# Nonlinear finite element analysis of FRP reinforced concrete structures

G Tharmarajah[1] & P Archbold[2]
[1,2]STRAIT, Structures & Materials Research Group, School of Engineering
Athlone Institute of Technology, Dublin Road, Co.Westmeath.
email: tgobithas@ait.ie, parchbold@ait.ie

ABSTRACT: FRP reinforcement is a non-corrosive alternative to steel and is gaining popularity for use in reinforced concrete structures exposed to corrosive environments. A major difference between the two reinforcing materials is their behaviour at failure. Steel tends to undergo ductile elongation, while FRP is a brittle material, which ruptures suddenly. Accordingly, while steel reinforced concrete members are generally designed to fail through yielding of the steel, FRP reinforced structures are designed to fail through compressive failure of the concrete. This crushing of the concrete represents plastic deformation. Accurate modelling of the failure of FRP reinforced concrete structures has proven challenging to researchers in the field. However, improvements in the development of material models, among other advances, mean improved accuracy from nonlinear finite element models is now achievable. This paper discusses the challenges of modelling reinforced concrete structures for concrete crushing failure. Results are presented from nonlinear finite element models of FRP reinforced concrete slabs, which were tested to failure and compared to the experimentally derived values.

KEY WORDS: NLFEA, BFRP, Concrete, DIANA, ANSYS.

## 1 INTRODUCTION

Nonlinear finite element analysis is increasingly used in civil engineering applications for structural investigations. The interest in using finite element analysis to solve complex structural problems can be traced back to 1941 when Alexander Hrennikoff first introduced a solution method to solve a plane elasticity problem using a finite element approach [1]. Since then the finite element approach has been widely used by engineers, physicist and mathematicians. Understanding of the behaviour of concrete and reinforcement materials and development of advanced material models to simulate this behaviour has given nonlinear finite element analysis the capability to expand to a wide range of applications.

Fibre Reinforced Polymer (FRP) is a generic title for a corrosion resistant composite reinforcement produced using various fibres and resin. The type of fibre used, determines the name of the material. The popular FRP types are Glass FRP, Carbon FRP, Aramid FRP and Basalt FRP. Along with its corrosion resistant attributes, FRP reinforcement materials demonstrate high tensile strength and light weight compared to traditional steel reinforcement. Moreover, where corrosion resistant reinforcement is required, FRP materials offer advantages over traditional alternatives such as stainless steel, primarily from an economic perspective.

One of the challenges associated with the use of FRP reinforcement is the lack of design guidance available to designers. The most prominent international code of practice in this area is produced by the American Concrete Institute [2]. Of particular concern is the fundamental difference between FRP and steel reinforcement at ultimate failure. Whereas steel tends to yield, leading to a gradual collapse, FRP composites are brittle, with a lower modulus of elasticity, leading to a sudden rupture without prior warning.

In order to address this failure mode associated with brittle reinforcement, ACI 440 recommends the use of over reinforced concrete sections, thereby forcing the structural member to fail through concrete crushing rather than FRP rupture. This is considered a marginally preferable failure scenario [2].

As the FRP reinforced structures are thus often designed to fail by concrete crushing and in-plane restrained slabs, such as deck slabs in beam and slab bridges show concrete crushing failure due to compressive membrane action ([3], [4]and [5], etc.), it is important to understand the behavior of the nonlinear finite element analysis (NLFEA) models where structures fail by concrete crushing. In such cases, the behavior of the concrete material model becomes of critical importance.

The authors are currently investigating the potential for use of FRP as an alternative to steel reinforcement in precast concrete products where the issue of durability of reinforcement is critical factor in terms of the product design and limitations. The steel and BFRP reinforced representative concrete sections tested with other unreinforced sections in a series of load tests are discussed here. The first author was also previously involved in the testing of in-plane restrained slab sections again reinforced with BFRP bars [6].

This paper presents the results of the experimental load tests on the concrete samples and compares the measured responses to those predicted from nonlinear FE models. The simply supported slabs were analysed using commercially available Ansys V11.0 [7] while the restrained slabs were analysed using both Ansys and DIANA 9.2 [8] for the service and ultimate behaviour of the test slabs.

## 2 EXPERIMENTAL INVESTIGATION

The concrete slab sections tested by the authors were subjected to four-point loading with simple supports as shown in Figure 1. The slab response to loading was measured in terms of mid-span deflection up until the point of failure. The serviceability performance of the slab sections in terms of crack generation before failure was monitored visually. Prior to load testing the slabs were all painted with white emulsion paint to emphasis any crack development that may occur. Figure 2 shows a slab being tested with cracks highlighted.

