

ENHANCEMENT OF SEISMIC PERFORMANCE OF REINFORCED CONCRETE STRUCTURES USING LINK COLUMN SYSTEM

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ABSTRACT

A novel Link column system has been proposed for the seismic response reduction of Reinforced concrete structures. This paper presents experimental and analytical results carried out on single bay RC frames with and without link column. A design procedure is proposed to ensure that plastic hinges form in the links of the link column at a remarkably lower story drift than when plastic hinges develop in the moment frame beams. The link columns are designed as cantilever column to resist the lateral loads. Three models were considered for the study, i.e. without link column (M1), with rigid connection between main beam and column (M2) and with hinge connection between main beam and column (M3). The experimental investigation consisted of cyclic load test on the three models. Analytical investigations using ANSYS was done to validate the experimental results. The test results showed that the energy dissipation of the link column frames increases significantly with a decrease in relative story drifts.

KEYWORDS: Non-Linear Pushover Analysis, Lateral Loading, Link Column Frame, Seismic Response, Energy Dissipation

INTRODUCTION

The increasing number of seismic activities has emphasized the need of seismic resistant structures to prevent the loss of human life. Generally, building owners are no longer satisfied with the performance of the building which provides only life safety due to seismic loading i.e. load imposed by an earthquake. Huge expectations are going on in the engineering field to reduce the economic loss by decreasing the structural damage and by ensuring continuity of the structural usage. In high seismic zone areas, the seismic design approaches currently include usage of ductile lateral load resisting system, where one of the most popular systems is moment resisting frames.

Moment resisting frames with traditional welded angle and bolted web connections were believed to be very ductile systems and were extensively used between the 1960s and the early 1990s. This belief was put into question during the Northridge earthquake (Bruneau et al., 1998; Behnam and Ronagh 2014), 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 result in significant repair costs. To enhance the seismic performance of the RC buildings, suitable local or global modification techniques have to be adopted. In the RC framed structure bracings are very often used to upgrade the stiffness, energy dissipation and lateral resistance (Ile and Reynouard 2003).

Later, the link beam concept in eccentrically braced frames depending on the inelasticity of specially designed links, to provide energy dissipation and ductility during earthquakes was developed. 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 (Park and Yun 2005). Bolted links would also allow for cost-effective designs of buildings located in lower seismic regions (Hines, 2009). But eccentrically braced frames have the disadvantages like it will occupy the significant space and suffers shortcomings when return to occupancy.

The Link Column Frame system (LCF) incorporates aspects of both 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 the recent developments in long-span bridge design (Nader et al., 2001). Later Dusicka et al., (2006) investigated the inelastic behavior of built-up shear links for seismic protection of bridges through the use of large-scale experiments, material investigation, and numerical analyses. Built-up shear links were shown to be effective hysteretic energy dissipaters. The link column frame (LCF) system has introduced in steel structures (Dusicka et al., 2009). In the Linked column frame (LCF) system, the dual columns which are placed at 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 and the elevation of the linked column frame is shown in Figure 1. This system consists of replaceable link beams which is intended to yield in shear and these beams are placed 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 to 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. Analytical investigations on the full scale models has been carried out by the authors and the performance of the system was satisfactory (Shelton and Hemalatha 2016).

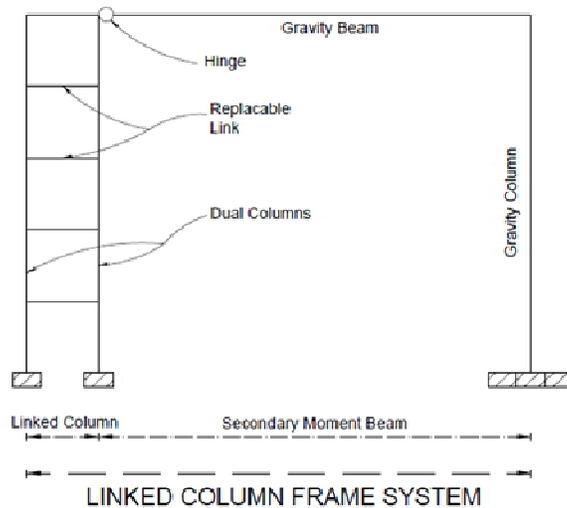


Fig. 1 Elevation of the Link Column Frame System

The objective of this research is to develop an innovative a new lateral RC system, that has the ability to provide immediate occupancy criteria after a major earthquake while maintaining the appeal of moment frame systems. The secondary frame system is designed as a sacrificial beam column system to yield in the inelastic range whereas the main system is in the elastic range. The link beams are designed as reinforced concrete members to resist shear and are connected to columns using dowel bars to form a hinge connection and transfer only the shear. The experimental program consisted of testing $1/3^{rd}$ scaled model of normal frame, linked column frame with a rigid connection in which the normal beam is connected in one end rigidly to the linked column and in another end to linked column frame with hinged connection i.e. the normal beam is made as hinged connection to the linked column. The response to cyclic loading was studied.

MODEL FOR THE STUDY

For analytical and experimental investigation single bay frame was considered. 1:3 ratio scaled down Reinforced concrete frame specimens were cast and later on two of the specimens were made of linked column with different connections. The static cyclic load tests were performed in order to find out the performance of the specimens in terms of their stiffness, strength, damage propagations, dissipated energy, ductility, over strength and performance factors. The testing of specimens was conducted in Structural Engineering Laboratory of Karunya University.

The reinforcement details and dimensions of a single bay and one-story test frames are shown in Figure 2. The beam–column connections are not designed for the confinement reinforcement. The

concrete was cast in site with a coarse aggregate size of 10 mm in diameter. In order to find out the material stress–strain relationships for concrete and reinforcement, the material tests were conducted. Concrete’s 28-day nominal compressive strength was 25 N/mm², and the nominal yielding and ultimate stresses of TMT rods were determined as 415 N/mm² and 500 N/mm², respectively.

