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THEORETICAL AND EXPERIMENTAL INVESTIGATION OF RC BEAM-COLUMN JOINT

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ABSTRACT

A theoretical model was developed in this investigation to predict the behaviour of full scale RC beam-column joint, the beam of which was of size 2000 mm × 610 mm × 915 mm reinforced with 36 mm diameter bars in 24 numbers placed on both sides equally and the column of which was 2915 mm × 610 mm × 915 mm containing reinforcement of 4 numbers of 36 mm bars along with smaller bars of sizes 25 mm, 16 mm and 12 mm diameters, respectively. The joint was replicated in the laboratory and tested under monotonic load applied on the beam. Based on the deflected profile of the column and beam, the joint was theoretically analysed following the principle of mechanics and reinforced concrete theory and the load-deflection and load-curvature of the joint were predicted. The trend in behaviour of the joint was found to be in close agreement with that of the measured values and observed to correlate well with that reported by other investigators.

Keywords: RC beam-column joint; Theoretical modelling; Prediction; load-deflection relation; Moment curvature; Compressive stress distribution.

I. INTRODUCTION

The design of beam-column connections of reinforced concrete (RC) moment resisting frames is an important part of earthquake resistant design. They are critical because they ensure continuity of a structure and transfer forces from one element to another. The flow of forces within a beam-column joint may be interrupted if the shear strength of the joint is not adequately provided (Hwang and Lee, 2000). Under seismic excitation, the beam-column joint region is subjected to horizontal and vertical shear forces whose magnitudes are typically many times greater than those within the adjacent beams and columns. If the joint shear strength is not carefully detailed, the beam-column joint may become the weak link (Hwang and Lee, 1999).

Seismic design provisions for beam-column joints are still controversial despite the great deal of research that has been conducted over the years (Shiohara, 2001). The seismic design standards of countries such as New Zealand, the U.S., Japan and those situated in Europe continent have seismic design requirements for beam-column joints that differ from each other in approach and detail. The major point of controversy relates to the anchorage of the longitudinal bars passing through the interior beam-column joints of the moment-resisting frames. In the seismic design, the objective is the formation of plastic hinges in the beam rather than in the columns. This approach helps to avoid soft storey collapse **mechanism, which means the column is strong** whereas the beam is weak. Normally, the plastic hinges form in the beams at or near their ends. Therefore, the stress in the longitudinal reinforcement in the beam approaches yield strength at the column faces. This results in high bond stresses along beam bars in the joint core. This is because the bars of the beam are stressed almost equal to yield in compression at one face of the column and in tension at the other (Hakuto et al., 1999).

During earthquakes, severe cyclic loading occurs. This leads to deterioration of the bond in the joint. If this is significant, the tension in the bar penetrates through the joint core, and the bar tensile force will be anchored in the beam on the far side of the joint. This means the compression reinforcement in the beam on one side of the column may actually be in tension, with a resulting loss in beam flexural strength and ductility. Moreover the stiffness of the frame will be reduced significantly (Shiohara, 2001).

The use of moment resisting frames is quite common in the construction of Nuclear Power Plant (NPP) structures. The NPP structures are built with reinforced concrete. In view of the increased incidence of seismicity in India, the safety of these structures against earthquake loading assumes greater significance. In order to sustain such loading, the beam-column joints in these structures must be ductile to undergo large rotation so that they can absorb the energy of the earthquake without getting damaged. In the past several decades, a number of investigations have been carried out to study the behaviour of beam-column joints subjected to seismic loading. However, these studies have been mostly confined to the behaviour of joints in multi-storeyed buildings of residential and commercial complexes (Lee and Woo, 2002). The sizes of the joints in these structures are generally smaller than those in NPP structures. The percentage of reinforcement used in the joints in NPP structures is far greater than that used in joints in ordinary multi-storeyed buildings. Therefore, full-scale joints identical to those available in the existing NPP structures, both in size and reinforcement detail, were cast in the laboratory and tested under monotonic loading to evaluate their strength and study their behaviour. Strain gauge instrumentation was extensively used to determine the stress distribution in the joints. Joints were tested under monotonic loading. Theoretical analysis of the joint based on reinforced concrete theory was also carried out to predict the deformation of the joint under loading. The paper presents a complete description of the experimental investigation about the testing of the joint under static loading and the theoretical prediction of deflection under monotonic loading. Both the results were compared and found to correlate well.

II. METHOD & MATERIALS

The NPP structures consisting of the RC beam-column joints were built before 1960 as per the code of practice prevalent at that time wherein provision for earthquake resistance design was not available. In order to evaluate the joints existing in the NPP structures, under static loading, they were prepared in the laboratory for testing with the same size and reinforcement detailing as per the original joints. The size of the beam of the beam was 2000 mm × 610 mm × 915 mm. It contained 5 percent steel placed equally at top and bottom. The longitudinal bars consist of 24 numbers of 36 mm diameters. The column size was 2915 mm x 610 mm x 915 mm. In the column 1.5 percent steel was used.

