Seismic Performance of RC Frames with Concentric Internal Steel Bracing

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Abstract

Steel bracing has proven to be one of the most effective systems in resisting lateral loads. Although its use to upgrade the lateral load capacity of existing Reinforced Concrete (RC) frames has been the subject of numerous studies, guidelines for its use in newly constructed RC frames still need to be developed. In this paper, the efficiency of using braced RC frames is experimentally evaluated. Two cyclic loading tests were conducted on a moment frame and a braced frame. The moment frame was designed and detailed according to current seismic codes. A rational design methodology was adopted to design the braced frame including the connections between the brace members and the concrete frame. Test results showed that the braced frame resisted higher lateral loads than the moment frame and provided adequate ductility. The adopted methodology for designing the braced frame resulted in an acceptable seismic performance and thus represents the first step in the development of design guidelines for this type of frames.

***Keywords**:* Reinforced concrete, moment frames, braced frames, cyclic load, testing, design, scaling.

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1. Introduction

Braced steel frames are commonly used to resist lateral loads. Their design guidelines are readily available [1, 2]. The use of bracing to upgrade the seismic capacity of existing RC frames has been the subject of several research investigations over the past three decades. Two bracing systems are typically considered, external bracing and internal bracing.

In external bracing, steel trusses are attached to the building exterior. Bush et al. [3] conducted cyclic loading tests on $\frac{2}{3}$ scaled models of a number of structures retrofitted using external bracing. They reported the efficiency of such a method in retrofitting existing RC buildings. Badoux and Jirsa [4] investigated numerically the behaviour of RC frames retrofitted with external bracing. They recommended using cables instead of steel sections for the brace elements to avoid buckling of the brace members, and thus increase the ductility of frames.

In internal bracing, steel trusses or bracing members are inserted in the empty space enclosed by columns and beams of RC frames. A number of researchers [5, 6, 7, 8] studied the effectiveness of using internal steel trusses to retrofit existing RC frames. They reported that such a method allows upgrading the seismic capacity of existing structures. Maheri and Sahebi [9] recommended the use of internal brace members over internal steel trusses. Nateghi-Alahi [10] successfully applied this technique to upgrade the seismic capacity of an existing eight-story building located in Iran.

Connections between the steel truss or bracing members and the RC frame are important to achieve the required lateral load capacity. A number of connections capable of transferring loads to the additional lateral load resisting elements were proposed by several researchers [11, 12, 13]. These connections relied on the use of adhesives, grout, or mechanical anchors. Maheri et al. [14, 15] proposed a connection that minimizes the eccentricity of the brace member force. This allowed transferring the brace force to the corner of the RC frame without producing local damage in concrete members. One of the benefits of using internal brace members instead of internal trusses is the reduction of the number of required connections and thus the construction cost.

Current seismic codes assume that the lateral loading system for newly constructed RC structures are either moment resisting frames, coupled walls, or shear walls. These systems can be designed to have low, moderate or high ductility. Steel bracing is generally not listed as one of the available lateral load resisting systems. Combined with the fact that previous studies were mainly conducted to evaluate the behaviour of non-ductile RC structures retrofitted by attaching bracing elements, this limits the use of steel bracing for new construction. However, using steel bracing for new construction has many advantages over the use of shear walls including: reducing the weight of the structure, and thus reducing seismic loads and increasing the ductility of the structure.

In this study, the use of concentric internal steel bracing for new construction was investigated experimentally. Two specimens representing a RC moment frame with moderate ductility and a braced RC frame were designed. Current seismic codes were used to design the moment frame. For the braced frame, a rational design methodology is proposed. Both frames were constructed and experimentally tested using cyclic loads. Their test results were compared and discussed. This allowed gaining an improved understanding of performance of braced frames and evaluating the proposed design methodology.

2. Choice of test specimens

A four-storey building with dimensions of 12.0 m by 12.0 m was considered for the design process. It was assumed that the building is located in a highly seismic area classified as category C in the International Building Code (IBC) [16]. Two lateral load resisting systems (Fig. 1), RC moment frames and braced RC frames, were considered. A midspan panel measuring 4.0 m by 3.0 m was isolated from the third floor of each frame. The gravity and elastic earthquake forces acting on these panels were determined in accordance with the IBC [16].

Modern seismic codes reduce the applied elastic seismic force by using a force reduction factor (R_d), which is the product of a ductility factor and an over-strength factor. This causes building deformations to exceed elastic limits. By using different R_d for each structural system and specifying guidelines for members and connections design, seismic codes assume that the ductility demand on each individual member or connection is lower than its capacity. This assumption comes from the relationship between the global structure ductility and the local member or connection ductility that is unique for each structural system. For the moment frame, the elastic earthquake force was reduced using a seismic force reduction factor for moment frames with moderate ductility. The same reduction factor was used for the braced frame.

A $\frac{2}{5}$ scaled model frame measuring 1.6 m by 1.2 m was found to be satisfactory. To keep stresses in the scaled model similar to that in the full-scale panel, the forces acting on the panels were also scaled down by a factor of $\left(\frac{2}{5}\right)^2$. Frames of similar size were tested by Maheri et al. [14]. Axial column forces for the braced frame were slightly higher than those for the moment frame as a result of the vertical component of the brace member forces. The boundary conditions for the tested specimens were chosen such that the distribution of the internal forces is similar to that in the full-scale frame. This was achieved by using two hinged supports at the ends of the bottom beam. Figure 2 shows the test specimens with the scaled design loads.

3. Design and construction of test specimens

The moment and braced frames were composed of top and bottom beams, and left and right columns. The cross-section dimensions of the beams and columns were chosen to be 140 mm by 160 mm. For both frames, the internal forces resulting from the scaled design forces were determined. These forces are then used to design the frames as explained below.

The moment frame was designed according to ACI 318-02 [17] and its detailing was done in accordance with the ACI special provisions for seismic design [17]. The top and bottom reinforcement of the beam sections were 2M10 (diameter of 11.3 mm). To satisfy the weak beam-strong column philosophy, the column longitudinal reinforcement was 4M15 (diameter of 16.0 mm). In the plastic hinge regions, the transverse reinforcement of beams and columns consisted of 6 mm steel wires spaced at 350 mm. The beam-column joint was transversely reinforced with two-6 mm wires. Details of this specimen are shown in Fig. 3.

