EXPERIMENTAL AND NUMERICAL ANALYSIS OF SHEAR DEFICIENT RC BEAM RETROFITTED WITH FERROCEMENT

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ABSTRACT

Strengthening of deteriorated existing concrete structures is necessary to extend their life span. Retrofitting has become popular in Bangladesh in recent time. Glass fiber reinforced polymer (GFRP), carbon fiber reinforced polymer (CFRP), ferrocement, etc. are the popular materials being used for retrofitting of the concrete structures. In the present study, an attempt has been made to determine the performance of the reinforced concrete beams retrofitted with ferrocement jacketing. The objectives of this study were to investigate the shear behavior of retrofitted beams experimentally and to develop a finite element model describing the beams and finally to investigate the influence of different parameters on the behavior of the retrofitted beams. The ANSYS program is used to model the beam & simulation of the beam behavior. The concrete model is created by SOLID65 and LINK8 is used for steel bar. The numerical analysis is performed to investigate the shear behavior of beam designed such a way that shear failure must be expected. The beams are loaded with third point loading until crack developed. The beam then retrofitted with ferrocement layer. The layer thickness is 12mm. Finally the retrofitted beam is loaded until failure. Hear contact and target element is used to create concrete-ferrocement interface. From the analyses the load- deflection relationships until failure, failure modes and crack patterns were obtained. The result shows that The ultimate shear load carrying capacity for the beams retrofitted with 12mm layer of ferrocement increased 16.67% compared with control beam.

Keywords: *Title, abstract, objective, results, conclusions*

1. INTRODUCTION

Reinforced concrete structures have to face different kind of load i.e. DL, LL, wind load, EQ load, vibration load etc. But a structure may not have enough strength to resist the load. Or the structure becomes too old to carry the loads. Then it exhibit distress and suffer damage, even before their service period is over due to several causes such as improper design, faulty construction, change of usage of the building, change in codal provisions, overloading, earthquakes, explosion, corrosion, wear and tear, flood, fire etc. Such unserviceable structures require immediate attention, enquiry into the cause of distress and suitable remedial measures, so as to bring the structure into its functional use again. In such circumstances there are two possible solutions: replacement or retrofitting. Full structure replacement might have determinate disadvantages such as high costs for material and labour, a stronger environmental impact and inconvenience due to interruption of the function of the structure e.g. traffic problems. When possible, it is often better to repair or upgrade the structure by retrofitting.

