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NUMERICAL MODEL FOR BIAXIAL LOADING OF REINFORCED CONCRETE STRENGTHENED WITH FIBER REINFORCED POLYMERS

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Abstract

One of the mostly used primary earthquake resistant mechanism in structural design is the use of shear walls that are to provide the earthquake resistance of the structure. Therefore, there is a sizable stock of older structures with shear wall that are deficient by the contemporary standards and codes. This fact leads to the obvious necessity of their strengthening and retrofit. This in itself lead to the development of many different methods of seismic strengthening and repair of shear wall structures. One of the latest and popular strengthening and retrofit techniques involves utilization of externally bonded fiber reinforced polymer (FRP) composites, which offer unique properties in terms of strength, lightness, chemical resistance, and ease of application. These techniques are especially attractive for their fast execution and low labor costs.

The extended use of these strengthening and retrofitting techniques introduced the need for efficient and accurate numerical modeling of the behavior of the FRP composites when used as strengthening and retrofitting material. Most widely used method for the simulation of the behavior of reinforced concrete strengthened with FRP composites is undouble the finite element method. Traditionally, most of the studies that include numerical modelling of FRP strengthened RC members with FEM use element overlaying, where one-, two- or even three-dimensional elements (solid or layered) that represent the FRP material are superimposed over the concrete elements, either with or without interface elements that represent the influence of the adhesive material or the bond between the FRP and the concrete.

The paper presents a material model of reinforced concrete strengthened with FRP that is formulated by extending the existing RC model by inclusion of the FRP material into the composite. The smeared approach was used in modeling the steel reinforcement, FRP strengthening as well as concrete cracks. This implies that perfect bond between the concrete, the steel and the FRP is assumed. The model is implemented into the code of the general-purpose finite element method program ANSYS as a user material model in order to test its results against the available experimental data. Results of the analysis of two different RC members strengthened with FRP are presented. The results are compared against the experimentally obtained data as well as against the numerical results from other finite element analyses employing more traditional approach into finite element modelling of such problems. Based on the presented results, it can be concluded that the proposed model, despite having certain deficiencies, is able to correctly simulate the behavior of the RC members strengthened with FRP in different configurations. Its ANSYS implementation enables its use in both research and practical purposes, facilitating the further research in this field as well as the practical applications in the construction industry.

Keywords: FRP; Strengthening; Numerical Model, ANSYS, Reinforced Concrete

1. Introduction

The conventional earthquake resistant design of reinforced concrete structures advises use of shear walls as effective way to add earthquake resistance to the reinforced concrete frames. A problem arises with structures erected decades ago following design rules which are by today's standards obsolete, inadequate and inefficient. Major earthquake events from around the world have shown the design deficiencies of these structures by inducing extensive damages in the structural members. Many of the old shear wall buildings are at risk of suffering damages from a major earthquake mostly due to their insufficient in-plane stiffness, flexural and shear strengths and ductility owing to the older design codes which didn't adequately estimate the demands that major earthquakes impose on the structures. This problem is ever increasing as the existing structures are getting older and their members gradually deteriorate.

Many different methods of seismic strengthening and repair of shear wall structures have been developed and tested in the last thirty years. Recently, state-of-the-art strengthening and retrofit techniques increasingly utilize externally bonded fiber reinforced polymer (FRP) composites, which offer unique properties in terms of strength, lightness, chemical resistance, and ease of application. Such techniques are most attractive for their fast execution and low labor costs.

Most widely used method for the simulation of the behavior of reinforced concrete strengthened with FRP composites is undouble the finite element method. The majority of the studies that included numerical modelling of FRP strengthened RC members with FEM use element overlaying, where one-, two- or even three dimensional elements (solid or layered) that represent the FRP material are superimposed over the concrete elements, either with (ex. Khomwan and Foster [1]; Wong and Vecchio [2]) or without (ex. Kheyroddin and Naderpour [3]) interface elements that represent the influence of the adhesive material or the bond between the FRP and the concrete.

