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Research Article

FINITE ELEMENT ANALYSIS OF THE MECHANICAL BEHAVIOR OF REINFORCED CONCRETE (RC) BEAMS STRENGTHENED BY FIBER REINFORCED POLYMERS (FRP)

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ABSTRACT

The reason for the extensive use of fiber reinforced polymers (FRP) as a reinforcement material in construction elements is due to its advantageous mechanical properties. The mechanisms underlying the failure of FRP reinforced concrete beams is still a phenomenon in question for researchers. The best means of determining the mechanical behavior of a construction element is usually by laboratory testing. But this process takes a lot of time and is costly. Only a limited number of specimens with certain sizes can be tested and decent experimental equipment and samples may not be easily available. The aim of this study is to develop a finite element model for simulating the behavior of FRP strengthened reinforced concrete (RC) beams reducing the need for experimentation. For this reason, in this study, test results carried out by the Swiss Federal Material Testing and Research Laboratory on FRP strengthened RC beams [7] were compared with results obtained by computer simulation using finite element method. The regions of damage occurrence obtained from the analysis were evaluated and the comparison of the failure load of the simulation with that of the test was made with the load-displacement graphs. The failure loads obtained by the simulations were in good agreement with the test data. Therefore finite element simulations can be used as a tool for predicting the mechanical behavior of FRP strengthened RC beams.
Keywords: FRP, de-bonding behavior, fracture, finite element method.

1. INTRODUCTION

The FRP material has many advantageous mechanical properties. These include high tensile strength, high strength-to-weight ratio, fatigue resistance, non-magnetic electrical insulation, and small creep deformation and corrosion resistance. In addition, the light weight and ease of application of the FRP lead to shortening of the rehabilitation period and the reduction of labor, thus compensating for the high costs. The realistic behavior of a construction element can usually be determined best by laboratory testing. But this process takes a lot of time and is costly. Besides it is not always easy to carry out laboratory tests due to difficulties in the availability of appropriate equipment and material procurement. Furthermore tests can be conducted on only a

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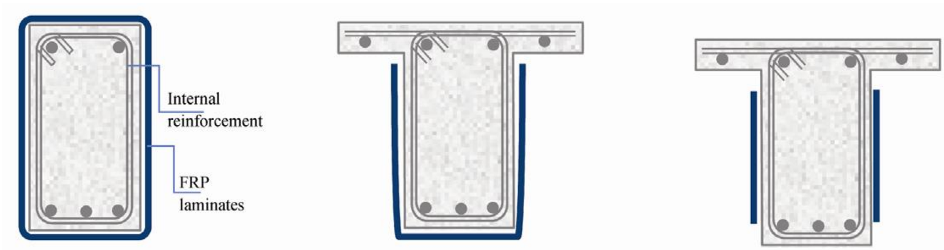
limited number of specimens with certain sizes, geometries and materials. However, with data from previous experiences (earthquake and similar situations) and tests, the behavior of the material, construction element and structure can be modeled in the computer environment with sufficient accuracy by the finite element method. These computer simulations can be carried out for a wide range of geometry and material combinations. In the finite element model material properties, sections, elements, interactions between the components and boundary conditions should be defined properly. Otherwise, the results obt ained from the computer model will not accurately reflect the real behavior.

Three typical FRP wrap forms are seen to be common in the literature (Figure 1). The first is the complete wrapping of the construction elements. This type of wrapping is known to be the most efficient way of FRP shear reinforcement. However, due to geometric restrictions the complete wrapping of construction elements, beams in particular may not always be possible. In this case, either the FRP U-wrapping scheme (wrapping on three sides) or the side bonding (two separate FRP sheets on opposite sides of the beam) scheme is used.



a) **b)** **c)**

Figure 1. Three typical FRP wrapping forms; a) complete wrapping, b) U-wrapping, c) side bonding [2]

The kind of FRP wrapping has an important effect on the failure mode of the reinforced element. Experimental studies show that de-bonding behavior is more predominant in wrapping schemes where the FRP is adhered to one side of the element compared with U-wrapping and complete wrapping schemes (Figure 2).

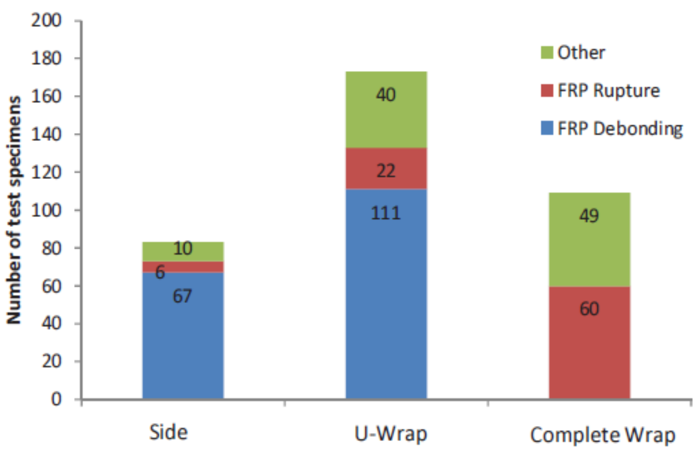


Figure 2. Distribution of failure modes according to the kind of FRP wrapping [2]

For the effective operation of the FRP reinforcement, a sufficient bond should be maintained between reinforced concrete and externally adhered FRP throughout the service life of the reinforced structure. This bond is expressed by the resistance of the FRP-concrete interface to the interface shear stresses in this area. The FRP-concrete interface plays an important role in maintaining the integrity and strength of the composite structure and in transferring stresses effectively from concrete to FRP material. De-bonding in the FRP-concrete interface may lead to early failure. In this way, the relative displacement between the FRP plate and the concrete beam is accumulated in this interface layer.

