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Experimental Study on Steel Semi-Rigid Beam-Column Joints under Quasi Static Loading

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ABSTRACT: *To Study the overall behavior of the Steel Beam-Column joint under Quasi-Static load Experimentally and to study the Ductility Characteristics of Beam-Column joint. Beam Column Joint is defined as that portion of the column within the depth of the deepest beam that frames into the column. The portion of the column that is common to both beam and column at the intersection is called beam-column joint. These are generally classified with respect to geometrical configuration and identified as interior, exterior and corner joints. The Quasi-static test are not Dynamic test in which the rate of application of load is very low so that the material strain-rate effects do not influence the structural behavior and inertia forces are not developed. This method adequately captures the most dynamic characteristics of the structure such as hysterics behavior, energy dissipation, stiffness degradation, ductility, hysteretic damping, the most distressed zones, and the lateral strength and deformation capacity. This data is also utilized to make the hysteretic model of component for the dynamic analysis of the structure.*

KEYWORDS: *Semi-Rigid Connections, Quasi Static Loading, Beam-Column Joints, ISMB Steel section*

I. INTRODUCTION

The main structural elements of steel framed multi-storey structures have to be conceived as the assemblage of three main structural components, namely, columns, beams and their joints. The capacity of steel frames to resist loads may be determined more by the strength and stiffness of joints than by the properties of the members by themselves. The stiffness and capacity of joints affect the number, location and extent of plastic hinges developing in frame members. This, in turn, determines the distribution local ductility within the frame and influences the overall ductility of the structures. Especially in unbraced frames, the stiffness of the joints may have a major effect on the deflections of the structure as a whole and on its stability. In practice, beam-to-column joints in conventional analysis and design of steel frameworks are usually assumed to behave either ideally pinned or fully rigid. The use of these extreme cases is due to two reasons (1) Low effort for dissemination of structural analysis methods that incorporate the joint flexibility. (2) Little knowledge about the moment versus rotation curves associated semi-rigid joints. Another reason that contributes to the continuous use of the simple and fixed solutions is related to the structural engineer’s natural resistance to alter the design process. However, the experimental investigations shows that the true behavior of joints lies in between that of ideally pinned and fully rigid and such joints are referred to as Semi-rigid joints.

Overestimating the joint rigidity may result in underestimating the lateral sway, story drift, and the probability of failure, while underestimating the joint rigidity can lead to underestimating forces developed in the beams and columns. Neglecting the real behavior of the joint in the analysis may lead to unrealistic predictions of the response and the reliability of steel frames. Thus, both of these extreme assumptions may be inaccurate and uneconomical. Based on these facts, the flexibility of joints is subject of several major building codes for steel structures e.g. British Standards (BS 5950), Eurocode 3(CEN), and the specifications of the American institute of steel Constructions (AISC-ASD, LFRD). Most of the Semi-rigid joints are proposed with high-strength bolts. Recently, extensive and increasing studies have been carried out on the high strength bolted joints in Steel Structures. The issues in high strength bolted joint as compared with welded alternatives are the stiffness, complex behavior and ductility, as well as construction cost. It is therefore of practical importance to investigate the real behavior of joints with respect to some rational test.

1.1 Beam-Column Joint

A beam-column joint is defined as that portion of the column within the deepest beam that frames in the column. The portion of the column that is common to both the column and beam at the intersection is called beam-column joint.

Beam-column joints are generally classified with respect to geometrical configuration and identified as interior, exterior and corner joints as shown in fig 1.1. With respect to the plane of loading, an interior beam-column joint consists of two beams on either side of the column and an exterior beam-column joint has a beam terminating on one face of the column.

Since the constituent materials of beam-column joint have limited strength, the joint have limited force carrying capacity, when forces larger than these are applied during earthquakes, joints are severally damaged. Repairing damaged joints is difficult, and so damaged must be avoided. Hence, beam-column joint gains importance and special consideration in various researches have been devoted to study the behavior for seismic loads.

