Modeling and strengthening of RC bridges by means of CFRP

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ABSTRACT: In the frame of the research work APVT-20-012204, three reinforced concrete girder bridges were diagnosed. The results of the diagnostic will be used for modelling and investigation of the real traffic load effects. The modified reliability levels for the evaluation of existing bridges were used for their evaluation (Koteš 2005, Koteš & Vičan 2006). The different remaining bridge lifetimes for various load models given in the Slovak standard were obtained by calculation.
Here, the paper is focused on just one bridge, which is the most corrupted. This bridge was numerically modeled using software ATENA. The material characteristics needed for modelling were obtained by diagnostic. The bridge should be strengthened due to low load-carrying capacity. The CFRP lamellas for strengthening of the girders subjected to bending will be used in the FEM model. The results of numerical analyses using the FEM model are used for the prepared experiment.

1 INTRODUCTION

The paper presents the objectives and partial results of research work no. APVT-20-012204 "Remaining service life and increase of concrete structure reliability". The optimum methods for increasing of existing concrete structure reliability by the application of the new technologies based on the technical diagnostics method, computer simulations and probabilistic evaluation of the concrete bridges, is the major aim of this project.

The attention of the project is paid to the project concentrates on existing concrete bridge structures, especially to bridge evaluation and the increase of reliability and serviceability for the remaining lifetime (Frangopol & Estes 1997, Nowak 1995, Vičan 1999). There are always applied modern approaches to the structure reliability evaluation in practice in many countries. The approaches are based on probabilistic interpretations and they are supported by computer simulations. The new materials and technologies are used to increase the load carrying capacity and serviceability of concrete bridge structures as well. Many European standards concerning such problems are based on theoretical and experimental test issues.

For successive accepting of the European standards in Slovakia, the research of degradation factors on concrete structures, their accurate evaluation and new technologies are needed to increase the load carrying capacity and serviceability of structures.

2 SELECTION OF CRITERIA AND THE DESCRIPTION OF THE BRIDGE

The criteria for the bridge choice were determined in order to fulfil the aim of the research work. The criteria were focused on the kind of bridge (from the viewpoint of the superstructure type, faults occurrence, accessibility of the bridge), location, importance in the traffic network and adequate traffic action on the bridge.

Three reinforced concrete girder bridges satisfied the determined criteria. The bridges are in the village Kolárovice, part of Škoruby, on the road I / 18 over the Kolárovice river (Fig. 1).

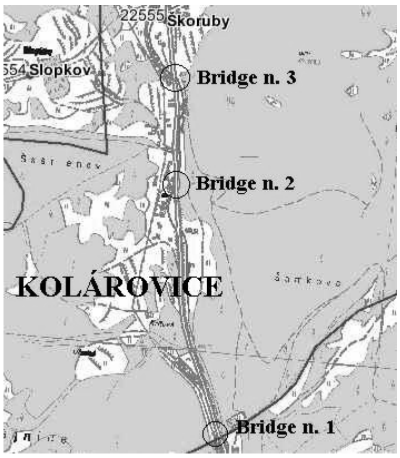


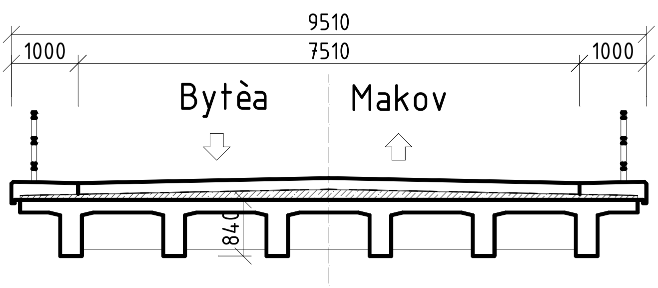
Figure 1. The location of the selected bridges in village Kolárovice.

All bridges have similar parameters concerning dimensions, material quality and traffic load, but the reinforcement corrosion and the significant bending and shear cracks were found just on bridge No. 3. Therefore, bridge No. 3 was selected for further investigation (Fig. 2).

Figure 2. Bridge superstructure.

The reinforced concrete single span girder bridge has a theoretical span of 10.006 m (the length of the bridge is 10.824 m). The width of the road is 7.51 m and the overall width of the bridge is 9.51 m. The bridge skewness is 45.22°. The superstructure consists of a bridge slab having a thickness of 0.19 m and of six main beams with dimensions of 0.325 / 0.84 m. The end diaphragms have dimensions of 0.58 / 0.84 m and three intermediate diaphragms having dimensions of 0.20 / 0.74 m to ensure the transverse load distribution (Fig. 3).

Figure 3. The bridge cross-section.



3 THE RESULTS OF THE DIAGNOSTICS

From the results of the bridge diagnostic follow that the type of the concrete is C 37 and the beams are reinforced by rebar of the type A (10210) in two layers (5ϕA30 in the lower layer and 2ϕA30 in the upper layer). Accordingly, the reinforcement corrosion was indicated. The corrosion caused the diameter loss from the initial value of 30 mm to the actual average value of 29.3 mm (the minimal measured value is 28.7 mm). The concrete cover is 30 mm. The measured values of geometric and material properties concerning all girders and slab are shown in Table 1.