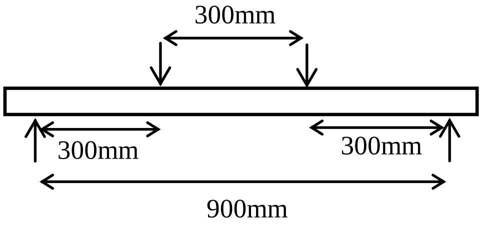


Figure 1: Test setup of simply supported slabs

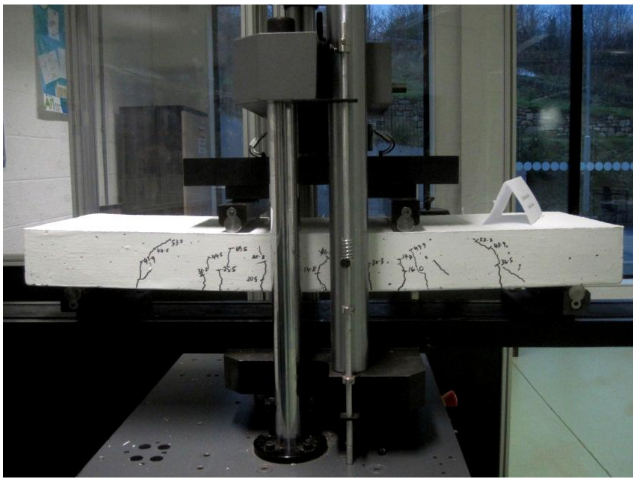

Figure 2: Simply supported slab being tested

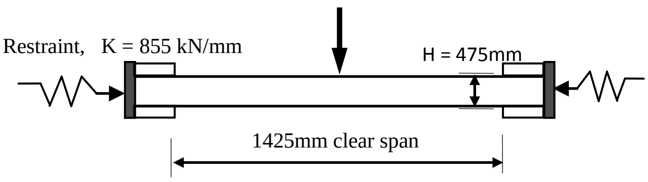


Figure 3: Test setup of restrained slabs (Reproduced from Tharmarajah et al., 2010)

The restrained slabs were subjected to single point loading and also were subjected to horizontal restraint as shown in Figure 3.

The steel reinforced simply supported slab was used as the reference sample. The details of the test slabs are given in Table 1.

Table 1: Details of test panels used for investigation

| **Test Slabs*** | **Rebar percentage** | **Effective Depth (mm)** |
|---|---|---|
| BFRP 0.6%_12_125 T&B* | 0.60% | 119 |
| BFRP 0.6%_16_300 T&B* | 0.60% | 117 |
| BFRP_60/SS† | 0.67% | 60 |
| STEEL_60/SS† | 0.67% | 60 |

** Dimensions: 1765mm (length) x 475mm (width) x 150mm (depth).*
*† Dimensions: 900mm (length) x 350mm (width) x 100mm (depth).*

## 3 NONLINEAR ANALYSIS

The simply supported slabs and in-plane restrained slabs were modelled with ANSYS. Also, the restrained slabs analysed with ANSYS are compared with the DIANA analysis results [9]. Both Ansys and DIANA are commercially available NLFEA tools which are widely used in nonlinear analysis of reinforced concrete structures.

### *3.1 Material model and numerical discretisation*

#### *3.1.1 Material Properties and Model*

The compressive and tensile properties of the concrete were obtained from compressive tests and tensile tests respectively carried out on concrete samples taken during the test slab production. Details of these concrete material properties are given in Table 2. Similarly, material properties for steel and BFRP were obtained from tensile strength tests carried out on BFRP and steel bars (Table 3).

Table 2: Concrete properties of the test slabs

| **Test Slabs*** | **Con. comp. strength $f_{ck,cube}$ N/mm$^2$** | **Tensile strength of conc. $f_t$ N/mm$^2$** |
|---|---|---|
| BFRP 0.6%_12_125 T&B | 69.3 | 3.77 |
| BFRP 0.6%_16_300 T&B | 66.1 | 3.13 |
| BFRP_60/SS | 57.1 | 3.55 |
| STEEL_60/SS | 57.1 | 3.55 |

Table 3: Properties of the reinforcement

| **Reinforcement** | **Tensile strength N/mm$^2$** | **Modulus of elasticity N/mm$^2$** |
|---|---|---|
| BFRP* | 920 | 54000 |
| STEEL | 460 | 210000 |

** The data used was obtained from the tests carried out by Tharmarajah et al. [6]*

#### *3.1.2 Concrete and Reinforcement*

The linear properties of concrete are the modulus of elasticity and Poisson's ratio. The modulus of elasticity was established from the compressive strength of the concrete. The compressive strength of the concrete was observed from testing of 100mm x 100mm x 100mm cubes. Young's modulus of elasticity can be calculated from the cube compressive stress $f_{ck,cube}$ using the formula $E_c$ = 4.73

($f_{ck,cube}$)^0.5 kN/mm2 [10]. Poisson's ratio of 0.2 was considered throughout the analysis.