Now-a-days several attempts have been made worldwide to increase the life of deteriorated concrete structures by suitable retrofitting techniques. The strengthening and enhancement of the performance of deficient structural elements in a structure or the structure as a whole is referred to as retrofitting. Repair refers to partial improvement of the degraded strength of a building after an earthquake. In effect, it is only a cosmetic enhancement. Retrofit aims to strengthen a building to satisfy the requirements of the current codes for seismic design. The seismic performance of a retrofitted building is aimed higher than that of the original building. The building may not be damaged or deteriorated. The various retrofitting techniques include steel plate bonding, polymer injection followed by concrete jacketing, use of advanced composite materials like FRP, Ferrocement etc. Of the various retrofitting techniques available, plate bonding is one of the most effective and convenient methods of retrofitting. Among the plate bonding techniques FRP plates are quite popular now-a-days. But it is observed that the use of FRP is restricted to developed countries or urban areas of the developing countries due to higher initial cost and requirement of skilled labor for their application. Thus, there is a need to develop an alternative technique, which is economical and can be executed at site with the help of semi-skilled labor available at site. Ferrocement as a retrofitting material can be pretty useful because it can be applied quickly to the surface of the damaged element without the requirement of any special bonding material and also it requires less skilled labour, as compared to other retrofitting solutions presently existing. The ferrocement construction has an edge over the conventional reinforced concrete material because of its lighter weight, ease of construction, low self-weight, thinner section and high tensile strength which makes it a favourable material for prefabrication also (Tumar, 2006). The flexural performance of deteriorated reinforced concrete beams repaired with ferrocement was found superior both at the service and ultimate load caring cases than those improved by conventional method (Andrew & Sharma, 1998). In another investigation, it was found that the crack width of the rehabilitated composite beams with ferrocement jacketing reduces by 36% (Kaushik & Dubey, 1994). An increase in the number of layers improves the cracking stiffness of the composite beams (Nassif, 1998, Vidivelli, 2001, Nassif, 2004). The beams are stressed up to 75 percent of safe load and then retrofitted with ferrocement jackets with wire mesh at different orientations. The results show that the percent increase in load carrying capacity for beam retrofitted with ferrocement jackets with wire mesh at 0, 45, 60 degree angle with longitudinal axis of beam, varies from 45.87 to 52.29 percent (Bansal, Kumar & Kaushik, 2008). Many experimental studies have been conducted in recent years to strengthen shear members by using various materials. By tested reinforced concrete beams to study the effectiveness of externally bonded precast ferrocement plates in strengthening beams showing shear distress. The relative efficacy of the bonding media (C-S mortar, epoxy) used in bonding the precast F.C Plates to the sides of beams were studied. Ferrocement was considered attractive for this application due to its thigh tensile strength, low weight economy in cost, long life of treatment and precise assessment of the additional strength gained by its use (Kaushik and Garg, 1994). Shear mode of failure in beams is undesired mainly being a brittle failure. By exploring the potentials of ferrocement in transferring the brittle mode to ductile mode, it was found that the strengthened beam showed a marked improvement in performance at service load, greatly improved ductility at ultimate with either a ductile shear failure or seemingly a transition from shear to flexure mode of failure.Moreover ferrocement wraps are more effective than ferrrocement strips (Rafeeqi, Lodi & Wadalawala, 1998). Tomar (2006) in an experimental study compared the shear performance of reinforced concrete beams repaired with conventional method and Ferrocement. He concluded that the beams retrofitted with wire mesh for different stress levels do not deboned when loaded to failure. The failure of composite is characterized by development of shear and flexural cracks over the tension zone. He also recommends that, Retrofitted beam corresponding to stress level of 60% has the highest load carrying capacity as compared to other specimen depicting that increasing stress levels contributes to strengthening in decreasing order.

Finite element method is a numerical analysis method that divides the structural element into smaller parts and then simulates static loading conditions to evaluate the response of concrete and pre stressed concrete members. The use of this technique is increasing because of enormous advancement of engineering and computer knowledge. This method responds well to non-linear analysis as each component possesses different stress strain behavior. This behavior is efficiently addressed by software ANSYS which provides number of elements for modeling of materials and apply loads to evaluate the response. This study will focus on correlating between the shear behaviour of ferrocement jacketing found in the experimental work & numerical analysis.

2. EXPERIMENTAL INVESTIGATION

2.1 Materials

The Ordinary Portland Cement conforming was used for the preparation of test specimens. The river sand of fineness modulus 2.35 was used for the preparation of concrete. Crushed stone in angular shape was used as coarse aggregate. After breaking into pieces, the aggregates were mixed as 5% from 25 mm to 20 mm, 57.5% from 20 mm to 10 mm, and 37.5% from 10 mm to 5 mm as per ASTM C33-93. The absorption capacity was determined as per ASTM C128 and unit weight as per ASTM C29. Normal tap water conforming drinking quality was used for the preparation and curing of concrete specimens. The concrete mix was designed according to ACI method 211 for slump value of 30 mm and 28 days cylinder compressive strength of 22 MPa (3200 psi). Design concrete mix of 1:1.5:3 and w/c of 0.5 was found to be appropriate for this requirement. Three cylinder specimens were cast and tested (at the age of 28 days) to determine compressive strength of concrete mixture. The average compressive strength of concrete was 22 N/mm². Steel bars of diameter 12 mm and 10 mm having 60 ksi yield strength were used as longitudinal reinforcement and 8 mm diameter bars were used as shear reinforcement. The wire mesh used in the ferrocement jacketing was 20 BWG (British Standard Wire Gauge) woven GI (Galvanized Iron) of 12 mm square openings.

2.2 Preparation of Testing Specimen

The experimental work comprises casting of total two sets of reinforced concrete (RC) beams having concrete of grade M22, cross sectional dimensions of 203mm x 203mm x 1000mm length. Two different configurations such as FRB and control beams were prepared for retrofitting of the beams using ferrocement. Total 8 no. of RC beams are cast and cured for 28 days by wet jute bags. First set of (4 no.) RC beams designated as FRB was retrofitted using single layer wire mesh and 12 mm thick cement mortar. Second set of (4 no.) RC beams (no retrofitting) treated as control beams.