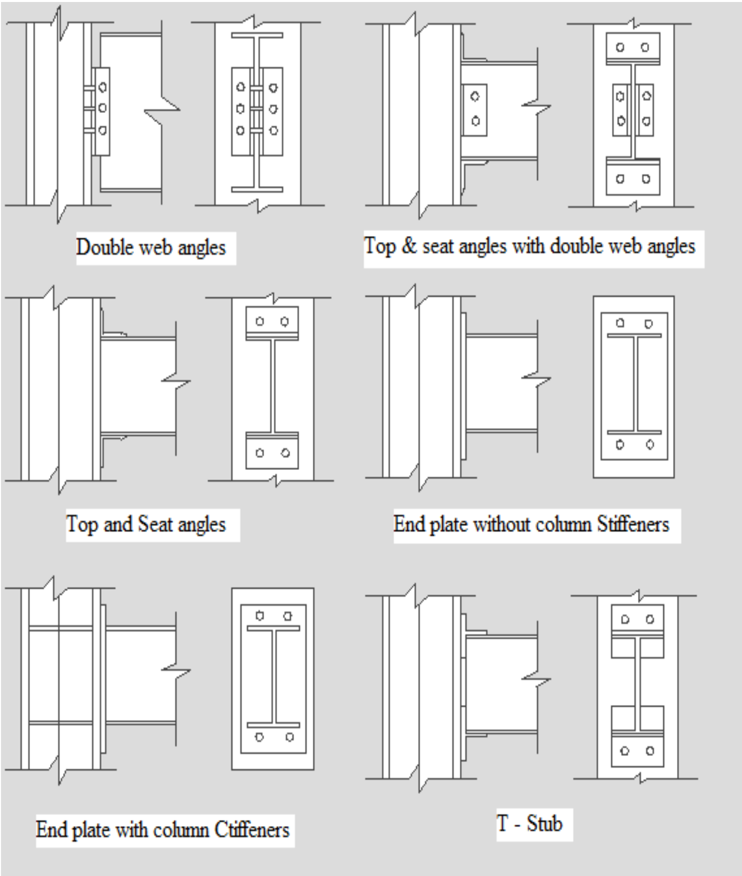

Fig 1.1 Types of Beam- To-Column Connections

1.2 Quasi Static Test

The Quasi-static test is not a Dynamic test in which the rate of application of load is very low so that the material strain-rate effects do not influence the structural behavior and inertia forces are not developed. The loading pattern and history must be carefully chosen to be general enough to provide the full range of deformations that the structures will experience under the earthquake excitation. This method adequately captures the important dynamic characteristics of the structure: hysteresis behavior, energy dissipation, stiffness degradation, ductility, hysteretic damping, the most distressed zones, and the lateral strength and deformation capacity. This data is also utilized to make the hysteretic model of component for the dynamic analysis of Structure.

1.3 Outline Of The Contents

The general introduction to beam-column joint, its importance in framed structure and the quasi static test are given in this chapter 1. Main objectives covered in the chapter 2. A brief review of literature available in the field of steel fibers in beam-column joints is given in the chapter 3. In chapter 4. Code requirements for the design of beam-column joints are discussed. Chapter 5 discusses the role of various parameters in the strength enhancement of beam-column joint. Experimental set-up, reinforcement details, design requirements and test procedure are also given in chapter 5. In chapter 6, results of experiments on eighteen beam column joints are shown. In chapter 7 conclusions and scope for future studies are given.

1.4 Summary

This chapter gives introduction to beam-column joint, behavior to earthquake, project objectives and a brief outline of the contents of the thesis.

II. OVERVIEW OF CODAL SPECIFICATIONS

2.1 General

Semi-rigid steel framing construction was first adopted by the American institute of Steel construction Specification for allowable stress design (AISC-ASD) as early as 1946, later in the Load and Resistance Factored Design (AISC-LFRD) Specification in 1986. The AISC-ASD Specification (AISC 1989) provides three connection types for construction and associated analysis assumptions: Type1, commonly designated as “rigid framing or continuous framing”, Type2, commonly designated as “simple framing”, Type3, commonly designated as “semi-rigid framing”.

The rigid frame assumes that beam-to-column connections have sufficient rigidity to maintain the original angles between intersecting members virtually unchanged. The simple framing assumes that the ends of the beam are connected for shear only and are free to rotate under load. The semi-rigid framing assumes that the connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the complete rigidity of the Type 1 and complete flexibility of Type 2.

The AISC-LFRD Specification (AISC 1986) classifies two types of constructions Type FR (fully restrained) and Type PR (partially restrained). Fully restrained constructions are basically same as the Type 1 construction in AISC-ASD. Partially restrained constructions comprises Type 2 and Type 3 of AISC-ASD and describes those conditions where the connections lack sufficient strength to transmit the bending moments between the beams and the columns or have insufficient rigidity to maintain the original angles between the beams and the columns.

When the partial connection restraint is taken into account in analysis and design procedures, the capacity of the connection for the desired restraint must be established by either analytical or empirical methods. Although both AISC-ASD and AISD-LFRD specifications recognize semi rigid construction in general, very little guidance as to how to incorporate such behavior into the analysis and design process.

EUROCODE3 (1990), the design specification of European community, defines three framing types: simple, continuous and semi-continuous. The specification stipulates the types of beam-to-column connections required for each type of framing. A classification system is established to classify the types of framing based on the moment-rotational behavior of the connections. The characteristics defined by the initial stiffness, ultimate moment resistance, and rotational deformation capacity must be evaluated based on theories supported by experimental tests. The specification requires connection behavior to be modeled rigorously and accounted for explicitly in frame design.