The plasticity of concrete in compression is crucial for structures where the failure is dominated by the failure of concrete. Unlike steel reinforced structures, where the failure occurs due to the yielding of steel, FRP reinforced structures are designed to fail by the concrete failure. In such cases, the post peak softening behaviour of the concrete is critical. The post peak stress strain behaviour of the concrete for both tension and compression needs careful investigation as it is difficult to establish [11]. Therefore, the strain hardening and softening of concrete was modelled using Thorenfeldt compression behaviour [12].

Steel and BFRP were modelled with Von Mises plasticity model with ideal plasticity. Although BFRP bars are brittle in nature, the ideal plasticity was considered as the slabs failed by concrete crushing. Since the stress on bars stays within linear region due to concrete rupture, it is appropriate to use ideal plasticity.

### *3.2 Numerical discretisation*

The restrained slabs had a single mid span loading and simply supported slabs were tested with four point bending. Since both are symmetric, half of the slab was considered for modelling.

Numerical discretization of the model can influence the results of the slab analysis. Especially the size of the finite element mesh can lead to either over estimated or under estimated failure load of a slab [13]. Small mesh size leads to reduced failure load as energy dissipation decreases and higher mesh size increases the failure load due to decreased crack progression [13].

While taking in to account the influence of the mesh size and aspect ratio, a 25mm element size was chosen with an aspect ratio of 2. A previous study by Duchaine and Champliaud [14] showed that, the error could increase with an increase on aspect ratio. Therefore, to minimize the error, it was decided to use an aspect ratio 2 to reduce the error to as low as 2%.

## 4 RESULTS AND DISCUSSIONS

### *4.1 Results*

A comparison for the in-plane restrained slabs analysed with DIANA and Ansys and the behaviour of simply supported slabs analysed with Ansys are discussed. Load versus deflection and the ultimate failure load are showed for both types of structures.

#### *4.1.1 Ultimate behaviour*

Table 4 presents the measured and predicted failure loads for the respective simply supported and restrained slabs. The nonlinear models were capable in predicting the ultimate failure load of the tests panels with a good accuracy. The comparison between the failure loads predicted by nonlinear analysis and experimental investigation is shown in Figure 4 for restrained slabs and in Figure 5 for simply supported slabs.

Table 4: Comparison of test results with NLFEA predictions

| **Test Slabs** | **Observed ultimate failure load kN** | **Predicted ultimate failure load (DIANA) kN** |
|---|---|---|
| BFRP 0.6%_12_125 T&B | 300.4 | 305.1(303.3) |
| BFRP 0.6%_16_300 T&B | 295.1 | 308.2(299.8) |
| BFRP_60/SS | 56.3 | 54.4(NIL) |
| STEEL_60/SS | 45.9 | 44.0(NIL) |

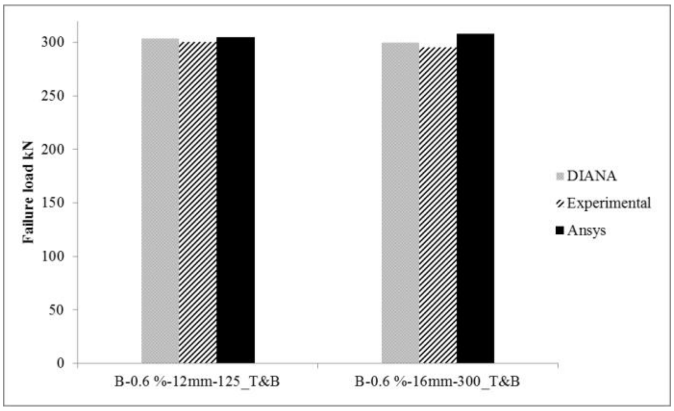


Figure 4: Comparison of ultimate failure load of the restrained slabs with DIANA and Ansys model predictions

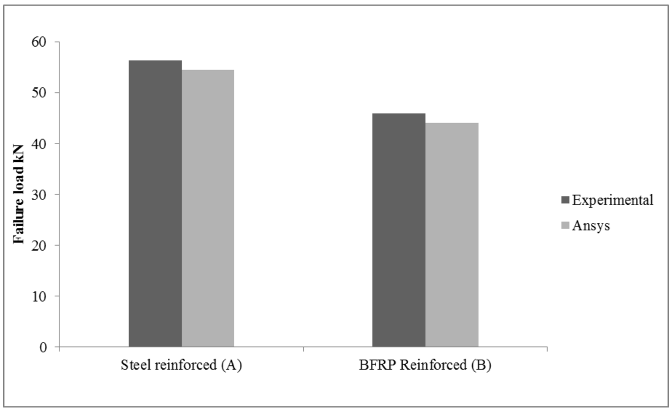


Figure 5: Comparison of ultimate failure load simply supported slabs with Ansys model predictions.

