



SEISMIC RETROFITTING OF EXTERIOR BEAM COLUMN JOINT USING FERROCEMENT

K.Ravichandran, C. Antony Jeyasehar

Civil and Structural Engineering, Annamalai University, Annamalainagar-608002

Accepted: 4 September 2012

Abstract

Evidences from previous earthquakes have shown that failure in beam column joints may cause disastrous collapse of structures, especially for building without seismic provisions. To extend the life span of beam column joints, strengthening is required and this can be effectively achieved using ferrocement. Ferrocement is a composite material of weld mesh and woven mesh embedded in mortar with various volume fractions. It is bonded to the beam column joint for strengthening. In this study, eight full scale reinforced concrete exterior beam column joints, two control specimen and six specimens strengthened by the proposed method, were constructed and tested under cyclic loading. The displacement is increased monotonically using a hydraulic push and pull jack. The hysteretic curves of the specimen have been plotted. The energy dissipation capacity of retrofitted beam column joints with various ferrocement configurations has been compared. In addition, comparisons were made between experimental and analytical results of control specimen and ferrocement retrofitted specimen. The experimental result indicates that the proposed strengthening method is effective to enhance the ultimate loading capacity, stiffness and energy dissipation.

Keywords: Beam column joint; Ferrocement; retrofitting; Volume fraction, Energy dissipation, Cyclic loading; Hysteretic curves.

1. INTRODUCTION

Beam column joints in reinforced concrete framed structure have been recognized as critical members transferring forces and bending moments between beams and columns. The change of moments in beam and columns across the joint region, under horizontal loading, induces high shear force in the joint as compared with adjacent members. This also aggravates bonding conditions for the longitudinal reinforcement of beams and columns in the joint region. Therefore, shear failure and bonding deterioration were traditionally considered as primary failure modes of beam column joints in moment resisting framed structures. Evidences from previous earthquakes have shown that failure in beam column joints may cause disastrous collapse of structures.

2. LITERATURE SURVEY

Although hundreds of thousands of successful reinforced cement concrete framed structures are annually constructed worldwide, there are large numbers of them that deteriorate, or become unsafe due to changes in loading, changes in use, or changes in configuration. Occurrence of natural calamities may also lead to review of engineering notations that make reworking of existing structures inevitable. The reworking is variously referred to as repair, rehabilitation, retrofitting and upgradation with distinct meaning attached to all these terminologies. The RCC beam column joints may require upgradation due to deficient detailing of reinforcing bars, insufficient column sections or due to increased loading on the structure. Upgradation of these

joint however is a challenging task that poses major practical difficulties. A variety of techniques have been applied to joint with most common ones being the construction of RCC and steel jackets. Sheela et al [1] suggest that the use of Ferrocement laminates in the joint region is one of the most effective way of strengthening system. Tsonos et al [2] suggested that the use of crossed inclined bars in the joint region is one of the most effective ways to improve the seismic resistance of exterior reinforced concrete beam column joints. Murty et al [3] have tested the exterior beam column joint subjected to static cyclic loading by changing the anchorage detailing of main reinforcement and shear reinforcement. The authors reported that the practical joint detailing using hairpin-type reinforcement is a competitive alternative to closed ties in the joint region. Jing et al [4] conducted experiment on interior joints by changing the beam reinforcement detailing pattern at the joint core. Diagonal steel bars in the form of “obtuse Z” were installed in two opposite direction of the joint. The authors found that the non-conventional pattern of reinforcement provided was suitable for joints in regions of low to moderate seismicity. Lakshmi et al [5] have developed analytical modeling of beam column joint subjected to cyclic loading by using ANSYS. Survey of existing constructions reveals that upgradation of structures is necessary due to the structure is inadequately designed for the present loading conditions and the structure is inadequately detailed for the present loading. This also includes those structures that are found deficient under seismic conditions. In this work, an attempt has been made to improve the confinement of core concrete without congestion of reinforcement in joints. The performance of exterior joint assemblages detailed for earthquake loads as per IS 13920:1993 [6] and detailed as per current Indian code of practice for concrete design IS 456:2000 [7] are compared with the retrofitted specimens. The experimental results are validated with the analytical model developed using finite element software package ANSYS.

3. EXPERIMENTAL INVESTIGATION

3.1 Ferrocement

Ferrocement is a composite material consisting of rich cement mortar matrix uniformly reinforced with one or more layers of very thin wire mesh with or without supporting skeletal steel. American Concrete Institute Committee 549 has defined ferrocement in broader sense as a type of thin wall reinforced concrete commonly constructed of hydraulic cement mortar, reinforced with closely spaced layers of continuous and relatively small diameter mesh. The mesh may be metallic or may be made of other suitable materials. Ferrocement possesses a degree of toughness, ductility, durability, strength and crack resistance which is considerably greater than that found in other forms of concrete construction. These properties are achieved in the structures with a thickness that is generally less than 25 mm, a dimension that is nearly unthinkable in other forms of construction and a clear improvement over conventional reinforced concrete. One can certainly call it a high technology material.

3.2 Experimental study of Ferrocement laminates

Preliminary experimental study was conducted to determine the elastic property for volume fraction of 1.38, 2.76 and 4.04. Ferrocement laminates of size 125 x 25 x 500 mm were cast with volume fraction of 1.38, 2.76 and 4.04. In the present investigation one layer consisting of weld mesh and woven mesh having volume fraction of 1.38 percent were used. The strength parameters are given in Table 1. The fiber mats kept in the mould and were grouted; the cement sand was mixed in a mortar mixer with super plasticizer for improving workability. Mixing

ratio of the cement and sand is given in Table 2. The modulus of elasticity for three different volume fractions is given in Table 3.

Table 1. Weldmesh and woven mesh Performance

Types of mesh	Wire diameter	Size of mesh openings	Cross sectional area of each wire	Ultimate tensile strength of reinforcement
Weldmesh	1.382 mm	20 mm	1.5 mm ²	476.86 N/mm ²
Woven mesh	0.798 mm	5 mm	0.5 mm ²	687.20 N/mm ²

Table 2 . Typical mortar composition

Sand/ cement	Water/ cement ratio	Superplastiziers/cement
0.66	0.30	0.025

Table 3. Composite properties

Modulus of elasticity Vf 1	Modulus of elasticity Vf 2	Modulus of elasticity Vf 3
0.67 X 10 ⁵ N/mm ²	1.23 X 10 ⁵ N/mm ²	1.82 X10 ⁵ N/mm ²

3.3 Casting of RC beam column joint

The beam column joint consisted of both column and beam 230×230 mm size. Eight specimens were cast out of which four are based on non ductile (Type A) and remaining four based on ductile detailing (Type B). In each case one specimen was considered as control specimen. All the Type A specimens had identical dimensions and were reinforced such that they would represent non-ductile detailed exterior joint of RC frame as per IS 456-2000 code recommendations. Reinforcement consists of four 12 mm diameter rebars in the column, two 12 mm diameter rebars in each side of the beam and 8 mm stirrups at a spacing of 150 mm in the column and beam uniformly (Fig. 1). All the Type B specimens had identical dimensions and were reinforced such that they would represent ductile detailed exterior joint of RC frame as per IS 13920-1993 code recommendations. Reinforcement consists of four 12 mm diameter rebars in the column, two 12 mm diameter rebars in each side of the beam and 8 mm stirrups at a spacing of 100 mm in the column and beam at the non anchorage zone and 8 mm stirrups at a spacing of 75 mm at the anchorage zone (Fig. 2). A short description of the specimens is given in Table 4.

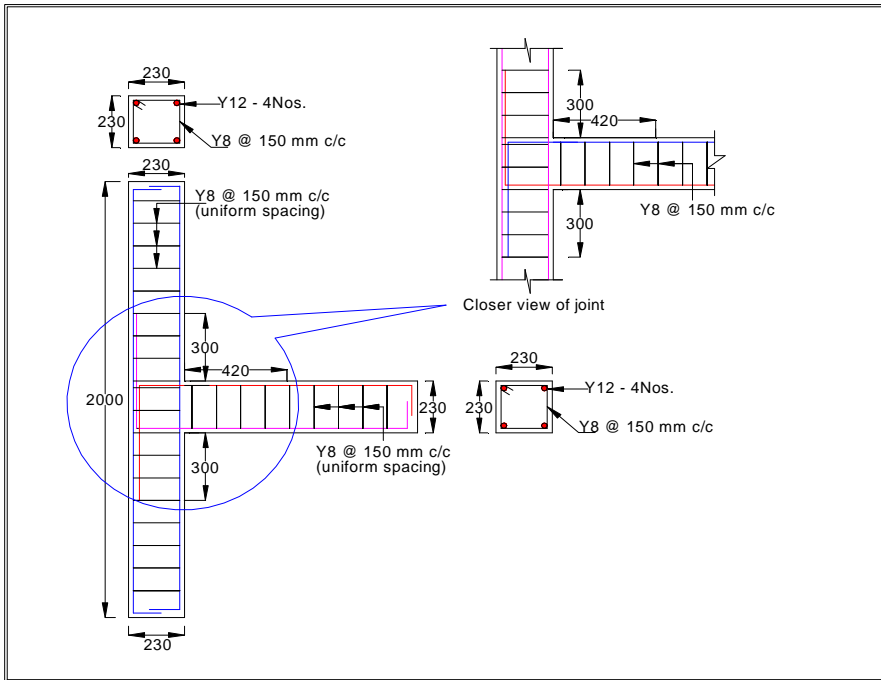


Fig. 1. Reinforcement Details for Non Ductile Joint (Type A)