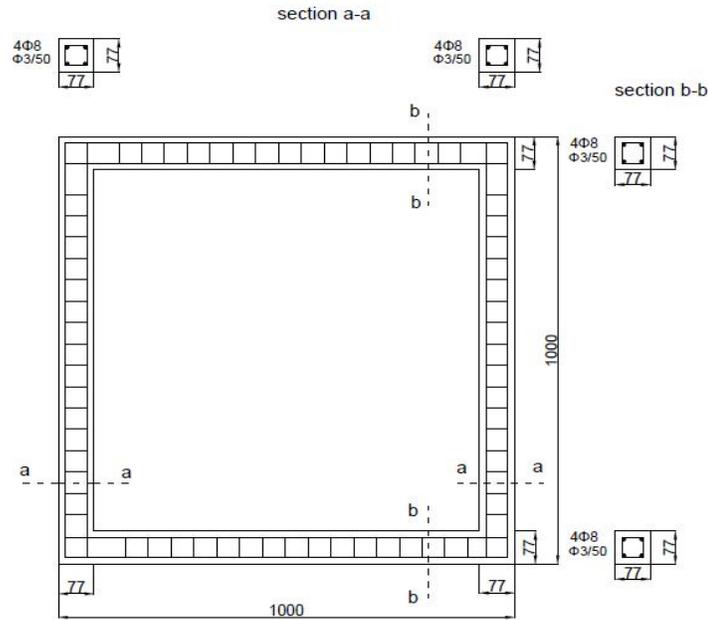


Fig. 2 Reinforcement details of bare frame M1 (All dimensions are in mm)

DESIGN PROCEDURE FOR LINK BEAM AND COLUMN

1. Design of Link Beam Length

The design of links was performed according to the Seismic Provisions (Manual, A.S. 2005). The design of the link lengths are done similarly to the link length in eccentrically braced frames and their yielding behaviour depends on their section properties and their length. As per AISC Seismic Provisions, links are divided into three categories based on their link length *e*, the link length should have to satisfy the following limit.

Shear links, which yield primarily in shear, should have:

$$e \leq 1.6 \frac{M_p}{V_p} \tag{1}$$

flexural links, which yield primarily in flexure, should have:

$$e \geq 2.6 \frac{M_p}{V_p} \tag{2}$$

and intermediate links, which may yield in a combination of shear and flexure, must/should have:

$$1.6 \frac{M_p}{V_p} < e < 2.6 \frac{M_p}{V_p} \tag{3}$$

The moment capacity *M_p* and a plastic shear *V_p* of the links are determined from the following equations

$$V_p = \tau_y A_v \frac{1}{\gamma_m} \tag{4}$$

$$M_p = Z_p \sigma_y \tag{5}$$

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 To reduce the bending, the link cross section should meet the seismically compact cross section requirements. For shear links, the inelastic deformation capacity defined as the link rotation angle in

AISC codes is limited to 0.08 radians and limited to 0.02 radians for flexure. The above equations were used to design the length of the links for one bay framed system mentioned in Figure 2 and the link length values drawn from above equation are given in Table 1.

Table 1: Length of Links

(Z) mm ³	(Av) mm ²	Shear link (mm)	Flexure link (mm)	Intermediate link (mm)
6750	196	160	258	200

To choose suitable links for the LCF system, the performance of various types of the links were studied using pushover analysis. The Linked column frame was subjected to IS 1893 (Part 1): 2002 specified lateral load pattern to obtain a pushover curve by using SAP 2000 V 10.0, a finite element-based computing tool. The nonlinear static analysis procedure is followed to calculate the building target displacement, likely to be experienced by the building for a given level of ground motion. In addition, the ATC, A., (1996) nonlinear static analysis procedure provide a tool for evaluating the damage in Beams and column elements, which were modeled as nonlinear frame elements by assigning concentrated M3 and P-M2-M3 plastic hinges respectively, at both ends. A numerical model for calculating the nonlinear response of reinforced concrete building structures was developed and is explained in detail in Keyhani and Shafiee (2016), Arjun and Kumar (2009). It is essential that all structural elements in the moment frame remain elastic when the link is at a plastic stage. The material properties used in SAP 2000 is shown in Table 2.

Table 2: Material Properties

Material property	Concrete M 25 grade	Steel Fe 415 grade
Weight per unit volume (kN/m ³)	23.56	76.97
Mass per unit volume (kg/m ³)	2548.3	7850
Modulus of elasticity (kN/m ²)	25E + 06	2E + 08
Characteristic strength (kN/m ²)	25,000 (for 28 days)	415,000 (yield)
Minimum tensile strength (kN/m ²)	-	485,800
Expected yield strength (kN/m ²)	-	456,500
Expected tensile strength(kN/m ²)	-	533,500

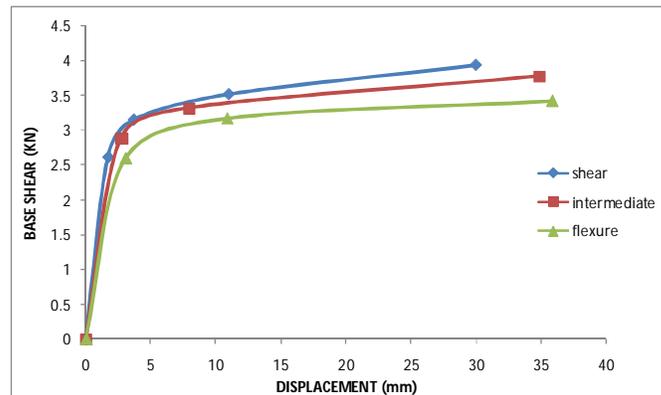


Fig. 3 Performance levels of various types of links

The results obtained from pushover analysis with respect to ATC, A., 1996 (Capacity spectrum method) shows that the performance-based capacity curve should satisfy the acceptance criteria for immediate occupancy and life safety limit states for various ground motions. The plastic rotations in relation to the building performance levels such as; immediate occupancy, life safety and collapse prevention are shown in the Figure 3 where these capacity curve gives information's on the damage of

elements and displacement profile at critical states, but they are not adequate in themselves to provide an assessment of the state of damage or proximity to collapse. There is a predominant decrease of 15.09% and 22.2% in roof displacement when the link length is designed for shear when compared with the link length designed for intermediate and flexure. The link beam length which is designed for shear performs better and the base shear is increased by 9.82% and 18.02% when compared to the link length designed for intermediate and flexure. So the link length which is designed for shear is taken for experimental work.