A rational design methodology for the braced frame is proposed and applied as explained below:

- RC beams, columns, and beam-column joints are to be designed according to standards for the design of RC elements. Detailing of steel reinforcement is to be done according to the general detailing requirements in these standards. Special seismic detailing of steel reinforcement is not required because of the expected reduced seismic demand. For the test specimen, it was chosen to use ACI 318-02 [17] to design the RC beams, columns, and beam-column joints. The top and bottom reinforcement of the beam and column sections were 2M10. The transverse reinforcement of beams and columns consisted of 6 mm steel wires spaced at 70 mm. The beam-column joint was transversely reinforced with one-6 mm steel wire. Details of this specimen are shown in Fig. 4. It should be noted that the total weight of steel reinforcement for the braced frame was 35% lower than that for the moment frame. Combined with the associated reduction in the workmanship required to install the stirrups, this is expected to result in a significant reduction in construction costs.
- The brace members and their connections are to be designed according to standards for the design of steel elements. Their design must satisfy the special seismic provisions in these standards. For the test specimen, AISC-LRFD [1] was used to design the brace members and their welded connections to the gusset plates. Their design was also checked using the AISC seismic provisions for steel structures [2].
- The connection between the gusset plates and the RC frame can be achieved by welding the gusset plates to steel plates that are anchored to the concrete frame. To calculate the forces acting on the anchors and the weld, it is proposed to use the uniform force method [1, 18]. The essence of the method is to select the geometry of

the connection so that moments are eliminated on the interfaces between the gusset plate and the steel beam, the gusset plate and the steel column, and the steel beam and the steel column. For concrete members, the moment at the interface between the beam and the column will not be eliminated since the location of centerlines of these members depends on the degree of cracking. The weld is to be designed according to standards for the design of steel elements. Its design must satisfy the special seismic provisions for steel structures. The anchors are to be designed according to standards for anchorage in concrete.

For the present test specimen, a total of eight steel plates were positioned on the inner corners of the RC frame. Each plate had four-5/8 inch headed studs as shown in Fig. 4. The studs were designed for the critical case of combined tension and shear according to Appendix D of ACI 318-02 [17]. The design ensured that concrete shear failure, bond failure, and connector shear failure are avoided.

The two designed specimens were constructed using self-consolidating concrete. Its compressive strength at the time of testing was 55 MPa.

4. Test setup

The specimens were tested using the setup illustrated in Figs. 5 and 6. As shown in these figures, the beams were oriented vertically and the columns horizontally. The specimens were pin jointed at the two ends of the bottom beam. They were subjected to constant gravity loads using two hydraulic jacks. Special rollers were manufactured to allow these jacks to slide on the concrete surface, and thus allow lateral deformation of the concrete specimen. An actuator was used to apply several cycles of loads using a displacement-controlled approach. In each cycle, the actuator was first pulled to a displacement d_1 of 5 mm (drift of 0.417%) then pushed to the same displacement. The value of d_1 was increased in the following cycles by increments of 5 mm. Strain gauges were used to monitor strains in the beam-column joint, the transverse reinforcement of the columns, and the longitudinal reinforcement of the beams. The locations of strain gauges on the test specimens are shown in Figs. 5 and 6. The following sections summarize the results of the experimental tests.

5. Sequence of failure of the tested specimens

The behaviour of the tested specimens was significantly different. For the moment frame specimen, the first observed crack occurred at a load of 30.0 kN and was a flexural crack in the bottom beam at the face of the column. By increasing the level of applied displacement, flexural cracks increased in number and width. No shear cracks were observed for this specimen. At a load of 37.5 kN, yielding of the bottom bars of the bottom beam initiated the plastic response. Failure occurred by plastic hinging at the ends of the top and bottom beams at a load of 55 kN. Figure 7 shows a photo of this specimen at failure. The diagonal members appearing in the figure were part of the instrumentation used to monitor the test and were not intended to provide any additional stiffness to the tested specimen.

The observed cracking load for the braced frame was 90.0 kN. Cracks observed in this frame were less in number and lower in width than that for the moment frame (Fig. 8). At a load of 105.0 kN, yielding of the brace member initiated the plastic response. Failure resulted due to buckling of the compressive brace, which was directly followed by plastic hinging of the ends of the bottom and top beams. The failure load for this specimen was 140 kN. It should be noted that the brace member connections, including welds and headed studs, behaved adequately.

6. Hysteretic behaviour

The lateral load-drift curves for the moment frame and the braced frame specimens are shown in Figs. 9 and 10, respectively. The initial stiffness of the braced frame was about 2.5 times that of the moment frame. The yield and failure drifts of the moment frame were 1.67% and 5.00%, respectively and those of the braced frame were 2.08% and 4.0%, respectively. This shows that the ductilities of the moment and braced frames were 3.0 and 1.9, respectively. The reduction in the ductility of the braced frame was compensated by a considerable increase in its lateral load capacity (over-strength) which was 140 kN compared to 55 kN for the moment frame. It is clear from the hysteretic behaviour that the pinching was less significant in the braced frame, indicating an overall better seismic performance.

7. Degradation of the lateral stiffness

The lateral stiffness was evaluated as the peak-to-peak stiffness of the frame load-displacement relationship. It is calculated as the slope of the line joining the peak of positive and negative loads at a given cycle. The lateral stiffness is an index of the response of the frame from a cycle to the following cycle. Figure 11 illustrates a plot of the lateral stiffness for the two tested specimens. Before buckling of the compressive brace, the diagram shows that the lateral stiffness of the braced frame was more than double that of the moment frame and that the rate of stiffness degradation for both specimens was almost equal. After buckling of the compressive brace, the lateral stiffness of the braced frame dropped and became comparable to that of the moment frame.

8. Energy dissipation

The ability of a structure to dissipate the ground motion energy is an accurate measure for its expected seismic performance. In this study, the energy dissipated by the two tested specimens during reversed cyclic load testing was calculated as the area enclosed by each hysteretic loop. Figure 12 shows a plot of the energy dissipated during a load cycle versus the lateral drift. It is observed that at low drift levels, the energy dissipated by the braced frame was less than that by the moment frame. This was mainly due to the initial high stiffness of the braced frame. At higher levels of drift, it is clear that the energy dissipated by the braced frame was much higher than that by the moment frame. This proves that the seismic performance of the braced frame is expected to be superior to that of the moment frame.

9. Transverse column reinforcement

The spacing of transverse bars in the columns of the moment frame and the braced frame were 35 mm and 70 mm, respectively. Figures 13, and 14 show the variation of strains measured in the stirrups located at a distance equal to 135 mm from the face of the beam-column joint. Unlike the case of the moment frame, the transverse column reinforcement of the braced frame did not reach the yielding strain. Moreover, the strains at failure in the transverse reinforcement of the columns of the braced frame were only 35%

of those of the moment frame. This indicates that forces in the brace member were mainly transferred to the beams and columns as axial forces, and thus a column shear failure is not expected.

