm) to obtain curves of the dependence of the loading force on the crack mouth opening displacement (CMOD).

2.3. Testing methods

To determine behaviour of the investigated strengthened and non-strengthened panels, a series of four-point bending tests were conducted. The span of the supporting pins was 2500 mm, the span of the loading pins was 800 mm. The loading force and deformation were monitored and recorded during the loading.

3. Numerical modelling of the four-point bending tests

All of the analysis was conducted using a finite element analysis software ATENA 5.6.1 Science with GiD interface. The actual geometry of the test setup was modelled in all cases. In the case of the fourpoint bending tests, the symmetry of the setup was considered, and only halves of the specimens were modelled. In all cases, the loading was introduced into the model as a pre-described deformation through steel distribution plates.

3.1. Numerical modelling of the reference non-strengthened panels

Firstly, the four-point bending test of the non-strengthened panels was modelled. As the information about its materials was limited, a careful calibration of the numerical model was crucial for further investigation of the UHPC strengthening method.

In order to determine whether it would be possible to omit the spatial effect, the RC panel was modelled as both a plane stress (2D) and a three-dimensional stress (3D) structure model type. The geometry was identical in both cases. However, due to the significantly lower computational demands of the 2D model, the mesh size could be set to 15 × 15 mm in the case of the 2D model, whereas the 3D model used 25 × 25 mm.

In order to obtain the loading curves, the loading force was monitored using a monitor placed under the force and the deflection was monitored on the lower surface in the middle of the span.

3.1.1. Material models

As mentioned above, the information about the RC original materials were limited. As a starting point, the material parameters of both concrete and steel bars were generated using the predefined materials in ATENA GiD software. Material Concrete EC2 which uses the CC3DNonLinCementitous2 material model was selected as a representation of concrete. As this material generates the parameters according to the Eurocode2, the category C30/37 was set.

The steel bars, both the original and the additional, were defined as Reinforcement EC2 using CCReinforcement set to the category B500B.

3.1.2. Parametric study and calibration of the FE models

Firstly, analysis of the 2D and 3D model with generated default material parameters, using the material models mentioned above, was run. Subsequently, the loading-deflection curves obtained from the analysis were compared in order to evaluate the differences between the 2D and 3D modelling.

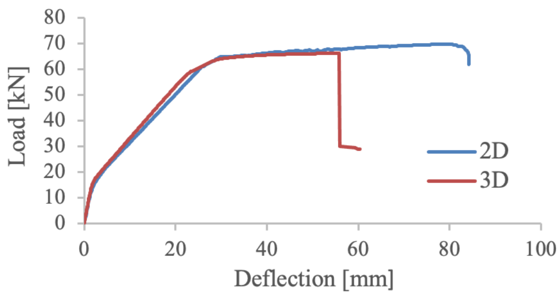

Figure 3. A comparison between the 2D and 3D loading curve - a reference panel with default material parameters.

As can be seen in Figure 3, generally, the curves did not differ significantly. The crack formation and the

Parameter	Units	Default settin	Multiplication factor
		Concrete material model	
Young modulus	MPa	32000	0.56
Tension strength	MPa	2.9	1.00
Compressive strength	MPa	−38	0.79
Fracture energy	MN/m	0.000073	1.14
Tension stiffening	–	no	yes
Critical compressive displacement	m	−0.0005	1.10
		Original steel reinforcement model	
Characteristic yield strength	MPa	500	0.84
Class	–	B	C

Table 1. Quality and Operations Assessment Schedules [7].

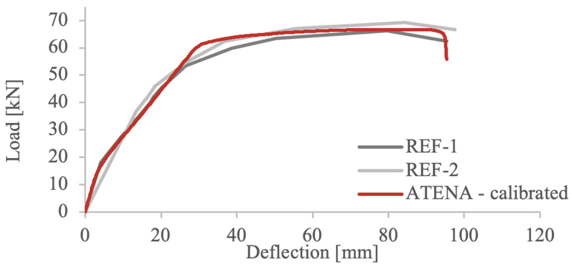

Figure 4. The experimental loading curves of the tested non-strengthened panels compared with the calibrated ATENA simulation.

