Experimental Investigation on Link Column Frame System for Reinforced Concrete Structures

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Abstract. The structural and non-structural components of a building is subjected to seismic forces that must have adequate strength and stiffness to minimize the inter-storey drift during structure excitations. One of the most efficient methods was proposed by Dusicka et al. [6] in the Linked Column Frame (LCF) system for steel structures. The main objective of this system is in utilizing the replaceable components that are positioned in such a way to protect the gravity load carrying system of the structure. In this paper, the concept of LCF is extends to Reinforced Concrete structures. The experimental investigation carried out in this research presents the behaviour of LCF with various connections which include rigid and hinged connection as per IS 12303-1987. Design of the LCF system should be checked to ensure that the plastic hinges developed in the links of the linked column should have reduced storey drift compared to the plastic hinges developed in the moment resisting frame system. In this experiment, cyclic load tests are carried out on a single bay frame with and without linked column. A significant reduction in the relative storey drifts along with an increase in energy dissipation of the linked column frame was noticed.

Keywords: Storey drift · Cyclic load · Plastic hinge · Gravity load · Energy dissipation

1 Introduction

Providing life safety is the first priority in the earthquake resistant design. Due to the rise of performance-based design, the researchers have started to focus on decreasing the damage in the building and minimizing the costs for repairing and replacement in the mild seismic events. Two basic criteria should be satisfied in the design of earthquake resistant structures. (i) During the minor seismic events, the structure must have sufficient stiffness to keep deflection below the limit. (ii) During the major earthquake, the structure should possess sufficient ductility to avoid collapse.

For the earthquake resistant structures, the researchers have implemented lateral load resisting system such as moment resisting frames and braced frames. Moment resisting frames with traditional welded angle and bolted web connections, which is considered to be very ductile systems and it was extensively used between the 1960s and the early 1990s. This belief was put into question during the Northridge earthquake

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[1], in many cases without any signs of plastic deformation in the beam. A conventional MRF is designed to yield and form plastic hinges with associated damages in beams and columns. These damages can result in significant repair cost. Later the link beam concept in eccentrically braced frames depends on the inelasticity of specially designed links to provide energy dissipation and ductility during earthquakes. Researchers have begun to examine the possibility of using a bolted link design so that after a seismic event the damaged sections could be replaced [2]. Bolted links would also allow for cost-effective designs of buildings located in lower seismic regions [3]. However, there are a few disadvantages of bracing systems, such as (a) the complete replacement of the buckled or damaged braces after a major earthquake may be labour intensive and expensive, and (b) it may be necessary to upgrade the existing foundations at the bracing locations.

The LCF system incorporates aspects of conventional systems such as moment resisting frames (MRFs) and eccentrically braced frames (EBFs), but combines them to achieve performance that can be designed for multiple design objectives. The idea behind the LCF was based on recent developments in long span bridge design to building construction [4]. Later Dusicka et al. [5] investigated the inelastic behaviour of built-up shear links for seismic protection of bridges through the use of large-scale experiments, material investigation and numerical analyses [5]. Built-up shear links were shown to be effective hysteretic energy dissipaters. The lateral load resisting system, link column frame (LCF) system has introduced in steel structures [6]. In the Linked column frame (LCF) system, the dual columns which are placed in specific areas and it is linked independently to the moment frame throughout its height. Under lateral loading caused by an earthquake, the displacement of the dual columns engages the links which are designed to yield in shear to control drift, dissipate energy and limit the forces which are transferred to the nearby structural members.

In this paper, lateral resisting system, i.e. the link column frame (LCF) system, is extended to Reinforced concrete frame which elevation was shown in Fig. 1. This system consists of replaceable link beams which are intended to yield in shear placed

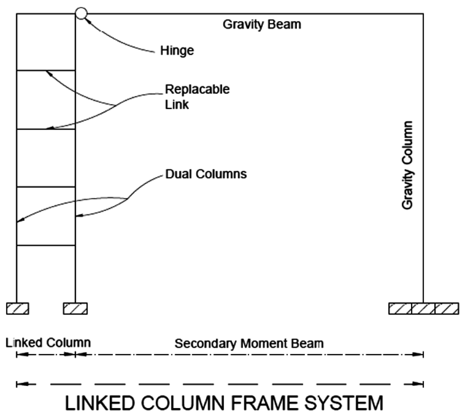

Fig. 1. Elevation of the link column frame system

between closely spaced dual columns and an adjacent flexible moment resisting frame in which the beam is restrained at one end and hinged at another end. The links act as a structural fuse which sacrifices itself by yielding to provide ductility, energy dissipation, and nonlinear softening behaviour while limiting the relative damage and inelastic behaviour of the structural members of the nearby moment resisting gravity frame. In the LCF system link beam will act similarly to links in eccentrically braced frames, i.e. they can yield in flexure or shear depending on their link length.