10. Transverse beam-column joint reinforcement

The beam-column joint of the moment frame was transversely reinforced with two-6 mm steel wires in accordance to the special seismic provisions of the ACI code [17]. For the braced frame, the stirrups of the column were continued in the joint resulting in one-6 mm wire in the joint area. Figures 15 and 16 show the variation of strains in the beam-column joint transverse reinforcement versus the applied lateral load. It can be observed that the strains in the one-6 mm wire of the braced frame were about 40% that of the two-6 mm wires of the ductile moment frame. Thus, the use of braced frames is expected to eliminate the undesirable shear failure of beam-column joints without the need for any special joint detailing. However, this needs further testing to reach final recommendations.

11. Longitudinal beam reinforcement

Figures 17 and 18 show the strain variation in the top longitudinal bars (2M10) of the top beam of both frames versus the applied lateral load. It is shown that maximum tensile strains in the longitudinal bars of the braced frame were about 50% that of those in the moment frame. This is mainly due to the change in the distribution of internal stresses in the frame members from bending and shear stresses to axial stresses.

12. Discussion

The use of braced RC frames as the main lateral load resistance system for RC structures is a promising technique. The lack of guidelines and provisions addressing the design of such frames is hindering their use. A comprehensive research program addressing design issues pertaining to braced RC frames is needed. Such a program is expected to result in seismic modification factors and design methodologies for connections, brace members, and concrete members.

The present study focussed only on comparing the behaviour of a conventional moment frame and a braced RC frame. A rational method was adopted to design the braced RC frame. The flow of forces in the brace member connections was determined using the uniform force method [1, 18], which has been mainly developed for steel structures. Although the connections behaved adequately in the conducted tests, individual component tests are needed to modify the uniform design method to be applicable to concrete frames. Additional tests on braced RC frames are needed to identify suitable seismic modification factors for their design. The results of these tests can also be used to calibrate numerical models that can be used to conduct parametric studies for multi-storey braced RC frames.

13. Conclusions

In this paper, an experimental investigation was conducted to assess the behaviour of braced RC frames. Two specimens, a conventional moment frame with moderate ductility and a braced frame, were designed using the same seismic force reduction factor. The following conclusions were drawn based on the results of the cyclic tests.

- A braced RC frame designed using the same force reduction factor as that of a conventional RC moment frame with moderate ductility would behave adequately during an earthquake event.
- The design of RC sections in a braced RC frame can be carried out using conventional RC design methods. General reinforcement detailing requirements are adequate and there is no need to use special seismic detailing.
- The brace members and its connections can be designed using a similar procedure to that for braces in steel structures.
- The uniform method proposed to predict forces transferred by brace members to beams and columns is adequate. It prevents failure from occurring in connections of the brace members and minimizes the moment at the interface of the beam and column.
- The use of braced RC frames as the main lateral load resisting system is a promising design alternative. Significant experimental and computational research is needed in this area to develop adequate design guidelines and provisions along with best construction practice for such frames.

Acknowledgement

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Figure captions

Fig. 1. Lateral load resisting systems.

Fig. 2. Loads acting on the scaled model frames.

Fig. 3. Detailing of the RC moment frame.

Fig. 4. Detailing of the braced RC frame.

Fig. 5. Schematic of the test setup.

Fig. 6. Photo of the test setup.

Fig. 7. Cracks observed in the moment frame at failure.

Fig. 8. Cracks observed in the braced frame at failure.

Fig. 9. Lateral load-drift curve of the RC moment frame.

Fig.10. Lateral load-drift curve of the braced RC frame.

Fig. 11. Degradation of lateral stiffness.

Fig. 12. Variation of the energy dissipation with lateral drift.

Fig. 13. Strain variation in the column transverse reinforcement (moment frame).

Fig. 14. Strain variation in the column transverse reinforcement (braced frame).

Fig. 15. Strain variation in the beam-column joint transverse reinforcement (moment frame).

Fig. 16. Strain variation in the beam-column joint transverse reinforcement (braced frame).

Fig. 17. Strain variation in the top beam reinforcement (moment frame).

Fig. 18. Strain variation in the top beam reinforcement (braced frame).

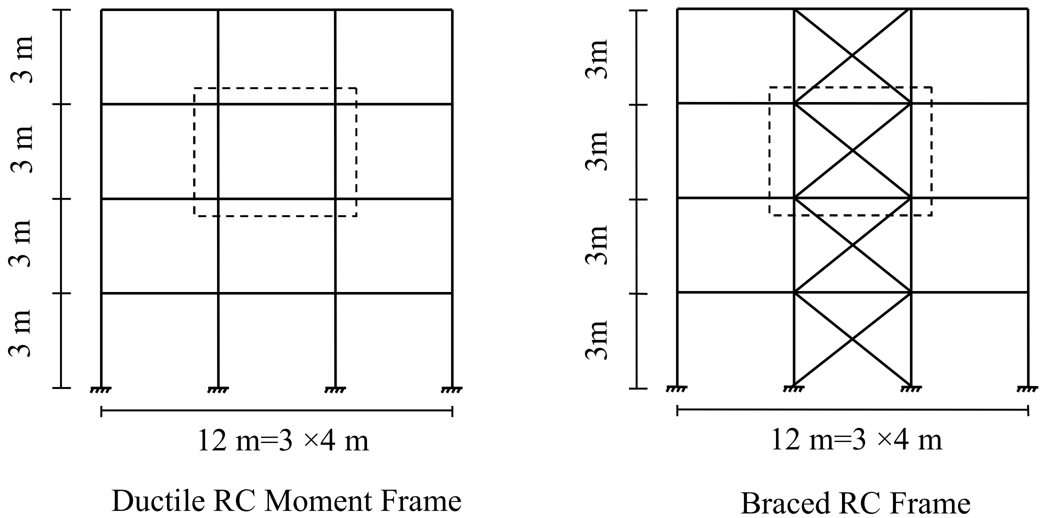

Fig. 1. Lateral load resisting systems.

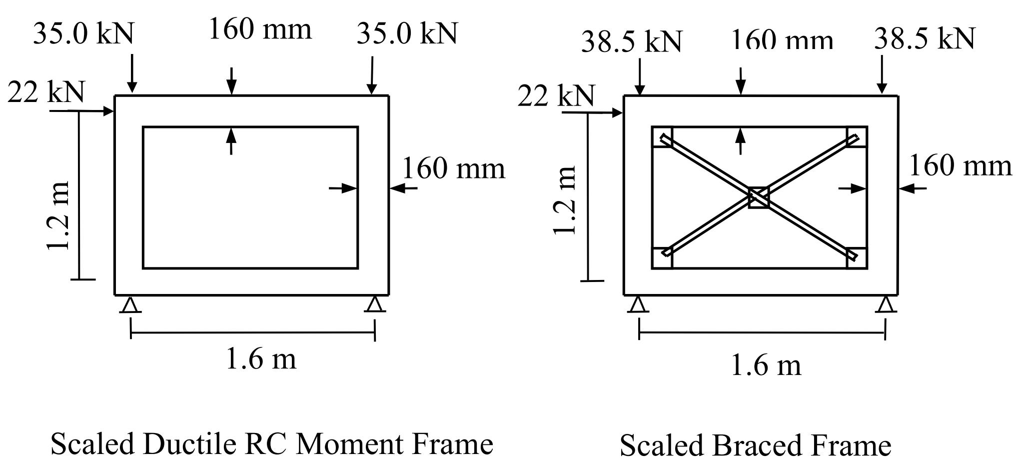

Fig. 2. Loads acting on the scaled model frames.