The deviation on failure loads predicted by DIANA and Ansys models were within 1% to 5% of the actual failure load of both simply supported and restrained test slabs.

#### *4.1.2 Service behaviour*

While the ultimate load behaviour of the slabs was accurately predicted by the nonlinear FE models, the load-deflection response prediction was not as successful. Previous investigations on FRP reinforced restrained slabs and nonlinear finite element analysis (NLFEA) of such slabs in

DIANA and Ansys showed that the nonlinear models predict a stiffer response than that observed during the experimental investigation [9]. This phenomenon was again observed on the simply supported slabs reinforced with both steel and BFRP bars. Figure 6 and Figure 7 show the load versus deflection behaviour of restrained slabs and simply supported slabs for both experimental and nonlinear evaluation.

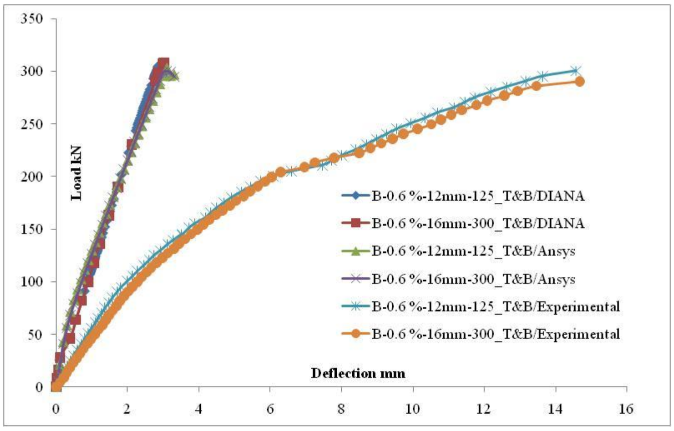


Figure 6: Load versus deflection behaviour of restrained slabs are compared with DIANA and ANSYS model results

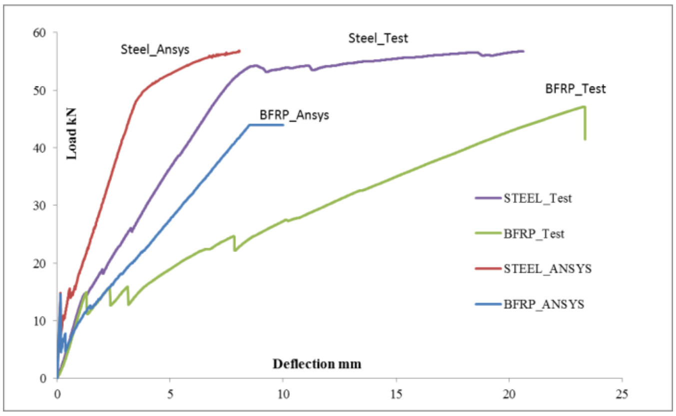


Figure 7: Load versus deflection behaviour of simply supported slabs are compared with Ansys results

In order to investigate this discrepancy between the predicted and recorded response, a comparison was carried out between the experimental results and nonlinear predictions against theoretical predictions (Equation 1 and Equation 2) within linear load levels (which is prior to the first crack formation). This shows that the nonlinear predictions give better agreement with the theoretical predictions than actual test results and both theoretical and nonlinear responses are much stiffer than experimental results (Figure 8). Deflection of four point bending was calculated using equation 1 while the deflection for single mid-point loaded restrained slabs was estimated using equation 2.