Table 1 Details of joints

Series	Specimen No.	Sizes (mm)						Reinforcement	
		Beam			Column			Beam	Column
		L	B	D	L	B	D		
I	B3KU1	2000	610	915	2915	610	915	24 Nos. of 36 ɸ mm	4 Nos. of 36 mm ɸ & other sizes
	B3KU2	2000	610	915	2915	610	915	24 Nos. of 36 ɸ mm	4 Nos. of 36 mm ɸ & other sizes
II	B3KC1	2000	610	915	2915	610	915	24 Nos. of 36 ɸ mm	4 Nos. of 36 mm ɸ & other sizes
	B3KC2	2000	610	915	2915	610	915	24 Nos. of 36 ɸ mm	4 Nos. of 36 mm ɸ & other sizes

Table 2 Concrete Strength at 28th day

Specimen No.	28th day cube strength (MPa)
B3KU1	41.144
B3KU2	33.550
B3KC1	31.607
B3KC2	42.328

Typical dimensions of the joint are shown in Fig. 1. Table 1 presents the reinforcement details of cast and tested joints. The specimens were cast with M25 grade concrete designated as per the Bureau of Indian Standards IS: 456 (2000: 100) that is equivalent to British Standards Institution BS 8110 (1987: 128). No admixture was used in mixing the concrete. A slump of 75 mm to 100 mm was achieved in concreting. The mix proportion adopted was

1:1.657:2.63 with a w/c ratio of 0.46. Concrete was compacted well by needle vibrator. Along with the test specimens, control cubes were also cast to evaluate the 28th day strength. Compressive strength at 28th day of all tested specimens is summarized in Table 2. Complete details of casting of specimens have been published elsewhere (Thandavamoorthy, 2006).

III. TESTING UNDER MONOTONIC LOADING

The joint was tested with the column in a vertical plane and beam in a horizontal plane using large size hinge assembly at each end of the column to achieve hinged boundary. The whole system was placed in the loading frame as shown in Fig. 2. The top plate of the hinge assembly was bolted to the cross beam of the loading frame and the bottom plate of the hinge assembly was bolted to the concrete floor.

Linear electrical resistance strain gauges of 5 mm size were pasted at the level of longitudinal steel of the column of the joint along its height on both its sides. The electrical resistance strain gauges were connected to the 40-channel digital data logger driven by software called AUTOSOFT-C. Mechanical brass pellets were fixed to the beam of joint, both in the compression and tension faces at every 100 mm centres for about 1m from the intersection of beam and column. The reading of the distance between the pellets was taken with the help of `Pfender` gauge.

Two hydraulic jacks of each 2000 kN capacity were arranged between the cross beam of the loading frame and a plate and bolted together as in Fig. 2. Under the jacks, an assembly consisting of a distributor beam of length equal to the width of the joint was also placed. A 2000 kN strain gauge based load cell and a hinge assembly was also placed on the beam to measure the load. This hinge assembly was directly placed on the joint. The jacks were connected to the electrically operated hydraulic pumping unit with the help of high-pressure rubber hoses. Mechanical dial gauges were mounted beneath the beam at load point. Along the height of the column also dial gauges were fixed at the centre as well as at the third points of column to measure its lateral displacement.

Load was applied on the joint in increments of 100 kN. At each load increment, readings of dial gauges, strain gauges and pellets were taken and recorded. In the case of the joint B3KU1 the first crack occurred on the tension side of the beam at 500 kN load at the intersection to a vertical length of about 100 mm (Thandavamoorthy, 2006). The growth of the crack was marked.