Experimental studies show that reinforced beams often fail suddenly with brittle behaviour due to the de-bonding between the FRP and the concrete interface, and therefore the full strength of the reinforcing zone cannot be used. The American Concrete Institute (ACI) design regulation, suggested the crushing of the concrete (before the steel yield) before the FRP de-bonding as the most acceptable failure mode.

In recent years, many researchers have conducted experimental studies to investigate the failure mechanisms of concrete structures reinforced with FRP laminates. If the failure modes are examined in detail; FRP rupture (Figure 3a): It is a mode of bending failure that occurs as FRP rupture after the steel reinforcement yield. Concrete crushing (Figure 3b): It is the state of crushing of the concrete in the pressure zone before or after the steel reinforcement yield. In this failure mode, the FRP remains intact. Concrete shear failure (Figure 3c): Generally, in concrete elements externally reinforced with FRP, FRP plates do not extend across the entire opening of the beam. As a result, there is a zone of stress intensity at the ends of the plate. This leads to cracks in the vertical edges of the plate ends, which lead to the propagation of the shearing cracks in the concrete body resulting in a sudden failure of the concrete. Concrete cover separation (Figure 3d): In this mode, the entire coating of the concrete separates as a result of crack formation at or near the end of plate, due to the intensity of the shear stresses. The cracks start first at the point close to the plate end in the concrete and then propagate to the level of the tensile reinforcement. When this level is reached, it moves horizontally along the bottom of the reinforcement. As the load increases, the horizontal cracks cause the concrete cover to be peeled and eventually the structure to fail. Plate end interfacial de-bonding (Figure 3e): This failure is due the concrete surface close to the concrete-adhesive interface due to high interfacial normal and shear stresses. When these stresses exceed the strength of the concrete near the end of the plate, it starts cracking and begins to move towards the middle zone. Mid-span de-bonding caused by flexural cracks (Figure 3f): This mode is initiated by the crack propagation near the concrete in parallel to the FRP plate and adjacent to the concrete-adhesive interface. The first de-bonding is caused by bending cracks in the central opening zone. And this de-bonding is propagated to one of the ends of the beam and consequently the failure occurs. Mid-span crack de-bonding caused by flexural/shear cracks (Figure 3g): Similar to the previous mode mechanism, the de-bonding of the FRP plate is triggered by the flexural/shear crack. Such cracks start as bending fractures away from the center of the beam. Initially, they grow vertically to a small part of the concrete cover and then change to become inclined. The de-bonding starts at the center of the opening and moves towards the ends of the plate, causing the entire structure to break in a brittle manner. Some concrete remains may be present on the stripped plate after the failure.

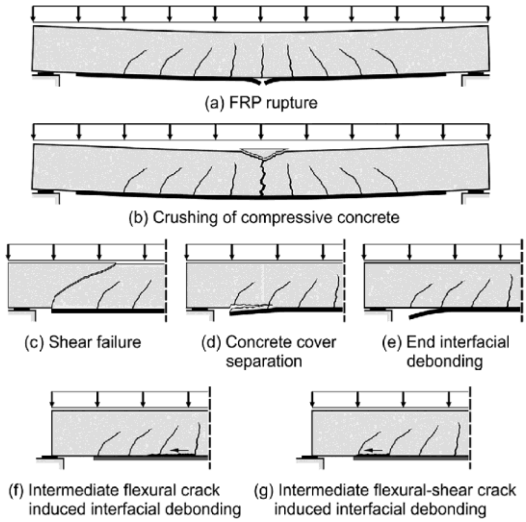


Figure 3. Failure modes in beam retrofitted for flexural strengthening [18]

