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Original Article

Numerical Modelling on the Assessment of Progressive Collapse of RC Framed Structures Subjected to Dynamic Loading

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Abstract - *Progressive collapse is one of the most devastating types of building failure, and it has attracted much attention in the past decades. A structure undergoes progressive collapse when a primary element fails, resulting in the failure of adjoining structural elements, which in turn causes further structural collapse and abrupt loss of structural stability, often leading to loss of life and property. This devastation has generated a worldwide concern regarding the risks of progressive collapse in multistory buildings. The structural elements of the building should be able to endure the removal of one or more structural members and transfer their load to surrounding elements to prevent disproportionate structure failure. In this study, the effect of sudden column failure on the adjacent structural members is analyzed for 2D models of single-storey, double-storey, 2-bay single, Double, Three, Four and Five storey frames under dynamic loading using time history analysis to obtain a better understanding of the probability of failure mechanism. The displacements of each joint in the X-axis and Z-axis direction are observed to achieve a concrete solution to this destruction. This study helps simulate real-world events by analyzing structures dynamically, identifying the weak points and ensuring collapse-resistant buildings.*

Keywords - *Column losses, Impact loading, Numerical models, Progressive collapse, RC frames.*

1. Introduction

Progressive collapse is a complex dynamic phenomenon that requires the structural system to effectively transfer and redistribute loads to avoid the failure of essential components and a complete breakdown. To ensure this, the design of beams, columns, and frame connections must incorporate ductility, redundancy, and continuity, enabling the structure to manage significant load shifts and mitigate collapse risks. The failure of Ronan Point 22 storeys apartment in London in 1968 was an eye opener for the structural design fraternity as the gas explosion on the 18th floor led to the collapse of entire panels below it. The 26-storey Skyline Tower collapse in 1973 in Virginia was the next progressive collapse failure due to premature removal of formwork. Alfred Murrah building, situated in Oklahoma City in 1995, consisted of nine storeys with 61m in height and 21m in width. It came under a blast attack, which destroyed a column, resulting in the collapse of other columns and beams adjacent to it, leading to collapse. The collapse of the Sampoong department store in 1995 in Korea was associated with changes in structural integrity to incorporate escalators and central air conditioning units. After the attack on Khobar Towers in 1996 and the World Trade Centre in 2001, authorities came up with updated guidelines in different codes to make structures progressive collapse resistant.

The alternate Load Path method is one of the methods proposed by GSA to analyse and design structures against progressive collapse. The different approaches for Alternate Load Path recommended by the General Services Administration (GSA) and the Department of Defence (DOD) are Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NLSP) and Nonlinear Dynamic Procedure (NLDP). This method involves removing one critical element at a time and checking the structure's potential to redistribute catastrophic loads, avoiding partial or complete collapse. The extensive numerical and experimental studies conducted by researchers to identify, analyse and design RC structures using the Alternate Load Path (ALP) method until recent times are: A study conducted on the advantages and disadvantages of different procedures such as LSP, LDP, NLSP and NLDP given by GSA. Effective results for each procedure were compared, but the NLDP procedure was found to be more time-consuming with greater system configuration for better results. Though studies have been done to understand the collapse analysis of the structures due to sudden failure of vertical elements, there has been little research in understanding the basic concept of failure, movement of joints, effect on adjacent members, and vibration in different directions, ultimately leading to failure. [1]. Studies have shown that modelled frames in high and moderate seismic zones along with shear walls demonstrated that sudden column failure could lead to progressive collapse, but high seismic zone design exhibited robustness against disproportionate collapse [2]. Full-scale beam-column elements analysis was conducted to evaluate analytical and experimental outputs. The corner column loss and middle column loss were studied specifically under dynamic loading, with corner column loss being more severe to progressive collapse [3]. The effect of infill walls for different collapse scenarios, including removing the corner column, edge column, edge shear wall, and internal shear wall, was studied. If the opening for windows in infill walls is more than 40%, then the progressive collapse is evident in those places [4]. Experimental studies were conducted on beam slabs using yield line theory.

The study investigated the effects of extra reinforcement and lap lengths at critical loading to mitigate progressive collapse under sudden vertical element loss. These specific areas resisted sudden failure with a small reinforcement increase [5]. On evaluating frames analytically and experimentally, it was observed that the side column connected to the bay was more vulnerable to progressive collapse than other columns. Experimental results gave more clarity than analytical results [6]. Experimental tests on two spans supported by a middle element attached to hinge connections at the ends have shown that the restraint by horizontal members with sufficient stiffness proves well in mitigating collapse. The span depth ratio if minimized to less than seven, helps in further resistance [7]. The effect of central column loss on steel frames connected through bolts performed well against deformation; however, the failure was mainly in places of hinge formation. It was also concluded that the increase in gravity load leads to plastic deformations and later collapse [8].

The behaviour of continuous beam and beam-slab specimens under central column failure has been investigated using 1/3rd scale tests. Slab thickness, reinforcement under seismic loading and beam height played an active role in resisting deformations. Continuous beam specimens fared much better when compared to beam slab specimens [9]. An analytical study studied the effect of edge column removal on reinforced concrete buildings designed per seismic zone standards. The damage due to gravity load is less, as the seismic loads act in vertical and horizontal directions. The structure deflects more under seismic loading when sudden collapse takes place [10]. A novel modelling approach incorporating service loads prior to member collapse. Small and large frames with varied bays and heights were analyzed to properly understand the cause of hinge formation, thereby providing some specific inputs to resist failure [11]. An experimental study on a 3d beam-slab frame under corner column and exterior column failure. However, this test setup was challenging in applying all required support reactions. They calculated the flexural capacity of the beam slab frame and punching shear resisted by the frames. The displacement in the exterior slab was greater when compared to the interior part of the slab [12]. Experimental analysis was conducted on a 1/3rd scale prototype with complete infill walls, varying in height and span, to assess their robustness against progressive collapse. The results indicated that infill frames relatively proved better than bare frames against vertical load resistance [13]. A numerical investigation on simultaneous removal of ground floor columns. The study revealed that the loss of two consecutive columns drastically reduced the bearing capacity of the framed structure [14].

The progressive collapse resistance of multi-storey modular buildings composed of corner-supported composite modules was investigated. Alternate load path analysis was conducted under three sudden column loss scenarios. The results showed varying failure mechanisms and overload factors, with recommended dynamic increase factors of 1.90 for 4-storey and 1.60 for 12-storey buildings [15]. Two risk-based robustness indices were evaluated for regular frame structures under abnormal loads. The obtained results confirmed the effectiveness of two indices for conventional, strengthened, and optimized structures. Developing reliable robustness indices for structural design is a matter of concern [1]. Research has demonstrated that high-performance ferrocement laminate and bonded steel plate can effectively strengthen the slab. The progressive collapse performance of the strengthened specimen was evaluated based on experimental results, encompassing the load-displacement relationship, ductility, and failure modes. Experimental results indicate that the slab-strengthening scheme shifted the location of the slab yield line and transformed the beams from brittle torsional failure to ductile flexural failure [16]. Regular and Irregular buildings were considered for the column removal scenario. Findings suggested that corner column removal tends to cause the most significant response, while plan irregularity can increase the risk of collapse. Their studies highlighted the importance of structural irregularity and robust design considerations for high-rise and irregular buildings [17].