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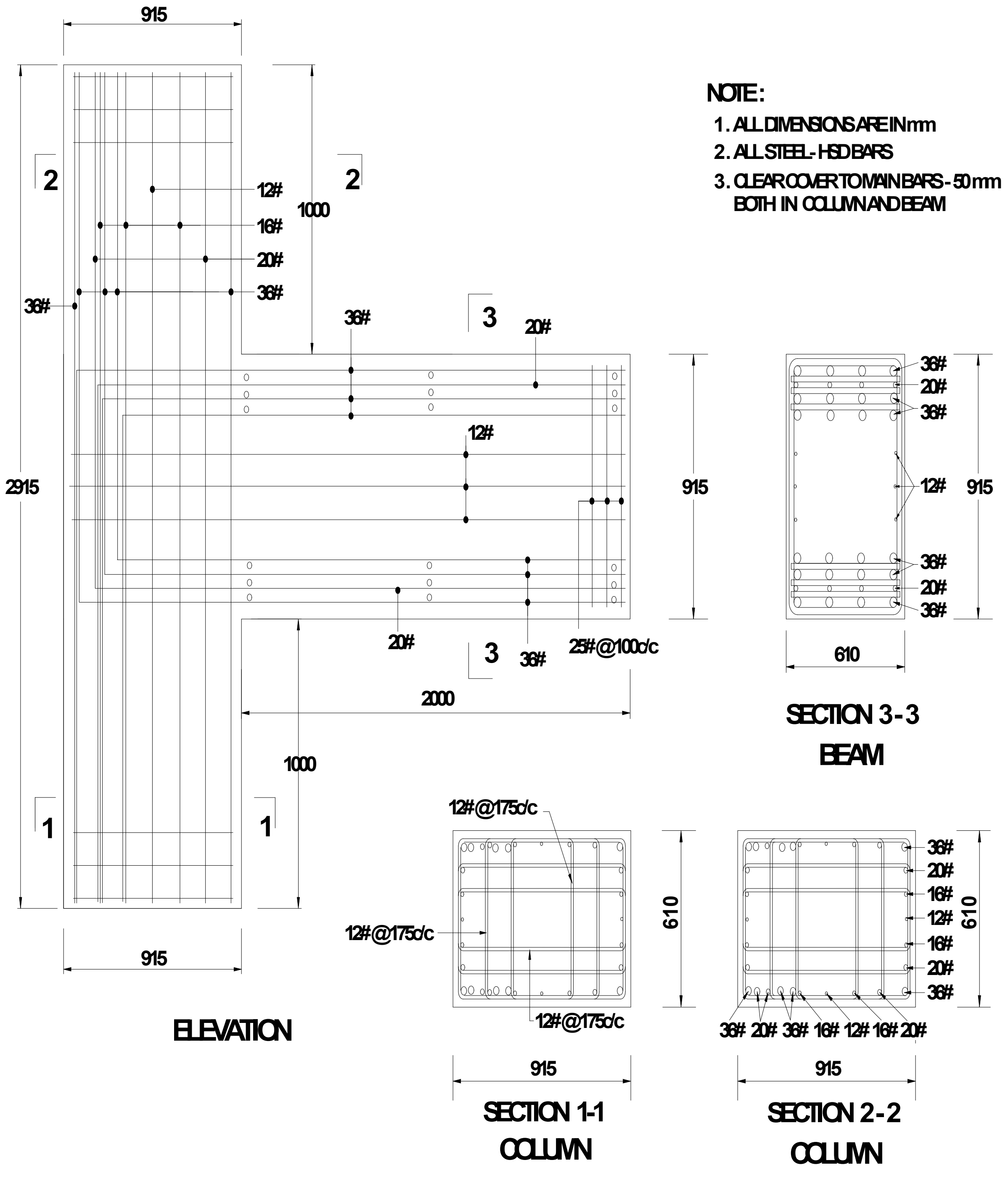

Fig. 1 Reinforcement details of join

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Fig. 2 Test arrangement of joint

IV. THEORETICAL MODELLING

The fundamental principles of mechanics were applied to derive expressions for the deflection of joint under the applied load of W on the beam. The free body diagram of the joint with applied load W and internal forces are shown in Fig. 3.

The capacity of the joint was determined based on the reinforced concrete theory. According to Zabulionis and Dulinskas (2008), in the case of the analysis of flexural members, different stress-strain diagrams for concrete in compression such as parabola with descending branch; parabola-rectangle and bi-linear may be adopted. The stress-strain curve for ordinary concrete has been defined by Hognestad (Oztekin et al., 2003). Braga et al. (2008) have stated that to evaluate the capacity of RC member the main codes in the world permit the use of simplified relationships for concrete under compression such as parabola-rectangle and equivalent rectangular distribution which are called stress-block. In accordance with the Indian Standard, the relationship in the distribution of the compressive stress and compressive strain in concrete may be assumed to be rectangle, trapezoid, parabola or any other shape which results in prediction of strength in substantial agreement with the results of test. An acceptable stress-strain curve is given in the Bureau of Indian Standards IS: 456 (2000: 100) that is equivalent to British Standards Institution BS 8110 (1987: 128). It has recommended a parabola-rectangular stress block as shown in Fig. 4 for the computation of compressive stress in concrete. In arriving at the stress block as depicted in Fig. 4 no factor of safety has been considered because it is not a design problem. Typical stress-strain diagram for concrete as recommended by the Bureau of Indian Standards IS: 456 (2000: 100) and shown in Fig. 4 is adopted for analysis. Here f_{ck} is the characteristic compressive strength of concrete and x_u is the depth of neutral axis. The stress-strain diagram for steel recommended by Bureau of Indian Standards IS: 456 (2000: 100) is shown in Fig. 5. In this f_y is the characteristic strength of steel and E_s is the modulus of elasticity of steel.