2. LITERATURE REVIEW

There are many analytical, numerical and experimental studies on FRP strengthened concrete beams in literature some of which are mentioned in this section. Triantafillou & Plevris (1991) proposed an equation to calculate the oscillation rate of the critical unit deformation energy for the interface. In the same study, the final value of the interface stress was calculated where the separation of the FRP plate started from the anchor zone. This amount is determined as a function of concrete compressive strength. Triantafillou and Deskovic (1992) observed that the main reason for the failure in the samples examined was crack propagation near the FRP-concrete interface. Therefore, it was proposed that the concrete should not contain loose particles in order to prevent any fracture. Yoshizawa et al. (1996) investigated the effect of different bonding conditions on the bonding capacity between concrete and CFRP layers. These different bonding conditions are the surface preparation, the type of the CFRP plate and the peel area. In their study, they observed that the bond strength increased with the CFRP having high rigidity and tensile strength. Furthermore, increasing the number of CFRP layers helped improve bonding capacity in the interface zone. The effect of surface preparation on the interface bond strength of the FRP plates was further investigated by Chajes and Finch Jr. (1996). From the test results, the mechanical wear of the concrete surface was considered to be the most efficient method to maintain high bond strength. The authors also suggested that in the case of a failure in the concrete-adhesive layer caused by cracking, the final bond strength varies in proportion of the square root of the compressive strength of the concrete. Taljsten (1997) conducted a tensile test on concrete prisms externally reinforced by steel or CFRP layers. The researcher defined the anchorage length as the minimum length of steel or FRP plate which contributed to the final load capacity and beyond that no real effect on the final bond strength was observed. A comprehensive parametric study was conducted by Arduini and Nanni (1997) to investigate the effect of different FRP parameters (such as bonding length, thickness and FRP and adhesive rigidity) in the case of FRP application as a reinforcement system in reinforced concrete beams. Malek et al. (1998) proposed a closed-form analytical model to estimate the distribution of normal and shear stress locations at the plate end. It is assumed that the material behavior in this model is linear elastic and has an isotropic structure. It was assumed that there is a perfect bond between FRP and concrete and that there is no shearing. The segregated crack approach was adapted to model the crack state in the concrete. The results from this model were then verified by using finite element analysis with the help of ABAQUS software. Grace et al. (1999) investigated the behaviors of concrete beams reinforced with different types of FRP laminates. The 14 beams in question have a simple bearing, rectangular cross-section, loaded with a load over the cracking load limit. Then the damaged beams were reinforced with CFRP or GFRP laminates and their final capacities were tested. In this study, the effect of the FRP reinforcement on the failure load and mode, the ductility and deformation of the beam were discussed. The authors also examined the effect of some FRP parameters, such as the number of FRP layers, epoxy type and reinforcement configuration, on the concrete capacity. It was observed that the combination of longitudinal and U-shaped FRP reduced beam deflection and increased load capacity. Ross et al. (1999), in the study, the beams on which the three-layer uniaxial CFRP laminates were bonded to the longitudinal edges were tested. The authors used a inelastic cross-section analysis procedure to estimate beam behavior and correlated the results with experimental data from the tests. Bizindavyi and Neale (1999) examined the load transmission mechanism of FRP laminates bonded to the concrete surface. The researchers conducted several pull-out tests in which various parameters were investigated. The type, thickness and geometric properties of FRP were evaluated and different types of concrete were included in the tests. The study also presented an empirical model to estimate the final load. Surface preparation was stated to be the dominant factor affecting the behavior of the bond. Mukhopadhyaka and Swamy (2001) reviewed some of the existing models to estimate the de-bonding behavior of the plate ends of FRP laminates in RC beams. There was a need to develop a more rational model for estimating the plate end peel mechanism of FRP composites, as the existing models were inadequate. The majority of previous research focuses on estimating shear and normal stress concentrations near the FRP plate end, whiles the study, presents the concept of interface shear stresses to predict plate shearing failure. According to the researchers' conclusions, the de-bonding occurs when the shear stress in the interface between the concrete and FRP plates reaches a certain limit. Maalej and Bian (2001) applied a experimental program to investigate the shear stress concentrations in the interface between the concrete and FRP plate. By conducting experiments on 5 beams with different FRP layers, the authors investigated the effect of the number of layers on the interface shear stress. As a result of their studies, they suggested modelling interfacial normal and shear stress concentration as a displacement function to the place of load. Another study was carried out by Ngujen et al. (2001) to estimate the load of the CFRP plate, which causes the failure of the concrete cover segregation. The authors analytically examined the composite motion between the RC beam and the FRP plate. Sebastian (2001) examined a wide range of data from an experimental program at the University of Bristol in England. A thin lubricated steel plate crack cracker is used in the middle opening. High interface shear stresses have been caused by stripping FRP plates. These stresses are attributed to the rigidity of the cracked concrete stress and to the corrosion of the internal steel reinforcement. The study by Leung (2001) was initially carried out to determine the relationship between crack opening width and bond stress. An equilibrium based on fracture mechanics including crack length, width and momentum was developed. This analytical model refers to the relationship between the final interface shear stress, (τmax) and the applied moment with respect to different materials and geometrical properties. The model was confirmed by the data obtained from the finite element analysis, and the results showed that the peel of FRP plate is more likely when there is a large crack gap, low adhesive thickness, low plate rigidity, and small contact area between plate and adhesive. Finite element analysis was performed by Yang et al. (2003) based on the segregated fracture simulation of concrete. The multiple segregated crack propagation during loading was simulated using a mixed-mode linear elastic fracture mechanics program. Niu and Wu (2005) examined the effect of FRP-reinforced RC beams on the peel behavior of multiple bending cracks. The authors investigated this peel mechanism by examining the effect of crack gap, local bonding capacity, interfacial rigidity and interfacial fracture energy. In the study, a finite element model based on nonlinear fracture mechanism was developed. Kishi et al. (2005) developed a three-dimensional elasto-plastic model. This numerical model, under the effect of diagonal bending-shear cracks, aims to examine the peel behavior of FRP layers externally bonded to RC beams. The interface elements are used to model crack openings, reinforcement displacement, and peel of the FRP plate. Ye and Yang (2005) developed a formulation for the determination of interface stresses in FRP reinforced concrete beams exposed to loading. They were reinforced with FRP, and they offered an elastic solution for the simply supported reinforced concrete beam exposed to the force pair. Wang (2007) analyzed the peel in the FRP-concrete interface under mixed-mode loading using an adhesive zone model. He used a nonlinear bond-shear model to examine the shear stress segregation state of the FRP-concrete interface. The closed form solutions of FRP stresses and interface stresses were found for a uniform sample for the entire peeling process. The present model can be used to effectively analyze the mixed-mode peel of the FRP-concrete interface. In their study, Toutanji et al. (2013) attempted to estimate the fracture mechanics-based model of the peel behavior of FRP in reinforced concrete beam supported with FRP material. They expressed the maximum transferable load as a function of material property and fracture energy. They determined the fracture energy for the FRP tensile test by the maximum interface shear stress and the corresponding shear. Colombi et al. (2014) evaluated wrapping and strip performances of different lengths. The de-bonding load was theoretically evaluated on the basis of fracture energy. Separate calibrations were made for windings and strips by two different statistical models to evaluate the effect of the reinforcement type and model assumption on the bonding load. Colombi et al. focused on the end peel failure in their study. Wu and Pan (2014) examined the effect of different parameters such as the rigidity of FRP plate, interface fracture energy and interface shear strength on effective bond length. They developed a simple analytical model for bond strength in case that the bond length was smaller than the effective bond length. They produced closed form solutions for tensile stresses, interface shear stresses and displacements along the bonded plate with the model they created. Chellapandian et al. (2017) in their study developed finite element models to predict the behavior of concrete columns strengthened by using various techniques under axial and eccentric loads. In their model they took into account the cohesive zone between FRP fabric and concrete.