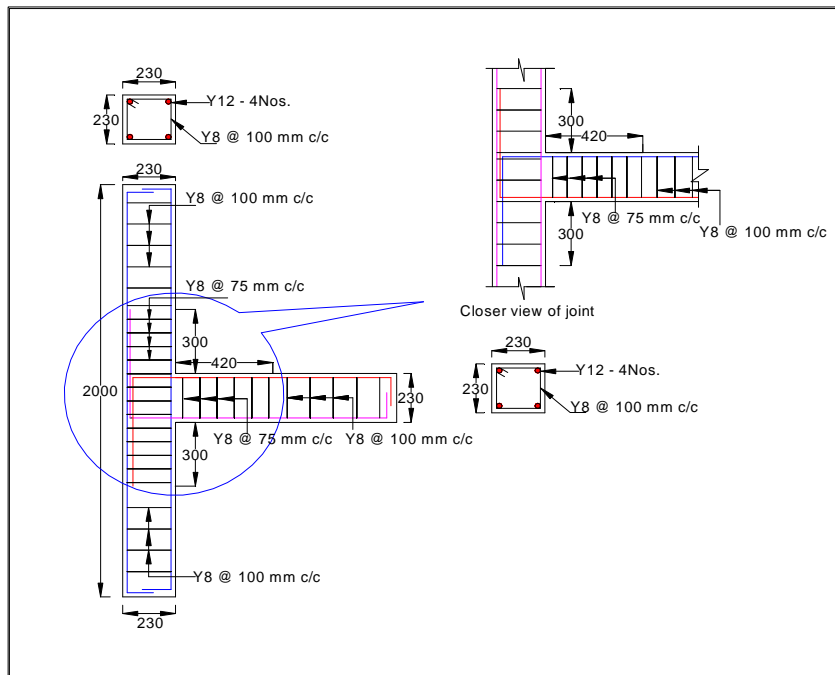


Fig. 2 Reinforcement Details for Ductile Joint (Type B)

Table 4. Description of test specimens

Sl. No.	Specimen Designation	Type of Reinforcement	Retrofitting Methodologies
1	NDA-1	non ductile	Control specimen
2	DDB-1	ductile	Control specimen
3	NDA-F1	non ductile	Ferrocement laminates, one layer consisting of weld mesh and woven mesh, volume fraction, 1.38.
4	NDA-F2	non ductile	Ferrocement laminates, two layer consisting of weld mesh and woven mesh, volume fraction, 2.76.
5	NDA-F3	non ductile	Ferrocement laminates, three layer consisting of weld mesh and woven mesh, volume fraction, 4.04.
6	DDB-F1	ductile	Ferrocement laminates, one layer consisting of weld mesh and woven mesh, volume fraction, 1.38.
7	DDB-F2	ductile	Ferrocement laminates, two layer consisting of weld mesh and woven mesh, volume fraction, 2.76.
8	DDB-F3	ductile </td <td>Ferrocement laminates, three layer consisting of weld mesh and woven mesh, volume fraction, .04.</td>	Ferrocement laminates, three layer consisting of weld mesh and woven mesh, volume fraction, .04.

3.4 Bonding of Ferrocement Laminates

Ferrocement laminates were used for externally strengthening the RC beam column joint and the laminate system has been cast in three parts as shown in Fig.3. After surface preparation, epoxy bonding systems were adopted to bond the laminates and bond line thickness of 2.0 mm was kept constant for all the test specimens.

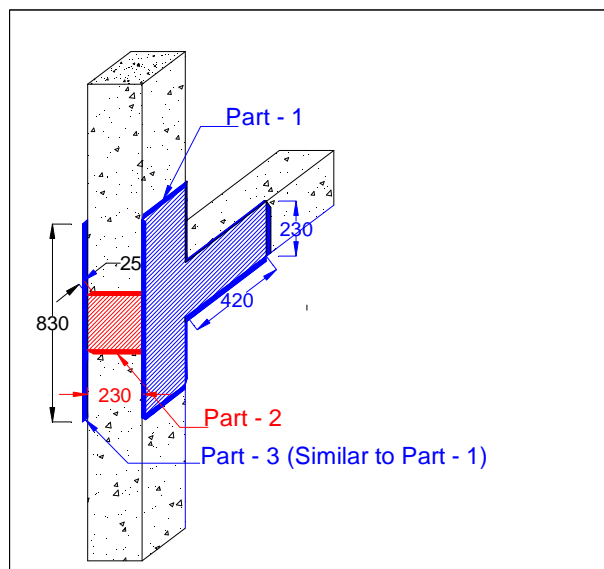


Fig. 3. Ferrocement Wrapping System

3.5 Testing Arrangement

The schematic view of the test set up is shown in figure 4. The joint assemblages were subjected to axial load and reverse cyclic loading. A 500 kN hydraulic jack mounted vertically to the loading frame was used for simulating the axial gravity load on the column. A constant axial load of 100 kN, which is about 20 percent of the axial capacity of the column was applied to the columns for holding the specimen in position and to simulate column axial load. Two ends of the column were given an external axial hinge support, in addition to two lateral hinge support provided at the bottom and top of the column. Another 500 kN capacity hydraulic push and pull jack was used to apply reverse cyclic load to the beam portion of the beam column joint. The point of application of the cyclic load was at 50 mm from the free end of the beam. The test was displacement controlled and the specimen was subjected to an increasing cyclic displacement up to the failure. The displacement increment was 5 mm, for push and pull for the test specimen. The specimens were instrumented with linear variable differential transducer having range ± 75 mm to measure the displacement at loading point.

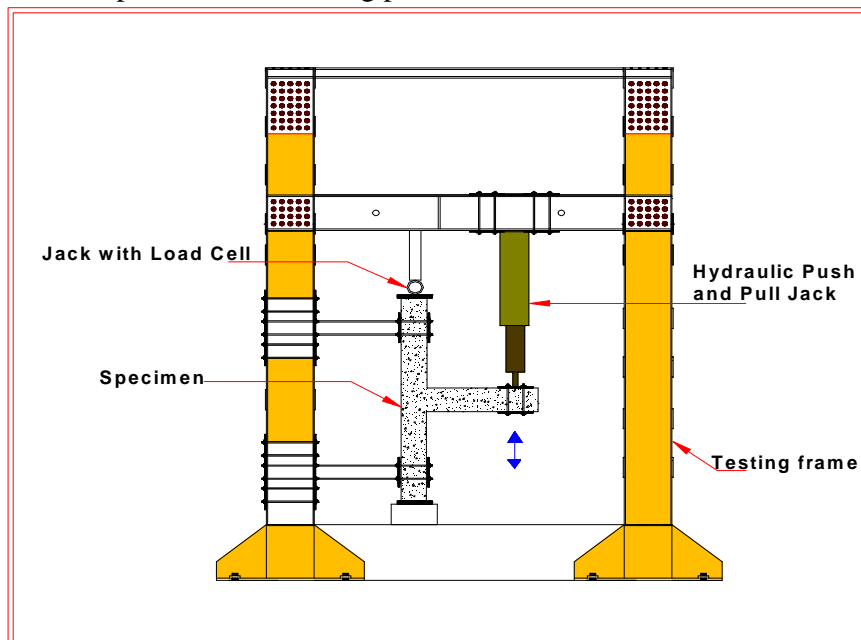


Fig. 4. Schematic View of Test set up

3.6 Beam Column Joint Control Specimen

Hysteresis behaviour of NDA-1 and DDB-1 specimen are shown in figure 5. For NDA-1, the maximum load observed is 25 kN in push and 18 kN in pull respectively and the specimen failed in 30 mm displacement. Based on the hysteresis behaviour energy dissipation and stiffness degradation per cycle are worked out. The total cumulative energy dissipation observed is 803.3kN mm (Table 5). The stiffness degraded from 2.9 kN / mm to 0.7 kN / mm. For DDB-1, the maximum load observed is 28.2 kN in push and 20.2 kN in pull respectively and the specimen failed in 35 mm displacement. The total cumulative energy dissipation observed is 1113.2 kN mm. The stiffness degraded from 3.4 kN / mm to 0.8 kN / mm. The increase in energy dissipation for ductile detailed specimen DDB-1 when compared to non ductile detailed specimen is 27.8 percent.