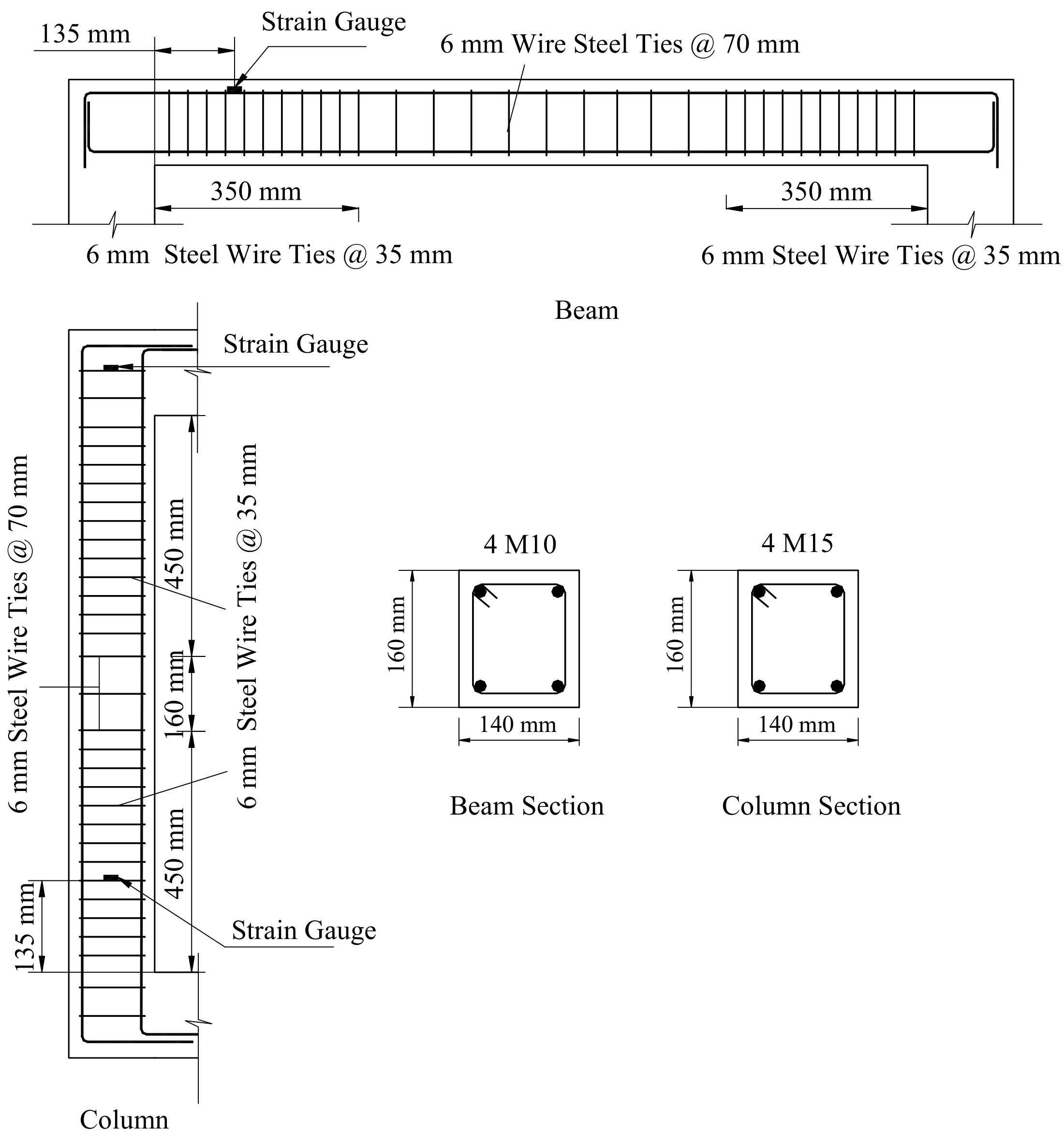

Fig. 3. Detailing of the RC moment frame.