Table 1. The measured values of geometric and material characteristics.

Variables	The mean value	The standard deviation
Strength of concrete – f_c.[MPa]	49.780	4.8689
Girder height – h [m]	0.8370	0.00356
Girder width – b [m]	0.3222	0.00371
Concrete cover – c [mm]	29.60	1.14
Bar diameter – ϕ [mm]	29.37	0.68
Slab height – h_d [m]	0.1864	0.00230

4 THE RELIABILITY-BASED EVALUATION OF THE BRIDGE STRUCTURE

In the frame of the research activities of the Department of structures and bridges at the University of Žilina, the modified reliability levels for the existing bridge evaluation were determined using a theoretical approach taking into account the conditional probability (Koteš 2005). Moreover, the influence of reinforcement corrosion as material degradation was investigated and incorporated into the theoretical approach. The obtained reliability levels depend on the age of the bridge and on the planned remaining lifetime.

The new modified reliability levels given in (Koteš 2005, Koteš & Vičan 2006) were used for determining lower partial safety factors for material and load effects, depending on the bridge age and its remaining lifetime. In the practical design, the reliability levels are transformed to the design values of the material resistance and load effects. In the partial safety factors method, the design values of the material resistance and load effects are determined by means of characteristic values and appropriate partial safety factors.

The selected bridge structure was estimated to a bending load-carrying capacity for recommended planned remaining lifetimes $t_r > 20$ years, $10 \leq t_r \leq 20$ years, $3 \leq t_r < 10$ years a $t_r < 3$ years. The actions given in the Slovak standard were intended as a characteristic variable load. The various lengths of the planned remaining lifetime were taken into account using various partial safety factors of materials (for concrete in compression $\gamma_{c,c}$, concrete in tension $\gamma_{c,t}$ and the reinforcement γ_s) and partial safety factors of loads (permanent loads and variable loads).

From the results of the bridge evaluation it followed that the remaining lifetime for load models given in the Slovak standard was less than 3 years and the load-carrying capacity is equal to 0.82<1.0 (taking into account the reinforcement corrosion). In order to increase the load-carrying capacity and elongation of the remaining lifetime, the urgency of bridge reconstruction or strengthening is evident.

5 STRENGTHENING RC STRUCTURES USING FRP REINFORCEMENT

The rehabilitation of RC structures using FRP (Fibre Reinforced Polymers) materials has become a growing area in the construction industry over the last few years. Many research projects in the world have been carried out to promote this efficient repair technique to extend the service life of existing concrete structures. FRP is a composite material generally consisting of carbon, aramid or glass fibers in a polymeric matrix (e.g. epoxy resin). Among many options, this reinforcement may be in the form of preformed laminates or flexible sheets. The laminates are stiff plates or shells that come pre-cured and are installed by bonding the plate to the concrete surface with epoxy. The sheets are either dry or pre-impregnated with resin (pre-preg) and cured after installation onto the concrete surface. This installation technique is known as wet lay-up (Kotula 2006).

The lightweight and formability of FRP laminates or sheets make these systems easy to install. And since the materials used in these systems are non-corrosive, non-magnetic, and generally resistant to chemicals, they are an excellent option for external reinforcement.

Externally bonded FRP laminates or sheets have shown to be applicable to the strengthening of many types of RC structures such as: columns, beams, slabs, walls, tunnels, chimneys and silos and can be used to improve flexural and shear capacities, and also provide confinement and ductility to compression structural members (Khalifa & Gold & Nanni & Abdel Aziz 1998, Khalifa & Nanni 2002).

Traditional methods such as for example different kinds of reinforced overlays, shotcrete or post tensioned cables are placed on the outside of the structure which normally needs much space. The FRP laminates or sheets do not require much space because they are very thin.

The bending strengthening of the structures is the most common way of the structure strengthening but shear strengthening is also often needed (Täljsten 2001).

6 SIMPLE DESIGN PROCEDURE

In the analysis, two types of beam models were considered. Firstly, the non-strengthened RC girder was calculated. The girder, which is three times reduced compared to the existing RC bridge girder in the village Kolárovice on the road I / 18, is shown in Figure 4. The 1/3 scale beam was considered because of the laboratory possibilities and the beam dimensions.

Next, the model of the strengthened RC girder with the externally bonded one MBrace® S&P CFK lamella 150 / 2000 was calculated (Fig. 5).