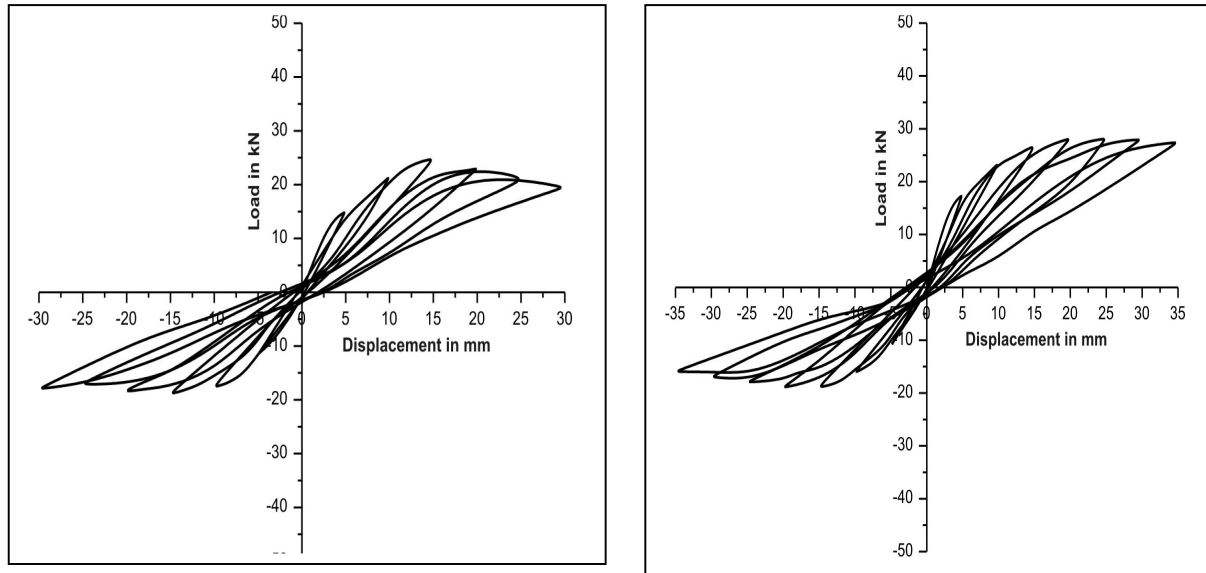


Fig. 5. Hysteresis behaviour of specimen NDA-1 and DDB-1

Table 5. Energy dissipation and stiffness at various displacements

Displacement mm	Energy dissipated (kN mm)		Stiffness (kN / mm)	
	NDA-1	DDB-1	NDA-1	DDB-1
5	20.6	21.4	2.9	3.4
10	80.4	83.3	2.3	2.3
15	179.9	206.2	1.7	1.8
20	334.1	356.9	1.2	1.4
25	550.3	582.2	0.9	1.1
30	803.3	839.3	0.7	0.9
35	-	1113.2	-	0.8

The cumulative energy dissipation and stiffness for NDA-1 and DDB-1 specimen are given in figure 6. The increase in energy dissipation of ductile detailed beam is 27.8 percent when compared with the non ductile detailed beam. The energy dissipation at first cycle of displacement for NDA-1 specimen is 20.6 kN mm and DDB-1 specimen is 21.4 kN mm. The stiffness at various cycle of loading, it can be seen that the stiffness degrades continuously in all the cycles. The stiffness at first cycle of displacement for NDA-1 specimen is 2.9 kN / mm and the DDB-1 specimen is 3.4 kN / mm.

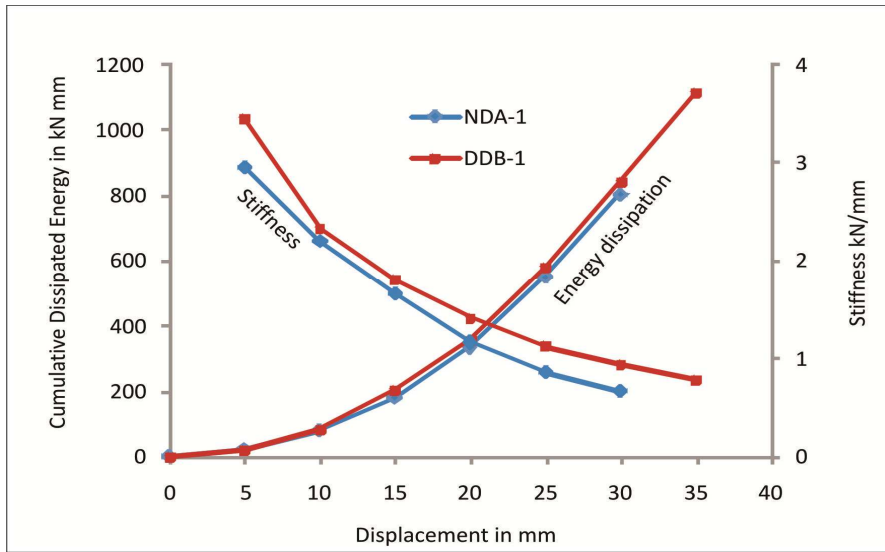
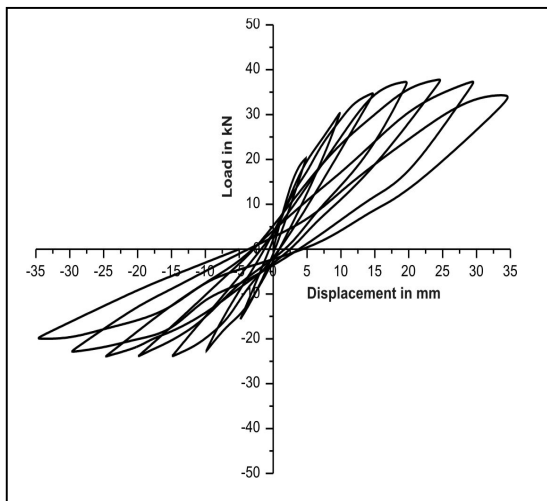


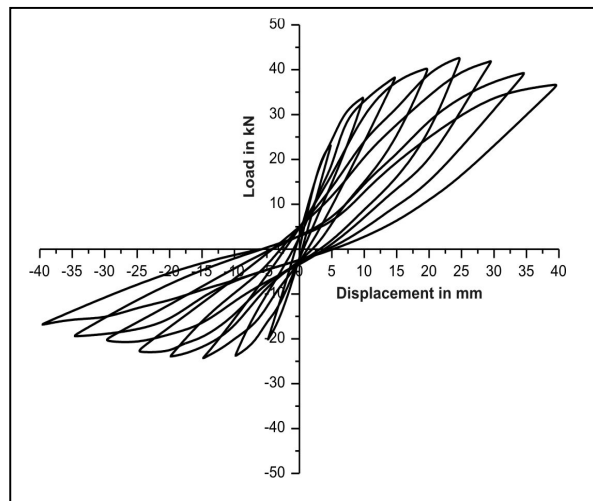
Fig. 6. Cumulative energy dissipation and stiffness Vs Displacement

3.7 Beam Column Joint Specimen Retrofitted with Ferrocement

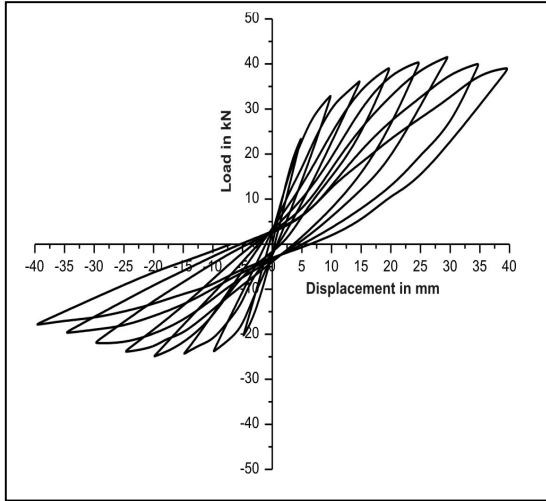
Hysteresis behaviour of ferrocement retrofitted specimen are shown in figure 7. For NDA-F1 the maximum load observed is 38.0 kN in push and 24.0 kN in pull respectively and the specimen failed in 35 mm displacement.