A different approach is presented in this paper. An attempt is made to formulate a new material model with an aim to simplify the modeling of FRP strengthened reinforced concrete members. The newly formulated material model is implemented into ANSYS [4] and tested using available experimental data.

2. Model formulation

In the analysis of RC structures plane stress problems make up a large majority of practical cases. Therefore, the numerical model presented here is based on the inelastic model for cyclic biaxial loading of reinforced concrete of Darwin and Pecknold [5] which was designed to be used for such type of structures (shear walls, beams, slabs, shear panels, shells, reactor containment vessels).

2.1 Concrete

The concrete is treated as incrementally linear, elastic material, which means that during each load increment the material is assumed to behave elastically. It is also considered to exhibit stress-induced orthotropic material behavior. The constitutive relationship for incrementally linear orthotropic material with reference to the principal axes of orthotropy can be written as:

$$\begin{Bmatrix} d\sigma_1 \\ d\sigma_2 \\ d\tau_{12} \end{Bmatrix} = \frac{1}{1-\nu^2} \begin{bmatrix} E_1 & \nu\sqrt{E_1 E_2} & 0 \\ \nu\sqrt{E_1 E_2} & E_2 & 0 \\ 0 & 0 & (1-\nu^2)G \end{bmatrix} \begin{Bmatrix} d\varepsilon_1 \\ d\varepsilon_2 \\ d\gamma_{12} \end{Bmatrix} = D_C \begin{Bmatrix} d\varepsilon_1 \\ d\varepsilon_2 \\ d\gamma_{12} \end{Bmatrix} \tag{1}$$

where $d\sigma_i$ and $d\varepsilon_i$ are the stress and strain increments, E_1 and E_2 are initial concrete stiffness modules in principal directions, $\nu = \nu_1 \cdot \nu_2$ is the "equivalent" poison ratio, $G = \frac{1}{4(1-\nu^2)}(E_1 + E_2 - 2\nu\sqrt{E_1 E_2})$ is the shear modulus and D_C is the concrete constitutive matrix in the principle directions. Before it can be used in the finite element procedure, the concrete constitutive matrix is transformed to global coordinates using:

$$D_C' = T^T D_C T \tag{2}$$

where T is the strain transformation matrix (Cook [6]). At the moment when the principle tensile stress exceeds the concrete tensile strength a "crack" forms perpendicular to the principle stress direction. This is modelled by reducing the values of $E$and ν to zero. This has an effect of creating a "smeared" rather than discrete crack. The constitutive equation for the cracked concrete then takes the form:

$$\begin{Bmatrix} d\sigma_1 \\ d\sigma_2 \\ d\tau_{12} \end{Bmatrix} = \begin{bmatrix} 0 & 0 & 0 \\ 0 & E_2 & 0 \\ 0 & 0 & \frac{E_2}{4} \end{bmatrix} \begin{Bmatrix} d\varepsilon_1 \\ d\varepsilon_2 \\ d\gamma_{12} \end{Bmatrix} \tag{3}$$

If the tensile strength in the other principle direction is exceeded then a second crack occurs and the constitutive matrix is then reduced to $D_C = [0]$. In order to keep track of the material degradation, the concept of "equivalent uniaxial strain" is used. It allows derivation of the actual biaxial stress-strain curves from uniaxial curves. The equation suggested by Saenz [7] is frequently used for this purpose:

$$\sigma_i = \frac{\varepsilon_{ui} \cdot E_0}{1 + \left(\frac{E_0}{E_S} - 2\right)\frac{\varepsilon_{ui}}{\varepsilon_{ci}} + \left(\frac{\varepsilon_{ui}}{\varepsilon_{ci}}\right)^2} \tag{4}$$

where E_0 is the tangent modulus of elasticity at zero stress, E_s is the secant modulus at the point of maximum compressive stress (σ_{ci}), and ε_{ci} is the equivalent uniaxial strain at maximum compressive stress, Fig. 1. The concrete biaxial strength envelope suggested by Kupfer and Gerstle [8].