The main objectives are: (a) to study the behaviour of the shear link beam, load-transferring system and RC frame of the strengthened specimen under lateral loading, and (b) to evaluate the overall performance of both strengthened and RC (bare) specimens in terms of lateral strength, lateral stiffness, energy dissipation, and damage levels. The experimental program consisted of testing 1/3rd scaled model of normal frame, linked column frame with a rigid connection in which the normal beam is connected rigidly to the linked column and linked column frame with a hinged connection i.e. the normal beam is made as a hinged connection to the linked column. The response to cyclic loading was studied.

2 Design Procedure

To achieve the desired performance at a specific seismic hazard level the linked column frame should be designed. The shear link in the linked column frame will play a major role while dissipating the energy. The basic design procedures are as follows (1) Determination of link length; (2) determination of link column member sizes; (3) determination of link beam member sizes.

2.1 Determination of Link Length

The design of links was performed according to the Seismic Provisions (UBC 1994) [7]. The design of the link length is done similarly to the link length in eccentrically braced frames and their yielding behaviour depends on their section properties and their length. Due to the huge unreliability's in predicting the inelastic behaviour of long links [8], response to link detailing for moment links in average seismic zones is not recommended. The intermediate links will provide combine bending with shear behaviour, and this makes some difficulties than long or short links.

The main intention of the link is to resist the shears induced and to yield under huge shear forces which are expected during seismic force. The capacity of the link is to yield in shear deformation during large displacements produces inelastic deformation in the link, its collapse mechanism was shown in Fig. 2. To achieve a link rotation of the links γ_p equal to 0.08 rad, the length of the link e should be

$$e < 1.3\frac{M_s}{V_s} \text{ (recommended upper limit)} \tag{1}$$

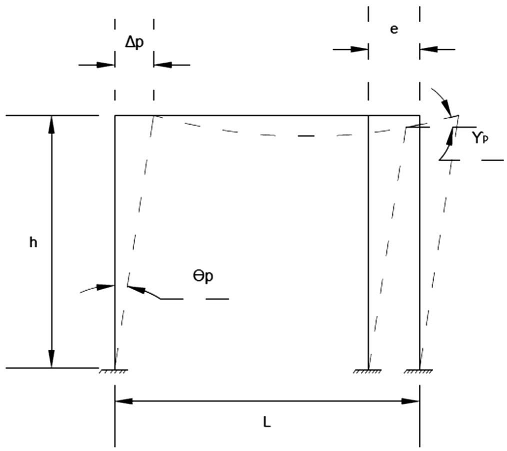

Fig. 2. Collapse mechanism of linked column frame

The moment capacity and plastic shear of a link, M_s, and V_s, are determined from the following equations

$$V_s = \tau_y A_V \frac{1}{\gamma_m} \tag{2}$$

$$M_s = Z_p \sigma_y \tag{3}$$

Where τ_y is the shear stress for the section, A_V is the shear area of the section, γ_m is the partial safety factor of the material, σ_y is the yield stress of the material, and Z_p is the plastic modulus respectively. The above equations were used to design the length of the links for one bay for the cross section mentioned in Fig. 4a and the values drawn from above equation are given in Table 1.

Table 1. Length of links

Z_p (mm^3)	A_V (mm^2)	M_s (N.mm)	V_s (N)	e (mm)
6750	196	10750	68.17	200

The axial force effect of the shear links ability needs to be satisfied if (UBC [7]):

$$\frac{P_u}{P_y} > 0.15 \tag{4}$$

where P_u is the required axial strength and $P_y = \sigma_y A_g$ is the nominal axial strength of the link (A_g is the total cross-sectional area of the link). The shear link beams are subjected to bending moments and large axial forces created by the yielded link. It is necessary to design the structural elements other than the link element should remains elastic. The beam in the main frame should be flexibly designed to create the link beam to act as a structural fuse, which makes the link to yield and dissipate energy. The beam in the moment frame should use larger cross section so that it can be in elastic stage under huge lateral forces transferred from the shear links.