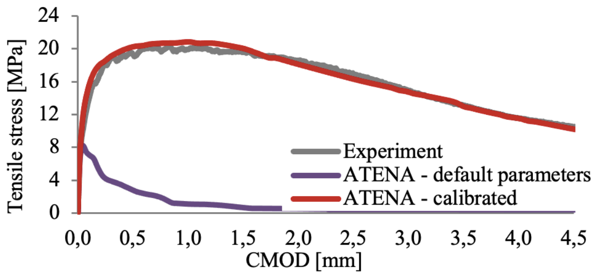

Figure 5. The experimental force-CMOD curve compared with the FE analysis results (with default material parameters and after the calibration procedure).

reinforcing bars yield occurred at almost the identical loading force (at approx. 15 kN and 63ăkN, respectively). However, before the yield point, the lower slope of the 2D curve indicated a slightly lower stiffness in the case of 2D analysis, especially after the crack formation. On the other hand, after the yielding, the slope of the 2D curve was higher compared to the 3D curve. Furthermore, the failure of the 2D model took place at significantly higher values of the deflection, resulting in a higher maximum load force.

On the base of a parametric study (the analysis was run several times, while changing the individual values of the default material models) the numerical models were calibrated (Figure 4) by setting the material parameter values to required values, so that the loading curve obtained from the FE analysis would correspond as closely as possible with the experimental measurements. In Table 1, the key default parameters together with the necessary multiplying coefficient for the changed values are listed.

3.2. Numerical modelling of the panels with a UHPC layer

After the calibration of the model of the non-strengthened panel, analysis of the strengthened specimens could take place. Conveniently, unlike the original materials, the newly introduced UHPC could have been subjected to several destructive tests. Thus, the information about its compressive strength and tensile behavior was much more certain.

To correctly determine its tensile material parameters, a three-point bending test was modelled in the ATENA software and calibrated using the conducted experiment. Subsequently, the strengthened panel was modelled using the parameters obtained from the numerical representation of the bending test.

3.2.1. The CMOD analysis of the applied UHPC

In order to define the UHPC material parameters, aăUHPC beam subjected to the three-point bending test was modelled using ATENA Science in 2D. In order to obtain the force-CMOD curves, the loading force and horizontal displacements in the notch mouth were monitored.

As a representation of the UHPC, CC3DNonLinCementitous2User was utilized as it allows the user to define its own fracture-plastic material model, and it is recommended for the fibre reinforced concrete (FRC) modelling. The calibration procedure was performed according to the software documentation. The tensile function, which have a crucial importance for the FRC materials, were carefully set so that the force-CMOD curves corresponded and closely as possible (Figure 5). Calibrated values of the material parameters can be seen in Table 2.

Parameter	Units	Calibrated values
Parameter	Units	Calibrated values
Young modulus	MPa	43000
Tension strength	MPa	6.5
Compressive strength	MPa	−133

Table 2. Calibrated values of the key material parameters based on the CMOD analysis.

3.2.2. Modelling of the four-point bending test of the strengthened panels

Based on the four-point bending test of the strengthened panels, the strengthened panels were modelled using ATENA Science as a plane stress structure model type. The geometry of the model remained identical, only the 30 mm UHPC layer was added. The connection between the old and new layer was considered perfectly solid, thus no mechanical characteristics were prescribed.

The material parameters of the steel bars and original concrete were set according to the calibration of the non-strengthened panels. The UHPC parameters were taken over from the calibrated CMOD analysis. Results of the analysis run with the parameters based on the prior calibration procedures can be seen in Figure 6.