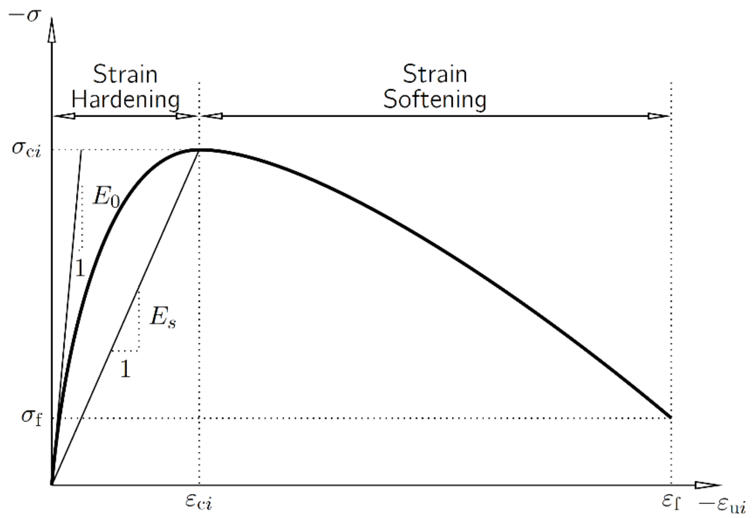

Fig. 1 – Equivalent Uniaxial Stress-Strain Curve in Compression, Hu and Schnobrich [9]

The development of the cracks in the concrete is considered as the main non-linearity inducing phenomenon in the concrete. If a crack occurs in the first principle direction, it is simulated by reducing the elasticity modulus in that direction to 0 and the constitutive matrix becomes:

$$D_C = \begin{bmatrix} 0 & 0 & 0 \\ 0 & E_2 & 0 \\ 0 & 0 & \frac{E_2}{4} \end{bmatrix} \tag{5}$$

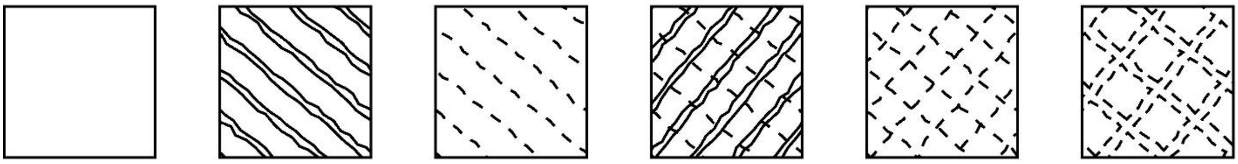

Fig. 2 - 6 possible crack configurations.

If then a crack occurs in the other principle direction, the concrete constitutive matrix is reduced to 0. This approach in crack modelling also means that cracks in the concrete are not considered as distinct material discontinuities, but rather as occurrence of many small cracks in the vicinity of the point considered, which is known as a "smeared" crack approach. Six different crack states can occur: uncracked, opened crack in the first principle direction, closed crack in the first principle direction, closed crack in the first principle direction and opened crack in the second principle direction, closed cracks in both principle direction and opened cracks in both principle directions (Fig. 2).

2.2. Reinforcing Steel

Generally, the reinforcing steel can be modelled as discrete or distributed. The model presented here considers the reinforcing steel to be distributed, or "smeared", throughout the concrete. A simple, bilinear model with strain hardening is adopted for the stress-strain behaviour of the steel. The constitutive matrix of the steel defined in the steel direction is

$$D_S = p_S \begin{bmatrix} E_{Steel} & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \tag{6}$$

with E_{Steel} the tangent stiffness of the steel and p_S the reinforcing ratio. Depending on the stress level in the steel, E_{Steel} can be either equal to the initial steel stiffness E_S or reduced by a strain hardening stiffness ratio δ, see Fig. 3. Before using it in the composite material matrix, D_S is transformed to the global coordinates using the strain transformation matrix T.