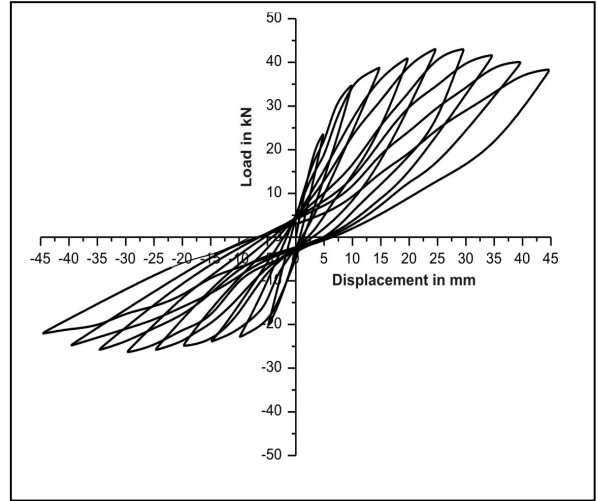
a) NDA-F1



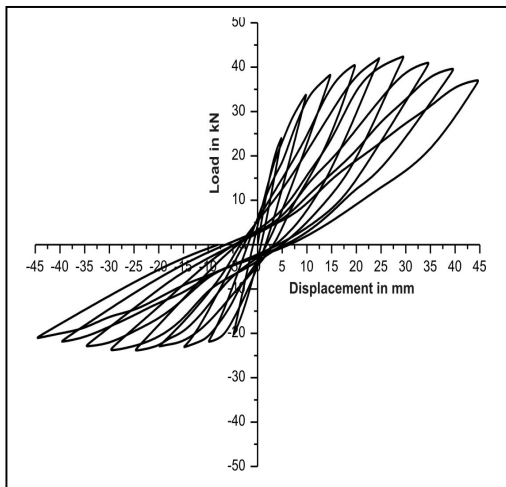
b) DDB-F1



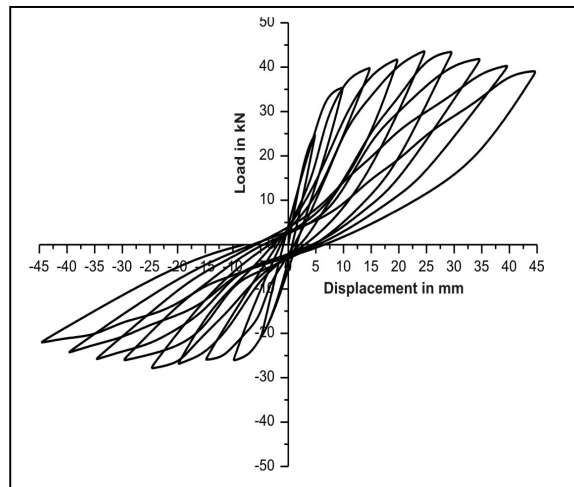
c) NDA-F2



d) DDB-F2



e) NDA-F3



f) DDB-F3

Fig. 7. Hysteresis behaviour of Ferrocement retrofitted specimen

Based on the hysteresis behaviour energy dissipation and stiffness degradation per cycle are evaluated and are given in Tables 6 and 7. The variation of cumulative energy dissipation and stiffness with displacement is shown in figures 8 and 9. The total cumulative energy dissipation observed is 1866.1 kN mm. The stiffness degraded from 4.1 kN / mm to 0.9 kN / mm. For DDB-F1, the maximum load observed is 42.9 kN in push and 25 kN in pull respectively and the specimen failed in 40 mm displacement.

Table 6. Energy dissipation at various displacements

Displacement mm	Energy dissipated (kN mm)							
	NDA-1	DDB-1	NDA-F1	DDB-F1	NDA-F2	DDB-F2	NDA-F3	DDB-F3
5	20.6	21.4	27.1	36.6	32.2	50.5	50.4	54.6
10	80.4	83.3	126.2	167.4	151.7	206.4	200.8	262.3
15	179.9	206.2	294.0	363.7	323.8	453.4	437.2	489.4
20	334.1	356.9	548.1	697.6	551.8	746.1	757.7	817.9
25	550.3	582.2	923.2	1161.2	940.7	1181.4	1173.5	1269.5
30	803.3	839.3	1407.1	1687.3	1419.2	1710.9	1655.5	1753.5
35	-	1113.2	1866.1	2194.5	1972.7	2246.6	2114.5	2289.6
40	-	-	-	2769.7	2570.8	2851.3	2607.3	2924.9
45	-	-	-	-	-	3495.9	3216.4	3621.2

Table 7. Stiffness at various displacements

Displacement mm	Stiffness (kN / mm)							
	NDA-1	DDB-1	NDA-F1	DDB-F1	NDA-F2	DDB-F2	NDA-F3	DDB-F3
5	2.9	3.4	4.1	4.7	4.8	4.8	4.9	5.0
10	2.3	2.3	3.1	3.4	3.3	3.5	3.4	3.6
15	1.7	1.8	2.3	2.6	2.4	2.6	2.6	2.7
20	1.2	1.4	1.9	2.0	1.9	2.1	2.0	2.1
25	0.7	1.1	1.5	1.8	1.6	1.7	1.7	1.8
30	0.7	0.9	1.3	1.4	1.4	1.4	1.4	1.5
35	-	0.8	0.9	1.1	1.1	1.2	1.2	1.2
40	-	-	-	0.9	1.0	1.0	1.0	1.0
45	-	-	-	-	-	0.9	0.8	0.9

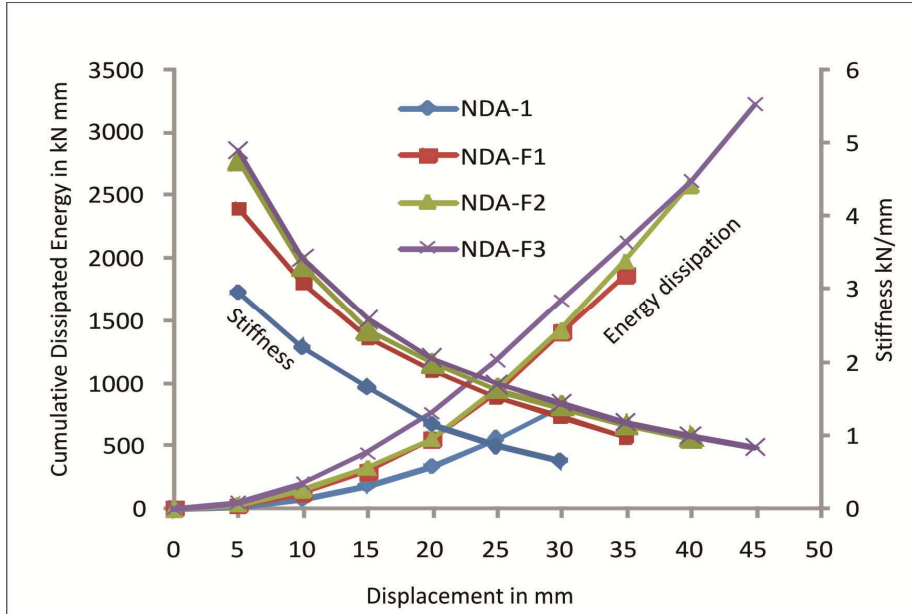


Fig. 8. Cumulative energy dissipation and stiffness Vs Displacement

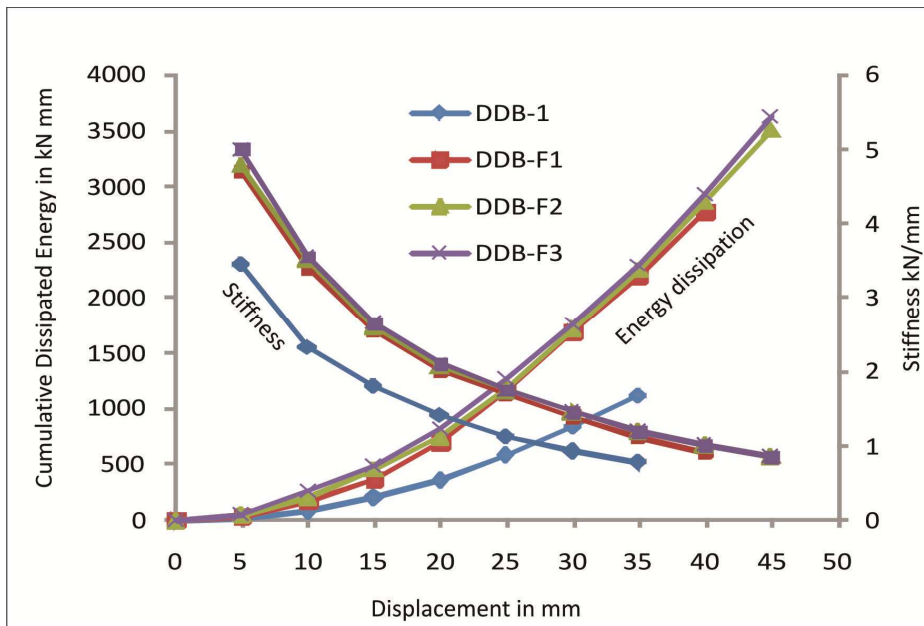


Fig. 9. Cumulative Energy dissipation and stiffness Vs Displacement

The total cumulative energy dissipation observed is 2769.7 kN mm. The stiffness degraded from 4.7 kN / mm to 0.9 kN / mm. For NDA-F2, the maximum load observed is 40.6 kN in push and 24.0 kN in pull respectively and the specimen failed in 40 mm displacement. The total cumulative energy dissipation observed is 2570.8 kN mm. The stiffness degraded from 4.8 kN / mm to 1.0 kN / mm. For DDB-F2 the maximum load observed is 43.4 kN in push and 26.0 kN in pull respectively and the specimen failed in 45 mm displacement. The total cumulative energy dissipation observed is 3495.9 kN mm. The stiffness degraded from 4.8 kN / mm to 0.9 kN /