3. FINITE ELEMENT ANALYSIS

Although experimental research constitutes the majority of data gathered on the behavior of FRP-reinforced RC elements, the fact that large-scale samples are too expensive and time-consuming to test is one of the disadvantages of experimental research. Analytical methods can only be used to model simple structures because of the difficulty in deriving boundary conditions and geometrical restrictions. In addition, it was found that most of the analytical methods used to model the interface relationship between FRP and concrete were not reliable. Therefore, many complex engineering problems cannot be solved analytically and more practical analyzes are often performed using numerical approaches.

One of the most widely used numerical methods is the finite element method. This method is based on the differential equation representing the physics of the problem. But the application of this method in finite element software begins by creating the geometrical model of the system. After this step the geometry is divided into discrete elements in a procedure called meshing. Then the material properties are assigned to the elements. Finally for a structural analysis the deformation of the structure is solved for the given loads and boundary conditions. The results obtained from this numerical modeling are compared with the actual data and the values obtained are verified. After reviewing the results of the analysis, the numerical model may need to be improved and the system is re-analyzed for the same loads. The process is continued until the values obtained as a result of the model analysis converge to the test data.

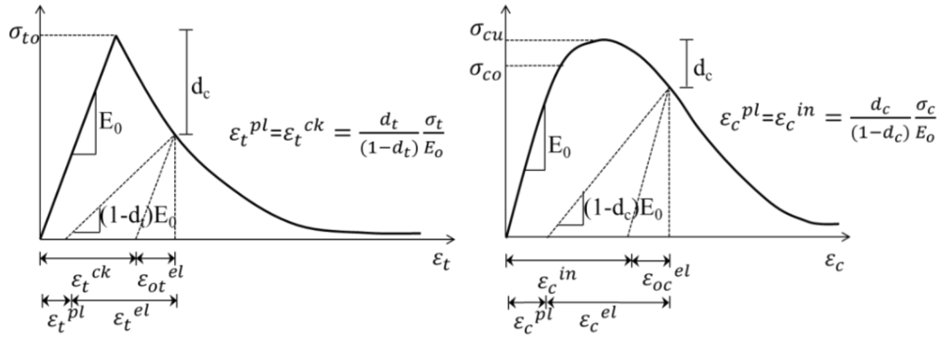
The finite element method includes the selection of elements (lattice, beam, 2D or 3D continuous), meshing and boundary conditions. The combined solutions of all elements form the solution of the whole part. This method is used to analyze structures with complex properties and various material properties. It also provides an effective analytical tool to study the structural behavior of reinforced concrete elements. Other mechanisms that are difficult to handle such as cracking, tensile rigidity, nonlinear material properties, interface behavior can be modeled rationally using the finite element method. There are many finite element software that can be used for the finite element analysis of reinforced concrete elements: ABAQUS software was used in this study for numerical analysis, due to its wide choice of materials and elements and its ability to model one, two and three dimensional projects.