$$\delta = \frac{FL^3}{192EI} \quad \text{--------------} \qquad (1)$$

$$\delta = \frac{Fa^2(3L-4a)}{6EI} \quad \text{--------------} \qquad (2)$$

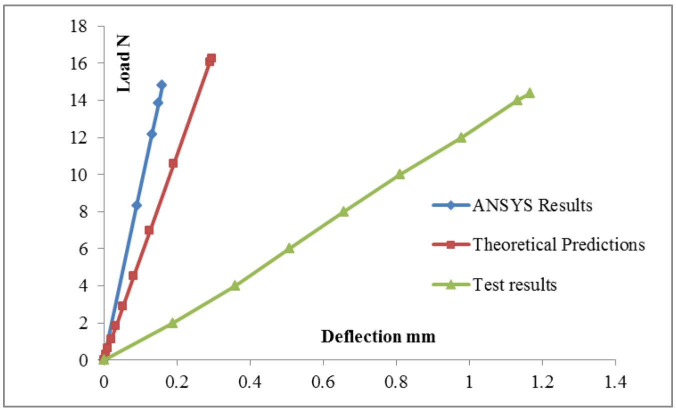


Figure 8: Ansys, theoretical and test results comparison of simply supported steel reinforced panel within linear load level (before the first crack). A similar phenomenon was noticed for restrained slabs.

Although the NLFEA models showed a stiffer response in predicting the load versus deflection behaviour, they did display good accuracy in predicting the crack load and crack pattern for the relevant test slabs.

### *4.2 Discussions*

Despite the stiff response of the nonlinear results in predicting load versus deflection behavior, the NLFEA models are capable of predicting the ultimate failure load of different types of reinforced concrete panels with good accuracy. Higher deflection noticed on experimental investigation leads to a question on what causes the variation between nonlinear predictions and experimental results. Since this issue has been noticed in various other occasions and in different research studies, the future research study will investigate the factors cause the fluctuation in load versus deflection predictions.

The restrained slabs and the slabs reinforced with BFRP bars are designed to fail by concrete crushing. Thus, ideal plasticity of the concrete cannot be used in such situations as ideal plastic conditions could cause misleading conclusions. Therefore, it is important to incorporate appropriate softening behaviour of the concrete in the post peak stress level. The current study considered a Thorenfeldt compression model to study the failure behaviour of FRP reinforced sections.

The results show that for both restrained and simply supported slabs, the failure load predictions using nonlinear models demonstrate a good correlation to the experimental investigation.

Although the test slabs showed good agreement in terms of failure load, the load versus deflection behaviour of the test panels obtained from nonlinear models showed a stiffer response compared to experimental investigation. A similar behaviour was noticed by several other researchers in different occasions.

A 1800mm length, 240mm depth and 150mm width beam tested by Hibino et al. [15] was modelled using DIANA FEA and the load versus deflection behaviour observed showed a good agreement with the test results of the beam. The same beam modelled by Parvanova et al [16] using ANSYS also gave a good agreement with test results. However, in another research publication, Al-Azzawi et al [17] compared Ansys results of 450mm (long) x 150mm (width) x 150mm (depth) beams and 600mm (long) x 200mm (width) x 50mm (depth) beams. The Ansys analysis carried out on both deep and shallow beams demonstrated that the deep beams with 150mm depth had a good agreement with the nonlinear results while the shallow 50mm deep beam showed relatively stiffer response to about 6 times of the actual stiffness of the test slab.

A research by Dirar and Morley [18] also observed that the DIANA FEA nonlinear models show a stiffer response in the linear stage. However a good agreement was noticed at the post cracking stage. Therefore, it could be noticed from the authors' nonlinear FE results and the results discussed by the other researchers on their nonlinear FE analysis, that the NLFEA models show variation in predicting the behaviour of the test model. In some occasions the models showed a good correlation with the test results and sometimes they didn't.

## 5 CONCLUSIONS

The following conclusions can be made from the studies.

1. Nonlinear FE models are capable of predicting the actual strength of the test panels regardless of physical and material characteristics of the test model.
2. Nonlinear investigation on experimentally tested model lead to an observation, that, in some situations the models predict stiffer response than experimental results.
3. The stiffer response found in nonlinear models for some slabs could be due to material properties used and requires further investigation.
4. Further research investigates beams with various depth and widths to study the difference between experimental and numerical results.

## 6 FUTURE RESEARCH

The future work will investigate the factors that cause the stiffer response of the nonlinear model in the linear and nonlinear region. Experimental investigations are expected to be carried out on beams with various depths and widths to evaluate the stiff response noticed on reinforced simply supported slabs for both nonlinear and theoretical evaluations.

The future research also expects to investigate the relationship between the compressive stiffness and the flexural stiffness of concrete in experimental, numerical and theoretical investigations.

## ACKNOWLEDGMENTS

The authors would like to gratefully acknowledge the support of Killeshal Precast Concrete and Enterprise Ireland for funding the project and Magmatech, U.K. for supplying the BFRP bars and technical staff at the Athlone Institute of Technology for their support in laboratory testings.

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