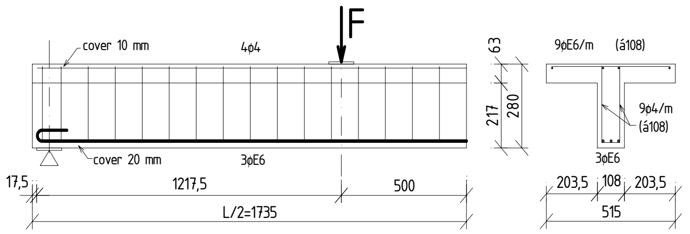


Figure 4. The three times reduced RC girder of RC bridge girder in the village Kolárovice on the road I / 18.

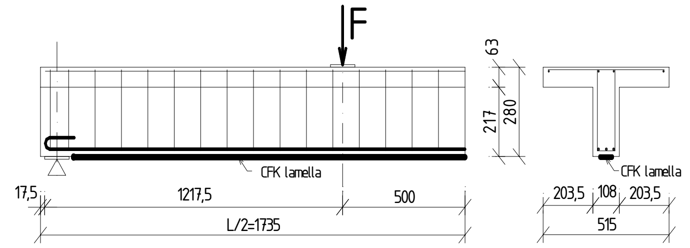


Figure 5. The three times reduced RC girder strengthened with one MBrace® S&P CFK lamella 150/2000.

Firstly, the resistance bending moment $M_{Rd,0}$ of the un-strengthened girder was calculated. The design moment $M_{Rd,0}$ was determined considering the existing geometry, reinforcement and concrete quality as well as the partial safety factors for material properties. The resistance bending moment $M_{Rd,f}$ of the strengthened cross section was calculated according to the assumption, that the degree of strengthening was $\eta \leq 2$.

The degree of strengthening is defined:

$$\eta = M_{Rd,f} / M_{Rd,0} \, . \tag{1}$$

In the following, Figure 6 shows the superposition of the strains and the internal forces acting on a reinforced concrete cross-section.

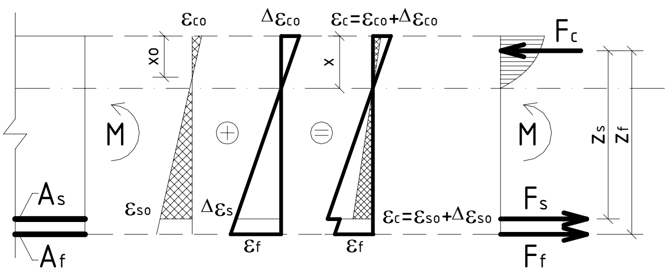


Figure 6. Superposition of the initial strain and additional strain after strengthening.

The required area of lamella and the resistance bending moment of the strengthened girder $M_{Rd,f}$ are derived from the condition of equilibrium $\Sigma F = 0$ and $\Sigma M = 0$ considering the mechanical behaviour of each material.

Internal forces are defined as:

$$F_s = E_s \cdot A_s \cdot \varepsilon_s \leq \frac{f_{yk}}{\gamma_s} \cdot A_s \tag{2}$$

$$F_f = E_f \cdot A_f \cdot \varepsilon_f \tag{3}$$

$$\varepsilon_f \leq \varepsilon_{f,lim} \tag{4}$$

$$F_c = \alpha_R \cdot b \cdot x \cdot \frac{\alpha \cdot f_{ck}}{\gamma_c} \tag{5}$$

where F_s = the force in reinforcement; F_f = the force in lamella; F_c = the force in concrete; E_s = the modulus of elasticity of reinforcement; E_f = the modulus of elasticity in fibre direction; A_s = the area of reinforcement; A_f = the area of lamella; ε_s = the strain in reinforcement; ε_f = the strain in lamella; $\varepsilon_{f,lim}$ = the limited strain in lamella; f_{yk} = the characteristic yield strength of reinforcement; f_{ck} = the characteristic compressive cylinder strength of concrete at 28 days; γ_s = the partial safety factor for the properties of reinforcement; γ_c = the partial safety factor for the properties of concrete; x = the neutral axis depth, b = the width of beam (girder); α = the reduction factor for concrete compressive strength; and α_R = the parabolic form parameter.

The presented simple design procedure is implemented in software S&P FRP Lamella (S&P FRP Lamella). The value of the resistant bending moment of the un-strengthened RC girder is equal to $M_{Rd,0} = 3.89\,\text{kNm}$ ($M_{Rk,0} = 4.48\,\text{kNm}$), to which the maximum force $F_{d,max,0} = 3.33\,\text{kN}$ ($F_{k,max,0} = 3.84\,\text{kN}$) corresponds. Adding one FRP lamella, the value of the resistant bending moment of the strengthened cross section is higher and the value is $M_{Rd,f} = 17.5\,\text{kNm}$ ($F_{d,max} = 14.98$ kN). It means that the degree of strengthening calculated from this program is $\eta = 4.51$. But, the formula (1) limits the degree of strengthening by value $\eta = 2.0$. This means, that the maximum applicable moment for strengthening is equal to value $M_{Rd,f,max} = 7.76\,\text{kNm}$ ($M_{Rk,f,max} = 8.94\,\text{kNm}$) and the corresponding maximum force is $F_{d,max} = 6.64\,\text{kN}$ ($F_{k,max} = 10.44\,\text{kN}$).