2.2 Summary

In this chapter, AISC-LFRD codal specification, EUROCODE3 are discussed.

III. EXPERIMENTAL INVESTIGATIONS

3.1 Introduction

This chapter presents a summary of the experimental work conducted in the present investigation to study the behavior Semi-rigid steel Beam-Column joint. The type of loading applied was Quasi-static loading in three cycles.

3.2 Experimental Program

Nine specimens were fabricated and tested under Static and Quasi-static cyclic loading. Details of the test program are provided in the following paragraphs.

3.3 Specimen Details

The specimen details are taken for the study of semi-rigid steel frame sections has been listed below. In this study three types of analysis on semi-rigid connections are done.

Type-1: semi-rigid steel frame connected without web angles.
Type-2: semi-rigid steel frame connected with web angle using 12mm and 16mm bolts.
Type-3: semi-rigid steel frame connected with web angle using only16mm bolts.

The beam and the column have a length of 600mm and 900mm respectively. The beam column connections are made by web angle of 40X 40 X 6 mm and the flange angle of 100 X 100 X 10 mm.

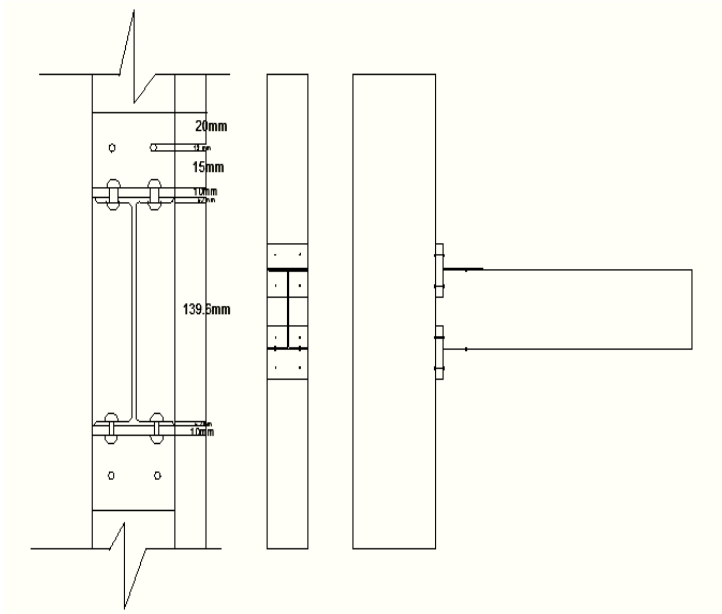

Fig 3.1 Details of Semi-Rigid Steel Frame Section Without Web Angle.

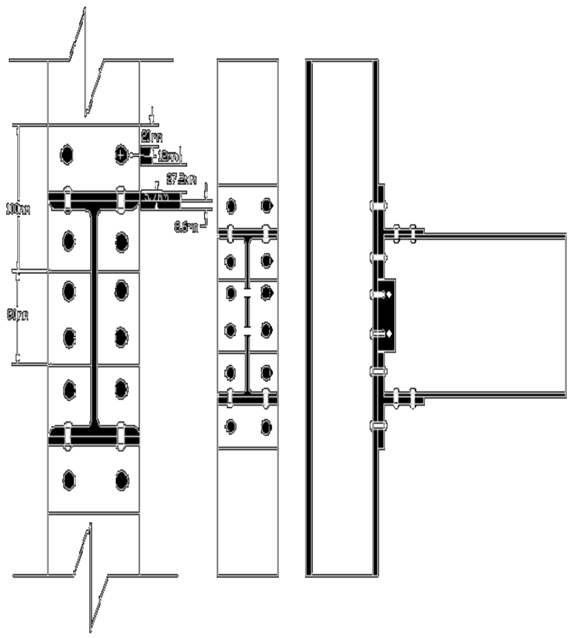

Fig 3.2 Details of Semi-Rigid Steel Frame Section with Web Angle using 12 mm Bolts.