2. Design of Link Column and Link Beam

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 (Part 1): 2002 the size of the link column was determined using these following relation Where the moment of inertia for the link column was derived from Lopes et al., (2015). Similar expressions for total and partial deformation can be obtained for different story levels.

$$I_{LC} = \frac{x^{4.6} Ph^3}{8E\Delta} \tag{6}$$

$$\Delta = \frac{PL^3}{3EI} \tag{7}$$

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 for 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 Figure 4. Lopes et al., (2012) and Malakoutian et al., (2013) numerically analyzed 6-story Linked column frame using time-history and pushover analyses, respectively.

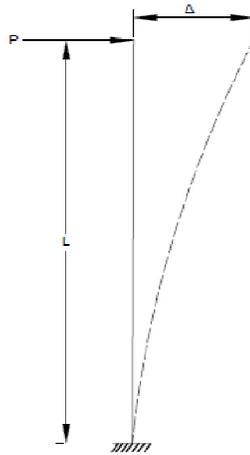


Fig. 4 Cantilever column of a one storey Linked column frame

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 load P is taken as 7700 N. The equivalent lateral force procedure is used to obtain total base shear as per IS 1893 (Part 1): 2002. The location of the building was in Zone V.

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 (Lopes et al., 2015).

$$I_L = 0.6 \frac{I_{LC} H}{h} \tag{8}$$

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 3 shows the dimensions of link column and link beam.

Table 3: Link Column and Link Beam Dimensions

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

EXPERIMENTAL PROGRAM

1. Specimen for Experimental Investigation

In order to carry out the experimental investigation, two reinforced concrete frames were taken with linked column and one frame was taken without link column system. Model 1 (M1) was cast as a bare frame without link column whose detailing is shown in Figure 2. Model 2 (M2) had link column with the rigid connection between the frame and the column, Model (M3) was designed with the hinged connection between the beam and the link column which are shown in Figure 5, 6. The dimensions of the specimen were scaled down into ratio 1:3 based on the availability of the facilities in the laboratory. The size of the link column was 57×57 mm and the link beam was 30×30 mm and 200 mm long intermediate links. The detailing of the three frames is shown in Figures 2, 5, 6 respectively. For hinged connection in linked column frame (M3), the beam of the moment frame is connected to the linked column using dowel bars. The dowel bars are used to transfer shear loads across construction and movement joints in concrete. With reference to the beam to column connections, the shear force V at the top of the columns were calculated from the resisting moment M_r of the section at the base of the columns with $V = \frac{M_r}{h}$ so that, introducing a γ_R factor, the force on the connection becomes,

$$H = \gamma_R V = \frac{\gamma_R M_r}{h} \quad (9)$$

and

$$R_d = 0.9n\phi^2 \sqrt{f_y f_{ck} (1 - \alpha^2)} \quad (10)$$

Where, n = no of dowels, ϕ = diameter of dowels, f_{ck} = characteristic strength of concrete, f_y = yield strength of steel, σ = normal tensile stress. 2 numbers of 20 mm ϕ bar were provided as dowel reinforcement for hinge connection. The dowel bar of 20 mm diameter was embedded in the column to a length equal to the development length.