It should be pointed out that the calculated values of the observed parameters did not include the real adhesion between the concrete surface and the lamella. This means that the adhesion factors between the concrete surface + the epoxy glue + the lamella surface are omitted - only the epoxy glue material is omitted in the software. So, in the next part of the paper we will focus on finding the influence of the adhesion between the concrete surface and the lamella given by the layer of epoxy glue on the real resistance bending moment M_{Rdf} and the maximum loading force F_{max}.

7 NUMERICAL ANALYSIS

7.1 *Description of 2D FEM models*

The ATENA software (ATENA ver. 3.2.6.) was used for the numerical analyses of RC girders, which are again three times reduced compared to the existing RC bridge girders. For this analysis, two types of half beam FEM models were made. The first type is a FEM model of a half RC girder (Fig. 4). The second type is a FEM model of a half strengthened RC girder with an externally bonded one MBrace® S&P CFK lamella 150 / 2000 (Fig. 5).

The material model of concrete „Concrete-SBETA Material“, derived from CEB-FIP MC 90, was applied for concrete. The basic properties of this material model are: tensile strength, fracture energy and the equivalent unaxial law. This material model provides objective results due to the formulations based on energetical principles and its dependency on the finite element grid is negligible. The main reinforcement in the RC girders was modeled as a uni-directional reinforcement element. This material element offers a uni-directional law for a stress–strain diagram in the reinforcement whose form can be: linear, bi–linear and multi–linear. In our case, the bi–linear law was used. The MBrace® S&P CFK lamella 150 / 2000 and the epoxy glue were modeled as the Plane Stress Elastic Isotropic element and alternatively as the 3D Bilinear Steel Von Mises element. The different sizes of the macro elements of the strengthened finite models of the RC girder web were used. The dimensions of these macro elements were: 15, 30 and 50 mm. The dimensions of the macro elements of the girder flange were 30 and 50 mm (Tab. 2.).

Table 2. The review of 2D FEM models.

Model	Descrip. size of FEM grid mm/mm	Type of CFK lamella (epoxy) element
Model 0(Fig. 4)	F 30/W 15	Non-strengthened
Model 1(Fig. 5)	F 30/W 15	Plane Stress El. Isotropic
Model 2(Fig. 5)	F 30/W 15	3D Bilin. Steel Von Mises
Model 3(Fig. 5)	F 30/W 30	3D Bilin. Steel Von Mises
Model 4(Fig. 5)	F 50/W 50	3D Bilin. Steel Von Mises

Caption: F – Flange, W – Web of RC girder

The non-strengthened FEM half model of the RC girder is shown in Figure 7.

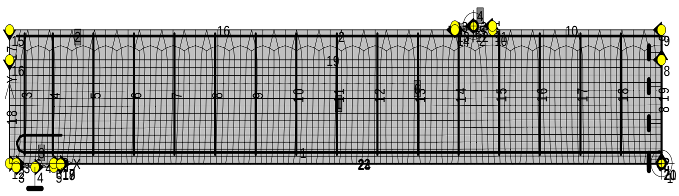


Figure 7. 2D FEM model of RC girder – Model 0.

7.2 *Description of 3D FEM models*

The same girders (Figs 5-6), which were modeled using 2D FEM models, were modeled by 3D FEM models in ATENA. The reason for the 3D modeling was to compare the results given from the 2D and 3D models and carried out the sensitivity analysis.

Two types of girders were again modeled – the first type was the non-strengthened FEM 3D model of the half RC girder and the second one was the FEM 3D model of the half strengthened RC girder with an externally bonded one MBrace® S&P CFK lamella 150 / 2000.

In the case of the 3D modeling, the fracture-plastic material model „3C Nonlinear Cementitious 2" was used because the material „Concrete-SBETA Material" was not available. This type of material is recommended for 3D concrete structures and element modeling (ATENA ver. 3.2.6.).

The reinforcement was modeled in the 3D model as well as in the 2D model. This means, that a unidirectional reinforcement element with a bi–linear law for the stress–strain diagram was used again. The reinforcement cage of the half 3D FEM model is shown in Figure 8.

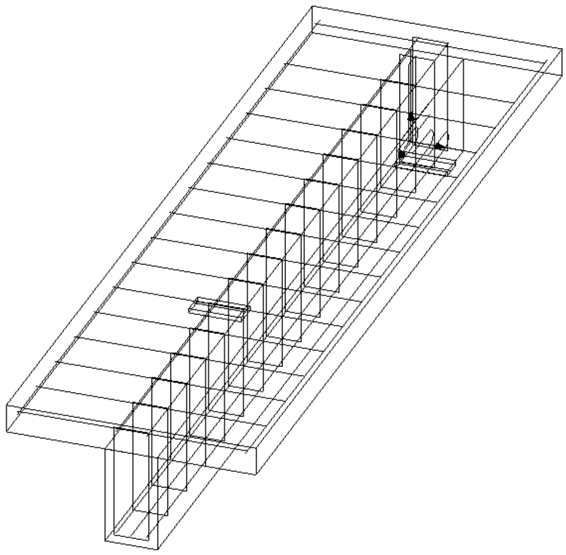


Figure 8. The reinforcement cage model of 3D FEM model.