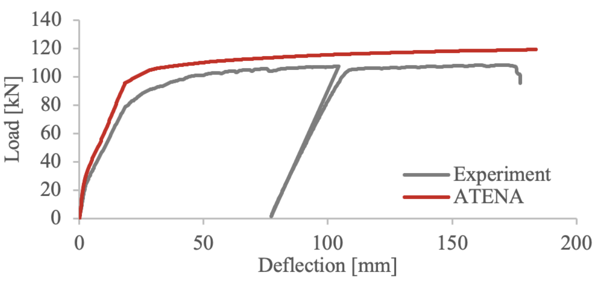

Figure 6. The experimental loading curve of strengthened panels compared with the ATENA simulation with material parameters based on the prior calibration procedures.

As can be seen, the load-deflection curve from obtained from the FE analysis did not match the experimental measurements completely. Similarly, the formation of the tensile cracks occurred around the same loading force and deflection. However, the stiffness of the cross-section after the cracking differed quite significantly, as the curve slope of the FE model is notably steeper. The FE model also showed approx. 12 % higher maximum loading force than the experimental measurements.

4. Discussion

The finite element analysis of strengthened prefab concrete panels presented in this paper pointed out several crucial factors which influence the design of the UHPC strengthening method. As previously stated in the introduction, the environmental benefits of recycling or renewal of no-longer necessary or unsatisfactory concrete structures are unquestionable. However, as is apparent from the experimental part of the study, the knowledge of the original materials and its geometrical arrangement in the structure is frequently incomplete. For this reason, the numerical analysis inputs are then based on several destructive or non-destructive tests, which provide only limited information about selected specimens.

As the parametric study of the non-strengthened panels conducted in our paper showed, the structure behaviour was fundamentally affected by the placement and characteristics of the original steel reinforcement. In our experimental program, the placement, and possibly the diameter, of the bars were slightly different in every specimen, and it was not possible to subject the bars to any mechanical measurements. Thus, although the FE model of the four-point bending test corresponded well with the experiment after the calibration, it was adjusted only to the chosen specimens. Therefore, the model could have been unsatisfactory for the further strengthened panels due to a different geometrical arrangement.

More known inputs were available for the modelling of the strengthened panels. The CMOD analysis of UHPC provided information about the material tensile properties and the compressive test determined its compressive strength. However, no load-deflection curve was known from the pressure test, only the maximum loading force. Thus, the compressive function (the stress-strain relationship) could not have been determined based on any measurements. The loading curve of the strengthened panel showed, that the calibrated model of the original panel and UHPC with defined tensile function based on the CMOD analysis did not match with the experimental measurements. This unsatisfactory result could have been caused by several factors.

Firstly, as stated above, the model fitted on the selected non-strengthened panels could have been unsuitable for the selected strengthened panels due to the different geometry or materials. Secondly, as mentioned in the introduction, the tensile properties of the strengthening layer in the compressive zone have only a limited impact on the load-bearing capacity. Thus, the setting of the compressive function based on the load-deflection curve could have been crucial and more experimental measurement is therefore needed. Further, no stresses due to shrinkage of the UHPC layer and mechanical properties of the connection between the original substrate and new material were considered. Although no visible deterioration of the layers was apparent during the bending test, the effect may not be negligible in the FE analysis and determination of the interface parameters will be necessary in future investigations.

5. Conclusions

Application of high-performance cementitious composites for reconstruction of concrete structures is aăstep towards sustainable construction. The reuse of damaged or no-longer satisfactory structural elements leads to saving both material and waste. However, an optimized design of the required strengthening is the key to ensure its advantageousness. When the non-linear behaviour of the materials is taken into account, it allows the design to be less conservative, thus more economic.

In this paper, the performed non-linear numerical analysis pointed out several crucial factors which influence the numerical simulation suitability. For a sufficient numerical model, the geometry of the recycled structures must be known as precisely as possible. Further, when modelling only a general geometry of the structures, the slight differences in the individual elements might affect the model accuracy greatly. However, most importantly, it is crucial to ensure inputs for the numerical analysis by conducting a sufficient number of mechanical tests of the materials present in the modelled structure.

Acknowledgements

FV20472 Application of high performance cementitious composites at rehabilitation of concrete structures, SGS19/149/OHK1/3T/11 Durability of concrete structure and assessment of its life cycle.

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