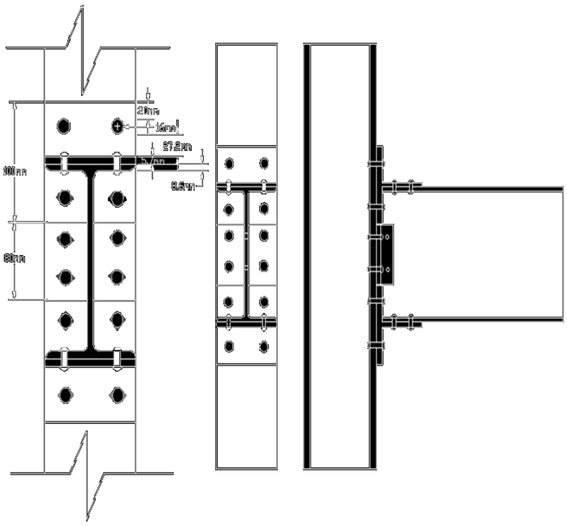

**Fig 3.3 Details of Semi-Rigid Steel Frame
Section with Web Angle using 16 mm Bolts.**

3.4 Specimen Testing And Instrumentation

The exterior beam-column joints were tested in upright position. Axial load of about 45kN was applied initially over the column (before the Quasi-static load was applied on the beam) with help of Hydraulic jack. Instrumentation was used to measure the necessary parameters. Linear Variable Displacement Transducers (LVDT) was used to measure deflection at the free end of the beam for every load increment. Strain gauges were pasted on top and bottom of the beam near the joint to measure the strain variations for variation in moments. Loading was applied at the free end of the beam under force control with hydraulic jack. Quasi-static load was applied in three cycles in both directions, till the specimen fails. Deflections of the beam, the strain variation in the beam, joint rotation were measured for each load increment. In each category, first cycle was started by loading from top, third cycle of loading was applied till failure. Fig 3.4 shows the experimental setup used in the testing of specimens.

3.4 Experimental Test Setup

3.5 Summary

The experimental and the specimen details are given in this chapter. Nine beam-column joints were tested to study the behavior of Steel semi-rigid beam-column joints under Quasi-static loads.

IV. RESULTS AND DISCUSSIONS

4.1 Introduction

The entire beam-column joint were subjected to Quasi-static cyclic load in three cycles. The support conditions of the specimen were assumed to be pinned on both ends of the column. The beam-column joints were tested keeping the column in vertical position. A constant axial load was applied initially before the application of cyclic loads. The test done was force controlled. Out of two specimens in each category of beam-column joint specimens was loaded for three cycles. Deflection at the free end of the beam was measured with LVDT. Various plots were drawn in order to compare the overall behavior of all the specimens and are shown below.

4.2 Load-Displacement Plot

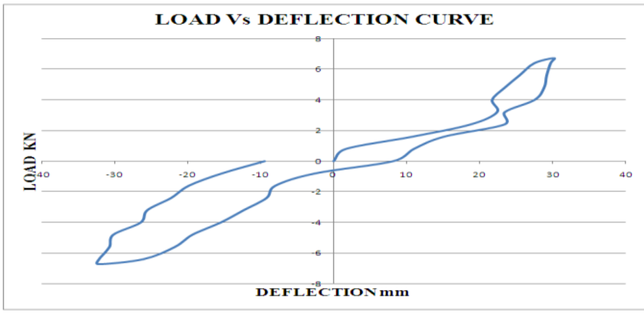

Fig 4.1 Load Deflection Curve for Type 1-sample 1

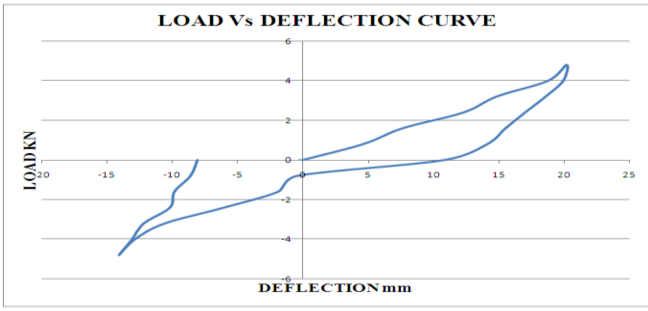

Fig 4.2 Load Deflection Curve for Type 1-sample 2

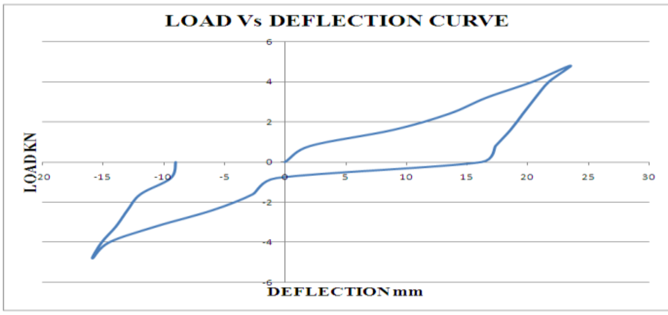

Fig 4.3 Load Deflection Curve for Type 1-sample 3

Fig 4.1, 4.2 & 4.3 shows the load deflection curve for steel semi-rigid frame without web connections under quasi-static load. From the above test results it is inferred that, after attaining a peak load of 6.7kN the same specimen is subjected to unloading condition. Under reverse loading case the load deflection curve for type 1 specimen shows a peak load of 6.7kN. The same cycle of loading – unloading and reverse loading – unloading has been carried out for three times. In this Type first cycle itself the specimen fails reaching the maximum deflection. The first cycle has peak load value of 6.7kN at maximum displacement of 30mm.