A cost-benefit analysis of the progressive collapse of reinforced concrete frames was done, balancing the added construction cost to reduce collapse probability. A four-storey RC frame was analysed, maintaining life cycle costs and benefits, considering factors like threat probability and time value for money. [18] Prior studies have either focused much on steel frames or static loading. Most researchers have directly considered three-dimensional structures rather than clearly emphasizing the structure's response under sudden loading considering a two-dimensional frame. Progressive collapse analysis ensures structural resilience against unforeseen events. Using advanced modelling techniques and robust design principles, civil engineers can minimize the risk of disproportionate failures and enhance public safety.

2. Methodology

Unlike prior studies focusing on linear static procedures this work adopts Linear Dynamic Analysis (LDA) with a time-history function to simulate abrupt column failure. Our method captures real-world dynamic effects, such as vibration-induced displacements, and compares responses across different storey frames. To understand the probable deflections and redundancy in the frames with varied bays and storey levels, 5 cases have been selected. The axial load on the column slated for removal is first calculated to model progressive collapse. An equivalent upward joint load then replaces this force. A time-dependent function was implemented to replicate the column's sudden failure. The load multiplier was initially set to 1 for a specified duration, simulating the column's presence. The upward force was abruptly removed by reducing the multiplier to zero, mimicking the column's elimination. The loading sequence was structured as follows: the full axial load (multiplier = 1.0) was maintained for 2.0 seconds. Subsequently, the load was linearly reduced over 0.2 seconds, reaching zero at 2.5 seconds. The load remained at zero until the simulation concluded at 4.0 seconds, allowing sufficient time for the structure to stabilize and exhibit its post-failure response. When conducting dynamic analysis, it is suggested that the vertical support should be removed over a period of not more than 1/10th of the output mode. This study focuses on initial elastic behaviour, and nonlinear effects are not included in this study. Beam-column connections are considered to be fully fixed, neglecting semi-rigid behaviour. A damping effect of 5% has been taken, which is typical for dynamic loads (as per IS 1893).

Two vertical elements have been proposed for removal.
1) Removal of Corner Column
2) Removal of the middle Column

The Reinforced Concrete Frames are putatively designed in the FE model per guidelines issued by IS 456:2000 and IS 1893 load combinations. Linear Dynamic Analysis is simulated for each case, and the displacements in the X-axis and Z-axis are analyzed.

3. Model Description

The modelled framed structure is a 2D reinforced 1 bay single storey, 1 bay double storey, 2 bay double storey, and 2 bay four storey having each floor of height of 3m and bay width of 3m presented in Figures 1, 2, 3, and 4. These reinforced concrete structures have been designed to resist both gravity and lateral loads in accordance with guidelines issued by Indian Standards.