4. LINEAR AND NONLINEAR MATERIAL MODELS

The data of the experiment carried out by the Swiss Federal Material Testing and Research Laboratory (1996) under the Swiss CFRP producer Stesalit AG were used. The analysis includes three rectangular and simple bearing beams, one of which is the control beam. Beams are generally composed of four types of materials: concrete, steel, FRP and adhesive. All the beams have the same geometry. The control beam named as B1 is not strengthened with FRP whereas the other two beams named as B2 and B3 are strengthened by FRP with different mechanical properties .

4.1. Concrete Material Model

In the recent numerical studies of the concrete materials, including this study, plasticity and damage development of the concrete parts are taken into consideration in the basic finite element modeling. The concrete damage plasticity model uses the concepts of isotropic stress and compression plasticity to represent the inelastic behavior of the concrete. These concepts reflect the assumption of the two mechanisms of failure. They are tension cracking and pressure crushing of concrete composites. The hardening variables correspond to the size of the damage in the concrete and the rigidity reduction parameter is used to characterize the uniaxial stress and pressure stress-deformation relationships under the applied loads. Concrete damage plasticity material model characterizes the axial tensile and compressive behavior of concrete as shown in Fig. 4. Stress-strain relationship is linear until the fracture strength (σ_{to}) is reached under axial tensile condition. When fracture strength is reached, the formation of micro cracks starts. Beyond this stress, strain-stress curve tends to decline (Fig. 4a). Under axial pressure, stress-strain curve exhibits a linear behavior until the initial yield stress (σ_{co}). After reaching the maximum stress (σ_{cu}) concrete behavior is characterized with strain hardening followed with strain softening (Fig. 4b). The damage in terms of tension (σ_t) and compression (σ_c) are defined via the below mathematical relationship.



a) Uniaxial tension b) Uniaxial compression

Figure 4. Concrete damage plasticity model stress-strain diagrams [9]

The main two failure mechanisms are tensile cracking and compressive crushing of the concrete material in the CDP model. The evolution of the yield (or failure) surface is controlled by two hardening variables linked to failure mechanisms under tension and compression loading, respectively. σ_{t0}, σ_{cu}, ε_t^{pl}, ε_c^{pl}, E_0, d_t and d_c in Fig. 4 refer the value of initial yield, the ultimate stress, tensile plastic strains, compressive plastic strains, the initial (undamaged) modulus of the material, the stiffness degradation variable in tension, the stiffness degradation variable in compressive, respectively [9].

4.2. Steel Material Model

It is an accepted approach in numerical analysis that the steel exhibits elastic-excellent plastic behavior (Figure 5). According to this approach, steel shows linear elastic behavior until it reaches yield strength (f_t), and after that, it exhibits plastic behavior. So the deformation continues without any increase of strength.

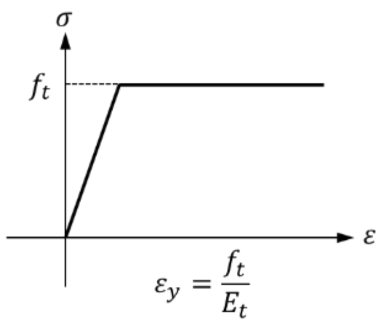


Figure 5. Stress-strain relationship for steel

4.3. FRP Material Model:

FRP material is considered to have linear elastic behavior. There is a linear increase between FRP strength and unit deformation (Figure 6).

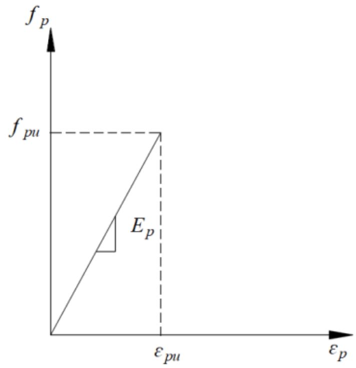


Figure 6. Stress-strain relationship for FRP [9]

4.4. FRP-Concrete Interface Model:

The FRP-concrete interface plays an important role in transferring stress from concrete to FRP. The cohesive behavior model was used to simulate the interface in the numerical analysis. The cohesive model describes the segregation surface and describes the FRP-concrete interaction by defining relative displacement at each contact point. The main parameters required to define the model are initial rigidity (K_0), shear stress and fracture energy (Figure 7). The parameters used in this study were calculated using Equation (3-5)

$$K_0 = 0.16 \frac{G_a}{t_a} + 0.47 \tag{3}$$

$$\tau_{max} = 1.46\, G_a^{0.165} f_{ct}^{1.033} \tag{4}$$

$$G_{fv} = 0.52\, f_{ct}^{0.26}\, G_a^{-0.23} \tag{5}$$

Here, ta is the thickness of the adhesive, Ga is the module of the adhesive, (GPa), f_{ct} is the tensile strength of the concrete (MPa).