mm. For NDA-F3, the maximum load observed is 42.6 kN in push and 24.8 kN in pull respectively and the specimen failed in 45 mm displacement. The total cumulative energy dissipation observed is 3216.4 kN mm. The stiffness degraded from 4.9 kN / mm to 0.8 kN / mm. For DDB-F3 the maximum load observed is 43.9 kN in push and 27.0 kN in pull respectively and the specimen failed in 45 mm displacement. The total cumulative energy dissipation observed is 3621.2 kN mm. The stiffness degraded from 5.0 kN / mm to 0.9 kN / mm. The increase in total cumulative energy dissipation for NDA-F1 when compared to NDA-1 is 56.9 percent, NDA-F2 is 68.7 percent and that for NDA-F3 is 75.0 percent. This shows ferrocement with volume fraction 4.04 performs better than ferrocement with other volume fractions. Further, the percentage increase in total cumulative energy dissipation for ferrocement retrofitted specimen is more than the percentage increase of energy dissipation in the case of ductile detailed specimen DDB-1 compared to non ductile detailed specimen NDA-1. This clearly indicates that ferrocement retrofitting can be used as a substitute for ductile detailing if it is absent in the existing structures. The increase in total cumulative energy dissipation for DDB-F1 when compared to DDB-1 is 59.8 percent, DDB-F2 is 68.1 percent and that for DDB-F3 is 69.25 percent. This shows ferrocement with volume fraction 4.04 performs better than ferrocement with other volume fractions. Further, the increase in total cumulative energy dissipation clearly indicates that the ferrocement retrofitting is an effective methodology for retrofitting of existing ductile detailed structures if the seismic zone is upgraded.

3.8 Crack pattern of control specimen

The crack patterns of all the tested specimen are shown in figure 10. All the specimen failed in the beam portion, yielding of steel has been observed at the point of failure. Strain gauges are bonded in the beam portion, but these strain gauges are deboned at the reach of two cycles and the allowable deflection.



a) NDA 1



b) DDB-1



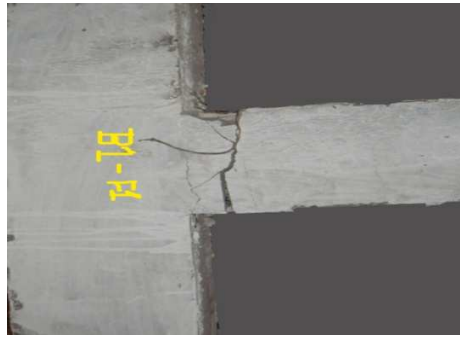
c) NDA-F1



d) NDA-F2



e) NDA-F3



f) DDB-F1



g) DDB-F2



h) DDB-F3

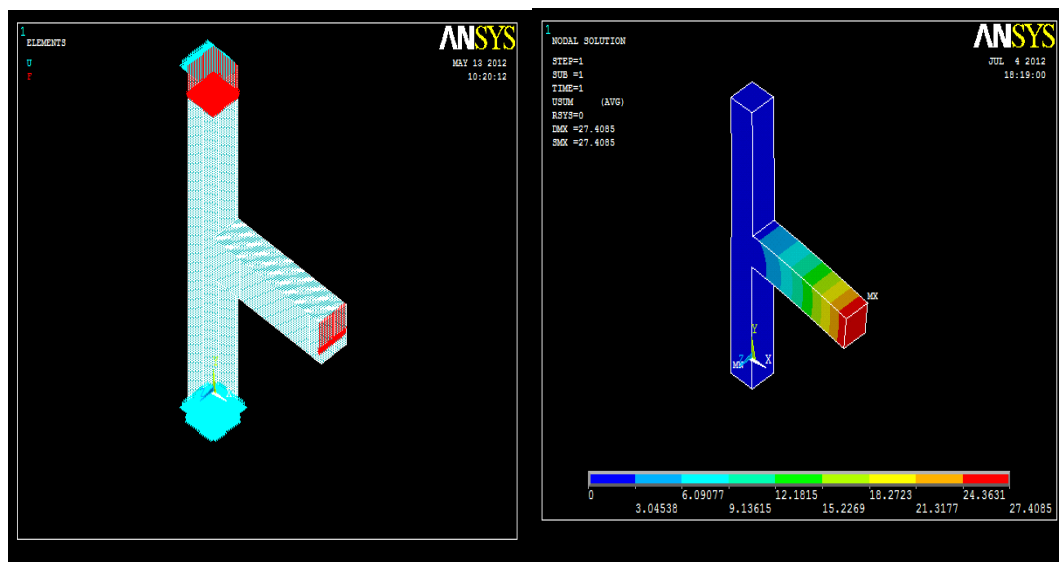
Fig. 10. Crack pattern of tested beam column joints

4 NUMERICAL ANALYSIS (ANSYS)

A full 3D finite element analysis has been carried out using ANSYS general purpose finite element software. The analysis presented in this paper assumes that the beam column joint is subjected to cyclic loading. Further experimental results are presented for beam column joint for comparison. The concrete has been modeled using eight noded solid element (SOLID 65) specially designed for concrete, capable of handling plasticity, creep, cracking in tension and crushing in compression. The characteristics of the adopted element being non linear, requires an iterative solution. In this analysis, the compressive strength of concrete (f_{ck}) is taken as 30.30 MPa and tensile strength of concrete (f_t) is considered as 3.5 MPa. The elastic modulus (E_s) is 25735 MPa. The reinforcing steel has been modeled using a series of two noded link element (LINK 8). The material properties associated with link elements include an initial yield stress 448 MPa. The ferrocement laminates has been modeled using eight noded multi layered sold element (SOLID 46).The material properties of ferrocement laminates, are listed in section3.2. The adhesive layer has been modeled using 3D isotropic elements (SOLID 45). The material property modulus of elasticity is (E_s) 1500 MPa.

4.1 Modeling of Beam-Column Joint

The boundary conditions were exactly simulated as in the test set up shown in figure 11 a. Horizontal and vertical restraints, representing a pin connection were applied at the top and bottom of the column. At the end of beams, only vertical displacement were provided to simulate the cyclic load conditions used in the test. A constant axial load of 100 kN was applied to top end of the column. The vertical displacement at the beam end was applied in a slowly increasing monotonic manner, with results recorded for every 5 mm vertical displacement up to failure. The deflected shape of the model is shown in figure 11 b.