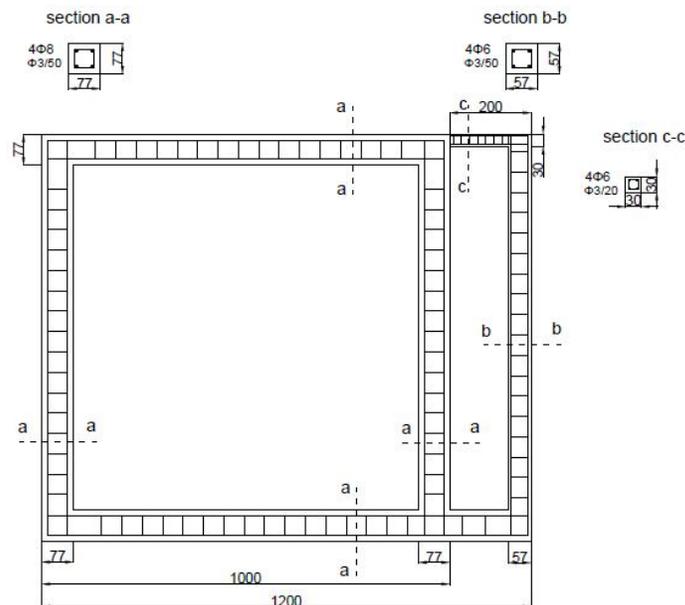


Fig. 5 Reinforcement details of rigid linked column frame M2

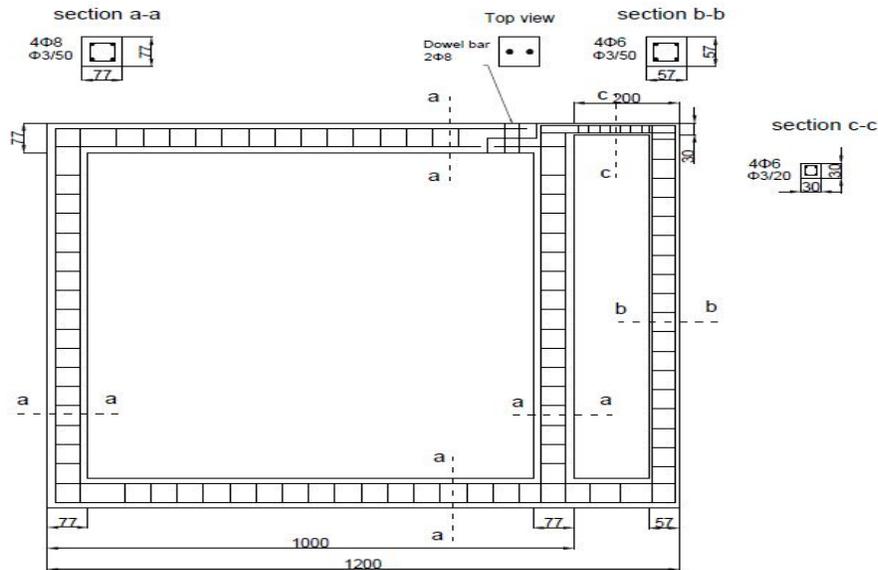


Fig. 6 Reinforcement details of hinged linked column frame M3

2. Test Setup and Instrumentation

In-plane, lateral load was applied at the end of the top beam along its centroidal axis through a servo-hydraulic actuator. The tests were conducted under load control. Load - drift response was obtained for each of the specimens. From each of these plots, the initial and residual stiffness for the frames were obtained up to its ultimate loading. The instrumentation for the tests consisted of Linear Variable Differential Transformers (LVDT's) and load cells and the displacement at the top of the column was measured directly through the actuator LVDT, since the test was conducted in load control mode.

A discussion of the response of the linked column frame, when subjected to in-plane cyclic lateral loading, is presented in this paper. Experimental observations of cracking patterns, mode of failure, nature of the load drift response, strength and stiffness characteristics and hysteresis behaviour of the linked column are discussed. The response characteristics for linked column frame is the compared with the corresponding characteristics for normal frames. At each load increment, the cracks were inspected and marked for ease of identification. The lateral loading sequence, as shown in Figure 8, was a load-controlled type of loading scheme.

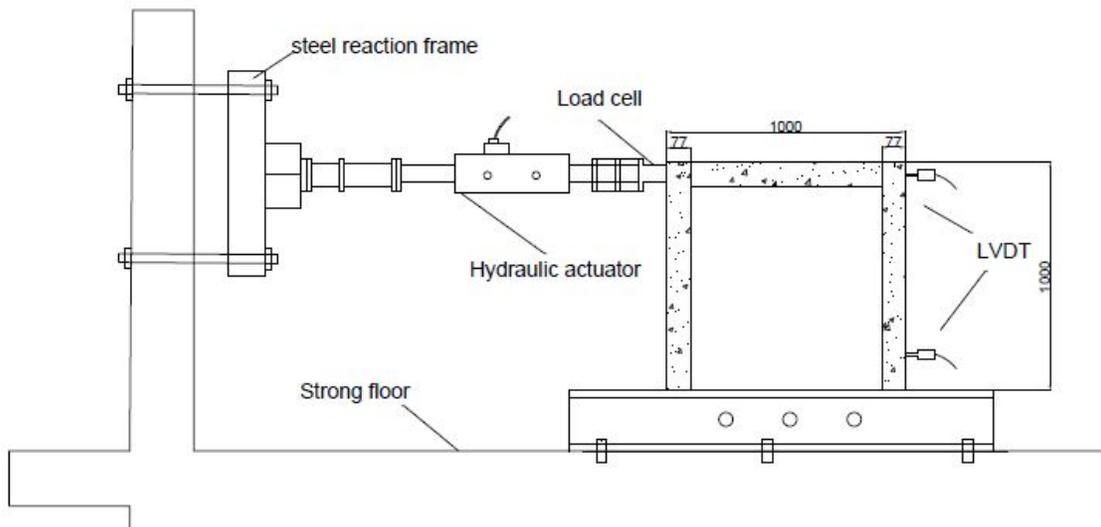


Fig. 7 The test setup (all dimensions are in mm)

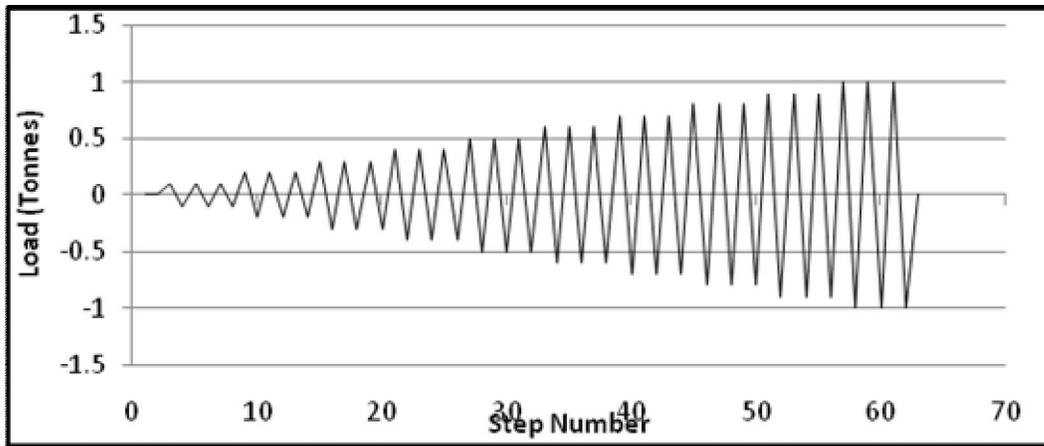


Fig. 8 Loading sequence

EXPERIMENTAL INVESTIGATION

Cyclic load test was conducted on the three frames and the behaviour of the specimens was studied. Discussions of the results are as follows

1. Bare Frame (M1)

M1 was designated as the reference frame to compare its performance against linked column frame specimens. The cyclic load was applied on the specimen. During the experimentation, shear cracks were formed at a force of 6 kN and the corresponding displacement was measured to be 5.583 mm. At the end of the 5th cycle, first yielding of the longitudinal reinforcement was observed at a displacement of about 8.56 mm. At this level, the restoring force was measured as 10 kN. The load vs deflection are shown in Figure 9.