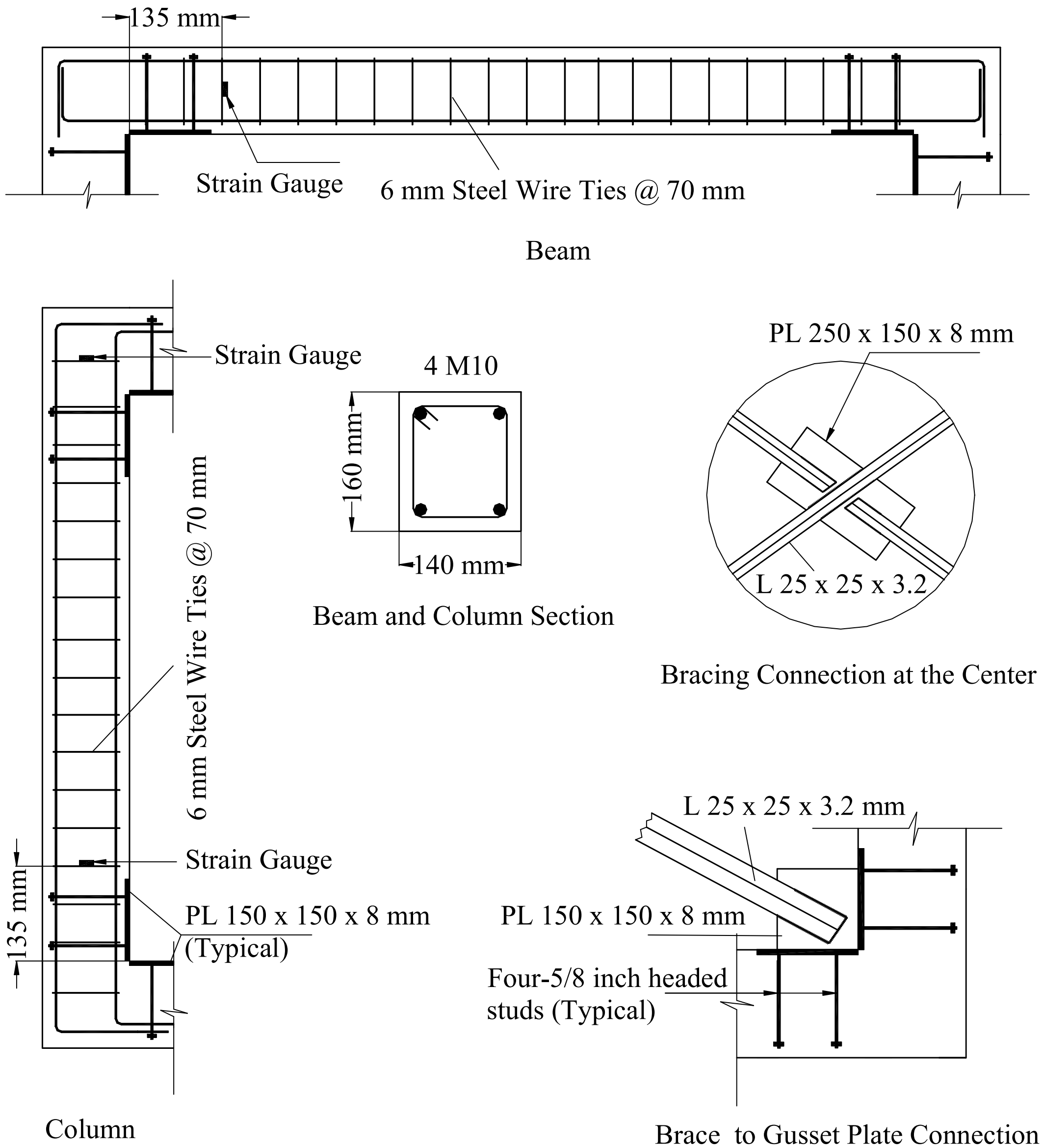

Fig. 4. Detailing of the braced RC frame.

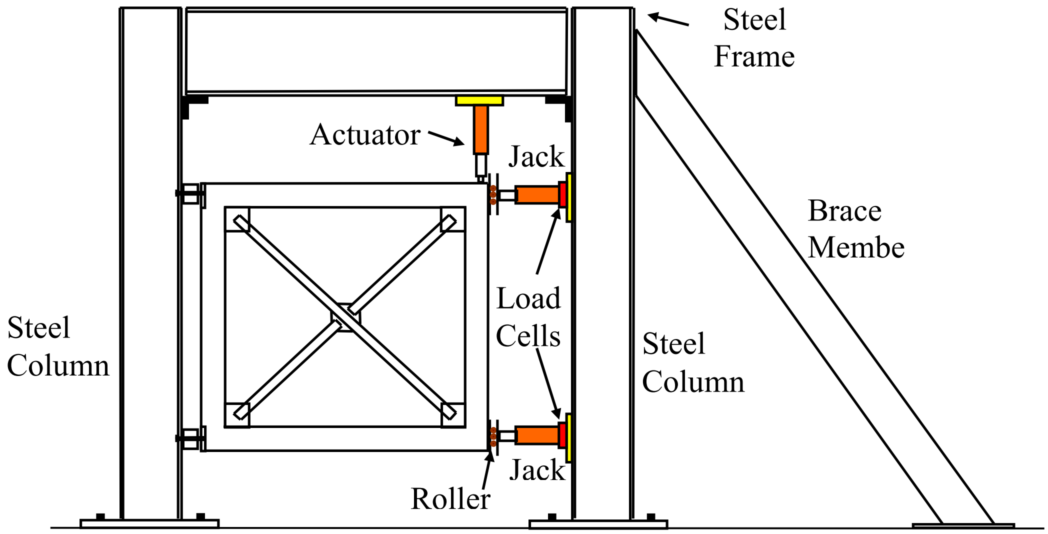

Fig. 5. Schematic of the test setup.

Fig. 6. Photo of the test setup.

Fig. 7. Cracks observed in the moment frame at failure.

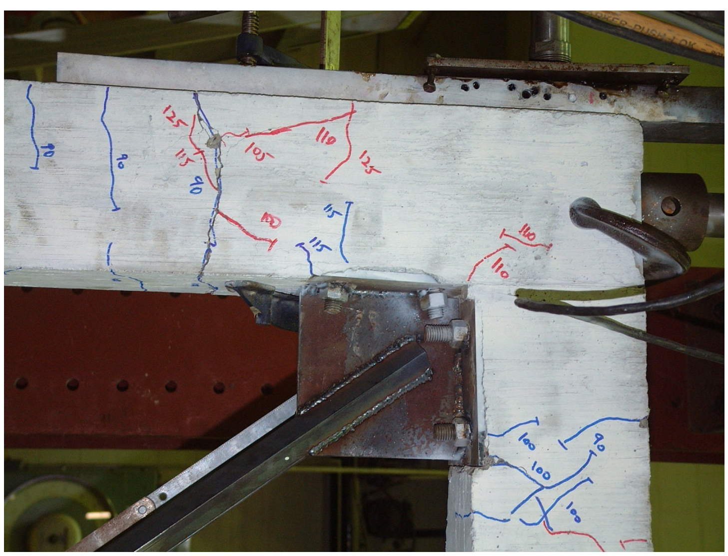

Fig. 8. Close up of cracks observed in the braced frame at failure.

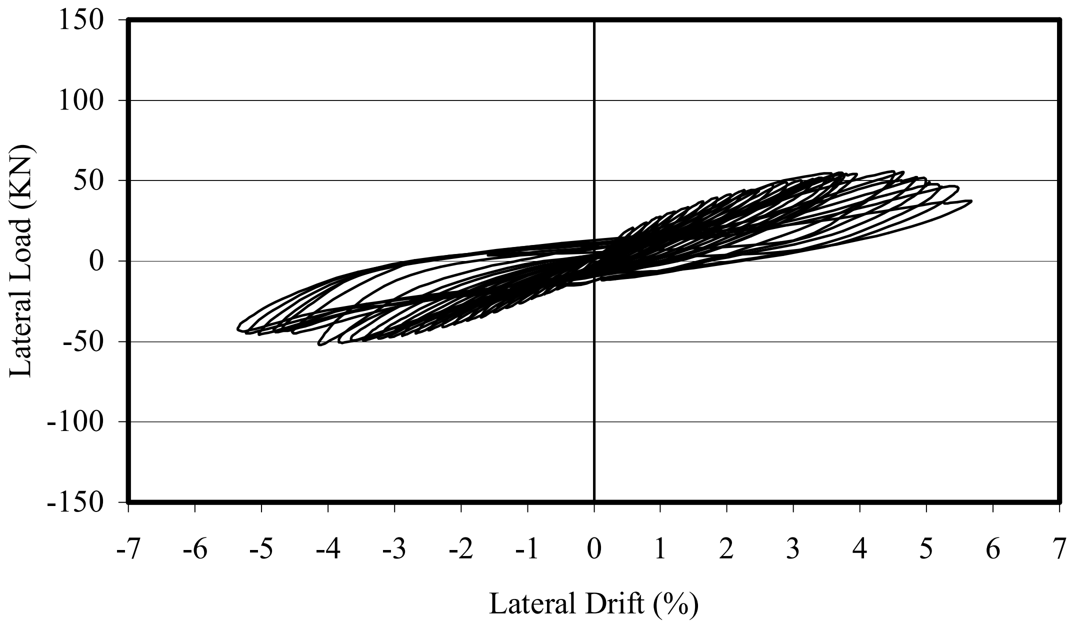

Fig. 9. Lateral load-drift curve of the RC moment frame.