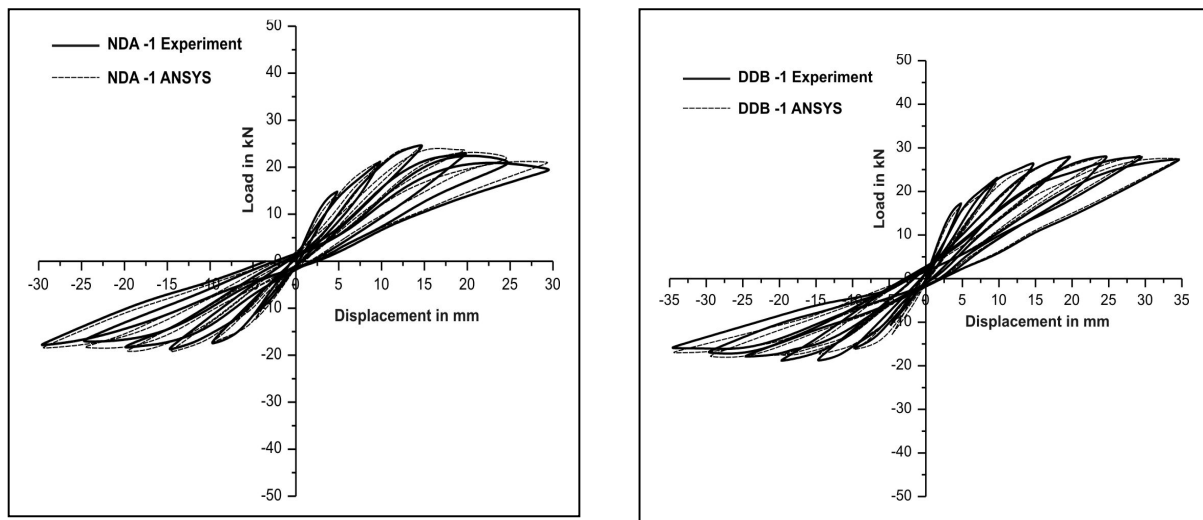
a) ANSYS model

b) Deflected shape

Fig. 11 ANSYS model and deflected shape of Beam column joint

4.2 Beam Column Joint- Control Specimen

Hysteresis behaviour of NDA-1 specimen obtained using ANSYS is shown in figure 12 along with the one obtained from experiment. The maximum load observed is 24 kN in push and 18.5 kN in pull respectively using ANSYS and 25 kN in push and 18.5 kN in pull respectively in the case of experiment and in both cases the specimen failed at 30mm displacement. Based on the hysteresis behaviour, energy dissipation and stiffness degradation per cycle are evaluated and are given in Table 8. The variation of cumulative energy dissipation and stiffness with displacement is shown in figure 13. The analytical value of total cumulative energy dissipation is 669.1 kN mm when compared to the experimental value of 803.3 kN mm. The stiffness degraded from 2.8 kN / mm to 0.6 kN / mm in the case of ANSYS and from 2.9 kN / mm to 0.7 kN / mm in the case of experiment. For DDB-1 specimen the maximum load observed is 27.5 kN in push and 19.5 kN in pull respectively using ANSYS and 28.2 kN in push and 20.2 kN in pull respectively in the case of experiment and in both cases the specimen failed at 35mm displacement. The variation of cumulative energy dissipation and stiffness with displacement for DDB-1 is shown in figure 14. The analytical value of total cumulative energy dissipation for DDB-1 is 954.5 kN mm when compared to the experimental value of 1113.2 kN mm. The stiffness for DDB-1 degraded from 3.4 kN / mm to 0.8 kN / mm in the case of experiment and analytical value from 3.3 kN / mm to 0.8 kN / mm. This indicates that the analytical behavior closely predicts the experimental behavior.



a)NDA-1

b) DDB-1

Fig.12 Hysteresis behaviour of specimen NDA-1 and DDB-1 (Experiment vs ANSYS)

Table 8 Energy dissipation at various displacements

Displacement mm	Energy dissipated (kN mm)				Stiffness (kN / mm)			
	NDA-1		DDB-1		NDA-1		DDB-1	
	Experiment	ANSYS	Experiment	ANSYS	Experiment	ANSYS	Experiment	ANSYS
5	20.6	17.3	21.4	18.0	2.9	2.8	3.4	3.3
10	80.4	69.5	83.3	71.8	2.3	2.1	2.3	2.2
15	179.9	163.8	206.2	175.4	1.7	1.6	1.8	1.7
20	334.1	287.4	356.9	304.2	1.2	1.2	1.4	1.4
25	550.3	451.9	582.2	489.5	0.9	0.8	1.1	1.1
30	803.3	669.1	839.3	696.4	0.7	0.6	0.9	0.9
35	-	-	1113.2	954.4	-	-	0.8	0.8

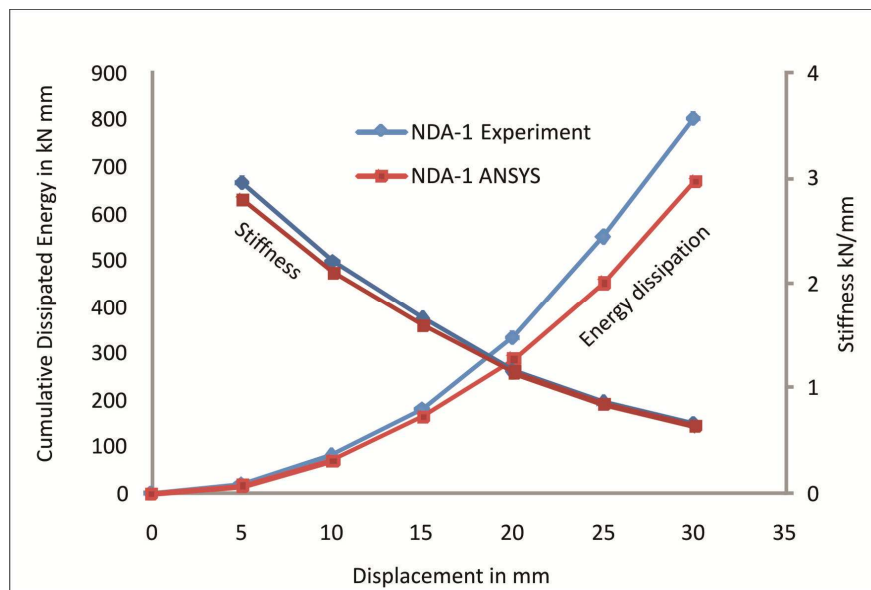


Fig. 13. Cumulative energy dissipation and stiffness vs Displacement

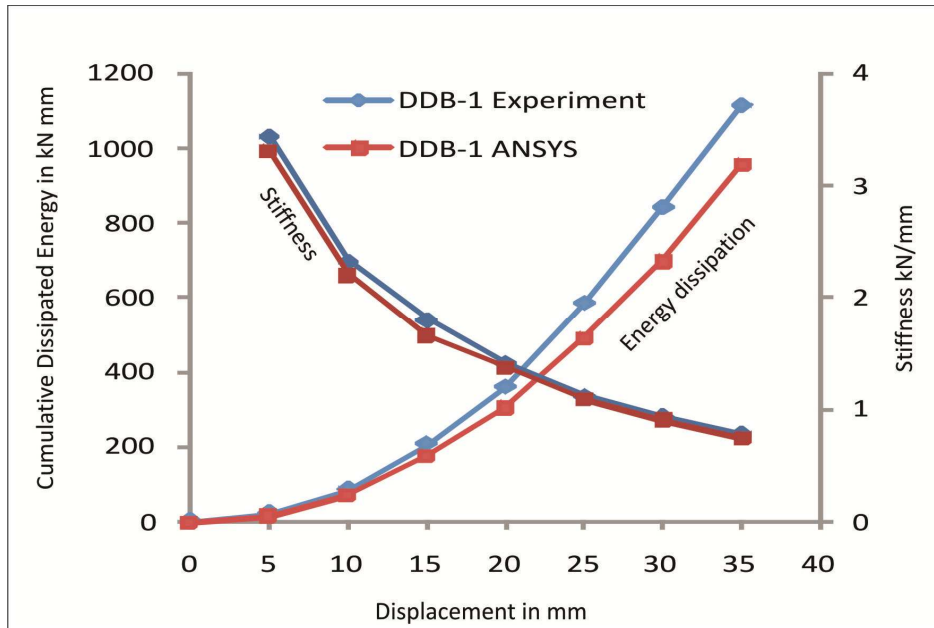
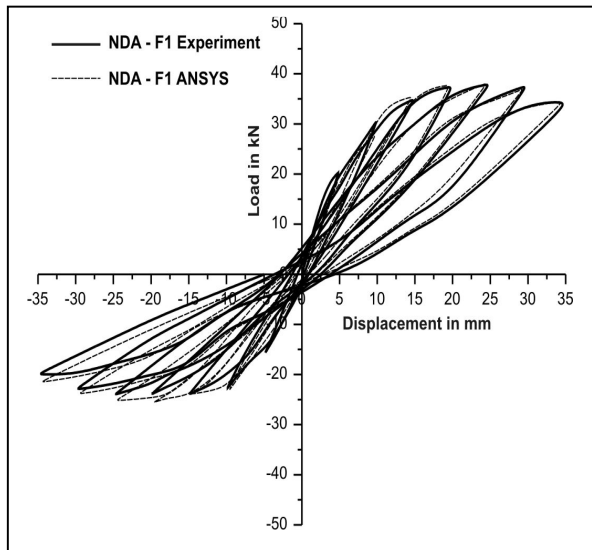


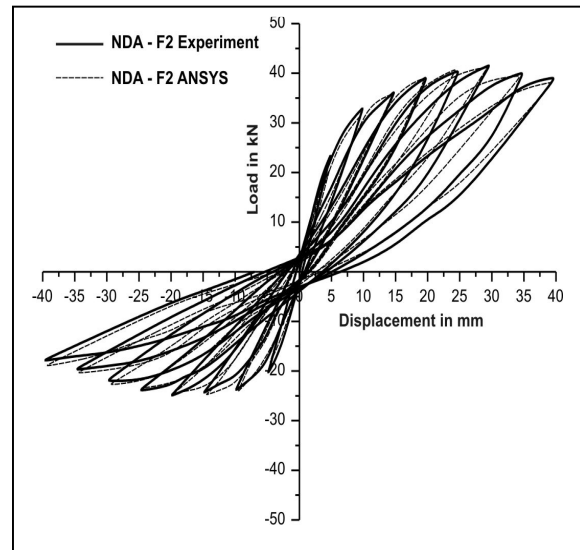
Fig. 14. Cumulative energy dissipation and stiffness vs displacement