2.2 Determination of Link Column Member Sizes

The link column was designed as cantilever column to take the entire lateral load on a particular floor. Based on the drift limitations as per IS 1893:2002 (Part-I) [9] the size of the link column was determined using the relation Where the moment of inertia for the link column and link beam was derived from Lopes et al. [10]. Similar expressions for total and partial deformation can be obtained for different story levels.

$$I_{LC} = \frac{x^{4.6} P h^3}{8E\Delta} \tag{5}$$

$$\Delta = \frac{PL^3}{3EI} \tag{6}$$

Where, Δ is the displacement, P is the lateral load, I_{LC} is the moment of inertia, E the Modulus of elasticity and I is the Moment of Inertia of the section, x is the number of stories, L is the storey height of a single linked column and h is the story height of the linked column frame. Consider one bay frame subjected to lateral seismic forces represented by a cantilever column represented in Fig. 3. Lopes et al. [11] and Malakoutian et al. [12] numerically analyzed 6-story Linked column frame using time-history and pushover analyses, respectively.

A one story Linked column frame with a storey height of 1000 mm and length of the link equals to 200 mm is considered for a total base shear of 7700 N. The lateral

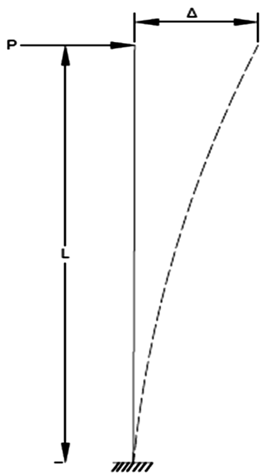

Fig. 3. Cantilever column of a one storey linked column frame

load P is taken as 7700 N. The equivalent lateral force procedure is used to obtain the total base shear as per IS 1893:2002 (Part-I). The location of the building was in Zone V.

2.3 Determination of Link Beam Member Sizes

The Vierendeel column approach is based on the assumption that the linked columns of the LCF building could be represented by a rectangular configuration with rigid joints. This approach is used to calculate the moment of inertia for the link beam.

$$I_L = 0.6\frac{I_{LC}H}{h} \tag{7}$$

Where H is the link length, For a preliminary link beam member sizing, assume $\theta = 0.02$ rad and also that the LCF should meet the design intent of 2.5% inter-story drift limits. Following Table 2 shows the dimensions of link column and link beam.

Table 2. Link column and link beam dimensions

	Breadth	Depth
Link column	57 mm	57 mm
Link beam	30 mm	30 mm

3 Experimental Study

The nominal 2 storey reinforced concrete building was designed and it was scaled down to 1:3 ratio according to our capacity of the experimental setup. Three specimens were casted, one normal frame and other two are linked column with different connections. To find out the performance such as stiffness, damage propagations, lateral strength and energy dissipation the static cyclic loading was done. The testing was carried out in Structural Engineering Laboratory of Karunya University.

3.1 Specimen for Experimental Investigation

The concrete was cast in site with a coarse aggregate size of 10 mm in diameter. The material tests for the concrete and reinforcement was done to find out the stress-strain relationship of them. Concrete's 28-day nominal compressive strength was found as 27 N/mm^2, and the nominal yielding and ultimate stresses of TMT rods were determined as 420 N/mm^2 and 500 N/mm^2, respectively. The beam–column connections are not designed for the confinement reinforcement. To carry out experimental investigation three reinforced concrete frames were taken with and without link column system. The detailing of the normal frame without link column and link column with a rigid connection between the frame and the column and the detailing of hinged connection between the beam and the link column which are shown in Fig. 4. For hinged