2.3 Reinforcing details of beams

All the beams are identical in dimension. For showing Reinforcement details of beam, longitudinal section and cross section of beam are shown in Figure 1.

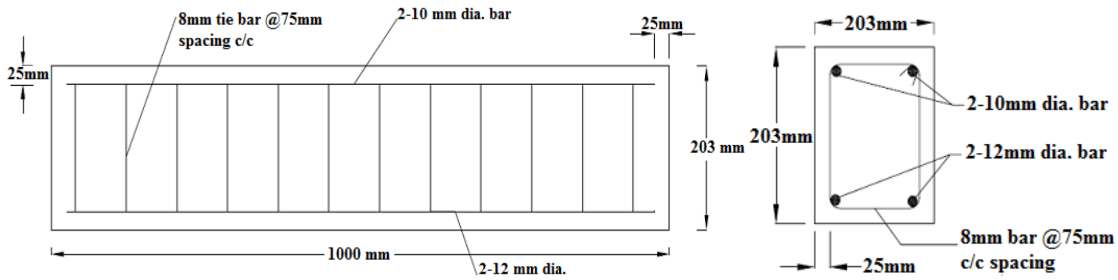


Figure 1: longitudinal section and cross section of beam

2.4 Retrofitting of beams

After 28 days of curing using wet jute bags RC beams were removed and the beams were dried for a few hours to obtain surface dry condition. In order to simulate damage, the beams were preloaded before applying ferrocement jacketing. Preloading was done with the same setup described in the section 2.5 and was loaded until appearing of first crack. Then, ferrocement jacketing was applied to different RC beams. Figure 2 shows the cross sectional details of the ferrocement retrofitted beams. The beams were retrofitted on two sides of RC beams using 1 layers of wire mesh and thickness of cement mortar layer was 12 mm. At first original beam surfaces are cleaned and wet by water. Thereafter, cement mortar having thickness half of the ferrocement layer is put on the beam surface. Ferrocement wire mesh is fixed on this wet surface as quick as possible. After fixing wire mesh, cement mortar is put again on the wire mesh to make required thickness of ferrocement wire. A gap of 3 mm was kept between RC beam specimen and ferrocement jacket at both the top and bottom of the specimen to avoid direct compression on the ferrocement jacket. Afterward, all the ferrocement jacketed beams were subjected to curing using wet jute bags for 28 days.

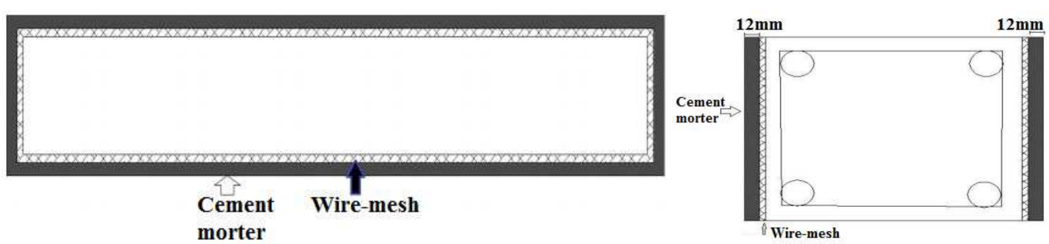


Figure 2: Top view and Cross Section of Retrofitted RC beam (one layer of wire mesh)

2.5 Testing Procedure

After curing, all the specimens were kept in room temperature for few hours to attain standard surface dry condition. The tests were performed on a Universal Testing Machine (UTM) with a capacity of 1000 KN. The control beams and the retrofitted beams were tested for shear. The testing procedures for all the specimens are same. The beams were subjected to third point loading to determine their load carrying capacity. This load case was chosen because it gives constant maximum moment and zero shears in the sections between the loads, constant maximum shear force between support and load. The moment was linearly varying between supports and load. The span between the supports was 970 mm and the load was applied at points dividing the length into three equal parts as shown in Fig. 3.1(a). Steel plates were used under the loads to distribute the load over the width of the beam. Deflection measuring gauge was used to measure the deflection at one third span, mid span and two third points as shown in Fig. 3.1(c). Deflections at one third, two third and mid span corresponding to load at 10 KN intervals were recorded during the test using deflection measuring gauges. Figure 3.1 shows the experimental setup of a beam.