In the 3D model, it was not possible to take into account the macro elements with a thickness less than 2 mm. So, it was a problem to model the layer of the epoxy glue having a thickness equal to 1 mm and the MBrace® S&P CFK lamella 150 / 2000 having a thickness of 1.2 mm. Therefore, these two macro elements were modeled as one macro element of a shell type having a thickness of 2.2 mm with two layers (layers model). The material characteristic of epoxy glue was assigned to the first layer of the macro element having a thickness of 1.0 mm and the material characteristic of the MBrace® S&P CFK lamella 150 / 2000 (3D Bilinear Steel Von Mises element) was assigned to the second layer of the macro element having a thickness of 1.2 mm.

Once again, the different sizes of the macro elements of non-strengthened and strengthened finite models of the RC girder web and flange were used. The dimensions of these macro elements were: 40, 50 and 100 mm (Tab. 3.). The detail of the epoxy glue and the lamella layers of the 3D model in ATENA is shown in Figure 9.

Table 3. The review of 3D FEM models.

Model	Descrip. size of FEM grid mm/mm	Strengthened/ non-strengthened
Model 5(Fig. 4)	F 40/W 40	Non-strengthened
Model 6(Fig. 4)	F 50/W 50	Non-strengthened
Model 7(Fig. 4)	F 100/W 100	Non-strengthened
Model 8(Fig. 5)	F 50/W 50	Strengthened
Model 9(Fig. 5)	F 100/W 100	Strengthened

Caption: F – Flange, W – Web of RC girder

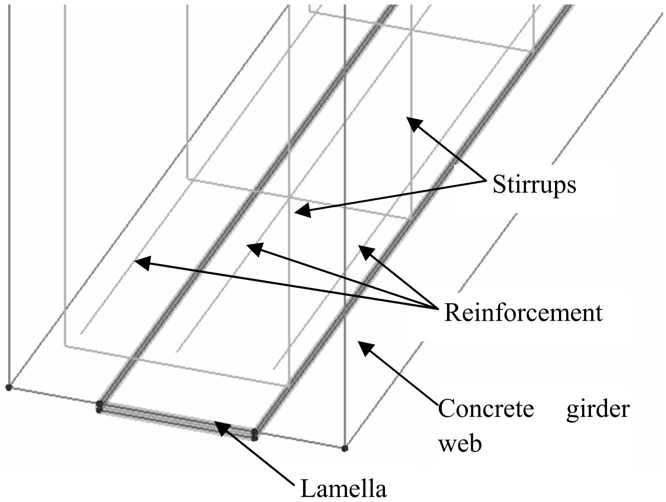


Figure 9. The detail of the strengthened 3D FEM model

8 RESULTS OF THE NUMERICAL SENSITIVITY ANALYSIS

8.1 *Result from numerical 2D FEM analysis*

The analysis was focused on the influence of the externally bonded MBrace® S&P CFK lamella 150 / 2000 on the bending capacity of the RC girder. The value of the additional bending capacity was controlled with the bending crack opening. The value of the deflection in the middle of the RC girder span was the next criterion for the estimation of the strengthening effect. According to the standard (STN P ENV 1992–1–1, 1999), the considered limited deflection is L / 250 which means w_{lim} = 13.88 mm and the limited value of the bending cracks width is ω_{lim} = 0.2 mm. The results from the numerical 2D FEM analysis are shown in Tables 4-5.

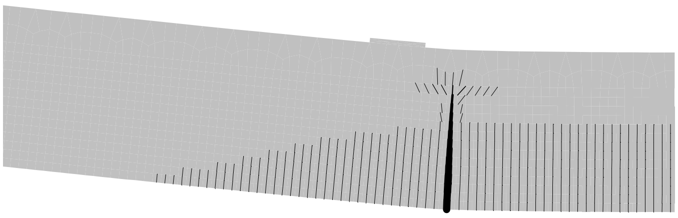
Table 4. Results from numerical 2D FEM analysis.