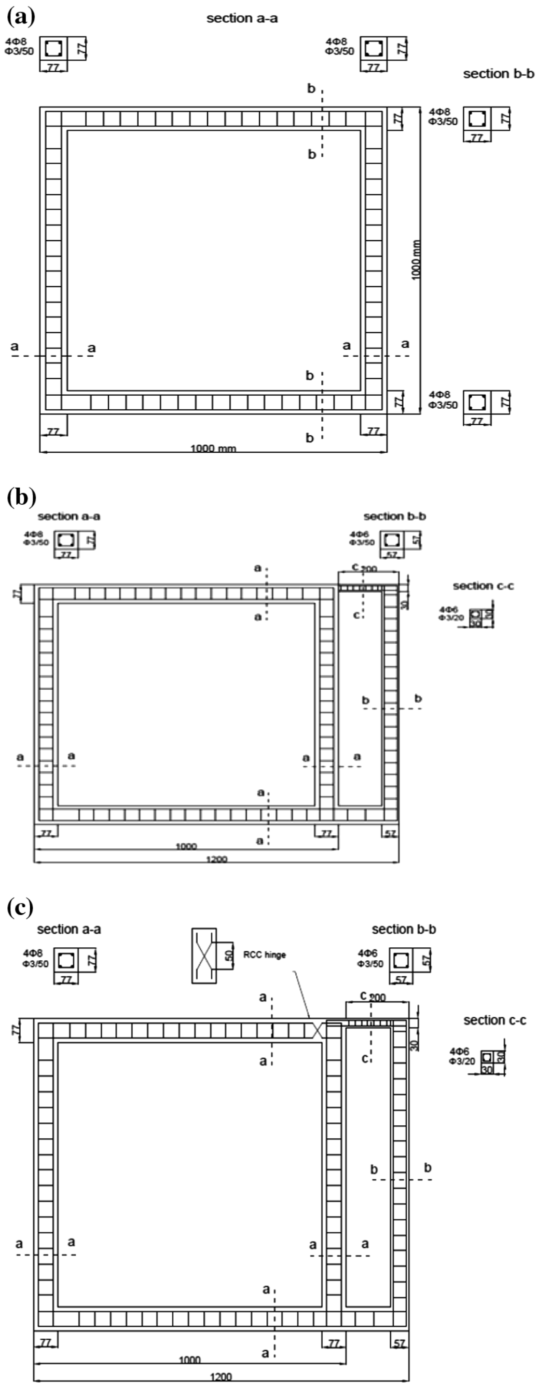

Fig. 4. a. Reinforcement details of normal frame. b. Reinforcement details of rigid linked column frame. c. Reinforcement details of hinged linked column frame

connection in linked column frame, the beam of the moment frame is connected to the linked column as hinge connection as per IS 12303:1987 [13]. The hinges were designed as Mesnager hinges which transmit the thrust and shear force and it will allow rotation.

The physical length of the real plastic hinge region is logically believed to have a certain intimate relationship with L_p. The conventional plastic hinge length L_p is considered as a virtual length over which a given plastic curvature is assumed to be constant for integration of cross-sectional curvatures along the RC member length [14], which is given by

$$L_p = 0.5d + 0.05z \tag{8}$$

where d = effective depth of the beam, z = distance from the critical section to the point of contraflexure from the above equation the length of the plastic hinge is taken as 50 mm.

3.2 The Test Setup

In-plane lateral load was applied at the end of the top beam along its centroidal axis through one end of the servo-hydraulic actuator. The other end of the actuator was supported by a strong steel reaction frame. The lateral load was applied to the beam of the moment frame by two channel sections, which is symmetrically placed on both sides of the beams and held together by high strength bolts at regular intervals. The footing was hardly placed in the strong floor by means of three bolted connections to avoid its possible vertical and horizontal movements due to the lateral loadings. The lateral support was given by the steel sections to avoid it's out of plane movements. Due to the limitation of the test setup as well as the difficulty in real-time force control, the axial load is not applied to the columns in this study. The purpose of this study is more focused on the behaviour of the link in the RC specimen. The test setup was shown in Fig. 5.