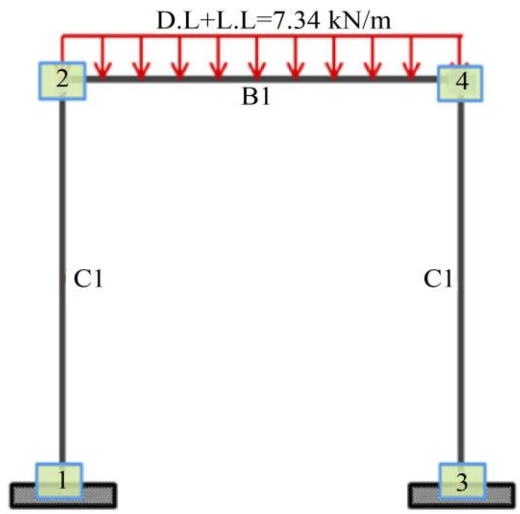

Fig. 1 Single-bay single-storey

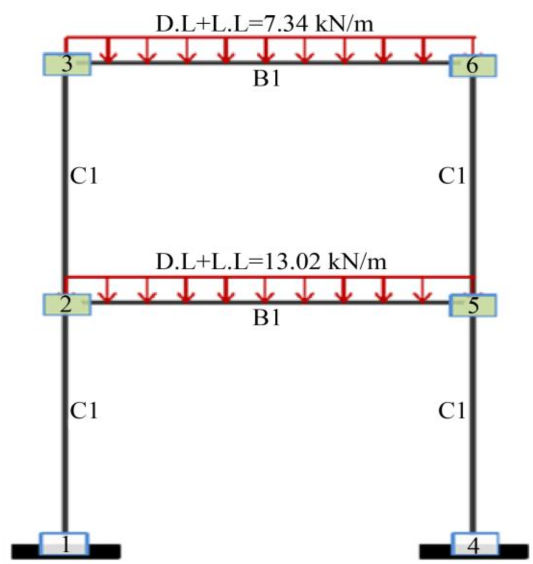

Fig. 2 Single-bay double-storey

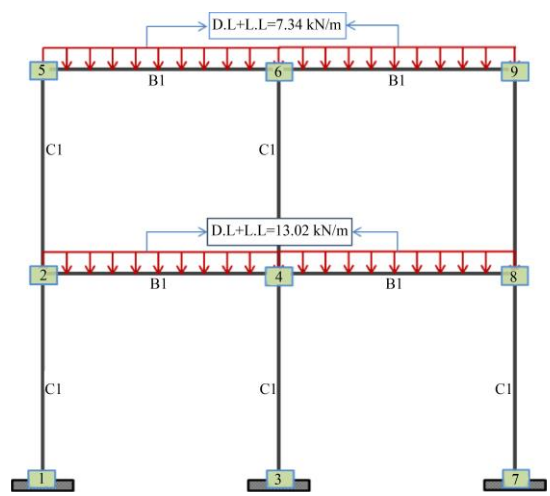

Fig. 3 Two-bay two-storey

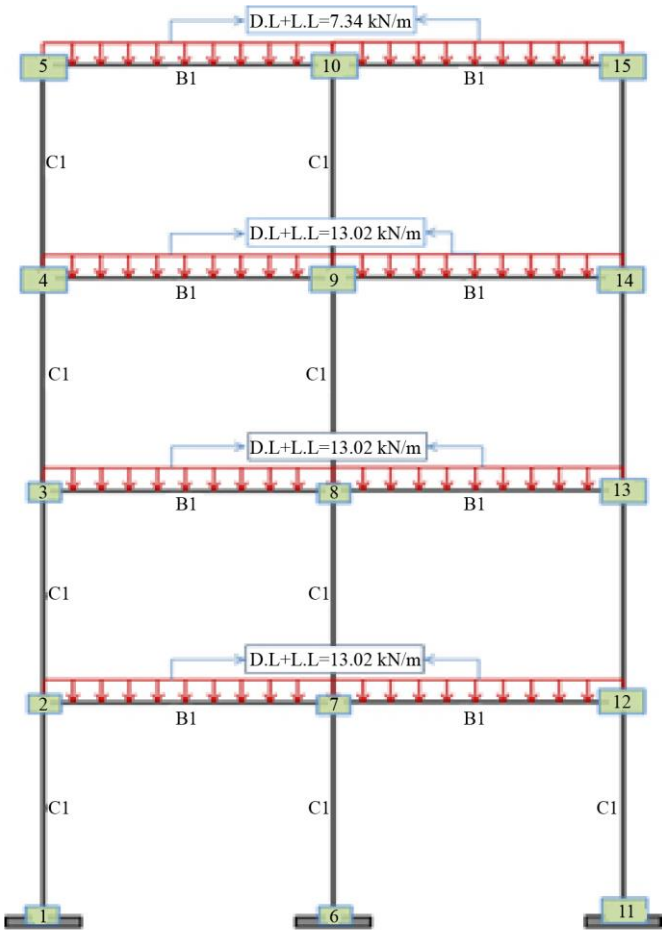

Fig. 4 Two-bay four-storey

4. Material and Section property

The material properties adopted are Elastic Modulus for steel and concrete, 200000 MPa, 27000 MPa for columns, and 25000 MPa for beams. Dead Load (DL) of 5.08 KN/m for the roof floor and 10.77 KN/m for other floors in addition to self-weight. A Live Load (LL) of 2.25 KN/m was taken for the roof floor and other floors. The size of beams and columns for all floors are 230X230 mm. The models were designed and checked as per IS 875, part 1 and part 2.

5. Results and Discussion

This current study investigates the probable response of the neighbouring columns and beams due to vertical element failure. Special emphasis is placed on understanding the vertical and lateral displacements at the point of column loss and its effect on neighbouring elements. With this context, linear dynamic analysis was performed to analyse and understand the outcome of changes in bays and storey height with the removal of the column at the bottom storey for four cases. To simulate the above phenomenon, the notable element is removed after a certain time's elapsed. In previous investigations, a linear force increase was maintained for five seconds until the load reached full capacity. In this investigation, the structure is loaded gradually from 0 to 2 seconds to reach equilibrium under service loads. Then, linear reduction of axial load is done after 5 seconds, i.e. from 2 to 2.5 seconds, until the vibrations regain stability. This is done under recommendation as per GSA guidelines. Studies reveal that the damage due to disproportionate collapse depends on the location of column failure. This proposed method provides insight into joint displacements under dynamic loading with sudden column loss. The simplicity of this method is very advantageous in understanding the progressive collapse of reinforced concrete framed structures.

5.1. Case 1: One Bay One Storey RCC Frame Structure

Figures 5(a) and 5(b) represent the dead load and live load on a 1-bay single-storey, and the right column was removed and replaced by the axial load.

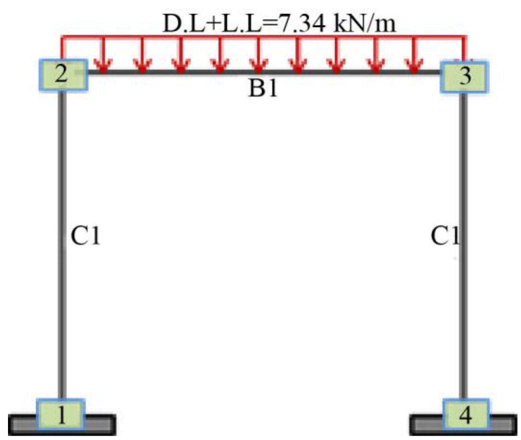

Fig. 5(a) Single-bay single-storey

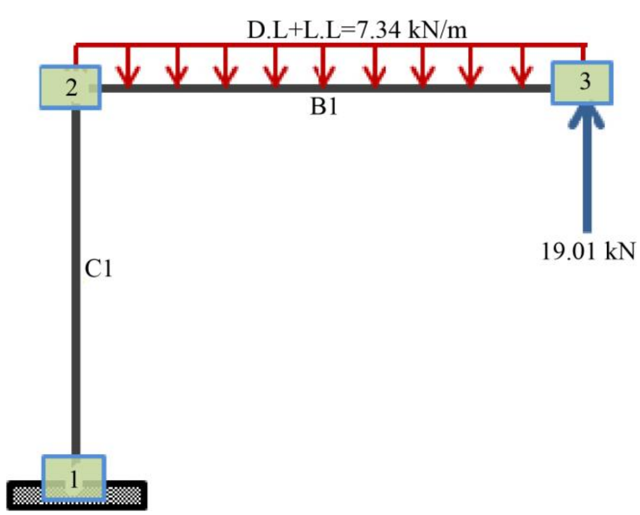

Fig. 5(b) Right column and replaced by axial load

5.1.1. Displacement Response along the X-Axis at Joint 2 and Joint 4

Dynamic linear analysis was performed when the right-side column (replaced by an axial force of 19.01 kN) in the first storey was suddenly removed using a Time history function. The result of the dynamic linear analysis is interpreted in the form of a graph in which two directional vibrations are plotted, displacement along the X-axis (m) and Time (sec), as shown in Figure 6(a). The joint on the top of the removed column vibrated and reached a maximum horizontal displacement of 0.0521 m. There are similar vibrations in joint 2 and joint 4, or the behavior of joint 2 and joint 4 is the same along the X-direction. The beam is acting as a cantilever, joint 2 is fixed, and joint 4 is free to move, but due to fixity the beam is acting as a rigid member and internal deformation of the beam is not seen in X-direction.

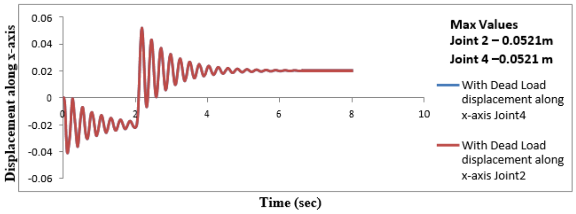

Fig. 6(a) Displacement of joints (2 and 4) along x-axis

5.1.2. Displacement Response along the Z-Axis at Joint 2 and Joint 4

The result of the dynamic linear analysis is interpreted in the form of a graph in which two directional vibrations are plotted, displacement along the Z-axis (m) and Time (sec), as shown in Figure 6(b). The joint on the top of the removed column vibrated and reached a maximum vertical displacement of 0.0000593m at joint 2 and 0.118m at joint 4 since joint 4 has a high vertical response when compared to joint 2, which can be seen in Figure 6(b). One of the major reasons for this behaviour is that the concrete column is rigid, whereas joint 4 is a free end due to the removal of a right-side column, and it undergoes deformation similar to a cantilever; the minute response of joint 2 is also seen in Figure 6(c), which shows the behaviour of joint 2 and 4 are similar but varies in their magnitude.