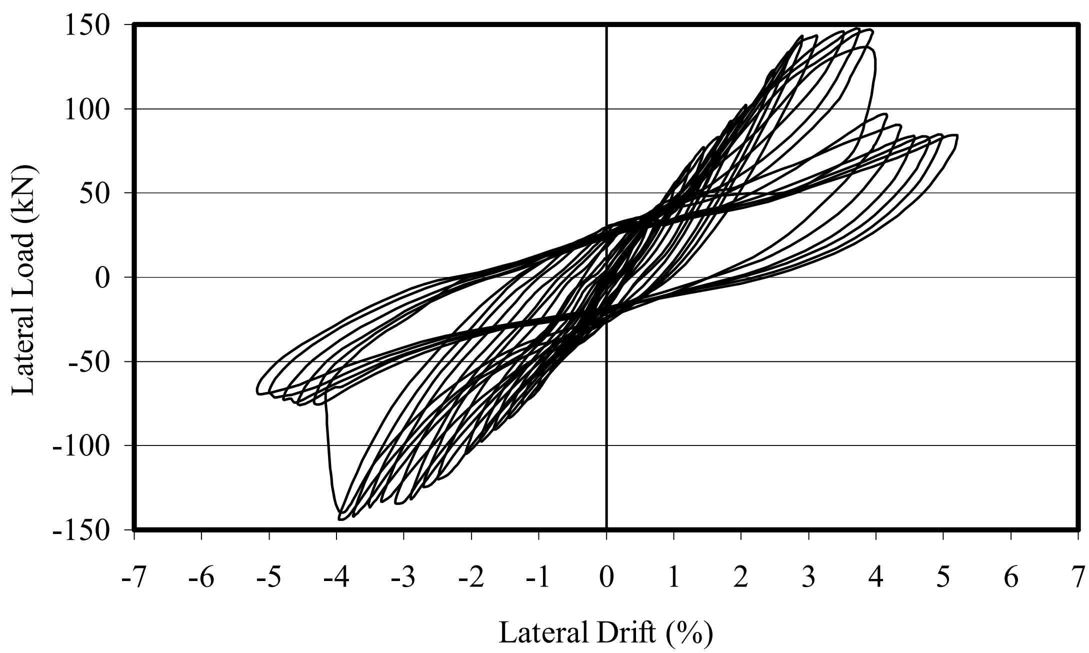

Fig.10. Lateral Load-drift curve of the braced RC frame.

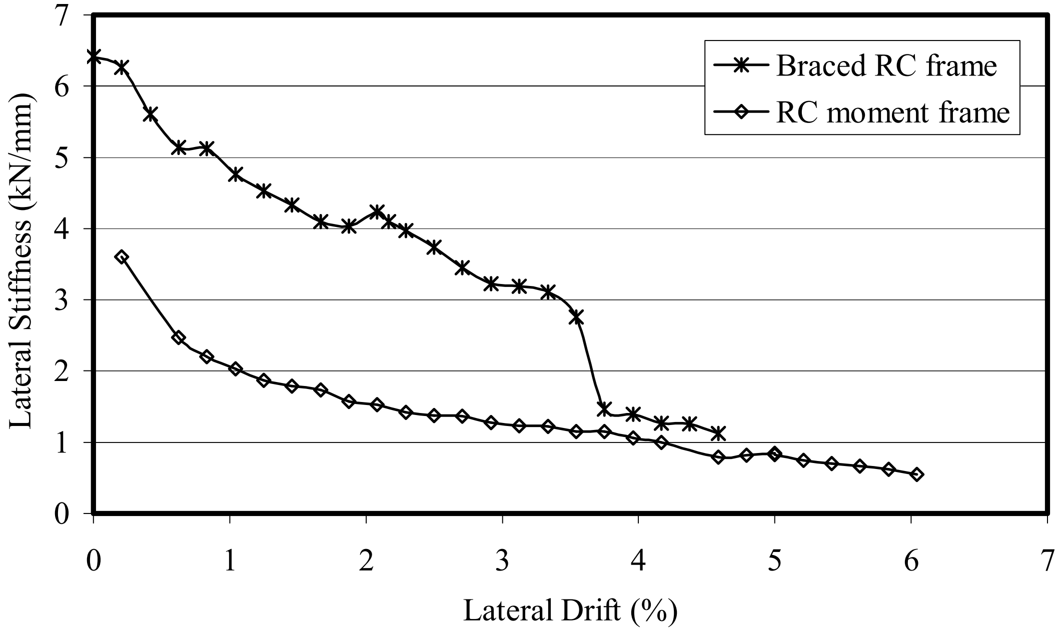

Fig. 11. Degradation of lateral stiffness.

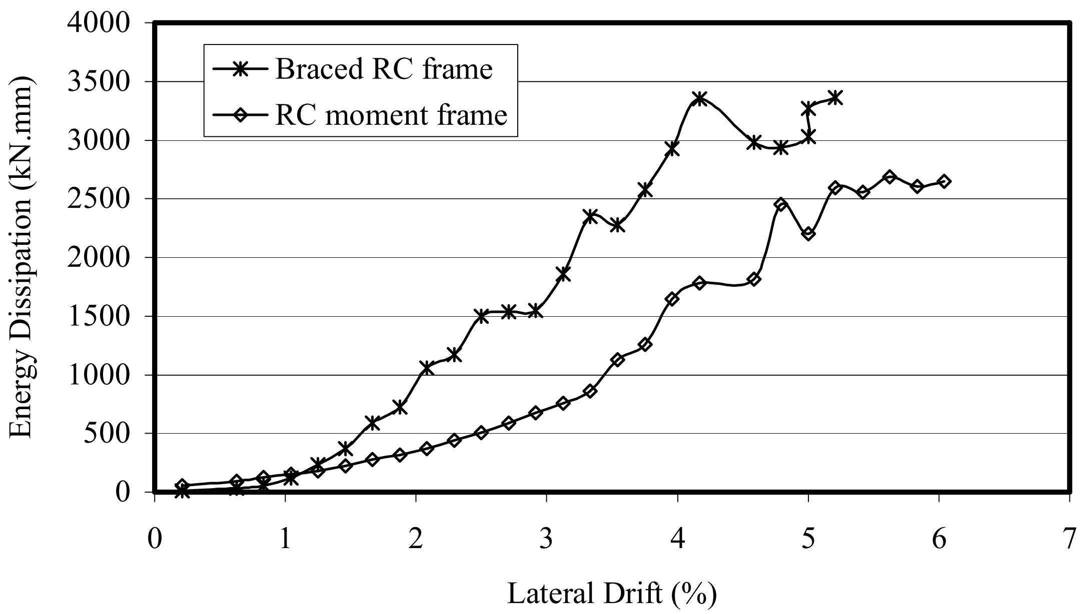

Fig. 12. Variation of the energy dissipation with lateral drift.