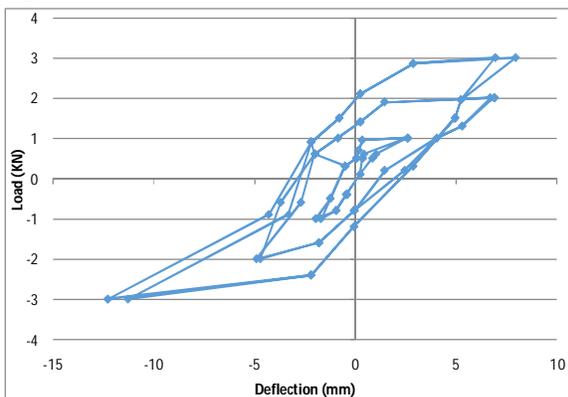


Fig. 9 Load versus Deflection and a damage pattern (M1)

2. Rigid Link Column Frame (M2)

In this frame system, the normal beam is rigidly connected to the linked column. Sixteen full cycles were applied to the frame. First flexural cracks were observed at 18 kN at a displacement of 9.6 mm and it occurred at the link joints. First shear crack was observed at 20 kN when the displacement was 10.26 mm. At the end of the 10th cycle, first yielding of the longitudinal reinforcement was observed, when the force was measured as 30 kN and displacement was 15.56 mm. The load vs deflection for M2 is shown in Figure 10.

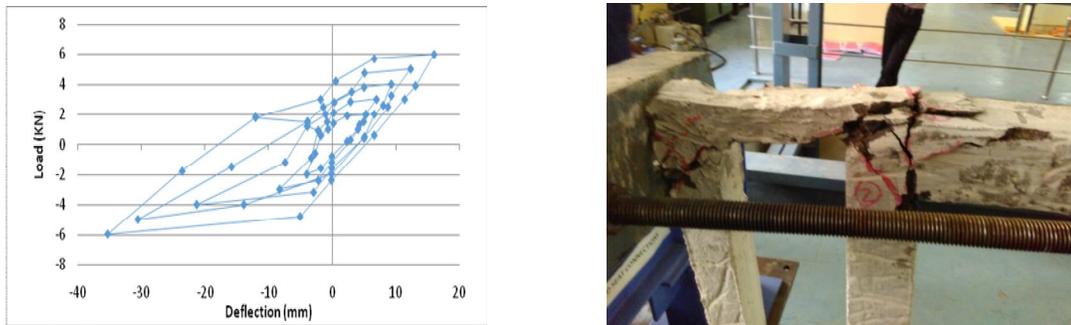


Fig. 10 Load versus Deflection and a damage pattern (M2)

3. Hinged Link Column Frame (M3)

In this frame system, the normal beam is flexibly connected to the linked column. Sixteen full cycles were applied to the frame. First shear cracks were observed at a force of 20 kN when the displacement was 15.6 mm which is occurred at the hinged joint. At the end of the 10th cycle, when force was 33 kN and displacement was 20.57 mm, first yielding of the longitudinal Reinforcement was observed. The load vs deflection for M3 are shown in Figure 11.

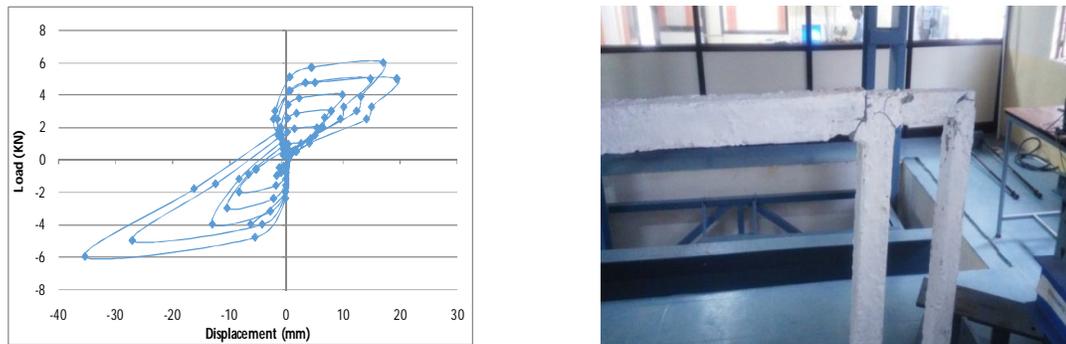


Fig. 11 Load versus Deflection and a damage pattern (M3)

4. Energy Dissipation

Based on principles of earthquake engineering, for a proper seismic behavior of structure, the input energy to the structure due to earthquakes need to be absorbed and/or dissipated, depending on the expected performance of the structure. Energy absorption and dissipation in structures is usually due to two main sources, the inherent damping of the structural system, and formation of plastic hinges in structural components and nonlinearity. The sum of the area enclosed by each hysteretic loop is used to determine the cumulative dissipated energy. The normal frame system was found to have 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 the rigid linked column 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 Figure 12.

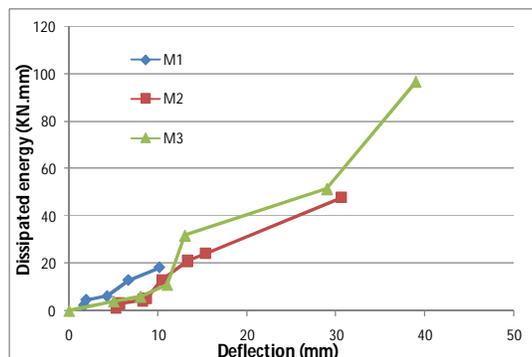


Fig. 12 Energy dissipation versus Deflection

5. Lateral Stiffness

The lateral stiffness is considered 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 hinged linked column when compared with the rigid linked column. Initial lateral stiffness was reduced by about 11.7%. The overall stiffness of rigid linked column frame is 1.59 times greater than the normal frame's stiffness, respectively. Variation of the lateral stiffness with respect to displacement for all specimens is given in Figure 13.