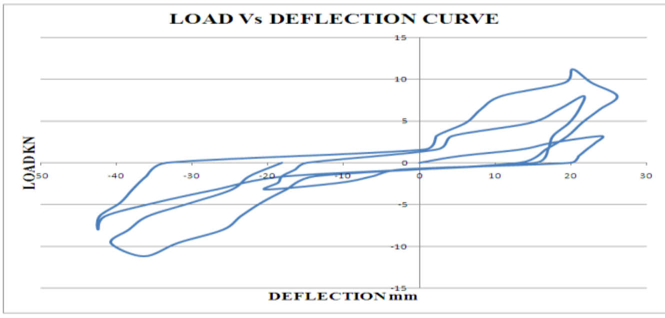

Fig 4.4 Load Deflection Curve for Type 2-sample 1

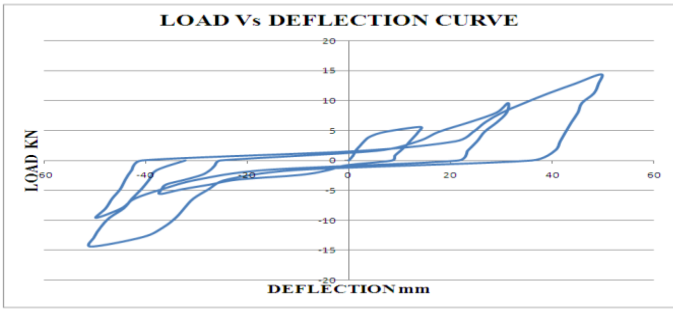

Fig 4.5 Load Deflection Curve for Type 2-sample 2

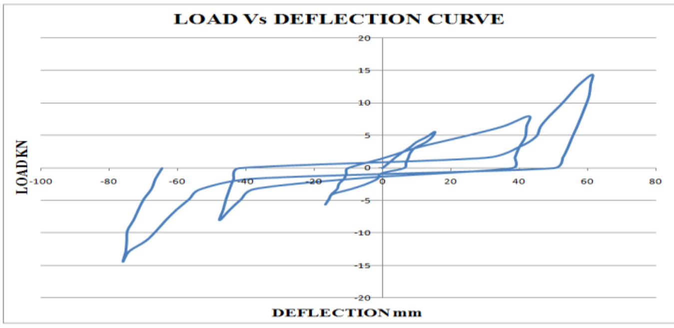

Fig 4.6 Load Deflection Curve for Type 2-sample 3

Fig 4.4, 4.5 & 4.6 shows the load deflection curve for steel semi-rigid frame with web angle using 12mm bolts under Quasi-static load. From the above test results it is inferred that, after attaining peak load of 15kN, the same specimen is subjected to unloading condition. Under reverse loading case the load deflection curve for type 2 specimen shows a peak load of 15kN. The same cycle of loading – unloading and reverse loading – unloading has been carried out for three times. All the three cycles of loading and reverse loading case shows almost the same peak load level of 15kN. The only difference is that, in the first cycle the peak load is achieved at 20mm displacement while in the second and third cycle the peak load is achieved at a displacement of 42mm and 76mm respectively.