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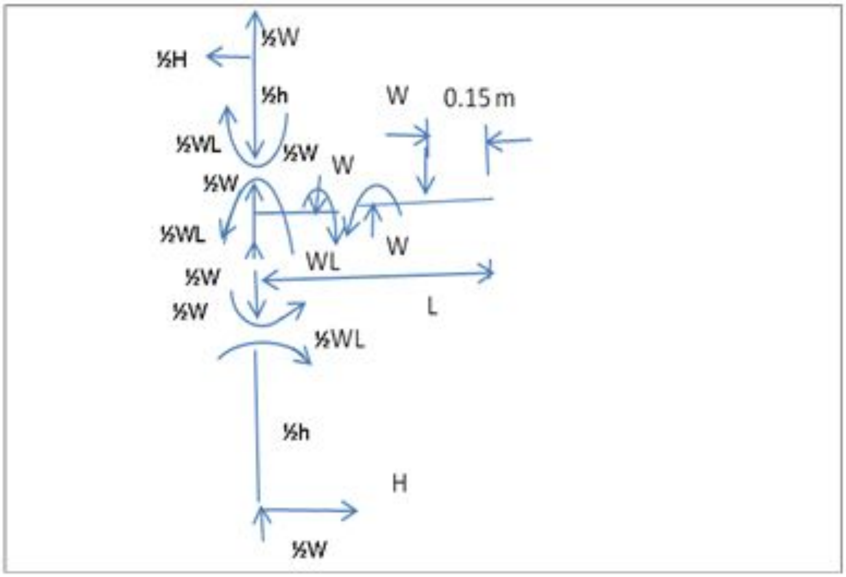

Fig. 3 Free-body diagram of the joint

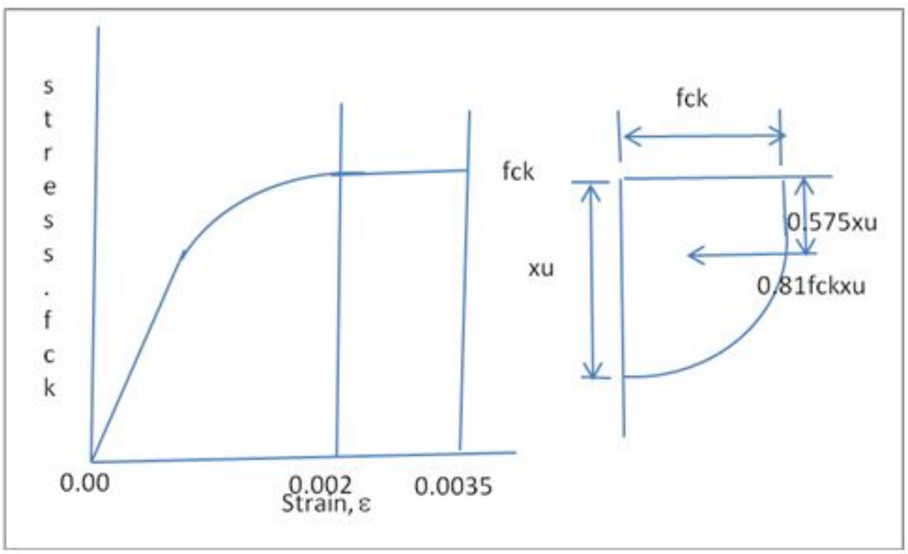

Fig. 4 Stress-strain relation for concrete and the stress block parameters

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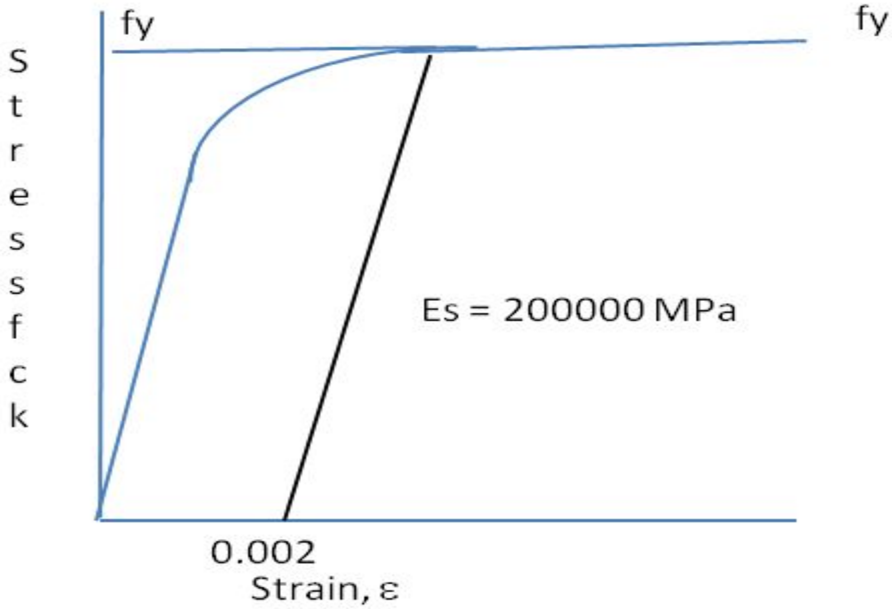