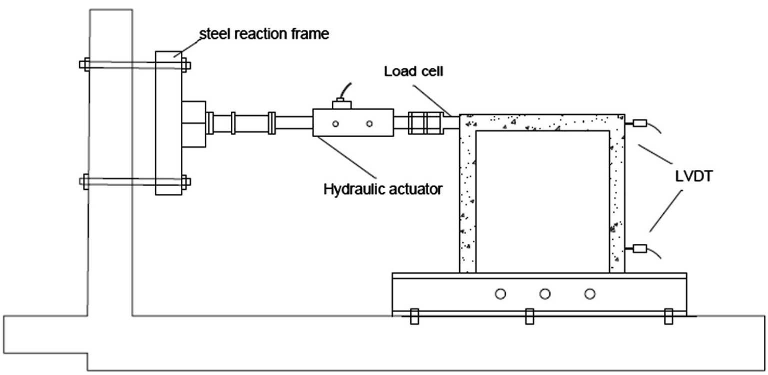

Fig. 5. The test setup

3.3 Loading Sequence

The loading scheme used for these specimens was a load controlled type which is shown in Fig. 6. The loading cycle upto 15 kN was given to the specimen. At particular load, each load cycles was repeated three times and followed by an increase of 1 kN.

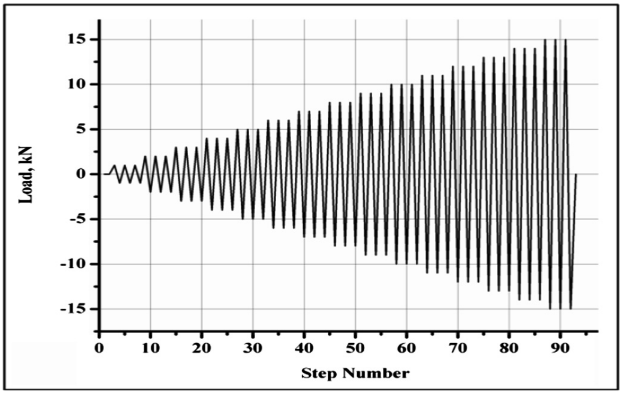

Fig. 6. Loading history

3.4 Instrumentation on Test Specimens

To monitor the lateral load and displacement applied to the test specimen the load cell and displacement transducer in the actuator were used. The lateral displacements at the top and bottom of the column of the test specimens were measured using two LVDT's (linearly varying differential transformers). The typical instrumentation scheme is shown in Fig. 5.

4 Experimental Results

A cyclic load test was conducted on the three frames and the behaviour of the specimens was studied. Discussions of the results are shown below.

4.1 Normal Frame

The normal frame was designated as the reference frame to compare its performance against linked column frame specimens. Fifteen full load cycles were applied to the frame. First shear crack were observed at the load of 18 kN and the corresponding displacement was 8.353 mm. At the end of the 5th cycle, first yielding of the longitudinal reinforcement was observed at 10.56 mm displacement. The maximum displacement was observed as 37.3 mm with a lateral load of 43.0 kN. The load versus deflection graph and the damage pattern were shown in Fig. 7.

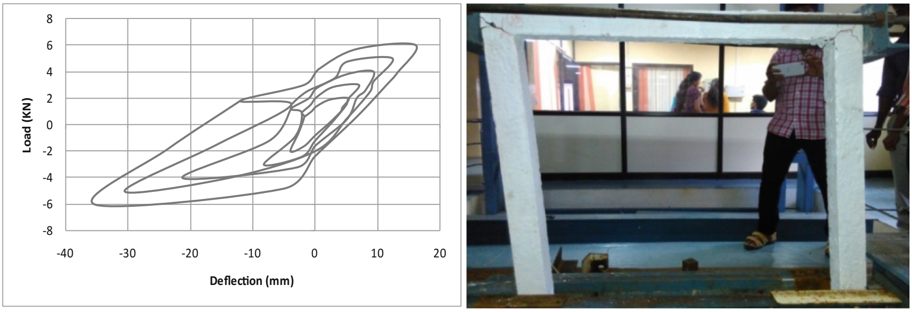

Fig. 7. Load versus deflection and a damage pattern of normal frame

4.2 Rigid Link Column Frame

This specimen is a link column frame in which the normal beam is rigidly connected to the linked column. Fifteen full load cycles were applied to the frame. First flexural cracks were observed at 18 kN at a displacement of 9.6 mm which is occurred at the link joints. First shear crack was observed at 26 kN when the displacement was 10.26 mm. At the end of the 8th cycle, first yielding of the longitudinal reinforcement was observed when the force was measured as 40 kN and displacement was 15.56 mm. The load versus deflection graph and the damage pattern were shown in Fig. 8.