2.3. FRP strengthening

The influence of the FRP strengthening is accounted for in the same fashion as the reinforcing steel. The material is treated as distributed, or "smeared" throughout the concrete. Its material behaviour is assumed to be elastic-brittle, having abrupt failure after reaching its maximal strength (Fig. 3). It is also capable of transmitting only tension stresses. The constitutive matrix of the FRP defined in the direction of the FRP fibres is therefore:

$$D_F = p_F \begin{bmatrix} E_F & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \tag{7}$$

with E_F the tangent stiffness of the FRP and p_F the "strengthening" ratio. Before using it in the composite material matrix D_F must also transformed to the global coordinates using the strain transformation matrix T.

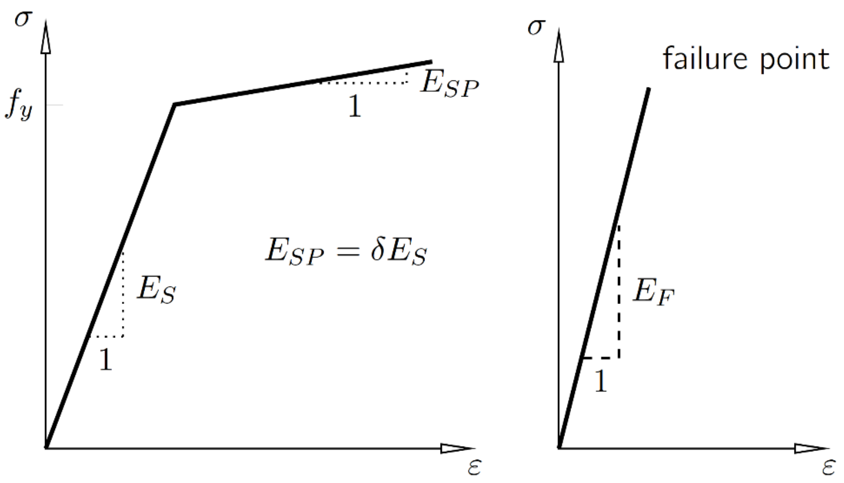

Fig. 3 – Stress-strain curves of reinforcing steel and FRP used in the model

2.4. Composite Material Constitutive Matrix

After defining the constitutive matrices of the constituent materials, the constitutive matrix of the composite material in the global coordinates is obtained by their summation:

$$D' = D'_C + \sum_{i=1}^{n} D'_{S,i} + \sum_{i=1}^{m} D'_{F,i} \tag{8}$$

where D', D'_C, D'_S and D'_F are the constitutive matrices if the composite material, concrete, steel and FRP in global coordinates, respectively, n is the number of different reinforcing steels and m is the number of different FRPs used for strengthening.

3. Verification

The material model briefly described in the previous section was coded and implemented into ANSYS in order to test its correctness and usability by comparing the results from numerical analyses with the available experimental data from the literature.

3.1. Monotonic loading – RC beam strengthened with FRP

Garden and Hollaway [10] performed four-point bending tests on 1m long RC beams strengthened with CFRP. The CFRP plates with a thickness of 0.82 mm were attached to the soffit of the beam. The beam with designation $3_{U,1.0m}$ was analyzed here (Fig. 4). Three different sections or parts of the beam (top, bottom and middle) were defined in order to accommodate the steel reinforcement and FRP strengthening placement in the actual beam cross-section (Fig. 5). Only half of the beam was model making use of the beam and load symmetry. The material properties for the reinforcing steel and the FRP strengthening for the three parts of the beam model are given in Table 1. The concrete definition for the whole beam was the same, having uniaxial compression strength of $f'_c = 43$ MPa, uniaxial tensile strength of $f_t = 3$ MPa, initial modulus of elasticity $E_0 = 40$ GPa, equivalent uniaxial strain at maximum strength $\varepsilon_{cu} = -0.0022$ and equivalent Poisson's ratio $\nu = 0.2$. Displacement control analysis was performed by applying series of vertical displacements at the point where the load was applied in the actual test. The obtained load-deflection curve is shown in Fig. 6. While the beam stiffness in the initial load increments was apparently overestimated, the rest of the curve stays very close to the shape of the experimental curve.