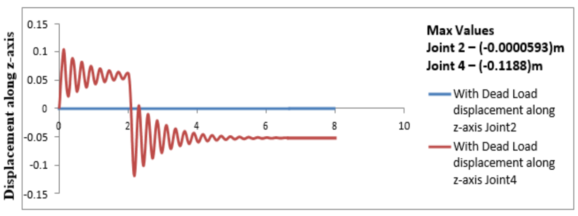

Fig. 6(b) Displacement of joints (2 and 4) along the z-axis

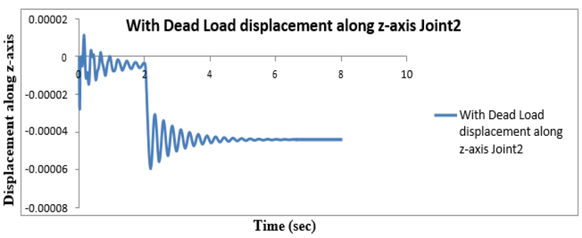

Fig. 6(c) Displacement of joints-2 along z-axis

5.2. Case 2: One Bay Two Storey RCC Frame Structure

Figures 7(a) and 7(b) represent the dead load and live load on a 1-bay single-storey, and on the right column, it was removed and replaced by an axial load.

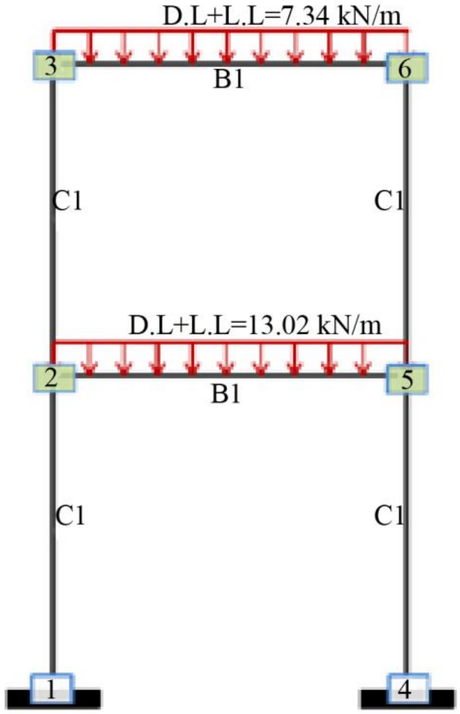

Fig. 7(a) One-bay double-storey

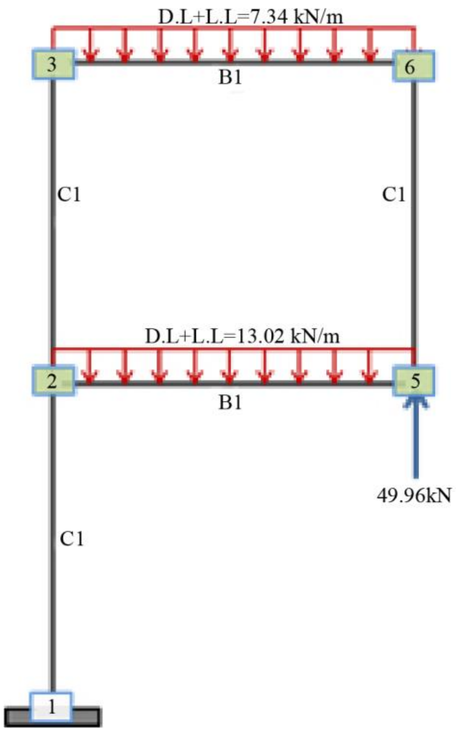

Fig. 7(b) Right column was removed and replaced by axial load

5.2.1. Displacement Response along X-Axis at Joint 2, Joint 5 and Joint 6

The response of these joints can be easily understood by the following deformed shape of the structure after performing linear dynamic analysis presented in Figure 7(c).

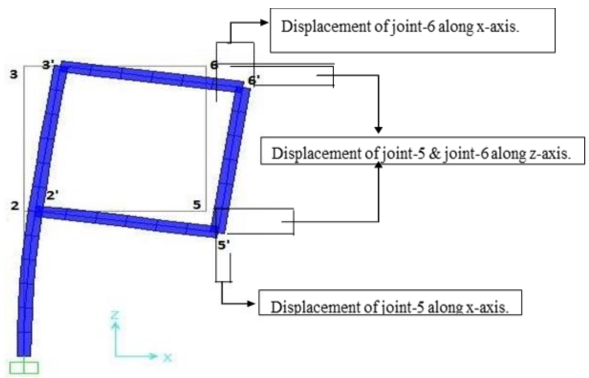

Fig. 7(c) Deformed shape after dynamic analysis

Dynamic linear analysis was performed when the right-side column (replaced by an axial force of 49.96kN) in the first storey was suddenly removed using a Time history function. The result of the dynamic linear analysis is interpreted in the form of a graph in which two directional vibrations are plotted, displacement along the X-axis (m) and Time (sec), as shown in Figure 8(a). Joint 5 on the top of the removed column vibrated and reached a maximum horizontal displacement of 0.158m, and there are similar vibrations in joint 2 (0.158m); at the same time, joint 6 reached a maximum horizontal displacement of 0.496m. The behavior of joint 2 and joint 5 are similar along a horizontal direction, as can be seen from Figure 8(b). The frame model is acting as a cantilever; joint 2 and joint 6 are rigid joints, whereas joint 5 is free to move; due to fixity, the beam is acting as a rigid member and internal deformation of the beam is not seen in joint 2 and joint 5.

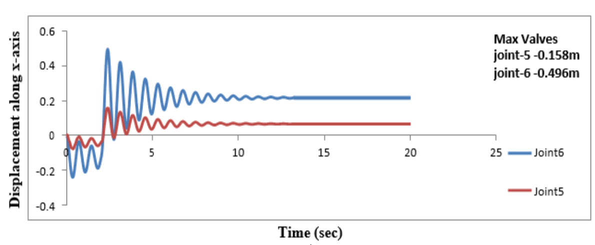

Fig. 8(a) Displacement of joints (5 and 6) along x-axis

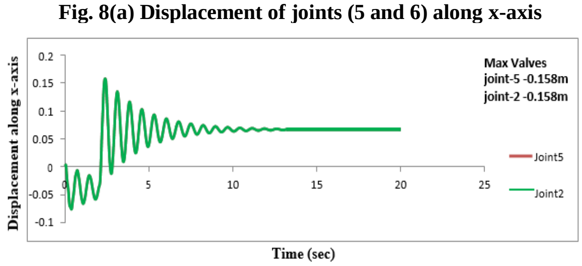

Fig. 8(b) Displacement of joints (2 and 5) along x-axis

5.2.2. Displacement Response along Z-Axis at Joint-5 and Joint-6

The result of the dynamic linear analysis is interpreted in the form of a graph in which two directional vibrations are plotted, displacement along the Z-axis (m) and Time (sec), as shown in Figure 9(a). It can be seen that the responses of both joints (5 and 6) are of the same magnitude (0.344m).