4.3 Beam Column Joint- Ferrocement Retrofitted Specimen (Non ductile)

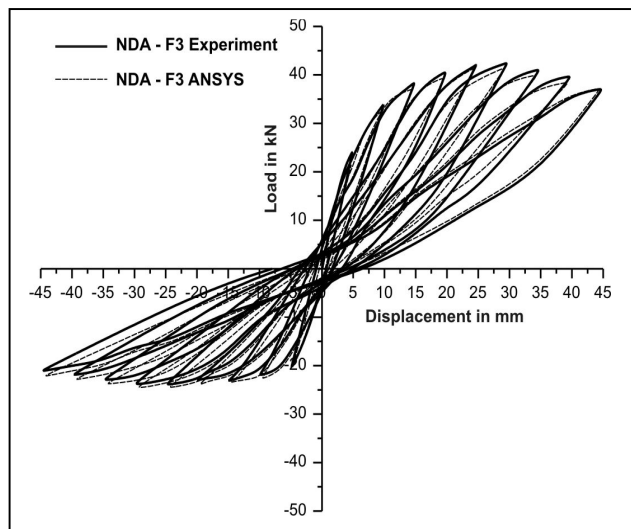
Hysteresis behaviour of NDA-F1 obtained using ANSYS is shown in figure 15 along with the one obtained from experiment. The maximum load observed is 37.5 kN in push and 23.5 kN in pull respectively using ANSYS and 38.0 kN in push and 24.0 kN in pull respectively in the case of experiment and in both cases the specimen failed at 35 mm displacement. Based on the hysteresis behaviour energy dissipation and stiffness degradation per cycle are evaluated and are given in Tables 9 and 10. The variation of cumulative energy dissipation and stiffness with displacement is shown in figure 16. The analytical value of total cumulative energy dissipation is 1514.8 kN mm. The stiffness degraded from 3.7 kN / mm to 0.9 kN / mm. The experimental value of total cumulative energy dissipation for NDA-F1 is 1866.1 kN mm. The stiffness of NDA-F1 degraded from 4.1 kN / mm to 0.9 kN / mm. For NDA-F2 the maximum load observed is 40.5 kN in push and 23.8 kN in pull respectively using ANSYS and 40.6 kN in push and 24.0 kN in pull respectively in the case of experiment and in both cases the specimen failed at 40 mm displacement. The analytical value of total cumulative energy dissipation is 1988.2 kN mm. The stiffness degraded from 4.2 kN / mm to 0.9 kN / mm. The experimental value of total cumulative energy dissipation for NDA-F2 is 2570.8 kN mm. The stiffness in NDA-F2 degraded from 4.8 kN / mm to 1.0 kN / mm. For NDA-F3 the maximum load observed is 41.0 kN in push and 24.2 kN in pull respectively using ANSYS and 42.6 kN in push and 24.8 kN in pull respectively in the case of experiment and in both cases the specimen failed at 45 mm displacement. The analytical value of total cumulative energy dissipation is 2501.8 kN mm. The stiffness degraded from 4.3 kN / mm to 0.8 kN / mm. The experimental value of total cumulative energy dissipation for NDA-F3 is 3216.4 kN mm. The stiffness in NDA-F3 degraded from 4.9 kN / mm to 0.8 kN / mm.



a)NDA-F1



b) NDA-F2



c) NDA-F3

**Fig. 15. Hysteresis behaviour of specimen NDA-F1,NDA-F2 and NDA-F3
(Experiment vs Analytical)**

Table 9. Energy dissipation at various displacements

Displacement mm	Energy dissipated (kN mm)							
	NDA-1		NDA- F1		NDA- F2		NDA- F3	
	Experimen t	ANSYS	Experimen t	ANSYS	Experiment	ANSYS	Experimen t	ANSYS
5	20.6	17.3	27.1	19.3	32.2	24.7	50.4	31.3
10	80.4	69.5	126.2	90.3	151.7	122.0	200.8	146.6
15	179.9	163.8	294.0	209.9	323.8	263.2	437.2	339.0
20	334.1	287.4	548.1	411.7	551.8	474.6	757.7	586.6
25	550.3	451.9	923.2	705.3	940.7	723.0	1173.5	884.4
30	803.3	669.1	1407.1	1121.5	1419.1	1121.8	1655.5	1227.1
35	-	-	1866.1	1514.8	1972.6	1520.9	2114.5	1581.9
40	-	-	-	-	2570.8	1988.2	2607.3	1996.9
45	-	-	-	-	-	-	3216.4	2501.8

Table 10. Stiffness at various displacements

Displacement mm	Stiffness (kN/ mm)							
	NDA-1		NDA- F1		NDA- F2		NDA- F3	
	Experiment	ANSYS	Experiment	ANSYS	Experiment	ANSYS	Experiment	ANSYS
5	2.9	2.8	4.1	3.7	4.8	4.2	4.9	4.3
10	2.3	2.1	3.1	2.8	3.3	3.2	3.4	3.3
15	1.7	1.6	2.3	2.3	2.4	2.4	2.6	2.5
20	1.2	1.2	1.9	1.9	1.9	1.9	2.0	1.9
25	0.9	0.9	1.5	1.5	1.6	1.6	1.7	1.6
30	0.7	0.6	1.3	1.2	1.4	1.4	1.4	1.5
35	-	-	0.9	0.9	1.1	1.1	1.2	1.1
40	-	-	-	-	1.0	0.9	1.0	0.9
45	-	-	-	-	-	-	0.8	0.8

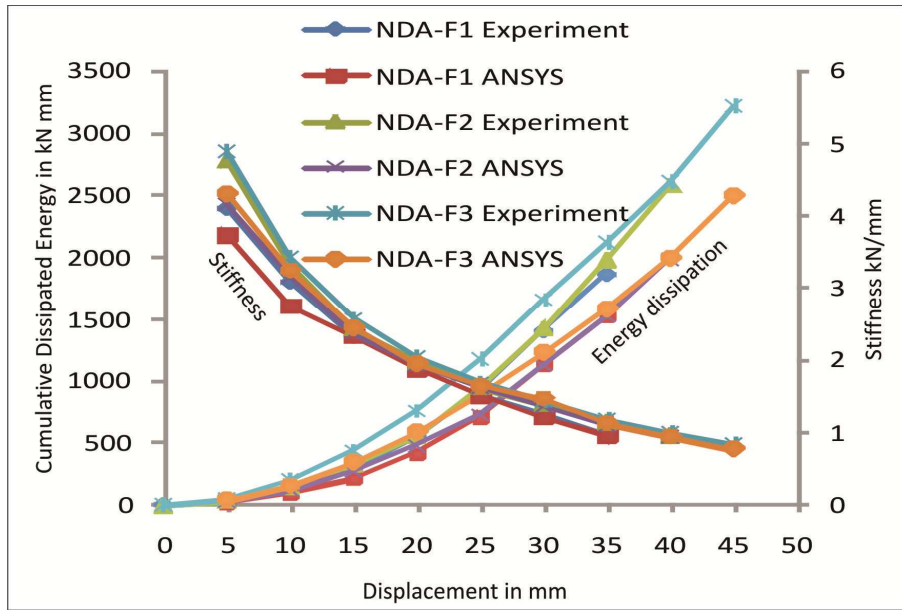
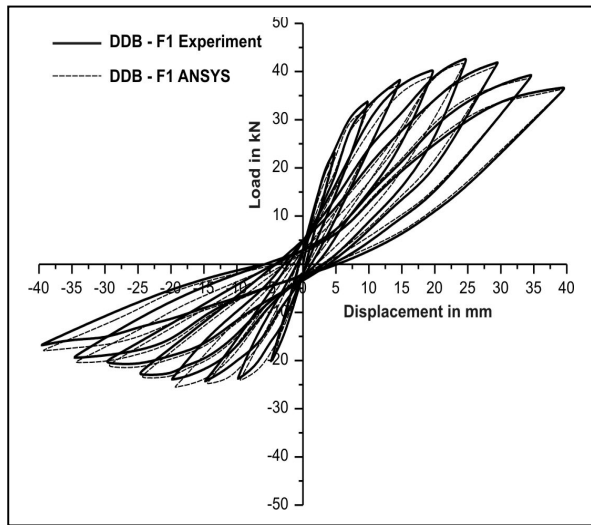


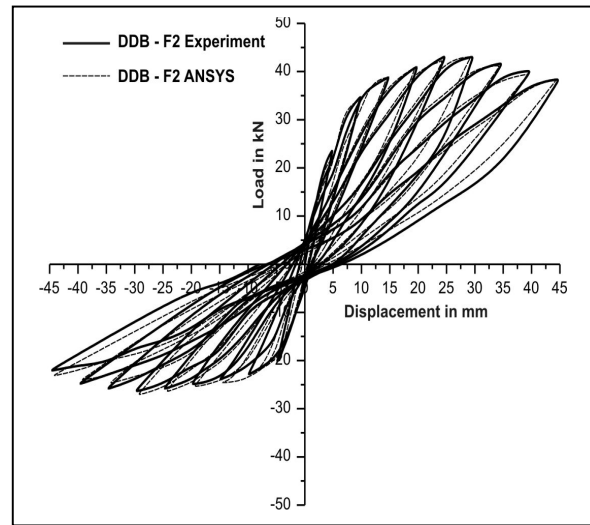
Fig. 16. Cumulative energy dissipation and stiffness vs displacement