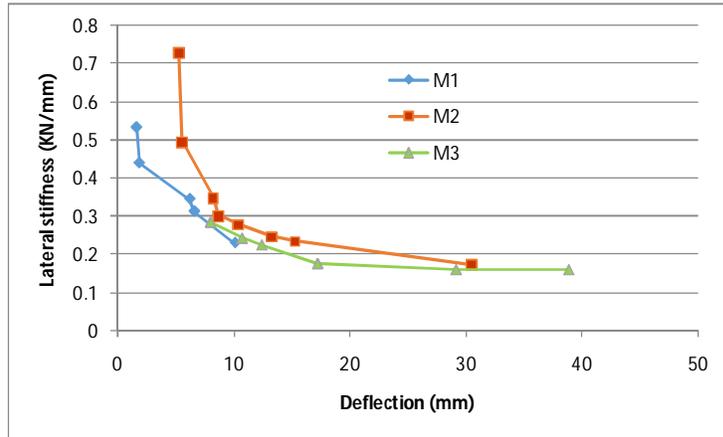


Fig. 13 Lateral stiffness versus Deflection

ANALYTICAL INVESTIGATION

ANSYS 15.0 WORKBENCH a nonlinear finite element analysis package is used to develop a 3D model of a frame with and without linked column. The finite element analysis is an assembly of finite elements which are interconnected at a finite number of nodal points. The main objective of this analytical study is to simulate the behavior of the linked column under cyclic load. LINK 8 element was used to model steel reinforcement and SOLID 65 was used for the 3-D modeling of concrete. This solid is capable of crushing in compression and cracking in tension where material properties provided in ANSYS are shown in Table 4.

Table 4: Material Properties

Material property	Concrete	Steel Fe 415 grade
Density (Kg/m ³)	2400	7850
Youngs modulus (N/m ²)	32527×10^6	2.1×10^{11}
Poissons ratio	0.15	0.3
yield stress (N/m ²)	-	498×10^6
Tangent Modulus (N/m ²)	-	847×10^6

In this analytical study, the square mesh was set up and the supports and frames were modeled as volume elements. It is the most important part of an analysis and can determine the efficiency and effectiveness of an analysis. The reinforcement meshing is a special case compared to the volumes. The necessary mesh attributes need to be set before each section of the reinforcement is stated in Bouboulas and Anifantis (2016). The stress distribution and Load-displacement hysteretic behavior for experimental and numerical from the ANSYS were shown in Figure 14.

Structures subjected to the strong ground motion should be designed to dissipate energy by interface friction, inelastic material behaviour, etc. Due to the continuous cyclic deformation, there is always decay in the attribute of such hysteretic behaviour. For the modeling and the design of earthquake

resistant structural systems, such decay should be taken into account. The availability of accurate constitutive models capable of representing deteriorating structural behaviour is the basic requirement to perform such analyses. Researchers have developed numerous hysteretic model. They are broadly divided into two types, smooth hysteretic model (SHM) and polygonal hysteretic model (PHM). The smooth hysteretic model (SHM) was adopted for sections of the frames, which considers the hysteretic characteristics such as strength deterioration, stiffness degradation and pinching created by Baber and Noori (1985). The SHM parameters of the bare frame and linked column frame were selected from Yuksel et al., (2010). In the analytical test, the cyclic load was applied at the top (i.e beam column joint) of the frame. In the analysis the structure, the frame is analyzed under lateral load (up to 10 kN), the structure is laterally loaded using loads rather than displacement which is consistent with the experiment.

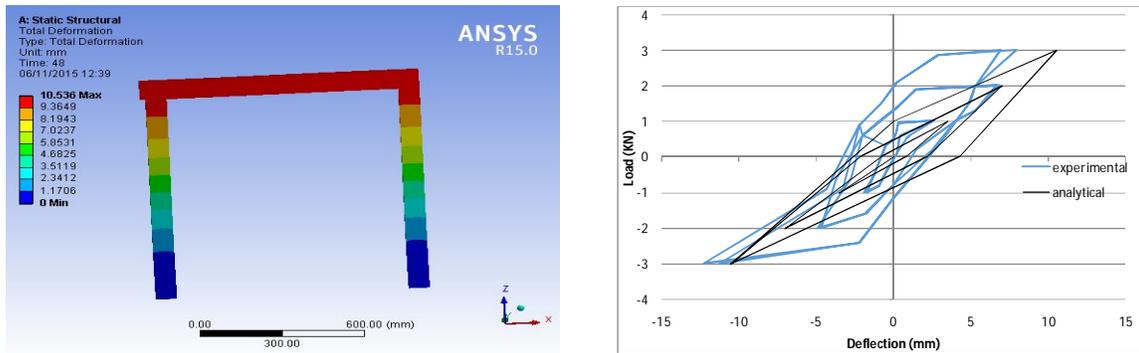


Fig. 14a Stress distribution and Load-displacement hysteretic behavior for experimental and numerical models of M1

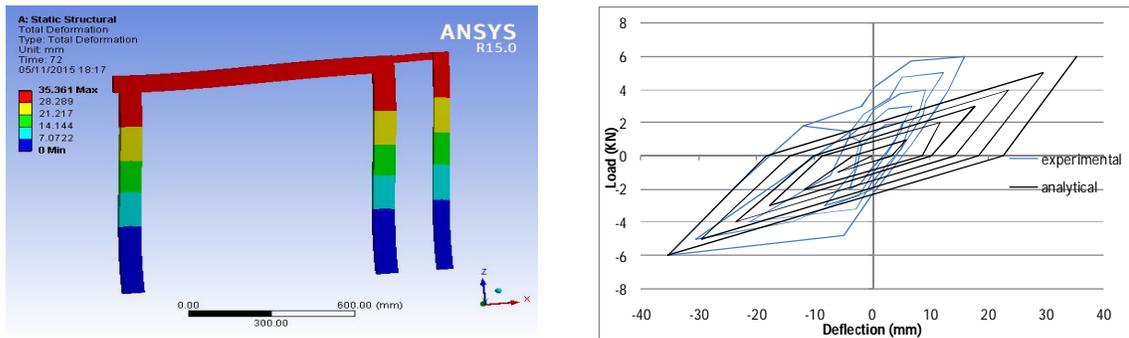


Fig. 14b Stress distribution and Load-displacement hysteretic behavior for experimental and numerical models of M2