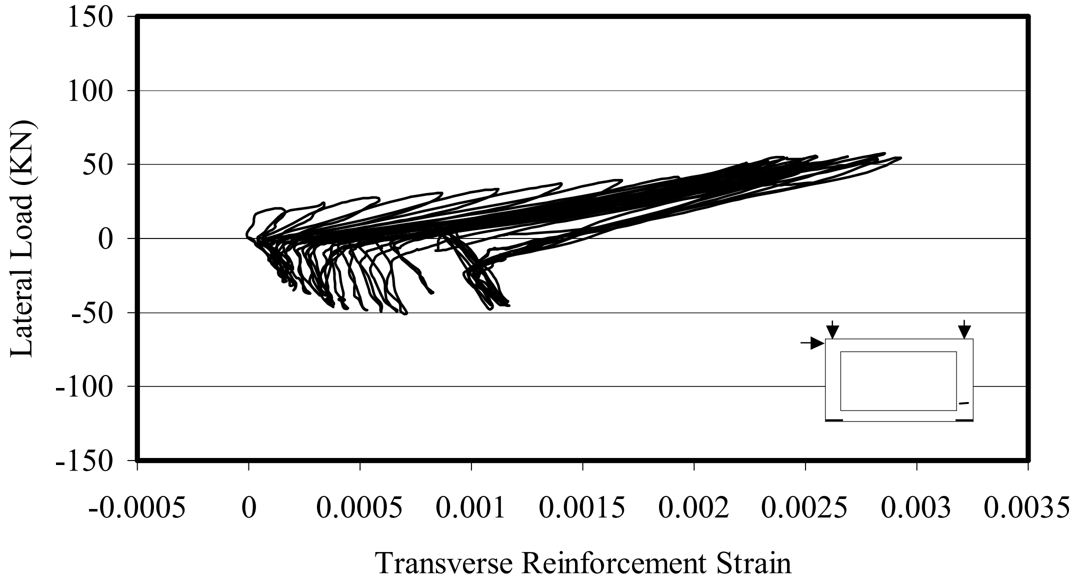

Fig. 13. Strain variation in the column transverse reinforcement (moment frame).

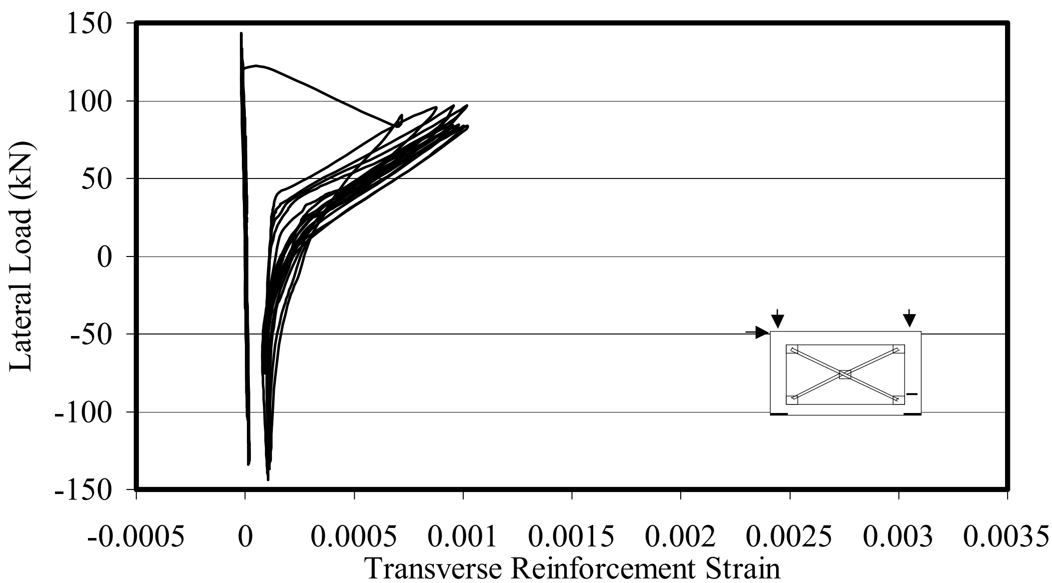

Fig. 14. Strain variation in the column transverse reinforcement (braced frame).

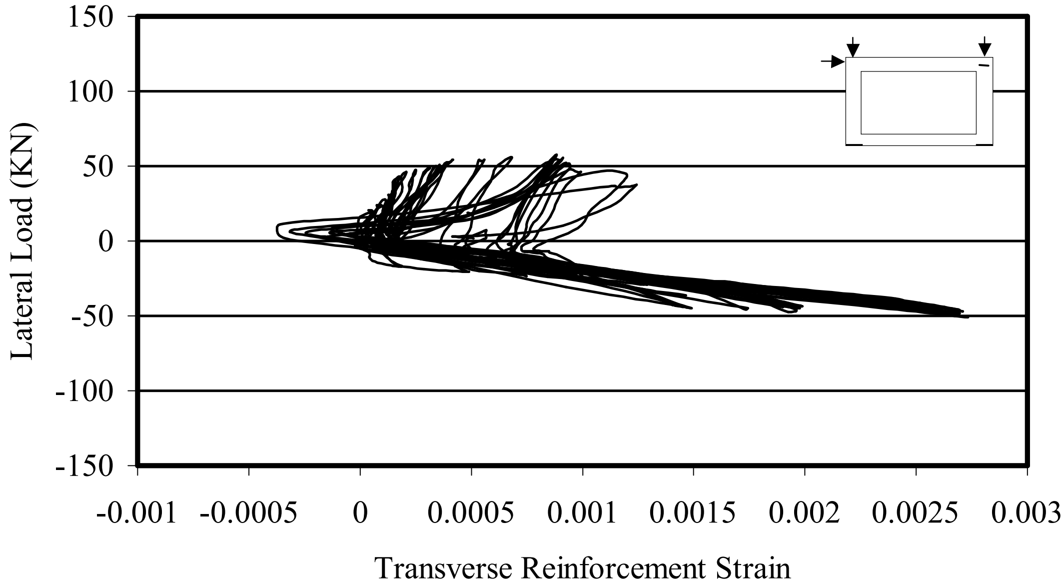

Fig. 15. Strain variation in the beam-column joint transverse reinforcement (moment frame).