Model	Bending cracks ω(i) [m]	Deflection in L / 2 w(i) [m]
Model 0	$1.10.10^{-6} < \omega_{lim}$	$1.356.10^{-3} < w_{lim}$
Model 1	$1.96.10^{-4} < \omega_{lim}$	$8.227.10^{-3} < w_{lim}$
Model 2	$1.97.10^{-4} < \omega_{lim}$	$8.235.10^{-3} < w_{lim}$
Model 3	$2.11.10^{-4} > \omega_{lim}$	$8.208.10^{-3} < w_{lim}$
Model 4	$2.67.10^{-4} > \omega_{lim}$	$9.685.10^{-3} < w_{lim}$

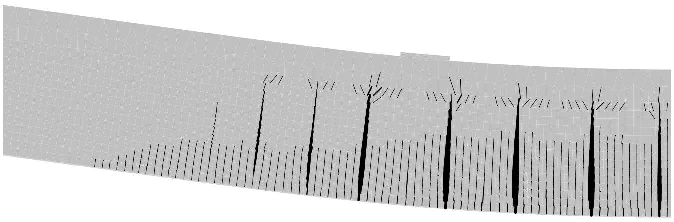
The comparisons between bending crack widths and deflections in L / 2 and their limited values are shown in Table 4.

Table 5. Influence of CFK lamella on bending capacity of RC girder – 2D FEM analysis.

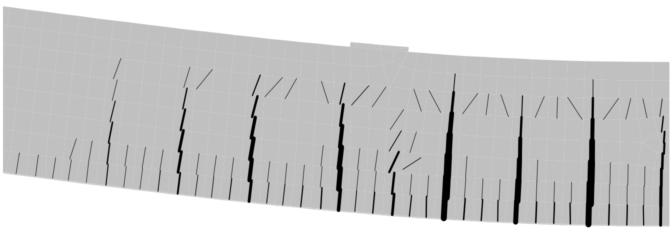
Model	Max. load F_{max} [kN]	Model(i)/Model(0) [-]
Model 0	7.685	1.00
Model 1	11.480	1.49
Model 2	11.490	1.49
Model 3	11.570	1.51
Model 4	12.110	1.58



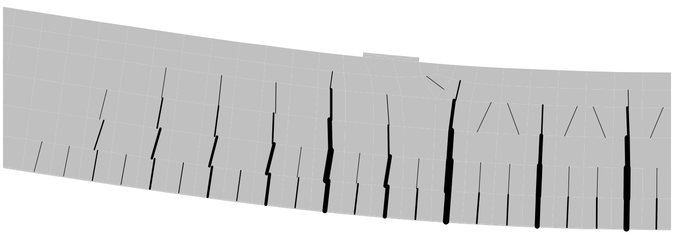
a) Model 0



b) Model 1-2



c) Model 3



d) Model 4

Figure 10. Pattern of bending cracks achieved from 2D FEM models.

The models of the strengthened RC girders indicate the positive effect of additional reinforcement (MBrace® S&P CFK lamella 150 / 2000) on the increasing bending capacity of the non-strengthened RC girders. However, the comparison of experiment measurements and these numerical calculations confirms or refutes the calculated increment of the bending capacity. Figure 11 presents the diagram of the dependency of loading – deflection in L / 2 of RC girders.

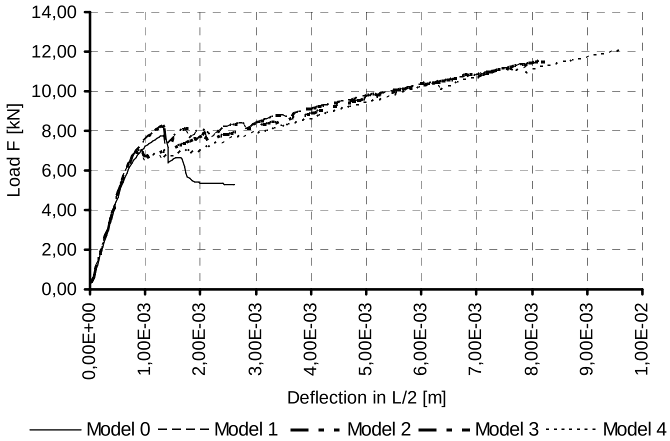


Figure 11. Dependency diagram of loading – deflection in L / 2 of RC girders – 2D FEM models.

8.2 *Result from numerical 3D FEM analysis*

In this 3D FEM analysis, the influence of the FEM grid size on the modeling results was detected not only in the case of the strengthened girder, but also in the case of the non-strengthened girder.

The maximum value of the bending crack width $\omega_{lim} = 0.2$ mm and the limited deflection in the middle of the beam $w_{lim} = 13.88$ mm (L / 250) were used as the criteria for the girder estimation. The results from the numerical sensitivity 3D FEM analysis are shown in Tables 6-7 and in Figures 12-13.

Table 6. Results from numerical 3D FEM analysis.

Model	Bending cracks ω(i) [m]	Deflection in L / 2 w(i) [m]
Model 5	$0.92.10^{-6} < \omega_{lim}$	$0.67.10^{-3} < w_{lim}$
Model 6	$1.76.10^{-5} < \omega_{lim}$	$0.67.10^{-3} < w_{lim}$
Model 7	$0.37.10^{-6} < \omega_{lim}$	$0.56.10^{-3} < w_{lim}$
Model 8	$1.98.10^{-4} > \omega_{lim}$	$5.03.10^{-3} < w_{lim}$
Model 9	$1.97.10^{-4} > \omega_{lim}$	$6.52.10^{-3} < w_{lim}$

Table 7. Influence of CFK lamella to bending capacity of RC girder – 3D FEM analysis.