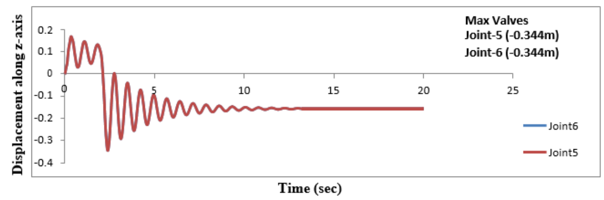

Fig. 9(a) Displacement of joints (5 and 6) along the z-axis

5.2.3. Displacement Response along Z-Axis at Joint-2 and Joint-5

In the case of joint-2 and joint-5, the response of joint-5 is of greater magnitude equal to 0.344m, as can be seen from Figure 9(b) since the column is lost just below joint-5 and hence can easily displace in vertical direction when compared to joint-2 which is having magnitude 0.000165m. Moreover, joint-2 acts as a rigid joint, which makes it difficult to undergo large displacement.

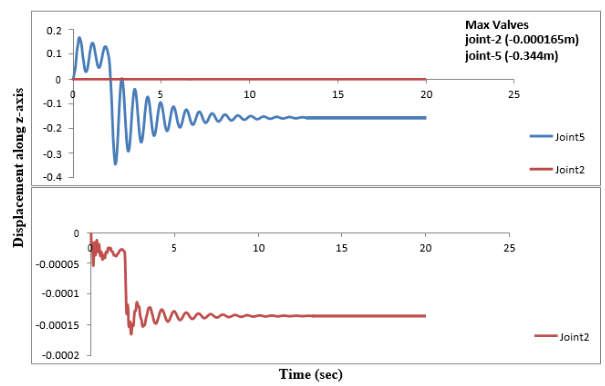

Fig. 9(b) Displacement of joints (2 and 5) along the z-axis

5.3. Case 3a: Two-Bay, Two-Storey RC Frame Structure (Removal of the Right Column)

Figures 10(a) and 10(b) represent the dead load and live load on a 2-bay single-storey, and on the right column, it was removed and replaced by an axial load. The top storey tends to move in the forward direction as the beam adjacent to the removed column pushes the remaining whole structure in the horizontal direction. Moreover, due to the weight of the superstructure, the structure also undergoes lateral deformations. This phenomenon can easily be understood through the shape of the structure after undergoing linear dynamic analysis. The graph depicts the response of joint 8 compared to joint 9 in a two-storey 2D frame along the x-axis under linear dynamic analysis. It is clearly observed that up to time 2 seconds there is initial disturbance along negative direction of x- axis, but it is more in joint 9 than in joint 8. After time 2 seconds, the disturbance is shifted to the positive direction of x axis since the column is removed after t=2 sec with maximum response in the form of horizontal displacement shown by joint 9.

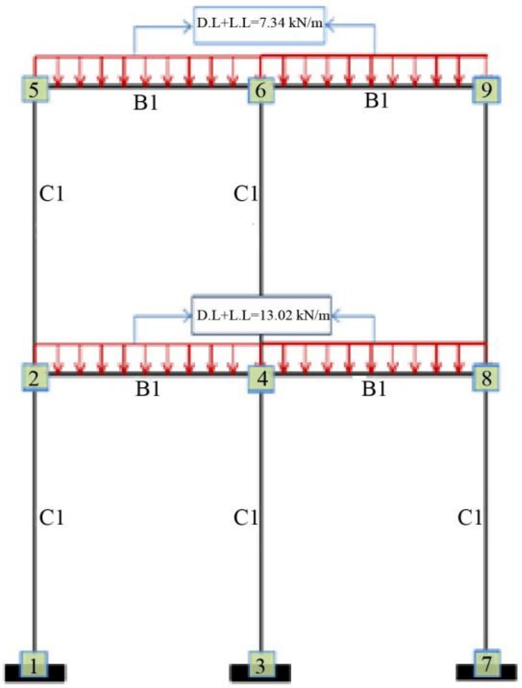

Fig. 10(a) Two-bay two-storey

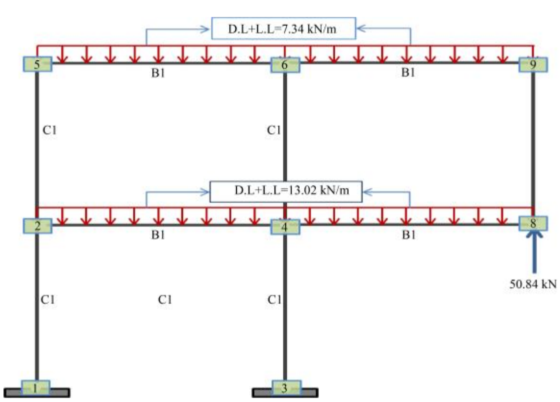

Fig. 10(b) Right column was removed and replaced by axial load.

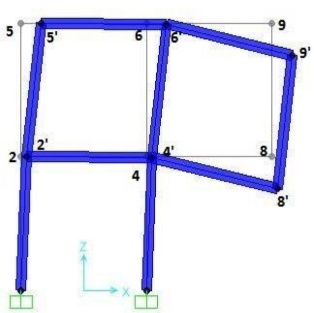

Fig. 10(c) Deformed shape after dynamic analysis

5.3.1. Displacement Response along X-Axis at Joint 2, Joint 4, Joint 8 and Joint 9

When the right-side column (replaced by an axial force of 50.84kN) in the first storey of a two-bay 2D two-storey frame was suddenly removed by using a Time history function, dynamic linear analysis is performed. The result of the dynamic linear analysis is interpreted in the form of a graph in which two directional vibrations are plotted, displacement along the X-axis (m) and Time (sec), as shown in Figure 10(c). Joint 8 on the top of the removed column vibrated and reached a maximum horizontal displacement of 0.0099m; at the same time, joint 9 reached a maximum horizontal displacement of 0.0236m. Joint 8 is free to end, but due to the removal of the right-side column, it undergoes deformation. There are similar responses (0.0099m) in joint 2, joint 4 and joint 8, or the behavior of joint 2, joint 4, and joint 8 is the same along X-direction as shown in Figure 10(d).