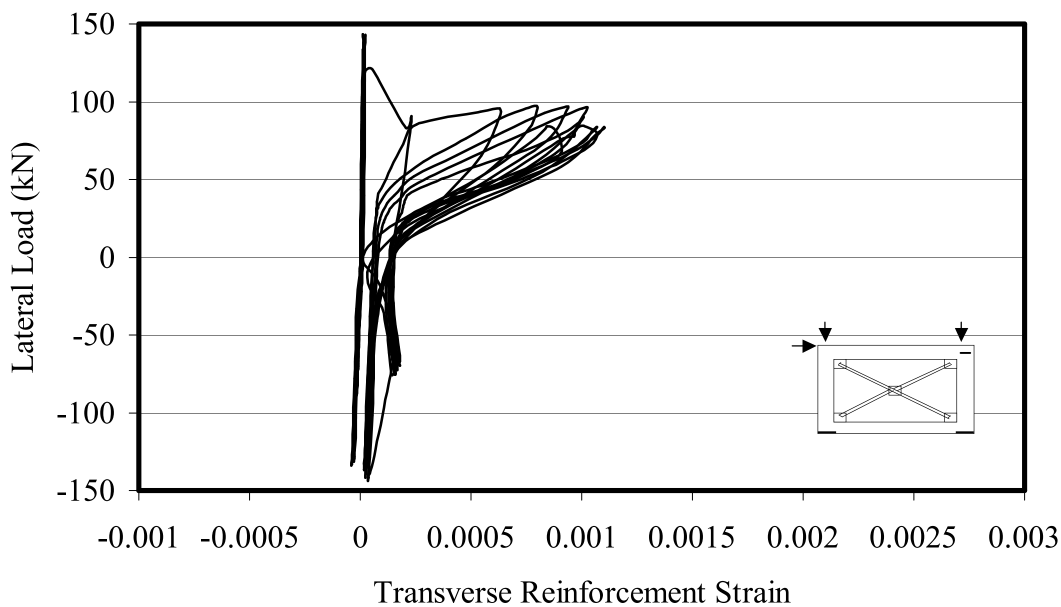

Fig. 16. Strain variation in the beam-column joint transverse reinforcement (braced frame).

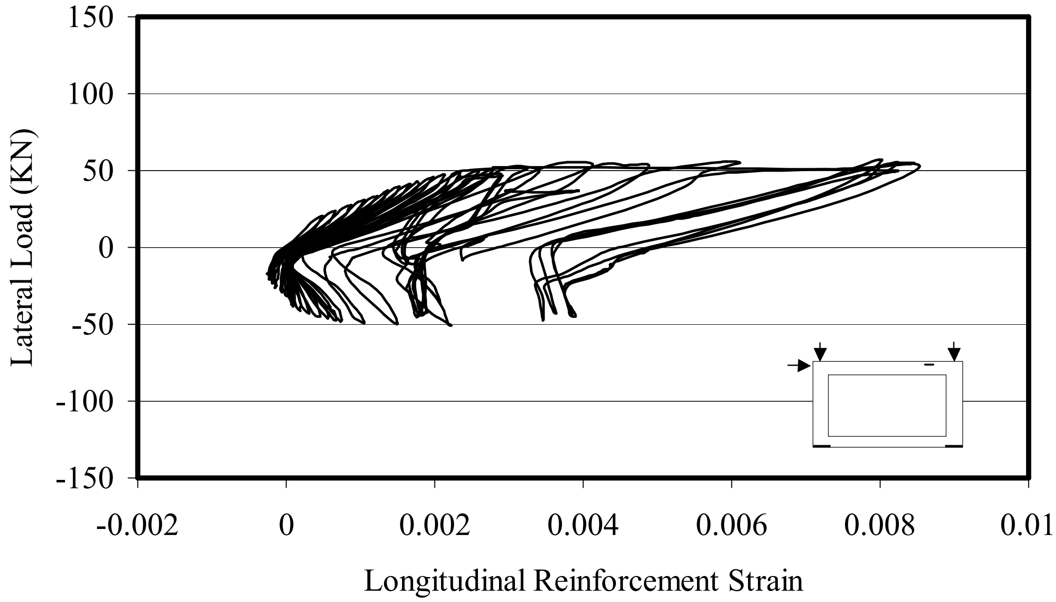

Fig. 17. Strain variation in the top beam reinforcement (moment frame).

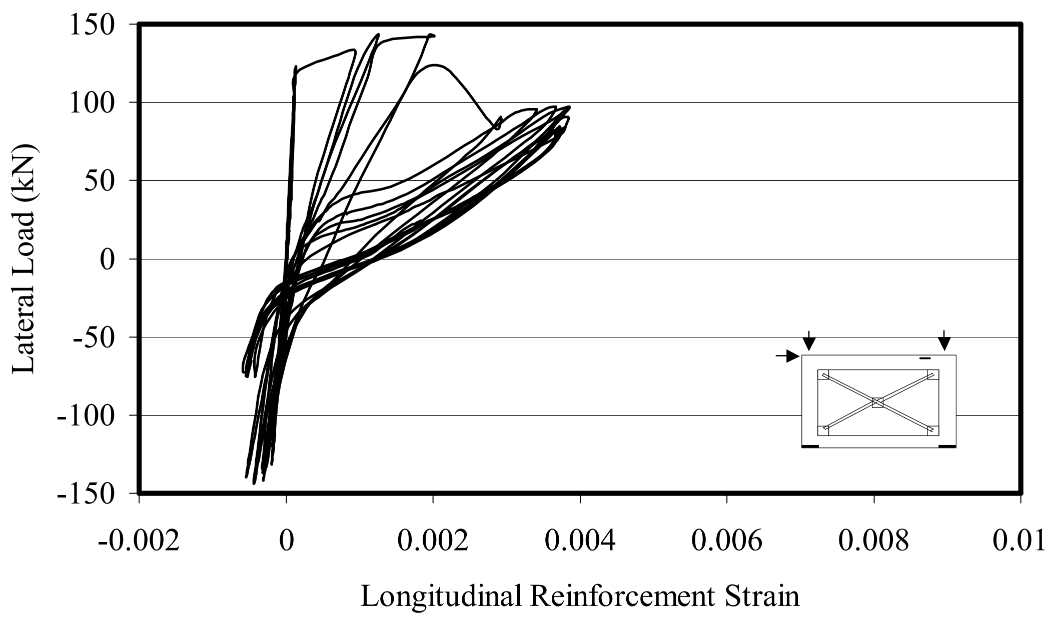

Fig. 18. Strain variation in the top beam reinforcement (braced frame).