Model	Max. load F_{max} [kN]	Model(i)/Model(5) [-]
Model 5	5.747	1.00
Model 6	5.288	0.92
Model 7	5.031	0.87
Model 8	8.453	1.47
Model 9	9.043	1.57

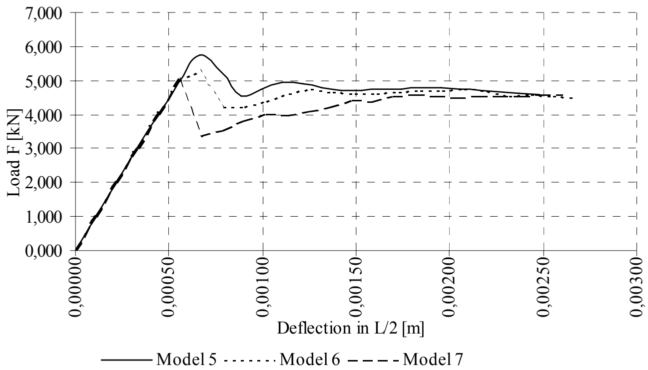


Figure 12. Dependency diagram of load – deflection in L / 2 of RC girders – 3D non-strengthened FEM models.

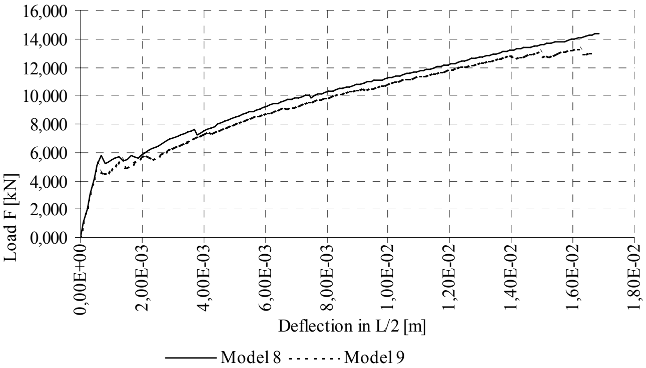
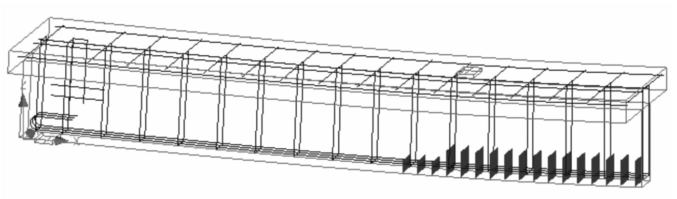
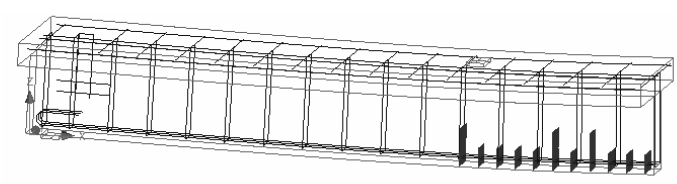
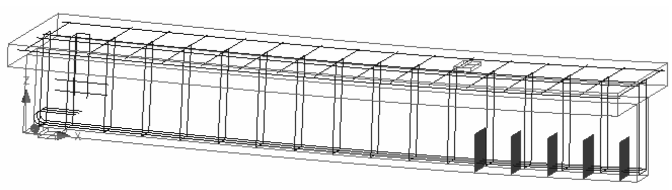
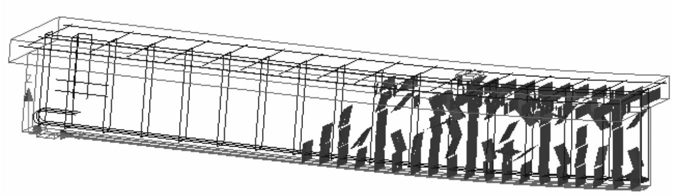


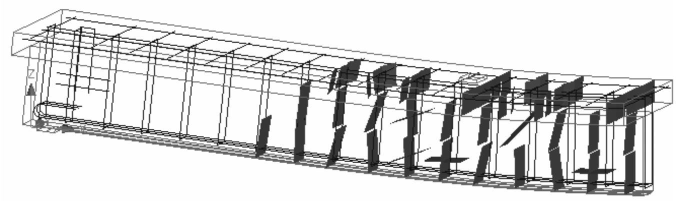
Figure 13. Dependency diagram of loading – deflection in L / 2 of RC girders – 3D strengthened FEM models.


a) Model 5


b) Model 6


c) Model 7


d) Model 8


d) Model 9
Figure 14. Pattern of bending cracks achieved from 3D FEM models.