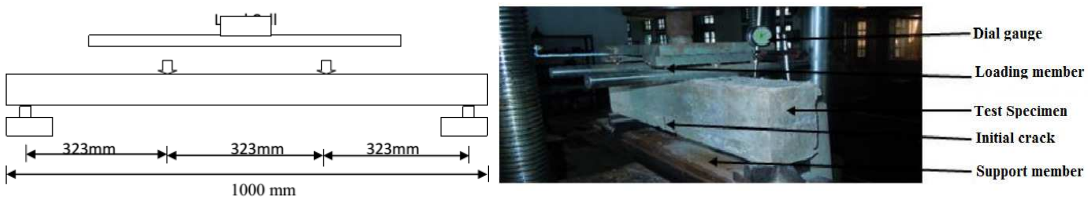


Figure 3: Experimental setup for testing of beams

3. NUMERICAL ANALYSIS

The model developed using ANSYS is capable of predicting shear capacity of concrete materials. It loaded as controlled beam to find out self-load carrying capacity and ultimate cracking load of beam.

3.1 Beam Modelling

For modelling of concrete the ANSYS used an element named as Solid65 which is non-linear model of brittle material similar to concrete. It was an eight node solid isoperimetric element with three degrees of freedom at each node. LINK8 element is used to model rebar. SOLID65 is used for the 3-D modelling of solids with or without reinforcing bars (rebar). The solid is capable of cracking in tension and crushing in compression. In concrete applications, for example, the solid capability of the element may be used to model the concrete while the rebar capability is available for modelling reinforcement behaviour. On the other hand, LINK8 is a spar which may be used in a variety of engineering applications. This element can be used to model trusses, sagging cables, links, springs, etc. The 3-D spar element is a uniaxial tension-compression element with three degrees of freedom at each node: translations in the nodal x, y, and z directions. The geometry, node locations, and the coordinate system for this element are shown in Figure 4 "LINK8 Geometry". The element is defined by two nodes, the cross-sectional area, an initial strain, and the material properties. The element x-axis is oriented along the length of the element from node I toward node J. The initial strain in the element (ISTRN) is given by Δ/L, where Δ is the difference between the element length, L, (as defined by the I and J node locations) and the zero strain length.

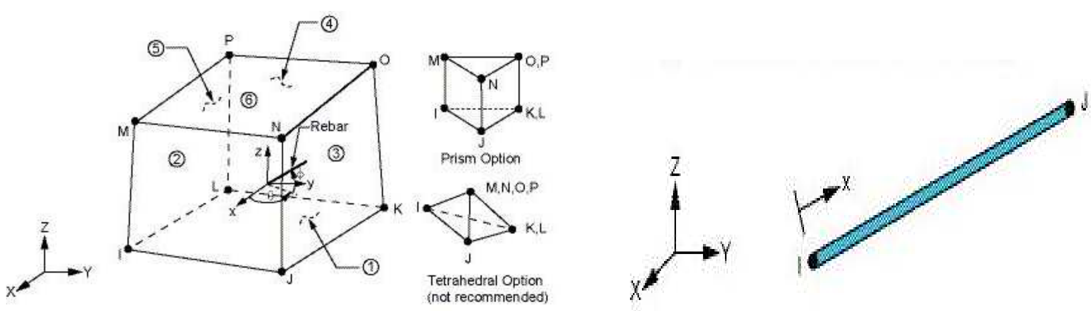


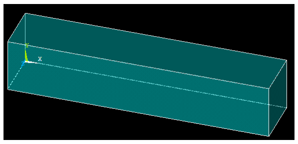
Figure 4: SOLID65 geometry and LINK8 geometry

3.2 ANSYS Modelling

In this program a model is developed by main three steps.