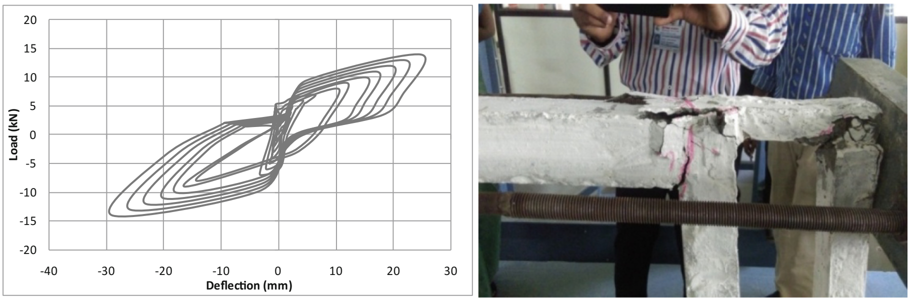

Fig. 8. Load versus deflection and a damage pattern of rigid linked column

4.3 Hinged Link Column Frame

This specimen is a link column frame in which the normal beam is flexibly connected to the linked column. Fifteen full load cycles were applied to the frame. First shear crack was observed at a force of 35 kN when the displacement was 7.6 mm which is occuring at the beam of the moment frame. At the end of the 9th cycle, first yielding of the longitudinal reinforcement was observed when the force was measured as 45 kN and displacement was 15.56 mm. The load versus deflection graph and the damage pattern were shown in Fig. 9.

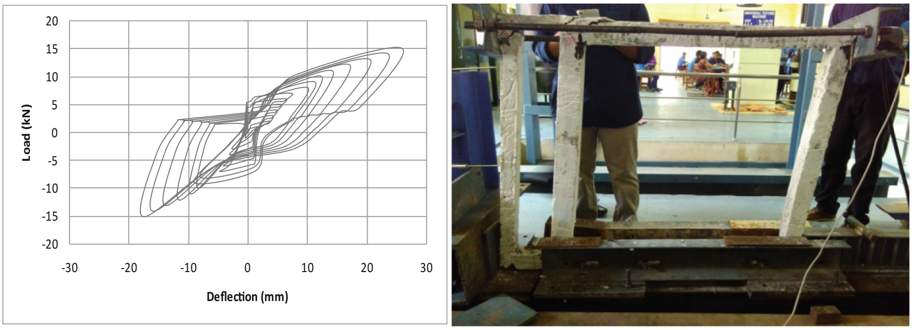

Fig. 9. Load versus deflection and a damage pattern of rigid linked column

5 Analysis of Test Results

The obtained test results are compared with each other in terms of general behaviour, strength, stiffness, cumulative damage, energy dissipation capacity, and ductility, over strength and performance factors.

5.1 Lateral Strength and the General Behaviour of the Specimens

The load versus deflection backbone graph is given in Fig. 10. The post yield hardening was noted in the hinged linked column with no degradation of lateral strength upto the displacement of 17 mm. Meanwhile, the backbone curve of the normal frame specimen was approximately linear upto the displacement of 7 mm after that both the stiffness and the lateral strength were decreased due to huge damage in the beam near the beam-column joint. The strengthened specimen has the maximum load carrying

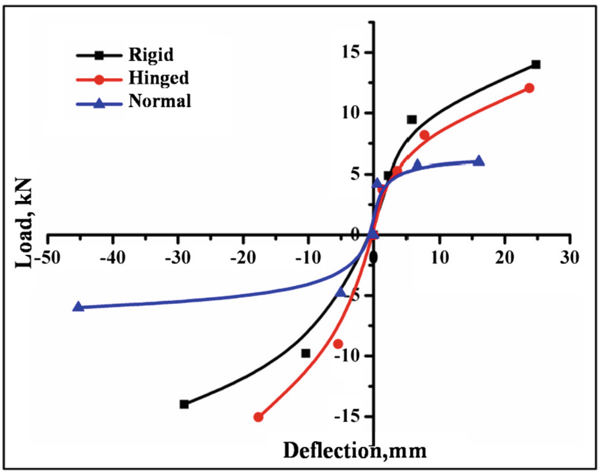

Fig. 10. Backbone curve

capacity of 71 kN at the displacement of 17 mm when compared with the normal frame. This magnification in the lateral strength of the hinged linked column frame was principally due to the lateral load sharing mechanism in the shear link. Due to the hinge connection in the moment frame, it won't take a load from the link beam and its overall strength of the linked column frame increases.