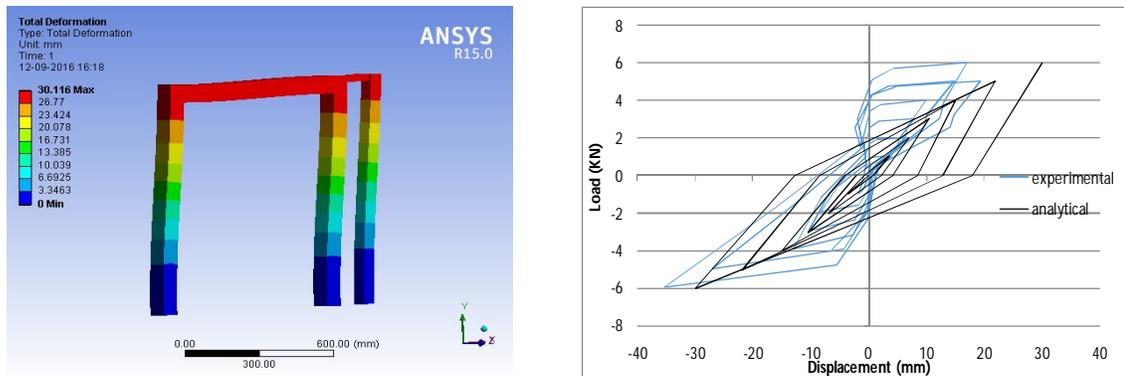


Fig. 14c Stress distribution and Load-displacement hysteretic behavior for experimental and numerical models of M3

The ultimate base shear capacity and the displacement obtained analytically are approximately 13% lower than those results obtained experimentally. For the bare, rigid and hinged linked column frames; the

ultimate base shear capacity is 10 kN, 35 kN, and 30 kN, respectively. Adding linked column is associated with increasing the stiffness, but some of this increasing stiffness can be caused by interaction between the gravity frame and the link beam. From the analysis of the frame under cyclic loads it was found that by increasing the load, stress intensity is increased and is maximum at the link beam which is indicated by the red colour.

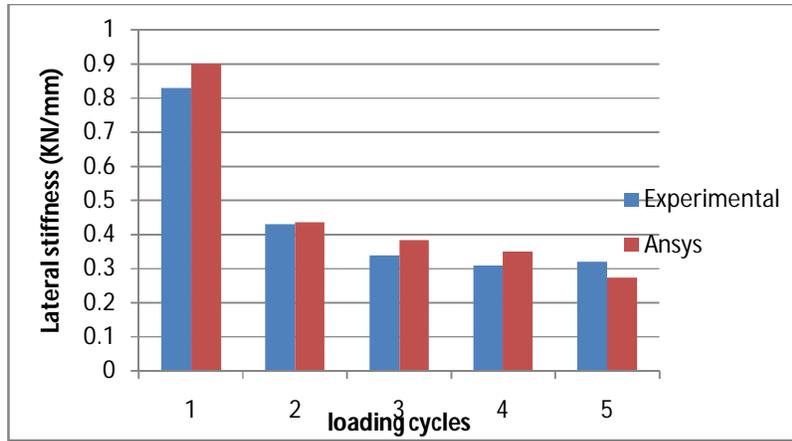


Fig. 15a Comparison of lateral stiffness for M1

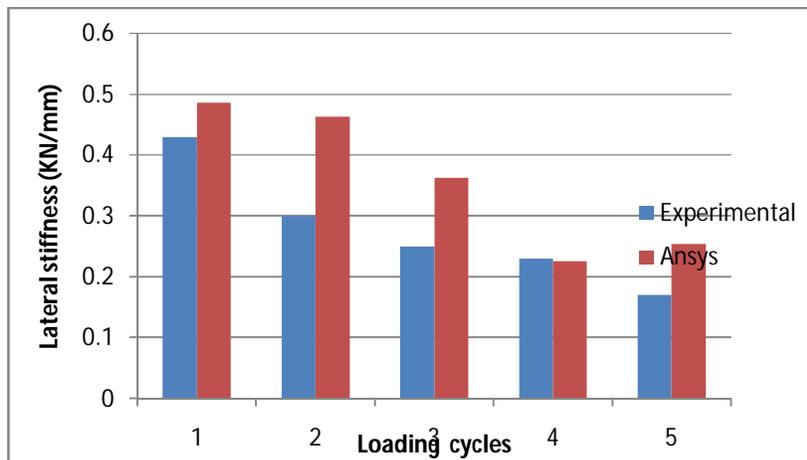


Fig. 15b Comparison of lateral stiffness for M2

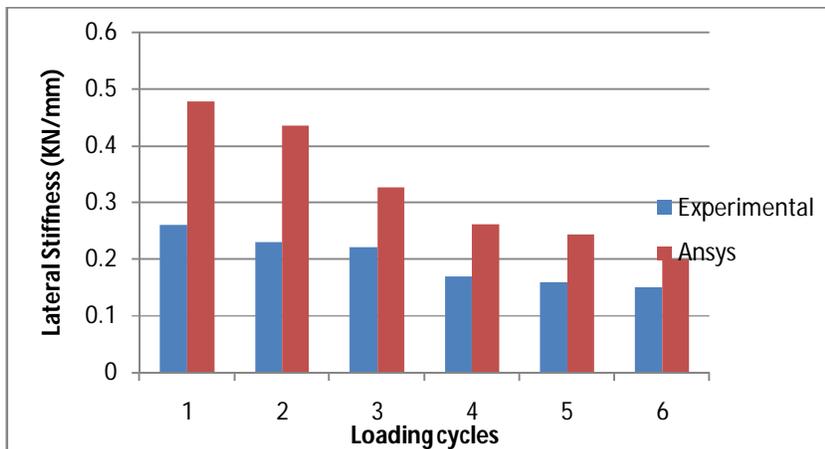


Fig. 15c Comparison of lateral stiffness for M3

The comparison of experimental and analytical lateral stiffness values are shown in Figure 15. The considerable decrease in story drift in M3 can prevent damage to drift sensitive non-structural elements of the building in moderate earthquake events. After the failure of the link element, the plastic hinges will begin to start in the moment frame. Because of the flexible connection in the M3, it will dissipate more energy than the other models. It is difficult to determine whether the test or FE results are more accurate. This is due to the fact that during the test a reasonably fine resolution is required if data acquisition is to capture the very moment at which the initial peak displacement drops. And also in experimental investigation, due to the fixidity conditions in the frame, the stiffness will be low when compared with analytical investigation.