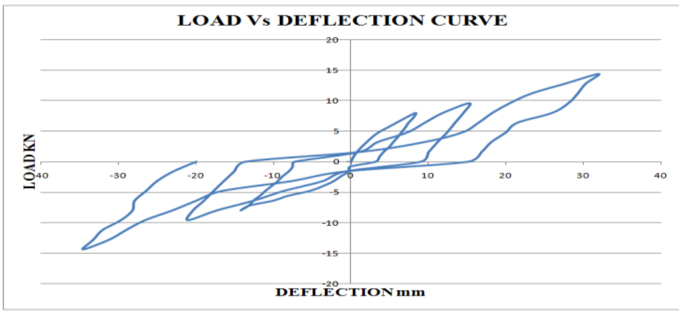

Fig 4.7 Load Deflection Curve for Type 3-sample 1

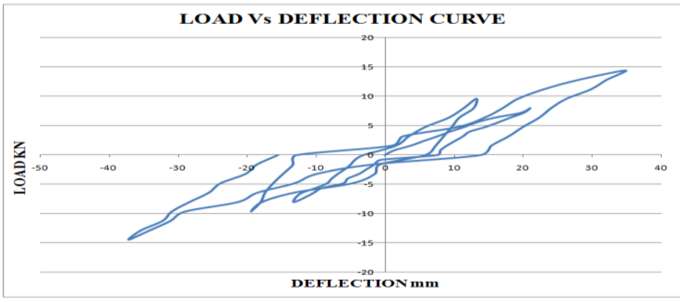

Fig 4.8 Load Deflection Curve for Type 3-sample 2

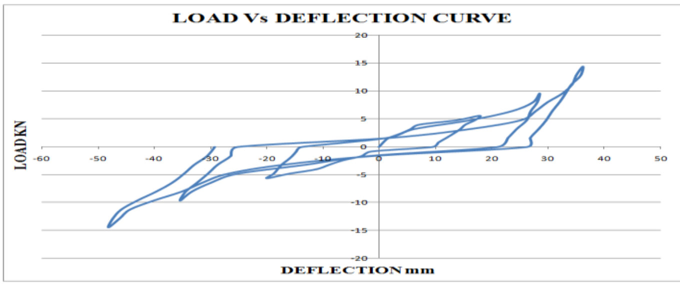

Fig 4.9 Load Deflection Curve for Type 3-sample 3

Fig 4.7, 4.8 & 4.9 shows the load deflection curve for steel semi-rigid frame with web angle using 12mm bolts under Quasi-static load. From the above test results it is inferred that, after attaining peak load of 15kN, the same specimen is subjected to unloading condition. Under reverse loading case the load deflection curve for type 3 specimen shows a peak load of 15kN. The same cycle of loading – unloading and reverse loading – unloading has been carried out for three times. All the three cycles of loading and reverse loading case shows almost the same peak load level of 15kN. The only difference is that, in the first cycle the peak load is achieved at 19mm displacement while in the second and third cycle the peak load is achieved at a displacement of 32mm and 48mm respectively.

4.3 Moment Curvature Relationship

The ductile behavior of an exterior beam column joint involves the formation of plastic hinge in the beam near the column face to investigate the flexural behavior of the beam, various sections of the top and bottom reinforcements were instrumented by electrical strain gauges.

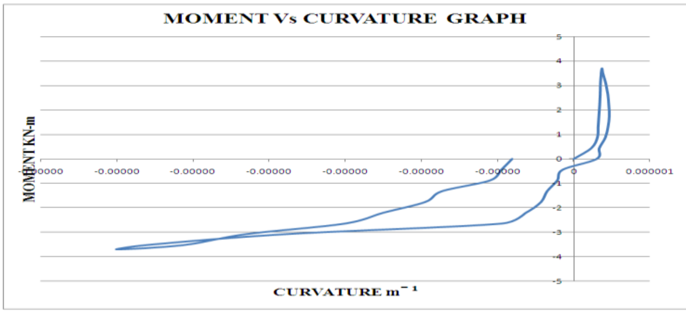

Fig 4.10 Moment Curvature Relationships for Type 1-sample 1

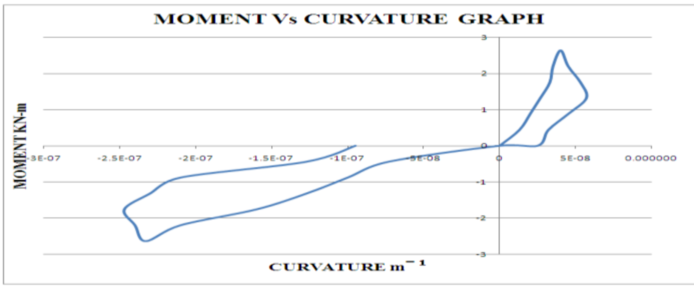

Fig 4.11 Moment Curvature Relationships for Type 1-sample 2

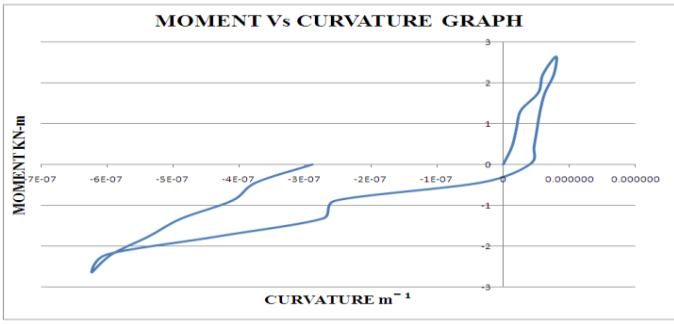

Fig 4.12 Moment Curvature Relationships for Type 1-sample 3

Fig 4.10, 4.11 & 4.12 shows the moment curvature curve for steel semi-rigid frame without web connections under Quasi-static load. From the above test results it is inferred that, after attaining a peak moment of 3.70kN-m, the same specimen is subjected to unloading condition. Under reverse loading case the moment curvature curve for type 1 specimen shows a peak moment of 3.70kN-m. The same cycle of loading – unloading and reverse loading – unloading has been carried out for three times. In this Type first cycle itself the specimen fails reaching the peak moment of 3.70kN-m that occurs at curvature less than .0000006m^{-1}.