Joint 2, joint 4 and joint8 are rigid joints, but due to fixity, the beam is acting as a rigid member, and internal deformation of the beam is almost equal in X-direction because the structural load is acting along vertical direction so there is almost equal response in X- direction. Joint 9 has a high horizontal response due to lateral force exerted from adjacent nodes.

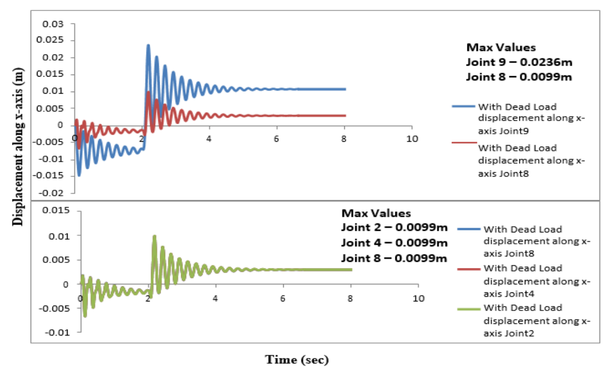

Fig. 10(d) Displacement of the joint (2, 4, 8 and 9) along x-axis

5.3.2. Displacement Response along Z-Axis at Joint 2, Joint 4, Joint 8 and Joint 9

The result of the dynamic linear analysis is interpreted in the form of a graph in which two directional vibrations are plotted, displacement along the Z-axis (m) and Time (sec), as shown in Figure 3(c). Joint 8 on the top of the removed column vibrated and reached a maximum vertical displacement of 0.02103m, and at the same time, there are similar vibrations (0.02103m) in joint 9, since the column is a rigid member, so joint 8 and joint 9 have same vertical response along Z - direction which can be seen in Figure 10(e). On the other hand, the response of joint- 2, joint-4 and joint-8 with respect to the z-axis varies. It is observed in Figures 10(f), 10(g) and 10(h) that joint-8(0.02103m) undergoes maximum displacement when compared to joint-2 (0.0000522m) and joint-4(0.000296m), respectively.

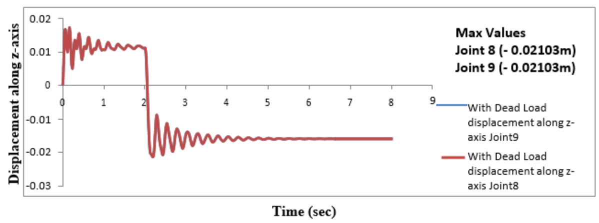

Fig. 10(e) Displacement of joints (8 and 9) along the z-axis

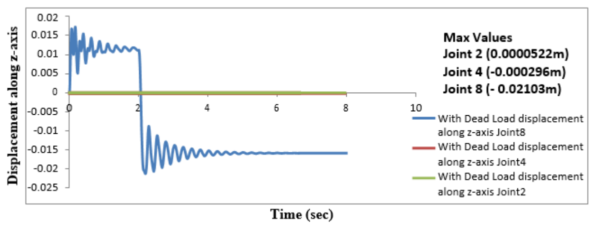

Fig. 10(f) Displacement of joints (2, 4 and 8) along z-axis

Since joint-8 is free to be displaced along the z-axis by exerting force from the superstructure, the response of joint-2(0.0000522m) and joint-4(0.000296m) is negligible when compared to joint-8 (0.02103m), which can be visualized from the Figures 10(f) and 10(g). Joint-2 shows the least displacement along the z-axis as it is rigidly connected and is 6m away from the removed column, whereas joint-4 is nearer to the removed column by just 3 meters. The displacement of joint-8 along the z-axis is 71.05 times the displacement at joint-4, whereas it is about 402.87 times the displacement at joint-2; this displacement occurs during the process of removing the column.

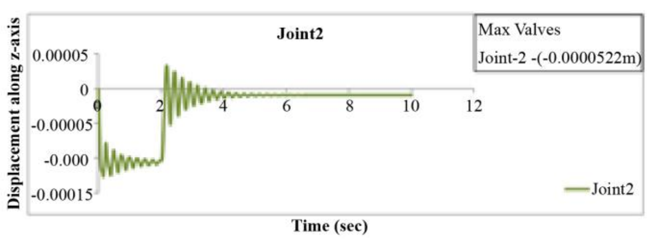

Fig. 10(g) Displacement of joints-2 along z-axis

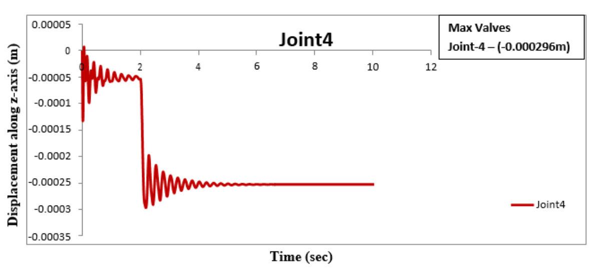

Fig. 10(h) Displacement of joints-4 along z-axis

5.4. Case 3a: Two-Bay, Two-Storey RCC Frame Structure (Removal of Middle Column)

Figure 11(a) represents the dead load and live load on the middle column removed and replaced by the axial load.

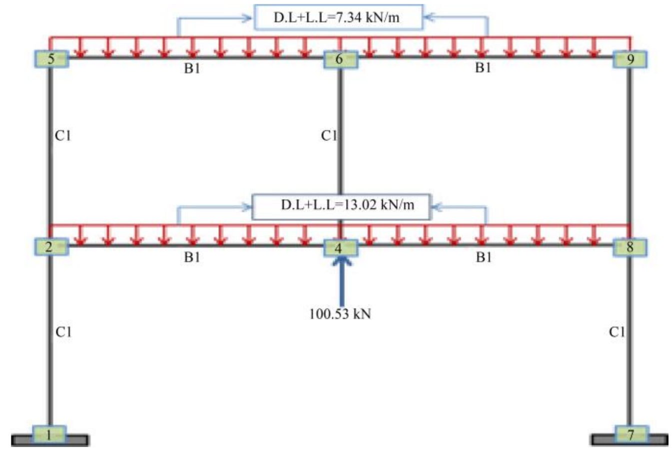

Fig. 11(a) Middle Column Removed & Replaced by Axial Load

5.4.1. Displacement Response along X-Axis at Joint 2, Joint 4, Joint 6 and Joint 8

Figure 11(b) represents the displacement of joints (2,4,6 and 8) along the x-axis. When the middle column (replaced by an axial force of 100.53kN) in the first storey of a two-bay 2d two-storey frame was suddenly removed by using a Time history function, dynamic linear analysis was performed.