Fig. 5 Stress-strain diagram for reinforcing steel

With the above information, reinforced concrete theory of balancing compression and tension across the section was adopted for further analysis. Based on the equilibrium of forces, the resulting moment of resistant of the section was computed to develop mathematical equations to calculate the deflection of the joint. After cracking, tension in concrete is assumed to be entirely resisted by the reinforcing steel. The provisions contained in the Bureau of Indian Standards IS: 456 (2000: 100) along with details shown in Fig. 4 were used to arrive at the stress distribution across the beam of the joint for various stages of loading. The deflection profile of the joint under the loading with nomenclatures is shown in Fig. 6 on an exaggerated scale. First the joint is axially compressed by $_1$ by its own self weight as given in Eq. (1) by the applied load W. This is shown in Fig. 6. Now the column rotates under the load W on the cantilever beam giving rise to an angle as shown in Fig. 6. This angle is expressed in Eq. (2). The deflection of the beam $_2$ due to this rotation is given in Eq. (3). The deflection of the cantilever beam is given in Eq, (4).

$$\delta_1 = \frac{Wh}{2AE} \tag{1}$$

$$\alpha = \frac{Ml}{2EI_c} = \left(\frac{WL}{2}\right)\left(\frac{h}{2}\right)\left(\frac{1}{2EI_c}\right) = \frac{WLh}{8EI_c} \tag{2}$$

$$\delta_2 = L\alpha = L\left(\frac{WLh}{8EI_c}\right) = \frac{WL^2 h}{8EI_c} \tag{3}$$

$$\delta_3 = \frac{WL^3}{3EI_b} + \left(\frac{WL^2}{2EI_b}\right) \times 0.15 \tag{4}$$

Total deflection of the joint is expressed as

$$\delta = \delta_1 + \delta_2 + \delta_3 \tag{5}$$

From the above individual expressions, the deflection at the free end of the beam was derived as given in Eq. (5).

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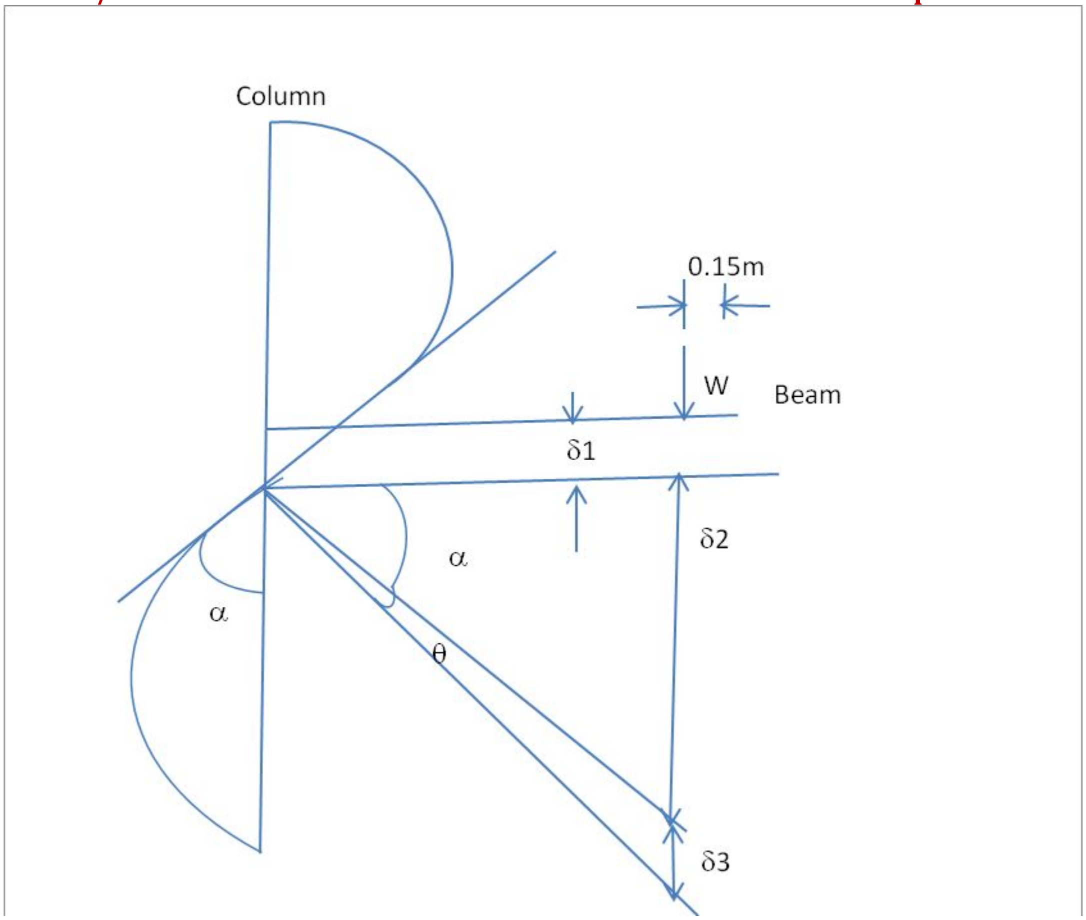