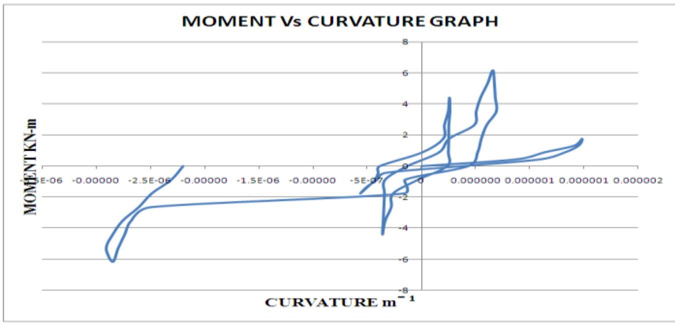

Fig 4.13 Moment Curvature Relationships for Type 2-sample 1

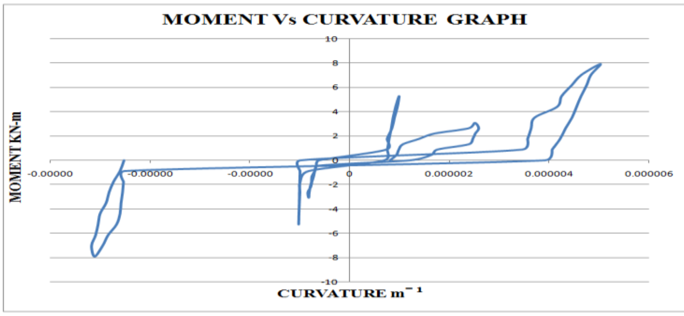

Fig 4.14 Moment Curvature Relationships for Type 2-sample 2

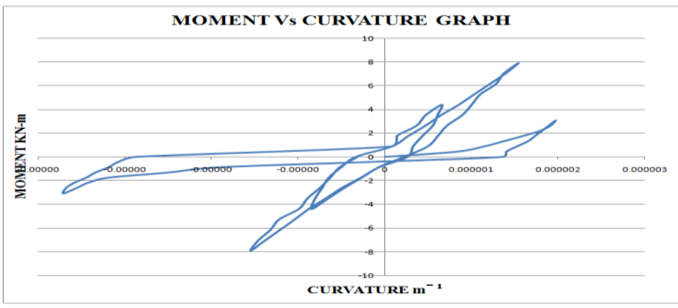

Fig 4.15 Moment Curvature Relationships for Type 2-sample 3

Fig 4.13, 4.14 & 4.15 shows the moment curvature curve for steel semi-rigid frame with web angles using 12mm bolts under Quasi-static load. From the above test results it is inferred that, after attaining a peak moment of 7.92kN-m, the same specimen is subjected to unloading condition. Under reverse loading case the moment curvature curve for type 2 specimen shows a peak moment of 7.92kN-m. The same cycle of loading –

unloading and reverse loading – unloading has been carried out for three times. All the three cycles of loading and reverse loading case shows almost the same peak moment level of 7.92kN-m. The only difference is that, in the first cycle the peak load is achieved at curvature greater than .000001 m^{-1} and less than .000002 m^{-1} while in the second peak moment is achieved at a curvature greater than .000002 m^{-1} and less than 0.000004m^{-1}. In the third cycle the peak moment is achieved at a curvature greater than .00000512 m^{-1}.