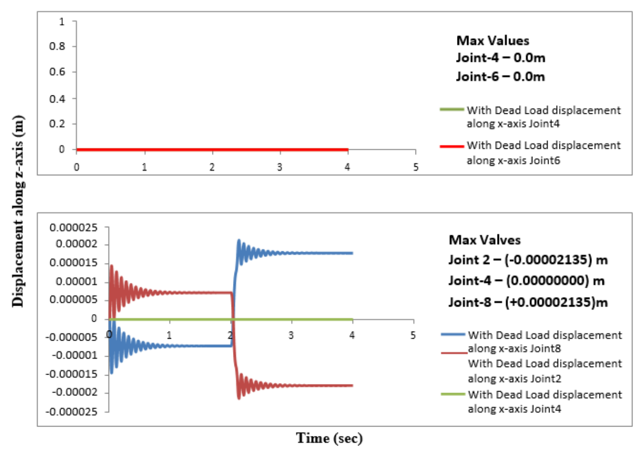

Fig. 11(b). Displacement of joints (2, 4, 6 and 8) along x-axis

The result of the dynamic linear analysis is interpreted in the form of a graph in which two directional vibrations are plotted, displacement along the X-axis (m) and Time (sec), as shown in Figure 11(c).

Joint-4 and joint-6 are exactly at the top of the removed column, so there is the least possibility of these joints undergoing any horizontal displacements as these joints are rigid, and as they lose their support, they will displace along the z-axis due to self-weight and gravitational pull whereas joint-2 and join-8 undergoes a maximum horizontal displacement of 0.00002135m. The behavior can be easily understood by the deformed shape of the structure after linear dynamic analysis, as shown below.

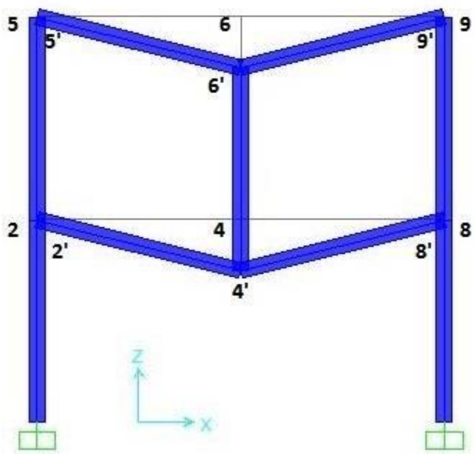

Fig. 11(c) Deformed shape after dynamic analysis

5.4.2. Displacement Response along Z-Axis at Joint 2, Joint 4, Joint 6 and Joint 8

Joint-4 and joint-6 show maximum response along a vertical direction, which is equal to 0.01027m. These joints undergo similar displacements due to their rigid behaviour. The response of joint-2 and joint-8, which is equal to 0.000146m, is negligible compared to joint-4 and joint-6, as shown in Figure 11(d). This behaviour is observed when the column is removed at time t=2sec up to time t=3sec, i.e. within a period of one second. Moreover, as the length of each member is 3m, beam member as well as column member, there will be a small deflection at joint-2 and joint-8 along the z-axis.

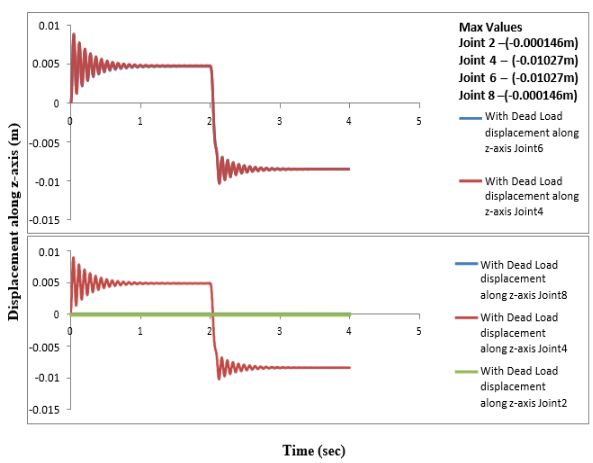

Fig. 11(d) Displacement of joints (2, 4, 6 and 8) along the z-axis

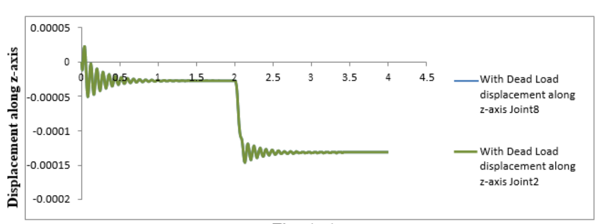

Fig. 11(e) Displacement of joints (2 and 8) along the z-axis

Since joint-4 is free to be displaced along the z-axis by exerting force from the superstructure, the response of joint-2(0.000146m) and joint-8(0.000146m) is negligible when compared to joint4 (0.01027m), which can be visualized in Figure 11(e).

5.5. Case 4(a): Two-Bay Four-Storey RCC Frame Structure (Removal of the Right Column)

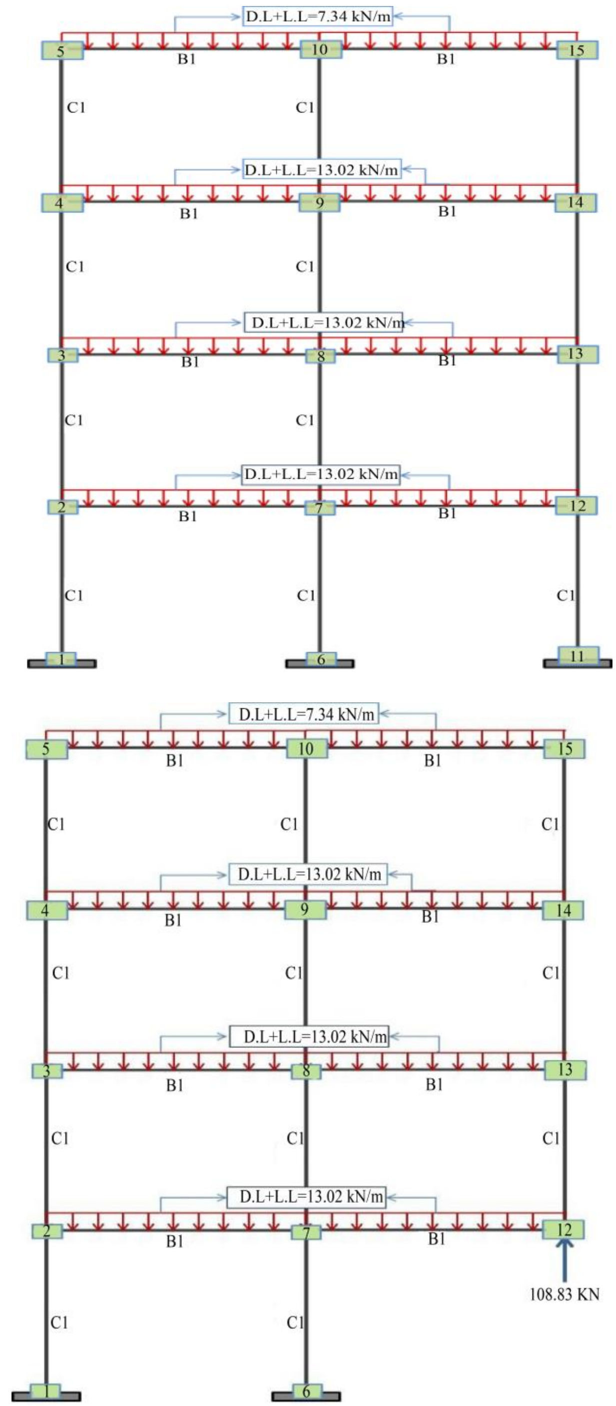

Fig. 12(a) Right column removed & replaced by axial load

Figure 12(a) represents the dead load and live load on two bay four storeyed with and without the right column and replaced by the axial load.