Fig. 6 Deflection profile of loaded joint

Similar deflection profile of column and beam under an applied load at the tip of the beam has been reported by Renton (1972). Liu and Park (1998) have reported similar column and beam deflection profile in the case of an interior beam-column joint. These reports corroborate the assumptions made in the analysis in this investigation.

V. RESULTS AND DISCUSSION

RC beam-column joints of beam size 2000 mm × 610 mm × 915 mm and column size 2940 mm × 610 mm × 915 mm existing in NPP structures in India were replicated in the laboratory and cast with M25 grade concrete with mix proportion of 1:1.67:2.63 and with w/c ratio of 0.45 in the laboratory and tested under monotonic loading till failure. Typical load-deflection relation of the joint is shown in Fig. 7. The relation is linear up to a maximum load of about 2100 kN at which the deflection attained a value of 40 mm. Afterwards the load-deflection curve remains asymptotic to deflection axis without any increase in load up to an ultimate deflection of 90 mm. Similar observation has been made by Ravi and Arulraj (2009) in their test in connection with the testing of retrofitted beam-column joint of the same grade of concrete in which, initially up to a load of 83% there is linearity and subsequently there is a slight slope up to the ultimate load. Same trend of load-deflection behaviour of closing joint analysed by ABACUS software package has been reported by Nabil et al. (2016). Aly et al. (2015) have reported from numerical analysis conducted on SIFCON repaired joint with load-deflection behaviour with initial linear variation at 91% of ultimate load with a small slope afterwards at the ultimate load. These observations though do not directly pertain to the dimensions of the joint adopted in this investigation the trend in behaviour indicate similarity in results which corroborate the outcome of the analysis carried out here.

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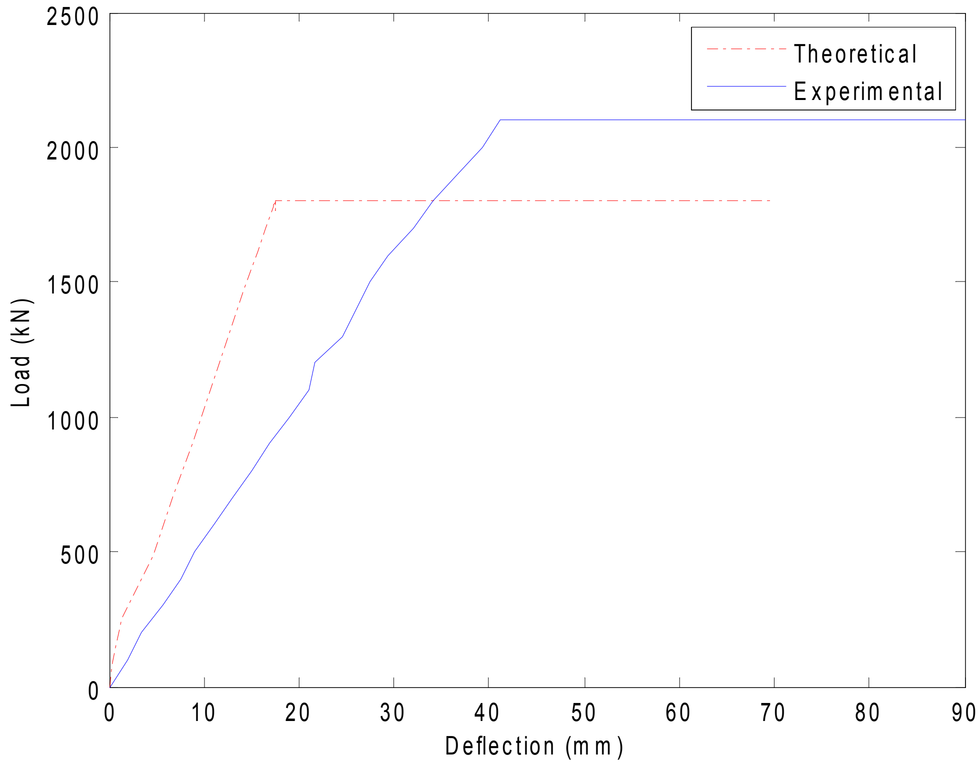