3.2.1 Preprocessing

In this step we have to define the problem. At first it has defined as the problem is a structural problem. The volume is created defining L=1000mm, B=203mm, W=203mm. Then define element SOLID65 for concrete and LINK8 for steel. The volume is mashes attributing SOLID65. LINK8 is attributed at defined node point. After giving material properties for steel and concrete such as elastic modulus E_b, ultimate uniaxial compressive strength f_c, steel grade, ultimate uniaxial tensile strength (modulus of rupture) f_r, Poisson's ratio for both steel and concrete, shear transfer coefficient, and compressive uniaxial stress-strain relationship the key points for the bars are specified to have the shear stirrups. The steel bar and stirrup formed using the selected node point and is selected and copied at required spacing and at required number of times. So finally the beam is modelled as shown in figure. After modelling RC beam the layer to be generate by volume option around the beam. The layer to be meshes by swipe for zone mesh and ferrocement properties to be attributed. The nodes of layer at support line to be constrained. The inner surface nodes of layer and the outer surface nodes of RC beam are selected and the contact pair is generated by contact wizard shown in figure 5.



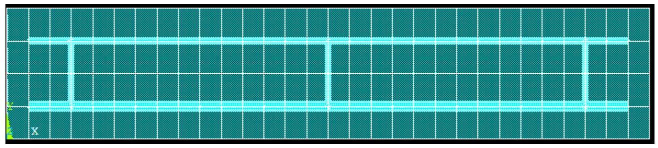


Figure 5: Concrete model and Concrete with steel bar model

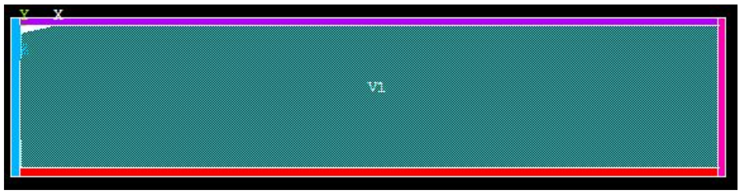
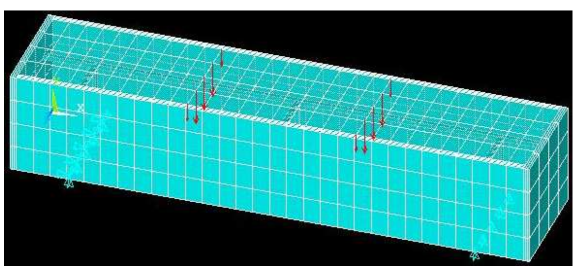


Figure 6: Retrofitted beam model with 12mm layer

3.2.2 Solution

After modelling the RC and retrofitted beam, we were assigned the loads. At the support points are constraints (translational and rotational) and finally solve the resulting set of equations. In nonlinear analysis, the total load applied to a finite element model is divided into a series of load increments called load steps. At the completion of each incremental solution, the stiffness matrix of the model is adjusted to reflect nonlinear changes in structural stiffness before proceeding to the next load increment. The ANSYS program uses Newton–Raphson equilibrium iterations for updating the model stiffness. In this study, for the reinforced concrete solid elements, convergence criteria were based on force and displacement, and the convergence tolerance limits were initially selected by the ANSYS program. For the nonlinear analysis, automatic time stepping in the ANSYS program predicts and controls load step sizes. Based on the previous solution history and the physics of the models, if the convergence behaviour is smooth, automatic time stepping will increase the load increment up to a selected maximum load step size. If the convergence behaviour is abrupt, automatic time stepping will bisect the load increment until it is equal to a selected minimum load step size. The maximum and minimum load step sizes are required for the automatic time stepping. Solution controls define weather the problem is linier or nonlinear. Here the main parameter is considering the problem as large displacement static.



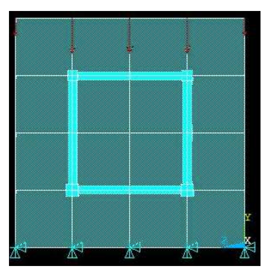


Figure 6: Isometric view and Side view of constrained model

3.2.3 Postprocessing

In this stage we got our required data such as lists of nodal displacements, element forces and moments, deflection plots, stress contour diagrams etc.