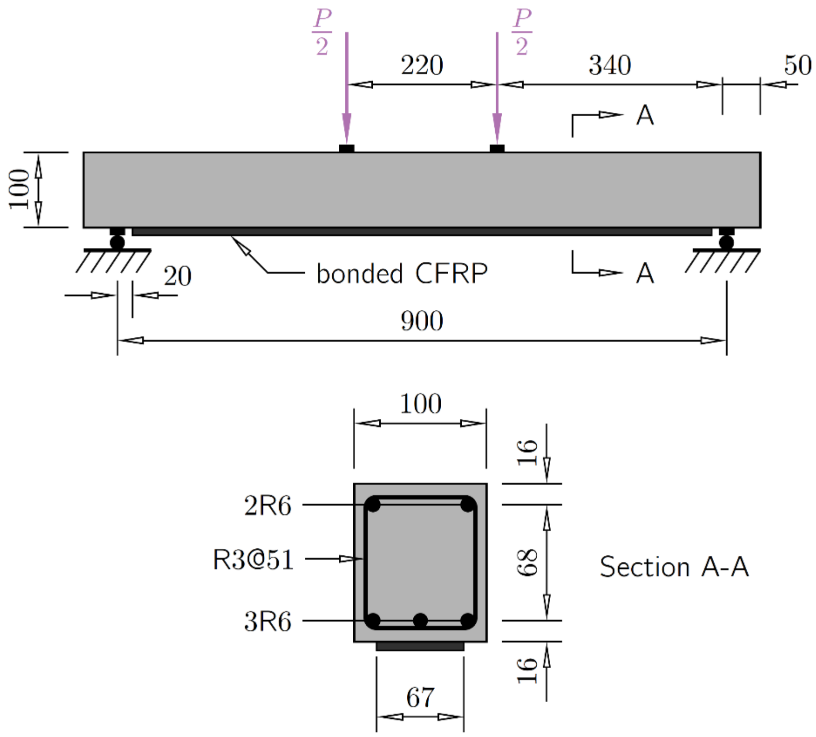

Fig. 4 – Test specimen $3_{U,1.0m}$, Garden and Hollaway (1998)

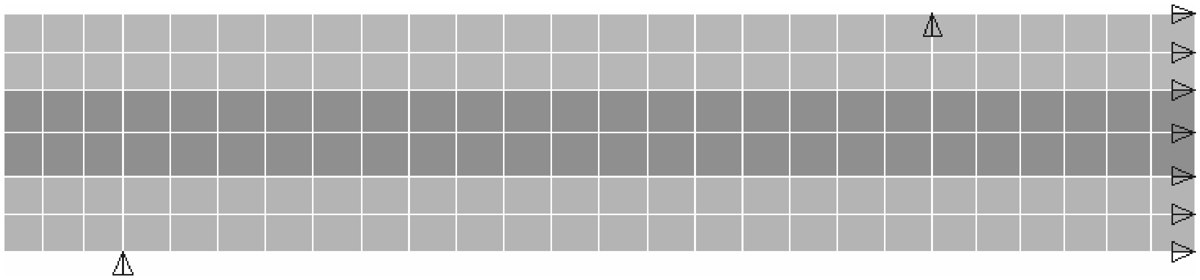

Fig. 5 – FEM model of test specimen $3_{U,1.0m}$ in ANSYS

Table 1 – Material parameters for the reinforcing steel and the FRP for the $3_{U,1.0m}$ beam used in the analysis