5.2 Energy Dissipation

The expected seismic energy is a measure of the structure ability to dissipate the seismic input energy. The sum of the area enclosed by each hysteretic loop is used to determine the cumulative dissipated energy. The normal frame is the specimen which has the minimum energy dissipation capacity. The linked column frame which has a flexible connection is the one which dissipates maximum energy when compared with the rigid connection. The hinged linked column frames dissipated 65% more energy than normal frame. This enables the plasticization to occur in links in lower drift compared to beams in higher drifts. The dissipated cumulative energy versus deflection relation for all specimens are shown in Fig. 11.

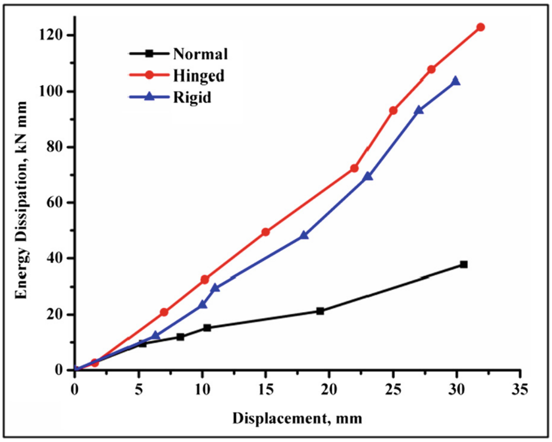

Fig. 11. Energy dissipation versus deflection

5.3 Lateral Stiffness

The lateral stiffness was defined as the slope of the line connecting the positive and negative peaks of a given load–displacement cycle. The presence of hinged connection reduces the lateral stiffness of hinged linked column frames. As expected, the lateral stiffness decreases for a hinged linked column when compared with the rigid linked column. The lateral stiffness is reduced by about 21.7%. The overall stiffness of the rigid linked column frame is 1.29 times greater than the normal frame's stiffness, respectively. A variation of the lateral stiffness with respect to displacement for all specimens is given in Fig. 12.

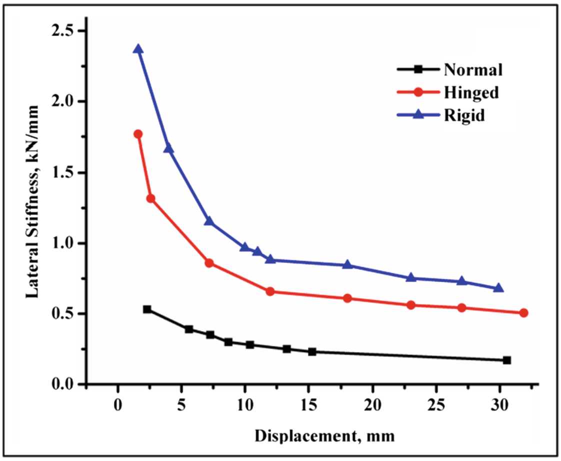

Fig. 12. Lateral stiffness versus deflection

6 Conclusions

The following conclusions can be drawn from the present study.

1. The toughening of reinforced concrete frame with a shear link as an energy dissipation mechanism which enhances the damping, lateral strength, and potential to dissipate energy. For the hinged linked column the load-deflection behaviour consists of hysteresis loop it shows with significant post-yield strain-hardening behaviour without any degradation in stiffness and strength.
2. The energy dissipation capacity of the specimens was increased by 70% for a hinged linked column and 62% for a rigid linked column when compared to the normal frame. The shear link is used to decrease the lateral load demand and hence the frame member's damage level should be controlled. The RC moment frame columns of the hinged linked column did not suffer any damage upto the displacement of 17 mm.
3. The shear link in the linked column is more reliable and effective in dissipating a large amount of energy. To establish its suitability under earthquake type loads the design methodology of the proposed shear link which act as a sacrificial element in reinforced concrete has been developed together with experimental studies. The cyclic loading results show that the link column frame system will have high strength at lower displacement.
4. The improving seismic performance was found in the building by providing linked column, which absorbs the lateral input energy more than the normal building. This method can be effectively used as new earthquake resistant construction. The cost of the construction should be reduced when the replaceable links are modelled with Reinforced concrete elements.

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