4. RESULTS

Load–deflection behavior of concrete structures typically includes three stages. In stage-I manifests the linear behaviour of uncracked elastic section. Then, Stage-II implies initiation of concrete cracking and Stage-III relies relatively on the yielding of steel reinforcement sand the crushing of concrete. In Nonlinear iterative algorithms, ANSYS8.0 utilizes the Newton–Raphson method for the incremental load analysis. The full nonlinear load–deformation response calculated using FE Analysis is plotted. A large amount of data has obtained from this analysis and only those relevant to this paper are presented here. The current evaluation utilizes data obtained from load steps and general postprocessor. In figure 7, from numerical analysis, NRCB i.e. numerical RC beam curve includes a linear response up to the load 38KN. The first appearance of a crack was noted at load 40.2KN. The mid span load deflection curve illustrates the nonlinearities of the concrete. The maximum load is shown in figure is 108.2KN. Here the maximum displacement has observed that is 11.8mm. On the other hand, from experimental analysis, ERCB i.e. experimental RC beam curve initial crack load and final crack load was found 51KN and 99.25KN respectively. On the contrary, In figure 8, the numerical load-deflection behaviour of retrofitted beam was describe by NFRCB curve and the experimental load-deflection behaviour of retrofitted beam was describe by EFRCB curve.

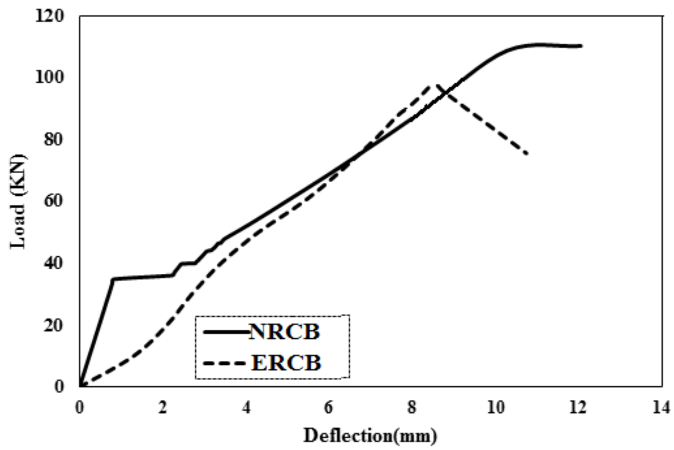


Figure 7: Comparison load-deflection behaviour of RC beam

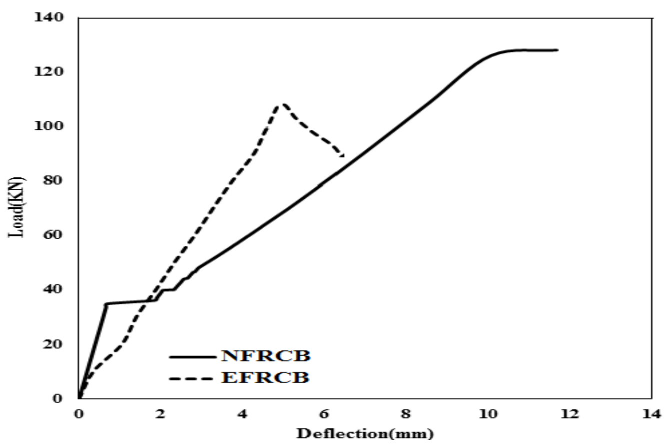


Figure 8: Comparison load-deflection behaviour of Retrofitted RC beam

Table 1: Comparison for Initial Cracking Load and Ultimate Cracking Load

Beam Designation	Control Beam		Retrofitted Beam		Percentage (%) Increase	
	Initial Crack Load (KN)	Ultimate Crack Load (KN)	Initial Crack Load (KN)	Ultimate Crack Load (KN)	Initial Crack Load (KN)	Ultimate Crack Load (KN)
Comparison with EFRCB	51	99.25	53.25	109	4.4	9.82
Comparison with NFRCB	40.2	108.3	41.4	126.19	2.8	16.67

Figure 9 and figure 10 illustrated the Initial crack behaviour and Ultimate crack behaviour of NRCB and NFRCB respectively.