Part	Steel #1 (horizontal)				Steel #2 (vertical)				FRP		
	f_y	E_S	δ	p_S	f_y	E_S	δ	p_S	E_F	ε_F	p_F
	MPa	GPa	%	%	MPa	GPa	%	%	GPa	%	%
Bottom	350	215	0	2.7	350	215	0	1.7	110	1.2	1.7
Top	350	215	0	2.7	350	215	0	1.7	-	-	-
Middle	-	-	-	-	350	215	0	1.7	-	-	-

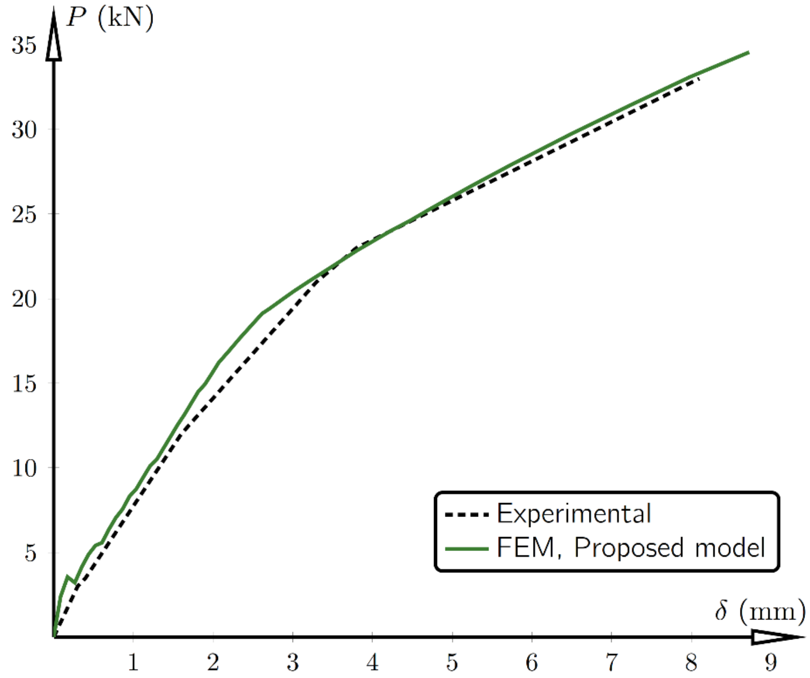

Fig. 6 – Load versus mid-span deflection of $3_{U,1.0m}$ beam

3.2. Cyclic loading – RC Wall strengthened with FRP

A series of reinforced concrete shear wall specimens were tested in cyclic loading conditions (Lombard [11]). The walls were constructed using 40 MPa concrete with identical reinforcement of 400 MPa, 10 mm reinforcing bars. The height of the walls from the base of the panel to the center of the cap beam is 2 m, the length is 1.5 m and the thickness is 10 cm. The vertical reinforcement consists of five pairs of 10 mm bars, spaced at 40 cm for a reinforcement ratio of 0.8%. The horizontal steel consisted of five pairs of 10 mm bars, spaced at 40 cm for a reinforcement ratio of 0.5%. Three of the test specimens included a control wall and two strengthened walls. The control wall was tested in its original state which provided a baseline for the evaluation of the repair and strengthening techniques. The two strengthened shear walls were strengthened by applying 0.11 mm carbon fiber sheets to the walls without pre-damage. The carbon fiber sheets had an elastic tensile modulus of 230 GPa and failure strain of 1.5%. The first specimen was strengthened with one vertical layer of FRP externally bonded to each face of the wall (Wall 1). The second specimen had one horizontal and two vertical FRP layers on each face of the wall (Wall 2). Both specimens were not loaded until the strengthening was applied.

This paper presents the results of the analysis of the specimen Wall 1. For the FEM model in this case triangular as well as quadrilateral meshes were tested. The preliminary analyses showed that using triangular mesh (Fig. 7) generally led to better solutions. A mesh of triangular, 6-node Plane183 elements with average size of 25 cm was used for the final results.