Fig. 7 Load-deflection of joint both theoretical and experimental

The deflection predicted by theoretical approach for the joint has been superposed in Fig. 7 for comparison. The trend of the theoretical load-deflection relation is similar to that of the experimental relation. However, the initial linear elastic curve attains a maximum load of 1750 kN at a deflection of 15 mm. Afterwards the relation between the load and deflection remains asymptotic to the deflection axis and attains a maximum displacement of about 70 mm. Peng et al. (2011) has reported that the theoretical flexural strength predicted by RC design codes in the world are significantly smaller than that measured in the experiment. The theoretical load-deflection relation shown in Fig. 7 indicates that the analytical model is quite stiff. From Fig. 7 it is quite obvious that there is a factor of safety of 1.4 for the collapse load above that predicted theoretically. From this it is quite obvious that the joint had been designed in a safe manner with adequate factor of safety.

The variation of curvature of the joint against the moment is shown in Fig. 8. The trend in this case is also similar to that of deflection of the joint. Initially the curvature varies linearly up to a moment of 4000 kNmm at which the curvature was 0.7×10^{-5}. Subsequently the moment remains the same and the curvature increases up to an ultimate value of 1.5×10^{-5}.

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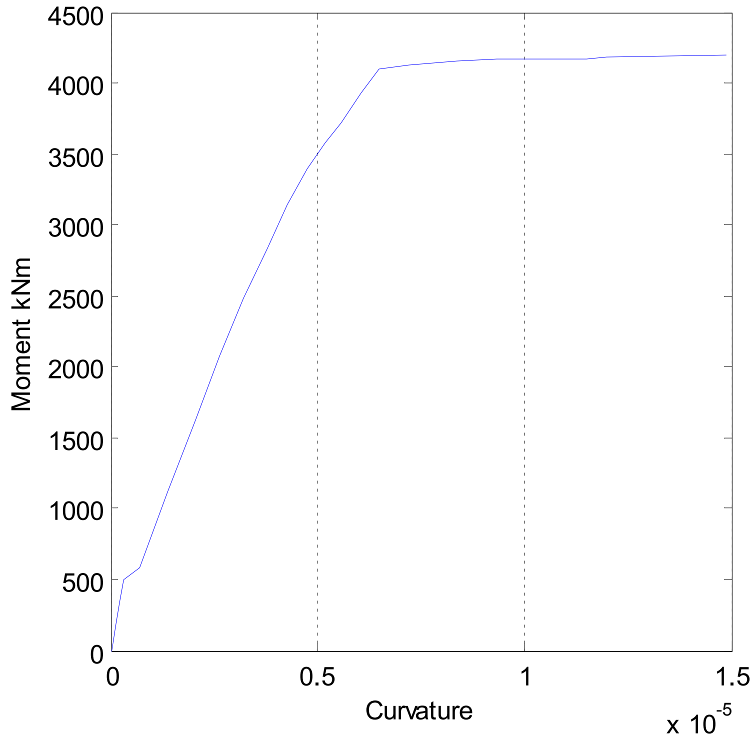

Fig. 8 Experimental moment-curvature relation

The compressive strain measured at the extreme fibre of the beam and that predicted theoretically is shown in Fig. 9. The compressive strain up to a load of 1000 kN is linear in both cases. There is a close correlation between measured and predicted values in this region. After the load of 1000 kN the load-compressive strain relation tend to be non-linear. The variation between the experimental and theoretical values in this relation is large at a load of 2500 kN. For loads below 2500 kN and above this the variation is small. At a micro strain of 4000 the theoretical load merges with experimental results.

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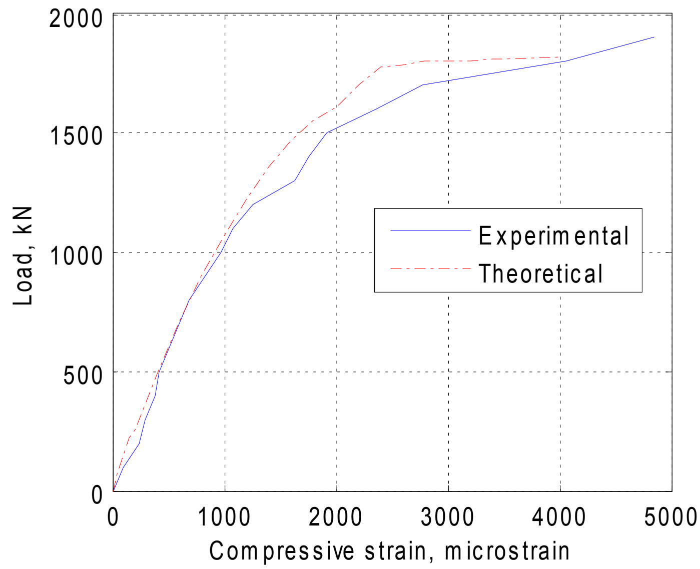