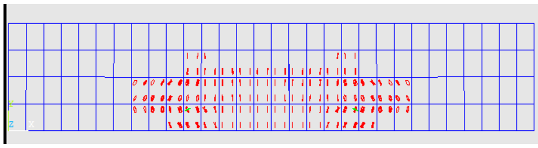
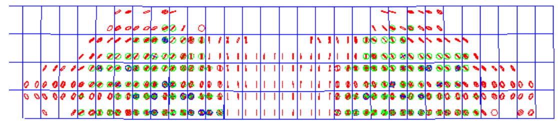



Figure 9: Initial crack behaviour and Ultimate crack behaviour of NRCB

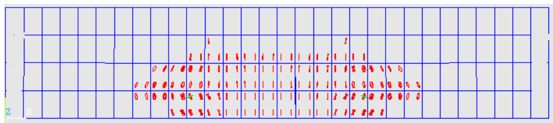
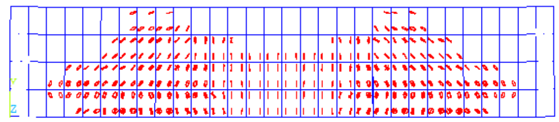



Figure 10: Initial crack behaviour and Ultimate crack behaviour of NFRCB

5. CONCLUTIONS

Based on the test results obtained from the analysis work, the following conclusions may be drawn out

- The failure of composite is characterized by development of shear and flexural cracks over the tension zone.
- The spacing of cracks is reduced for beams retrofitted with ferrocement layer for different stress levels indicating better distribution of stress.
- From experimental study, the initial crack forming load for the beams retrofitted with 12mm increased 4.4% and ultimate shear load carrying capacity increased 9.82% when compared with control beam.
- From numerical study, the initial crack forming load for the beams retrofitted with 12mm increased 2.8% and ultimate shear load carrying capacity increased 16.67% when compared with control beam.
- With the increase in load, the number of crack increase and spread over the entire length of the Beam.
- In a retrofitted beam the crack initiate at the main beam first.

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REFERENCES

Tumar, A.(2006), "Retrofitting Of Shear Deficient R.C. Beams Using Ferrocement Laminates", M.Sc. Thesis, Thapar Institute Of Engineering and Technology, Patiala.

Andrews, G. and Sharma, A.K., (1998), “Repaired Reinforced Concrete Beams” ACI, Concrete International, pp. 47-50.

Kaushik, S.K. and Dubey, A.K., (1994)“ Performance Evaluation of RC Ferrocement Composite Beams” Proceedings of Fifth International Symposium, UMIST, pp 240- 256, 1994 International Journal of Engineering, Volume (2) : Issue (1).

Nassif, H.H., Chirravuri, G. and Sanders, M.C., (1998), “ Flexural Behavior of Ferrocement / Concrete Composite Beams”, Proceedings of the Sixth International Symposium on Ferrocement, Universty of Michigan, Michigan, USA, pp. 251 – 258.

Vidivelli, B., Jeyasehar, C.A., and Srividya, P.R., (2001), “ Repair and Rehabilitation of Reinforced Concrete Beams by Ferrocement” Seventh International Symposium on Ferrocement and Thin Reinforced Cement Composites, National University of Singapore, 27-29, pp. 465- 471.

Bansal, P.P., Kumar, M. and Kaushik, S.K., (2008), “Effect of Wire Mesh Orientation on Strength of Beams Retrofitted using Ferrocement Jackets”, International Journal of Engineering (IJE), Volume - 2 Issue – 1, pp. 8-19, CSC Journals, Kuala Lumpur, Malaysia.

Kaushik .S.K. and Garg.V.K., (1994), “Rehabilitation of RC Beams Using Precast Ferrocement Bonded Plates” Department of Civil Engineering ,University of Roorkee.

Rafeeqi, S.F.A, Lodi, S.H., Wadalawala, Z.R. (1998), “Behavior of Reinforced Concrete Beams Strengthened in Shear “ Prof and Co Chairman ,Dept of Civil, NED Univ. of Engg. and Tech , Karachi(Pakistan).

Nassif, H.H. and Najm, H, (2004), “Experimental and analytical Investigation of Ferrocement Concrete Composite”, International Journal of Cement Composites and Lightweight Concrete; Vol. 26, pp.- 787-796.