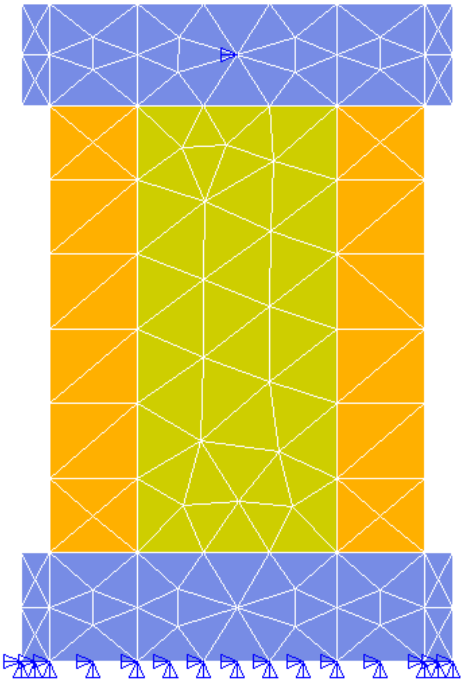

Fig. 7 – Triangular Element Mesh of the FEM Model

Five different sections of the wall with different properties were defined: cap-beam, foundation block, two side section (‘columns’) and a middle section (‘panel’). Since the cap-beam and the foundation block are significantly stiffer that the wall and their actual purpose is to provide the load transfer and anchorage for the tested wall, they were modelled as rigid elements. The confining effect of the stirrups in the ‘columns’ was approximately accounted for by slightly increasing the concrete compressive strength in those regions, taking it to be 46 MPa in the ‘columns’, and 40 MPa in the ‘panel’. The other concrete parameters were taken as: tensile strength of 4 MPA, initial elasticity modulus of 35 GPa, equivalent uniaxial strain of 0.35% and equivalent Poisson’s ratio of 0.2. The steel material parameters were taken as: yield strength of 400 GPa, elasticity modulus of 200 GPa and strain hardening stiffness ratio of 1.8. The reinforcement ratio in vertical direction is 0.8%, and in horizontal direction 3% (in the ‘columns’) and 0.5% (in the ‘panel’). The FRP material parameters were taken as: elasticity modulus of 230 GPA, ultimate strain at failure 1.5% and “strengthening” ratio of 0.22 for both the ‘columns’ and the ‘panel’ for each applied layer of the FRP strengthening, in the corresponding direction.

The cyclic load was applied at the middle of the top beam as a series of small displacements. The force and displacement at the same point were taken as results of the performed analyses. These were compared with the available experimental data.

The resulting hysteretic loops are shown in Fig. 8. To measure how the numerical results compare to the experimental data the energy dissipated at each cycle (which corresponds to the area of the hysteretic loop) was calculated. The calculated energy dissipation is given in Table 2. The results indicate quite good correspondence with the experimentally acquired data.

It should also be noted that although the cyclic loading analyses yielded good results, the solution showed significant sensitivity on the input parameters (element type and size, load step sizes, material data). Non-convergent load-step solutions frequently occurred leading to premature failure of the model. To obtain good and stable solution the model needed to be calibrated by performing several parametric analyses which would yield the most appropriate set of input parameters. As the final results show, once stable solution is reached, the simulation shows satisfactory correspondence to the test results.