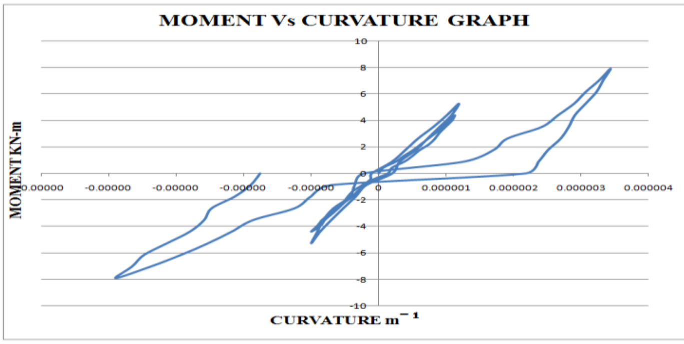

Fig 4.16 Moment Curvature Relationships for Type 3-sample 1

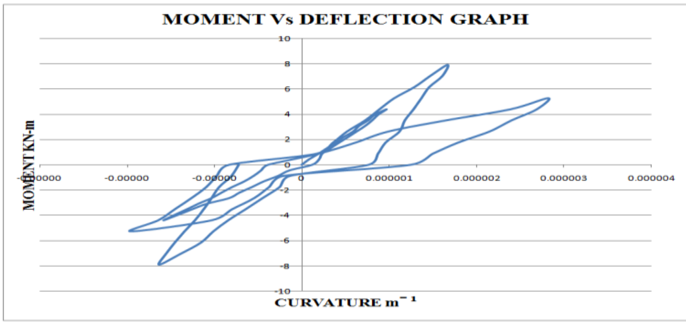

Fig 4.17 Moment Curvature Relationships for Type 3-sample 2

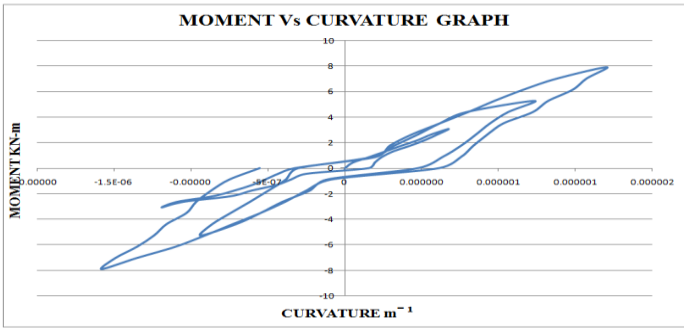

Fig 4.18 Moment Curvature Relationships for Type 3-sample 3

Fig 4.16, 4.17 & 4.18 shows the moment curvature curve for steel semi-rigid frame with web angles using 12mm bolts under Quasi-static load. From the above test results it is inferred that, after attaining a peak moment of 7.92kN-m, the same specimen is subjected to unloading condition. Under reverse loading case the moment curvature curve for type 3 specimen shows a peak moment of 7.92kN-m. The same cycle of loading – unloading and reverse loading – unloading has been carried out for three times. All the three cycles of loading and reverse loading case shows almost the same peak moment level of 7.92kN-m. The only difference is that, in the first cycle the peak load is achieved at curvature greater than .0000005 m^{-1} and less than .000001 m^{-1} while in the second peak moment is achieved at a curvature greater than .000001 m^{-1} and less than .000002m^{-1}. In the third cycle the peak moment is achieved at a curvature greater than .0000039 m^{-1}.

4.4 Discussions

The load deflection curve of type 1 specimen shows peak load value on first cycle itself without conducting second and third cycle test. While the load deflection curve of type 2 and type 3 specimen shows almost the same peak load level. The semi-rigid steel frame connections using web angle only 12mm bolts is found to carry more deflection than other two types. The semi-rigid steel frame connections using without web angle only 12mm bolts is found to carry more curvature than other two types. The semi-rigid steel frame consisting of web angle carries more moment.

4.5 Summary

In this chapter the experimental results on testing of beam-column joints are presented. Load deflection plots and moment curvature plots are drawn and discussed.

V. CONCLUSION

The experimental study to access the connections in the beam column joint for semi-rigid behavior was carried out. Tests were conducted on three sets of specimens, each set having three identical specimens. The test specimens were subjected to quasi-static loading in three cycles. Based on the test results, the following conclusions are drawn.

- All the specimens behaved as semi-rigid connections.
- Specimens without Web angle exhibit very poor deflection characteristics compared to connections with web angle. Specimen without web angle has poor load carrying capacity of maximum 6.7kN attained at 30mm vertical displacement. While the specimen using only 12mm bolts show high load carrying capacity of maximum 15kN attained at 70mm vertical displacement. But the specimen using only 16mm bolts has same load carrying capacity of 15kN with 48mm vertical displacement.
- Specimens without web angle exhibit very poor curvature characteristics compared to connections with web angle. Specimen without angle has a poor moment carrying capacity of maximum 3.70 kN-m attained at curvature less than .0000006m^{-1}. While the specimen connected with web angle using only 12mm bolts shows high moment carrying capacity of 7.92kN-m attained at curvature less than .00000512m^{-1}. But the specimen connected with web angle using only 16mm bolts has intermittent moment carrying capacity of 7.92kN-m attained at curvature less than .0000039m^{-1}.
- The performance of the joint depends on the number of bolts used in the connection.
- Failure of connections is brittle and on the application of quasi-static load there is premature failure of connections.

5.2 Summary

In this chapter, conclusions are drawn from the experimental results. Type 2 specimens are found to behave better compared to Type 1 and Type 3 specimens under quasi-static loads.

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