4.4 Beam Column Joint - Ferrocement Retrofitted Specimen (Ductile)

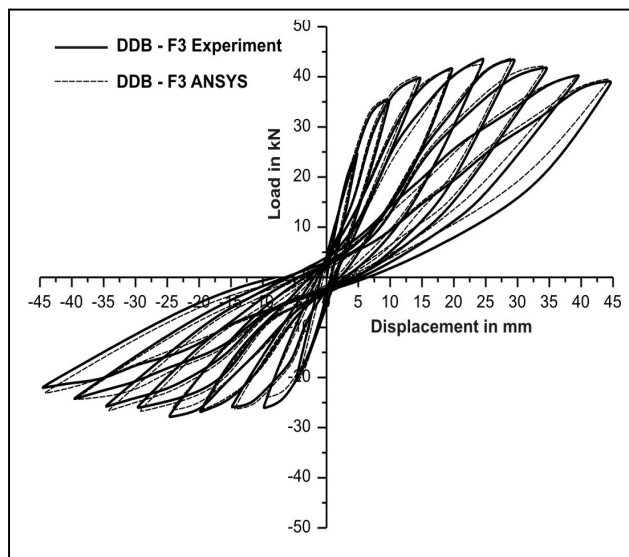
Hysteresis behaviour of DDB-F1 obtained using ANSYS is shown in figure 17 along with one obtained from experiment. The maximum load observed is 41.2 kN in push and 23 kN in pull respectively using ANSYS and 42.9 kN in push and 25 kN in pull respectively in the case of experiment and in both cases the specimen failed at 40 mm displacement. Based on the hysteresis behaviour energy dissipation and stiffness degradation per cycle are evaluated and are given in Tables 11 and 12. The variation of cumulative energy dissipation and stiffness with displacement is shown in figure 18. The analytical value of total cumulative energy dissipation is 2333.9 kN mm and the stiffness degradation from 4.4 kN / mm to 0.9 kN / mm. The experimental value of total cumulative energy dissipation for DDB-F1 is 2767.9 kN mm and the experimental stiffness of DDB-F1 degraded from 4.7 kN / mm to 0.9 kN / mm. For DDB-F2, the maximum load observed is 42.4 kN in push and 25.5 kN in pull respectively using ANSYS and 43.4 kN in push and 26 kN in pull respectively in the case of experiment and in both cases the specimen failed at 45 mm displacement. The analytical value of total cumulative energy dissipation is 2779.5 kN mm and the stiffness degradation from 4.6 kN / mm to 0.8 kN / mm. The experimental value of total cumulative energy dissipation DDB-F2 is 3495.9 kN mm. The experimental stiffness for DDB-F2 degraded from 4.8 kN / mm to 0.9 kN / mm. For DDB-F3, the maximum load observed is 43.2 kN in push and 26.2 kN in pull respectively using ANSYS and 43.9 kN in push and 27 kN in pull respectively in the case of experiment and in both cases the specimen failed at 45 mm displacement. The analytical value of total cumulative energy dissipation is 3159.7 kN mm and the stiffness degradation from 4.8 kN / mm to 0.8 kN / mm. The experimental value of total cumulative energy dissipation DDB-F3 is 3621.2 kN mm. The experimental stiffness for DDB-F3 degraded from 5.0 kN / mm to 0.9 kN / mm.



a) DDB-F1



b) DDB-F2



c) DDB-F3

Fig. 17. Hysteresis behaviour of specimen DDB-F1.DDB-F2 and DDB-F3
(Experiment vs analytical).

Table 11. Energy dissipation at various displacements

Displacement mm	Energy dissipated (kN mm)							
	DDB-1		DDB- F1		DDB -F2		DDB -F3	
	Experiment	ANSYS	Experiment	ANSYS	Experiment	ANSYS	Experiment	ANSYS
5	21.4	18.1	36.6	33.4	50.5	30.9	54.6	38.9
10	83.3	71.8	167.4	145.4	206.4	141.9	262.3	214.6
15	206.2	175.5	363.7	326.0	453.4	305.4	489.4	422.3
20	356.9	304.2	697.6	574.5	746.1	585.1	817.9	731.5
25	582.2	489.6	1161.2	901.7	1181.4	957.6	1269.5	1124.9
30	839.3	696.5	1687.3	1329.3	1710.9	1370.0	1753.5	1557.7
35	1113.2	954.5	2194.5	1781.2	2246.6	1818.9	2289.6	2046.8
40	-	-	2769.7	2333.9	2851.3	2294.5	2924.9	2594.0
45	-	-	-	-	3495.9	2779.5	3621.2	3159.7

Table 12 Stiffness at various displacements

Displacement mm	Stiffness (kN/ mm)							
	DDB-1		DDB-F1		DDB-F2		DDB-F3	
	Experiment	ANSYS	Experiment	ANSYS	Experiment	ANSYS	Experiment	ANSYS
5	3.4	3.3	4.7	4.4	4.8	4.6	5.0	4.8
10	2.3	2.2	3.4	3.2	3.5	3.4	3.6	3.6
15	1.8	1.7	2.6	2.5	2.6	2.6	2.7	2.7
20	1.4	1.4	2.0	1.9	2.1	2.0	2.1	2.0
25	1.1	1.1	1.8	1.6	1.7	1.7	1.8	1.7
30	0.9	0.9	1.4	1.4	1.4	1.4	1.5	1.4
35	0.8	0.8	1.1	1.1	1.2	1.2	1.2	1.2
40	-	-	0.9	0.9	1.0	0.9	1.0	0.9
45	-	-	-	-	0.9	0.8	0.9	0.8

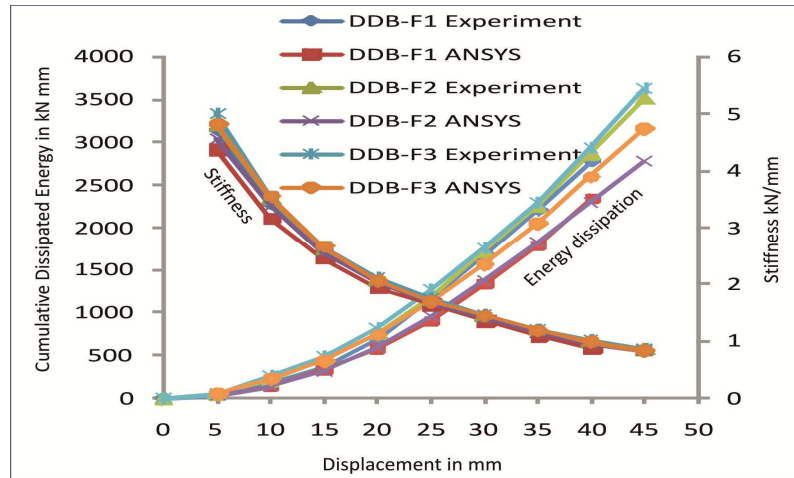


Fig. 18. Cumulative energy dissipation and stiffness vs displacement

5. CONCLUSIONS

Based on the experimental work and the numerical analysis, the following conclusions are drawn:

- (i) The composite materials ferrocement can be efficiently used for seismic retrofitting of reinforced beam column joint.
- (ii) The deficiency in cumulative energy dissipation in the case of non ductile reinforced beam column joint can be made good by ferrocement strengthening.
- (iii) The increase in cumulative energy dissipation having volume fraction of 4.04 is 75.6 percent for non ductile and 69.2 percent for ductile reinforced concrete beam column joint strengthened by ferrocement.
- (iv) ANSYS modeling closely predicts the experimental behavior of beam column joint.

6. REFERENCES

1. Sheela S, Anu Geetha B. Studies on the Performance of RC Beam-Column Joints strengthened using Different Composite Materials, Journal of Institution of Engineers India Section A, 93(2012) 63-71
2. Tsonos AG, Tegos IA, Penelis G. Gr. Seismic resistance of type 2 exterior beam column joints reinforced with inclined bars. ACI Structural journal, 89 (1992) 3-12.
3. Murty CVR, Rai DC, Baipai KK, Jain SK. Effectiveness of reinforcement details in exterior reinforced concrete beam column joints for earthquake resistance, ACI Structural journal, 100(2003)149-155
4. Jing LI, Pam HJ, Francis TK. New details of HSC beam column joints for regions of low to moderate seismicity. 13th World Conference on Earthquake Engineering, Vancouver, Canada, (2004) Paper No 449.
5. G.A. Lakshmi, Anjan Dutta, S.K. Deb. Numerical studies of strengthening of beam column joint under cyclic excitation using FRP composites. Journal of structural Engineering.

35(2008) 59-65

- 6 . IS 13920: 1993, Indian Standard Ductile detailing of reinforced concrete structures subjected To seismic Forces, (Bureau of Indian Standards), New Delhi, India, 2002.
7. IS 456:2000, Indian Standard Plain and Reinforced Concrete Code of Practice, (Bureau of Indian Standards), New Delhi, India, 2000.