The models of the strengthened RC girders, like in the 2D FEM models, indicate the positive effect of additional reinforcement (MBrace® S&P CFK lamella 150 / 2000) on the increasing bending capacity of the non-strengthened RC girders. Figure 12 presents the diagram of the dependency of load – deflection in L / 2 of the non-strengthened RC girders (model 5-7) and Figure 13 presents the diagram of the dependency of load – deflection in L / 2 of the strengthened RC girders (model 8-9). In Figure 14, the influence of the element grid size on the crack pattern (position, number, orientation, …) is shown. The results show that the cracks in the ultimate limit state are not only in the web of the beam, but they reach the flange and they are going almost through the whole flange, because of very small compressive part of the concrete cross-section (3 mm).

9 CONCLUSIONS

The values of the resistance bending moments achieved from the simple design procedure (characteristic value), 2D and 3D FEM models are shown in Table 8.

Table 8. Non-strengthened RC girder – comparison of the results.

Model	Max. load F_{max} [kN]	Resistance bending moment $M_{R,0}$ [kNm]
SDC*	3.84	4.48
2D FEM model 0	7.685	8.976
3D FEM model 5	5.747	6.712
3D FEM model 6	5.288	6.176
3D FEM model 7	5.031	6.192

* SDC – simple design procedure, characteristic value

The value of the resistance bending moment calculated by software S&P FRP Lamella using the code approach is the smallest among all values. Higher values achieved from the ATENA are highly influenced by using the model of concrete material, which better or more realistically describes the real girder behaviour. The characteristic parameters of concrete and reinforcement used in the code approach do not include any properties of real concrete and reinforcement, such as for example tension stiffness, softening law, shear retention factor, crack orientation (fixed or rotated), debonding as a failure mode etc. Next, the influence of the FEM element grid size on the achieved value of the resistance bending moment was demonstrated. This influence was also demonstrated in the case of the crack pattern, this means on position, number and orientation of the cracks. It is valid that the value of the resistance bending moment and the maximum load increase with the decrease of element grid size. The number of cracks and the bending crack width are higher.

Next, the 3D FEM modeling gives the results, which are closer to the code approach than the 2D FEM modeling. The resistance bending moment (maximum force) from the 2D FEM model is 2 times higher than the resistance bending moment achieved from the simple design procedure (code approach). In the case of the 3D FEM model, these values are 1.31 – 1.43 times higher.

The results of the strengthened RC girder are shown in Table 9. From the results it can be seen that by using the additional reinforcement (MBrace® S&P CFK lamella 150 / 2000) is increased the bending capacity of the RC girder.

But, the limit degree of the girder strengthening is higher than the value $\eta = 2.0$ neither by using the 2D FEM analysis nor by using the 3D FEM analysis (Tabs 5, 7).

Table 9. Strengthened RC girder – comparison of the results.

Model	Max. load F_{max} [kN]	Resistance bending moment $M_{R,0}$ [kNm]
SDC*	6.64	7.76
2D FEM model 1	11.480	13.409
2D FEM model 2	11.490	13.420
2D FEM model 3	11.570	13.514
2D FEM model 4	12.110	14.144
3D FEM model 8	8.453	9.873
3D FEM model 9	9.043	10.562

* SDC – simple design procedure, characteristic value

From the results in Table 4 and Table 6 it is seen, that the deflections in L / 2 of the non-strengthened RC girders made in the 2D and the 3D FEM model are smaller than the deflections in L / 2 of the strengthened RC girders. But, these deflections cannot be compared, because they were achieved at the various maximum loads F_{max} (Tab. 5 and tab.7). As a matter of fact, the deflections of the strengthened RC girders are smaller than the deflections of the non-strengthened RC girders if the same maximum loads F_{max} are considered.

Once again, the 3D FEM modeling of the strengthened RC girder gives the results, which are closer to the code approach than the 2D FEM modeling. In this case, the resistance bending moments (maximum force) from the 2D FEM model are 1.73 – 1.82 times higher than the resistance bending moment achieved from the simple design procedure (code approach). In the case of the 3D FEM model, these values are 1.27 – 1.36 times higher.

It is necessary to point out that the maximum resistance bending moment M_{Rd} and the maximum load force F_{max} obtained from the ATENA 2D and 3D FEM models are the values corresponding to the limited values. However, the limited values do not correspond to the Ultimate limit state (ULS), but they correspond to the Serviceability limit state (SLS). This means, that the girder does not collapse due to overloading (collapse of the concrete or failure of the lamella or reinforcement), but the girder limit state is defined by exceeding the limited bending crack width.

From the mentioned reasons, the debonding do not appear in the 2D or 3D FEM models because the limited bending crack width was achieved before the debonding in models appears. Also, the debonding is not implemented in software S&P FRP Lamella (S&P FRP Lamella).

The real models of the girders are being prepared at this time and will be tested in the future. So, the achieved results from the ATENA analysis will be compared with the results achieved from the real girder testing. The comparison of the results will be presented as soon as possible. Moreover, the reliability-based solution of the strengthened RC girder is prepared (Omishore & Kala 2006).

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