CONCLUSION

The results of experimental and analytical investigations carried out to study the feasibility of implementing sacrificial link beam and column system for seismic resistance of reinforced concrete structures are presented in this paper. The following are the conclusions drawn.

- Energy dissipation capacity of the models was increased by 86% for M3 and 62% for M2 when compared to M1 was introduced. The Lateral stiffness increased by 70% for M3 and 63% for M2 when compared to M1.
- In Model M3, the failure occurred in the links and the damage to the main structure was very less.
- In M2, the failure was predominant in the joint between the main beam and column.

The shear link in the linked column is more reliable and effective in dissipating a large amount of energy. A methodology to design the proposed shear-link sacrificing element in reinforced concrete configuration has been developed together with analytical studies which establish its suitability under earthquake-type loads. Results from the cyclic loading show that the cracks initially occur at the link beam. After the yielding of link beam only the cracks are begin to occur at the main frame.

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. In addition, the cost of the construction should be reduced when the replaceable links are modeled with Reinforced concrete elements.

ACKNOWLEDGEMENT

The authors are thankful to Ministry of Earth Sciences (MoES), GOI for sponsoring the research work. The authors also extend their sincere thanks to Karunya University for facilitating the research work.

REFERENCES

1. Arjun, C.R. and Kumar, A. (2009). "Artificial Neural Network-Based Estimation of Peak Ground Acceleration", ISET Journal of Earthquake Technology, Vol. 46, No. 2, pp. 19-28.
2. ATC, A. (1996). "40, Seismic Evaluation and Retrofit of Concrete Buildings", Applied Technology Council, report ATC-40. Redwood City.
3. Baber, T.T. and Noori, M.N. (1985). "Random Vibration of Degrading, Pinching Systems", Journal of Engineering Mechanics, ASCE, Vol. 111, No. 8, pp. 1010-1026.
4. Behnam, B. and Reza Ronagh, H. (2014). "A Study on the Effect of Sequential Post-Earthquake Fire on the Performance of Reinforced Concrete Structures", International Journal of Structural Integrity, Vol. 5, No. 2, pp. 141-166.
5. Bouboulas, A. and Anifantis, N. (2016). "Three-Dimensional Finite Element Modeling for Post-Buckling Analysis of Cracked Columns", International Journal of Structural Integrity, Vol. 7, No. 3, pp. 397-411.
6. Bruneau, M., Uang, C.M. and Sabelli, S.R. (1998). "Ductile Design of Steel Structures", McGraw Hill Professional.

7. Dusicka, P., Itani, A.M. and Buckle, I.G. (2006). "Built-Up Shear Links as Energy Dissipators for Seismic Protection of Bridges (No. MCEER-06-0003)", Multidisciplinary Center for Earthquake Engineering Research.
8. Dusicka, P., Berman, J.W. and Purasinghe, R. (2009). "Steel Frame Lateral System Concept Utilizing Replaceable Links", In NZSEE Conference.
9. Hines, E.M. (2009). "Eccentric Braced Frame Design for Moderate Seismic Regions", In Structures Congress 2009: Don't Mess with Structural Engineers: Expanding Our Role, pp. 1-10.
10. Ile, N. and Reynouard, J.M. (2003). "Lightly Reinforced Walls Subjected to Multi-Directional Seismic Excitations: Interpretation of CAMUS 2000-1 Dynamic Tests", ISET Journal of Earthquake Technology, Vol. 40, No. 2-4, pp. 117-135.
11. IS 1893: (2002). "Indian Standard Criteria for Earthquake Resistant Design of Structures Part 1: General Provisions and Buildings", Bureau of Indian Standards, New Delhi.
12. Keyhani, A. and Shafiee, A. (2016). "Effectiveness of Seismic Retrofit for Existing Concrete Buildings using Nonlinear Static-Analysis", Indian Journal of Science and Technology, Vol. 9, No. 2.
13. Lopes, A.P., Dusicka, P. and Berman, J.W. (2012). "Design of the Linked Column Frame Structural System", Stessa, pp. 311-317.
14. Lopes, A., Dusicka, P. and Berman, J. (2015). "Lateral Stiffness Approximation of Linked Column Steel Frame System", In Structures Congress 2015, pp. 2408-2420.
15. Malakoutian, M., Berman, J.W. and Dusicka, P. (2013). "Seismic Response Evaluation of the Linked Column Frame System", Earthquake Engineering & Structural Dynamics, Vol. 42, No. 6, pp. 795-814.
16. Manual, A.S. (2005). "Seismic Provisions for Structural Steel Buildings", ANSI/AISC, pp. 341-05.
17. Nader, M., Abbas, S. and Ingham, T. (2001). "Seismic Safety Design of the New San Francisco-Oakland Bay Bridge", In Structures 2001: A Structural Engineering Odyssey, pp. 1-11.
18. Park, W.S. and Yun, H.D. (2005). "Seismic Behaviour of Steel Coupling Beams Linking Reinforced Concrete Shear Walls", Engineering Structures, Vol. 27, No. 7, pp. 1024-1039.
19. Shelton, J.J. and Hemalatha, G. (2016). "Behavior of Linked-Column System Subjected to Seismic Force", Indian Journal of Science and Technology, Vol. 9, No. 6.
20. Yuksel, E., Ozkaynak, H., Buyukozturk, O., Yalcin, C., Dindar, A.A., Surmeli, M. and Tastan, D. (2010). "Performance of Alternative CFRP Retrofitting Schemes Used in Infilled RC Frames", Construction and Building Materials, Vol. 24, No. 4, pp. 596-609.