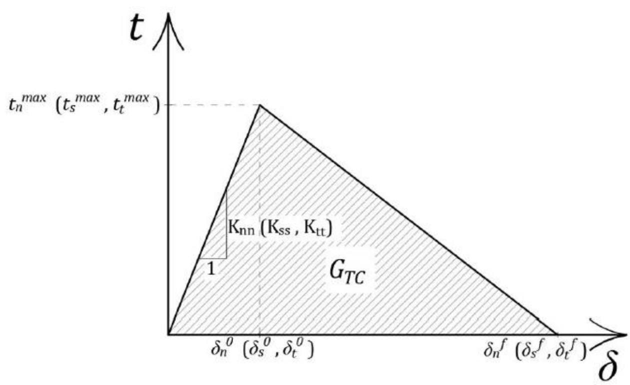


Figure 7. Behavior of concrete and FRP interfaces in tension and shear [12]

5. FINITE ELEMENT MODELING OF BEAMS:

The control beam is a simple bearing beam with a rectangular cross section with dimensions of 250×150×2400 mm. The cover width is 20 mm. There is a distance of 2000 mm between the bearings and the beam is subjected to a four-point loading test. There is a distance of 667 mm between the loading points. 3 pieces of 8 mm reinforcement were used as tension and pressure reinforcement. There is a distance of 150 mm between the binders and their diameter is 6 mm (Figure 8).

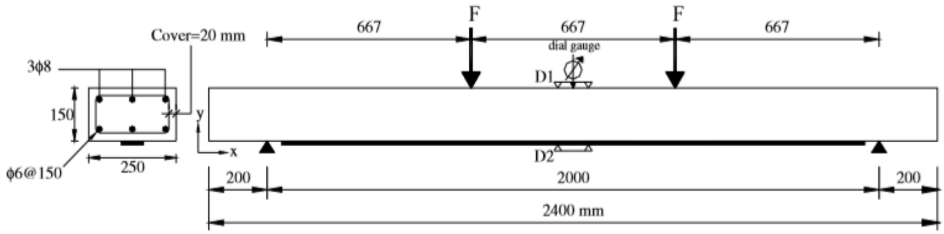
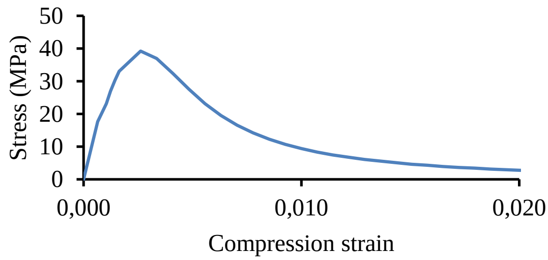


Figure 8. Geometric properties and cross-section of tested beams [7]

The compressive strength of the concrete used in the experiment is 39.2 MPa. The elasticity module of concrete was taken as E= $4700\sqrt{fc}$ =29426.65 MPa and poison's ratio was taken as υ=0.18. The behavior curves under pressure and tensile force are as in Figure 9. The elasticity module of the steel was taken as 200000 MPa, yield strength as 485 MPa, poison's ratio as 0.3. The FRP material was therefore not used control beam B1. Reinforced beam B2 had a FRP elasticity module of 305 GPa, tensile strength of 1300 MPa, while beam B3 had a FRP elasticity module of 214 GPa, tensile strength of 2000 MPa. The poison's ratio was 0.3 for both FRP materials. The strengthening of the beam is also used FRP material of 1.2 mm thickness and 50 mm width. The elasticity module of adhesive material was taken as 12800 Mpa.



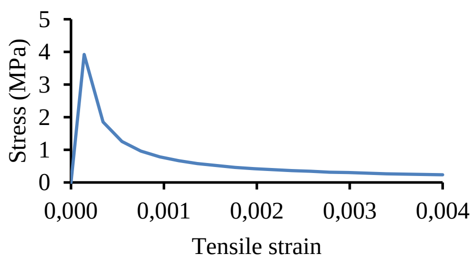


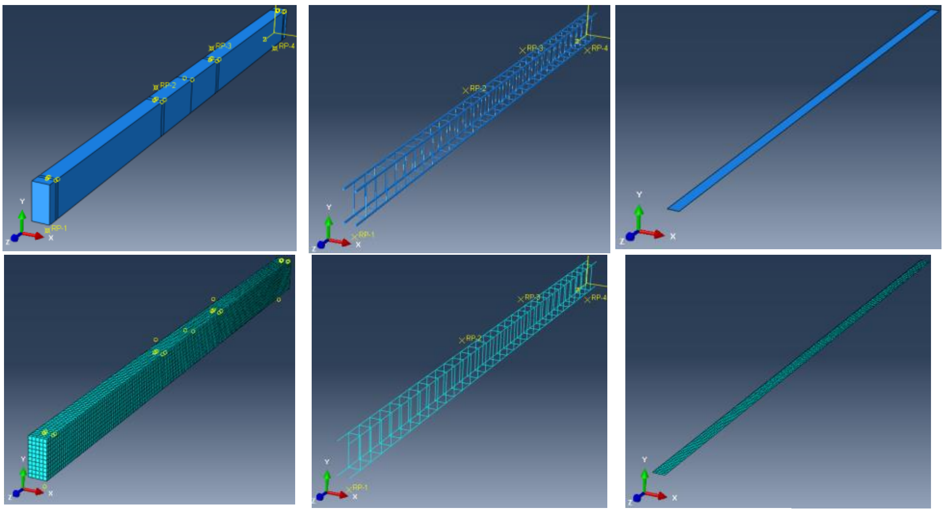
Figure 9. Behavior of the concrete strength under compression and tensile

CDP model was used to determine the nonlinear behavior of concrete material. The CDP model uses stress-strain curves to express nonlinear behavior of concrete under axial tension and head behavior. In the CDP model, four fundamental material parameters such as dilatation angle, eccentricity, σ_{bo}/σ_{c0}, K_c should be defined to accurately simulate the non-linear behavior in addition to the definition of stress-strain relationship. These parameters are given in Table 1 [9].