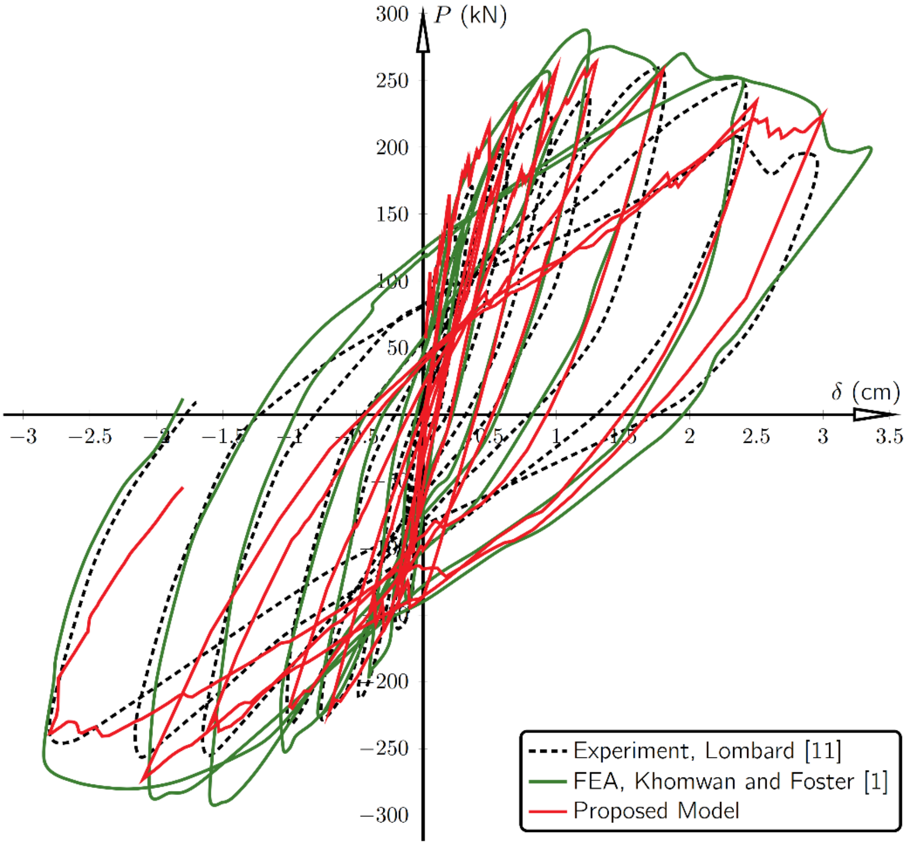

Fig. 8 – Load-Deflection Curves for Test Wall 1

Table 2. Comparison of Energy Dissipation per Cycle for Wall 1 (in Nm)

Loop #	Experimental	FEM	Ratio	Difference
3	622.25	494.81	0.80	20%
4	1412.25	1442.94	1.02	2%
5	2268.10	1964.96	0.87	13%
6	4603.00	3511.48	0.76	24%
7	6960.88	5797.95	0.83	17%
8	8660.70	8161.54	0.94	6%

4. Summary and Conclusions

The paper presents an attempt to formulate material model which will correctly simulate the behaviour of reinforced concrete members in plane stress strengthened with FRP materials. The presented model builds up on the concepts of an earlier model reinforced concrete model of Darwin and Pecknold. It uses the uniaxial strain approach in modelling of biaxially loaded reinforced concrete and the distributed approach of modelling the cracking behavior, the reinforcing steel and the FRP strengthening material.

The proposed model is subsequently implemented into the code of the general finite element method program ANSYS as a user material model in order to test its results against the available experimental data. Two different analyses were presented: An analysis of RC beam strengthened for bending by externally bonded FRP strips on the soffit side if the beam and analyses of three RC wall strengthened by externally bonded FRP wraps on its sides. The results are compared against the experimentally obtained data as well as against the numerical results from another finite element analysis performed by other authors employing more traditional approach into finite element modelling of such problems. Based on the presented results, it can be concluded that the proposed model is able to correctly simulate the behaviour of the RC members strengthened with FRP in different configurations. Its ANSYS implementation enables its use in both research and practical purposes, facilitating the further research in this field as well as the practical applications in the construction industry. However, since the results showed that the solutions were significantly sensitive to the input parameters, they have to be thoroughly checked before adopting them as a correct solution. Further investigations need to be carried out in order to determine if this limited reliability of the results is due to model formulation deficiencies or maybe its software implementation.

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