Fig. 9 Load Vs. Compressive strain relation

Variations of tensile strain at top of the beam and compressive strain at the bottom of the beam in this investigation are shown superposed in Fig. 10 against the applied load. The relationship in both cases indicates linearity up to a load of 1750 kN. Both the quantities in this linear range do not agree with each other as is evident from Fig. 10. After a load of 1750 kN both the curves merge together and become asymptotic to strain axis. The ultimate compressive strain terminates at 4000 micro strain whereas the tensile strain goes up to 7000 micro strain. The maximum compressive strain in concrete recommended by the Bureau of Indian Standards IS: 456 (2000: 100) for design is 3500 microstrain. The value obtained in this investigation is 4000 microstrain which almost coincides with the value given in the code. This enhances the credibility of the results obtained in this investigation. The variation of the tensile strain against load is similar to the trend exhibited by load-deflection relation.

VI. CONCLUSIONS

Four numbers of full scale beam-column joints identical to that existing in the NPP structures with beam of size 2000 mm × 610 mm × 915 mm reinforced with 5 percent steel placed equally at top and bottom, and column of size 2915 mm × 610 mm × 915 mm reinforced with 1.5 percent steel were cast successfully in the laboratory and tested under monotonic loading. The concrete of grade M25 with a mix proportion of 1:1.657:2.63 with w/c ratio of 0.46 was used for the preparation of the joints. No admixture was used. A slump ranging from 75 mm to 100 mm was achieved in this investigation. Weigh batching was used in the proportioning of the ingredients. In the testing of joints both ends of the column were provided with hinges to simulate the reality of the behaviour of the column in a multistorey structure. This is perhaps for the first time full-scale RC beam-column joint of realistic size with heavy reinforcement was cast and tested in the laboratory to have a basic understanding of its behaviour.

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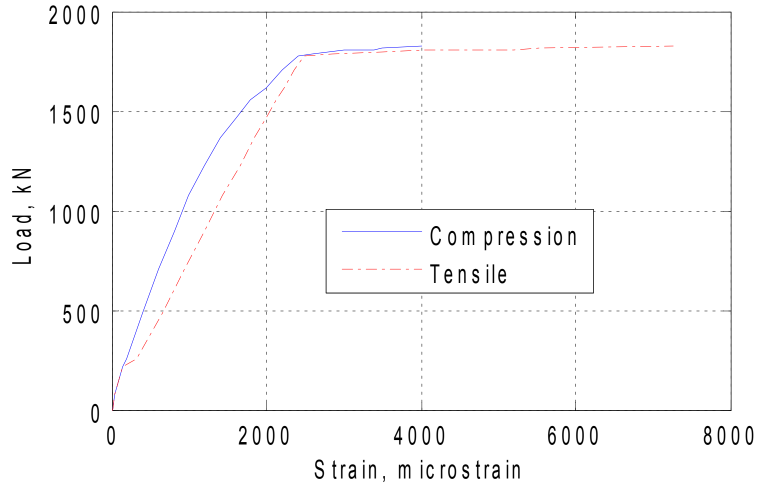

Fig. 10 Comparison of Compressive and Tensile strains

The tested joint was theoretically analysed using the principle of mechanics and. reinforced concrete theory. Necessary equations to predict the deflection of the joint were developed from the deflection profile of the joint induced by the load applied at the tip of the beam. The load-deflection relations of joint, both measured and predicted theoretically, were superimposed. It was observed that the trend in behaviour of both cases was identical. Both the curves show initial linearity up to certain load and subsequent constant load till failure. The theoretical behaviour indicated a stiff model. The load-deflection relation also shows large ductility of the joint. The theoretical moment-curvature relation indicates an initial linearity up to a moment of 4000 kNmm with subsequent constant regime till an ultimate curvature of 1.5×10^{-6}.

The experimental and theoretical relations of compressive strain against load are initially linear up to 1000 kN. Subsequently the curve becomes non-linear till the peak load of 2000 kN. Between a load of 1000 kN and 1750 kN agreement between measured and predicted values was not good. This mismatch is due to the transition from the parabolic stress to rectangular stress distribution in the stress block. This is clearly indicated by theoretical distribution of compressive stress across the section. The theoretical stress distribution is almost similar to the stress-strain curve for concrete. The experimentally measured compressive strain is also almost of the same shape excepting the curve above 1750 kN where there is a small deviation.

The theoretically predicted compressive and tensile strain distribution in the joint follow the pattern of its load-deflection behaviour with initial linearity up to a load of 1750 kN after which the load remaining constant, strain increases till failure load.

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