Table 1. Parameters of concrete damaged plasticity [9].

Dilatation angle	Eccentricity	σ_{bo}/σ_{c0}	Kc	Viscosity
35	0.1	1.16	0.667	0.0001

Concrete is modeled using continuous elements. In addition, such elements are used in the case of plasticity properties and large deformations. C3D8R element was used to model the concrete beams in the finite element model. Steel reinforcements are modeled using truss elements. The truss elements are thin structural elements that can only transmit axial forces and do not transmit moments or transverse loads. Finite element model of steel reinforcement is used T2D2 elements. FRP laminate is modeled as a shell element (Figure 10).



(a) (b) (c)

Figure 10. Finite element models, a) Concrete part, b) steel reinforcement system, c) FRP

Considering that there is a perfect bonding at the concrete-steel interfaces, steel reinforcement was modeled so as to be embedded in concrete. This is achieved within Abaqus by means of the embedded region constraint which basically means there is no relative motion between steel and concrete at the interfaces. A cohesive contact property was assigned for the contact region between the FRP laminate and the concrete. The cohesive contact property used in the model allows debonding of the laminate if a certain stress state is exceeded. By applying the boundary conditions so as to replicate the test conditions, simulation results were obtained. Since the tests involve slow application of loads static finite element analyses were done. Simulations were carried out for all three beams. The failure load values of the beams obtained from the finite element analyses and the tests are given in Table 2.

Table 2. Comparison of analysis results and experiment results

Beam no	Experimental failure load (kN)	FEM failure load (kN)
Control beam	15.6	15.9
Beam B2	30	31.2
Beam B3	31.4	35.7

The control beam has reached failure, after the reinforcement yields, by crushing the concrete with pressure. FRP material ruptured in B2 beam and FRP has de-bonded in B3 beam due to cracks occurring in middle beam of beam. The elements in the concrete where damage occurs (red elements) at the B2 and B3 beams according to the simulations are shown in Figures 11-12.

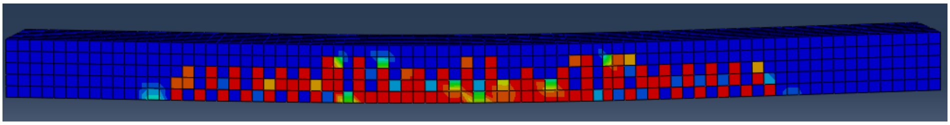


Figure 11. Damage in B2 beam

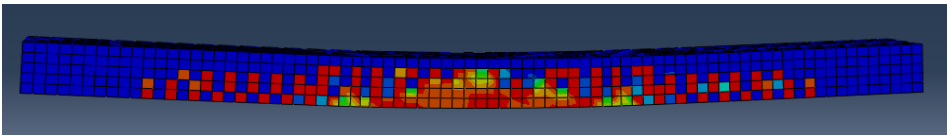


Figure 12. Damage in B3 beam

Load-displacement graphics obtained by the numerical analyses and tests are given in Figures (13-15)

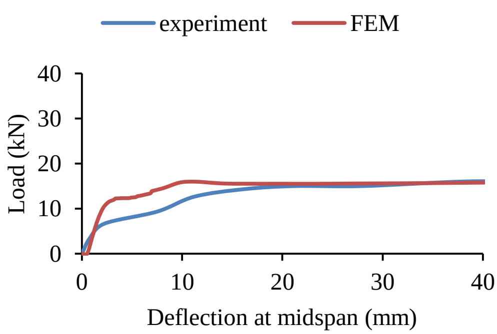


Figure 13. Load-deflection diagram of control beam B1

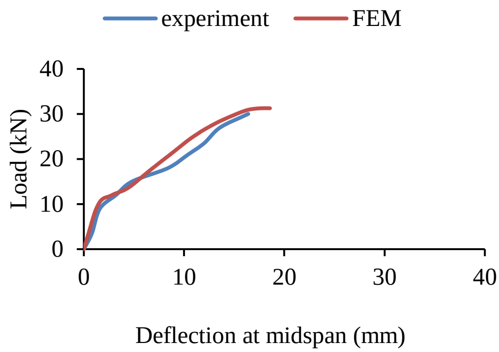


Figure 14. Load-deflection diagram of B2 beam

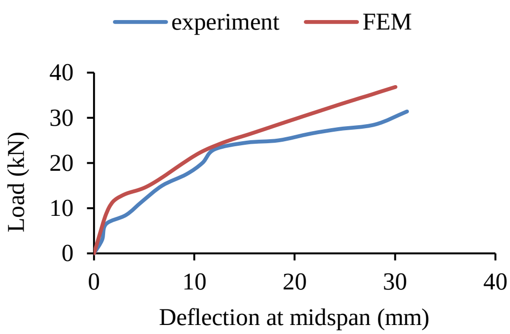


Figure 15. Load-deflection diagram of B3 beam

6. CONCLUSIONS

The aim of this study is to develop a finite element model for simulating the behavior of FRP strengthened RC beams reducing the need for experimentation. For this reason, results of tests carried out by the Swiss Federal Material Testing and Research Laboratory on FRP strengthened RC beams [7] were compared with results obtained by computer simulation of these tests using ABAQUS program. The regions of damage occurrence obtained from the analysis were evaluated and the comparison of the failure load of the simulation with that of the test was made with the load-displacement graphs.

The results of this study can be summarized as follows:

1. The load-displacement curves obtained from the finite element model analysis of three beams, one of which was control beam, the other two were strengthened with beams with different elasticity modulus and tensile strength, showed good agreement with experiment results. This means that the modeling is successful.

2. The load capacity obtained from the finite element model is higher than the load obtained from the experimental study due to the model beam is more rigid. In computer modeling, it is assumed that the material properties obtained from experiments are homogeneous throughout the structure. But in reality there’s always a deviation in the material behavior from idealized material behavior due to imperfections such as voids that occur during the manufacturing and cannot be accounted by for the mathematical model. Generally these imperfections cause a reduction in the strength of the real structure when compared with the strength of the idealized mathematical model such as the FEM.

3. Concrete is a heterogeneous and anisotropic material. But since it is very difficult to build a more realistic material model a homogeneous and isotropic material definition was chosen, which is one of the reasons of the small discrepancy between test results and simulation results.

4. The definition of boundary conditions can also lead to a backlash between the computer model and the test results. In the computer model, support definitions are applied perfectly, without errors. On the other hand, when the test setup is created, the support conditions of the beam cannot be exactly matched to the assumptions. It is inevitable that there is a discrepancy between the test beam and the model beam in terms of the bearing conditions and this will lead to a difference in the results.

5. The modulus of elasticity of FRP material used in B2 beam is higher than the modulus of elasticity of FRP used in B3 beam. This explains the reason why the B3 beam is more tolerant than the B2 beam when it reaches the failure load. The high modulus of elasticity resulted in the strengthening of the reinforced beam by rupturing the FRP material.

6. The higher tensile strength of FRP used in B3 beam than used in beam B2 increased the magnitude of the B3 beam failure load. In other words, it has had a positive effect on the strength of the beam.

7. When modeling B2 and B3 beams, 5 and 10 MPa were used as well as 7 MPa for the shear stress of the interface in order to see the effect of bonding strength on the load bearing capacity. It was observed that the use of different values did not cause a change in the load-displacement relationship (Figure 16-17).

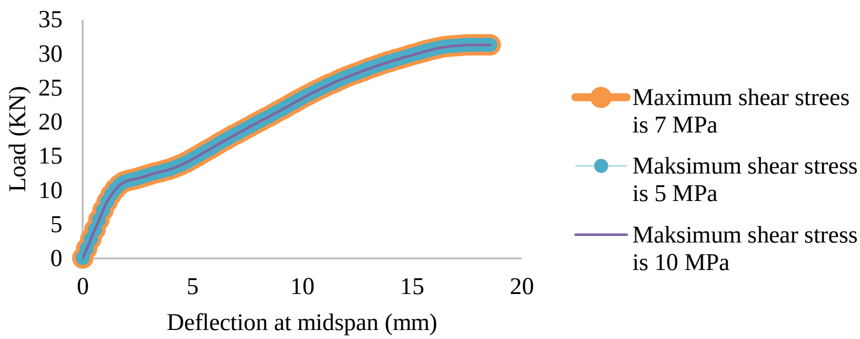


Figure 16. The effect of interface maximums shear stress τ_{max} value to load-deflection relationship in B2 beam

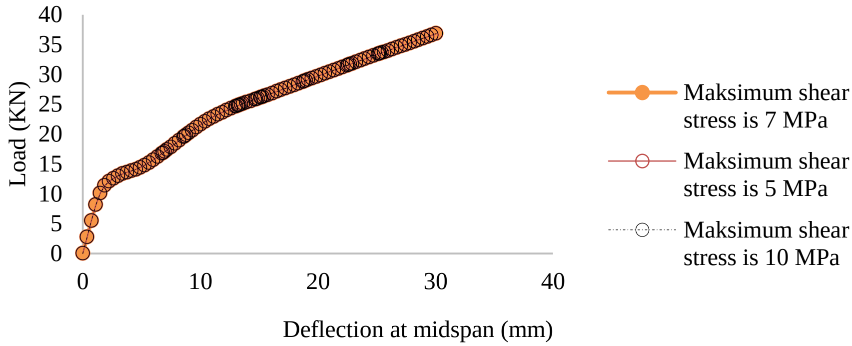


Figure 17. The effect of interface maximum shear stress τ_{max} value to load-deflection relationship in B3 beam

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