5.5.1. Displacement Response along X-Axis at Joint 2, Joint 7, Joint 12, Joint 13, Joint 14 and Joint 15

It can be predicted from Figure 12(b) that the response of joint-2,joint-7, and joint-12 are similar in magnitude and equal to 0.0047m, but the behavior changes for the extreme right joints(13,14). The displacement of joint-13(0.0156m) along the x-axis is 3.3 times greater than the displacement of joint-12; similarly, the displacement of joint-14(0.0261m) and joint-15 (0.0364m) is 5.5 and 7.7 times the displacement of joint-12. It can be concluded that the farther the joint is from the removed column, the higher the response along the x-axis. This response can be clearly understood by observing the deformed shape of the structure obtained after dynamic analysis, as shown in Figure 12(c).

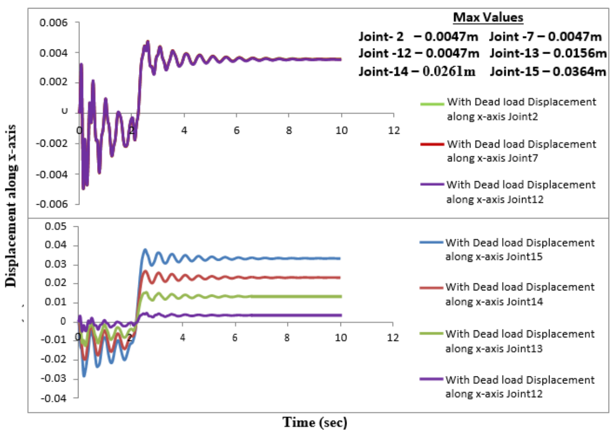

Fig. 12(b) Displacement of joints (2, 7,12,13,14 and 15) along x-axis

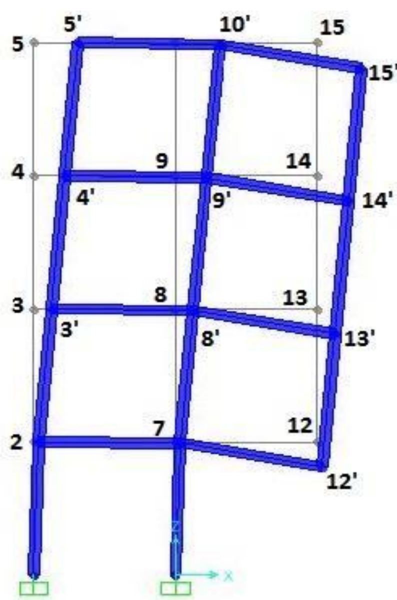

Fig. 12(c) Deformed shape after dynamic analysis

5.5.2. Displacement Response along Z-Axis at Joint 2, Joint 7, Joint 12, Joint 13, Joint 14 and Joint 15

The response of joint-2, joint-7 and joint-12 along the z-axis varies in magnitude, with joint-12 showing a maximum displacement of 0.0207m and joint-2 showing a minimum displacement of 0.0000388m, but the behavior of all three joints are nearly similar, which can be predicted from Figure 12(d) and Figure 12(e) respectively. The only difference is that as the right corner column is removed using the time-based function, the dead load of the superstructure, which was taken by the removed column, does not have any alternate path to redistribute the superstructure loading; hence, the joints above the removed column displace more compared to other adjacent horizontal joints. It is also observed that the displacement of joint-12 along the z-axis is 4.4 times the displacement along the x-axis. Similarly, the displacement of joints (13, 14 and 15) along the z-axis is 1.33, 0.793, and 0.56 times the displacement along the x-axis. The displacements of joints (2 and 7) along the z-axis are 0.008 and 0.129 times the displacement along the x-axis, which is negligible; hence, the displacement will be maximum only for those joints which are just above the removed column. Since the magnitudes of joint-2 and joint-7 are negligible compared to joint-12, which can be visualized in Figure 12(f),

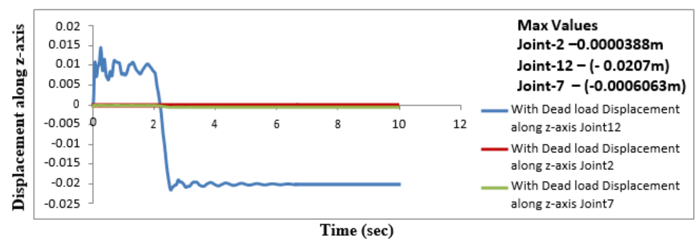

Fig. 12(d) Displacement of joints (2, 7 and 12) along the z-axis

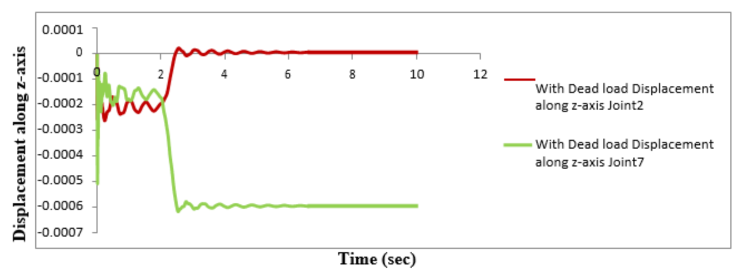

Fig. 12(e) Displacement of joints (2 and 7) along the z-axis

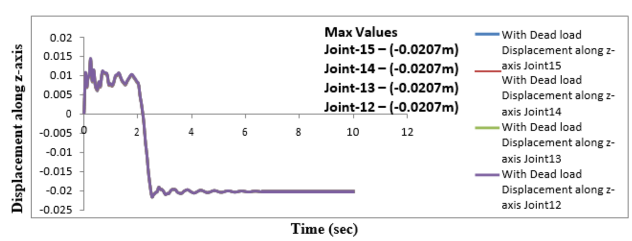

Fig. 12(f) Displacement of joints (12, 13, 14 and 15) along z-axis

5.6. Case 4(b): Two-Bay Four Storey R.C.C Frame Structure (Removal of Middle Column)

Figure 13(a) represents the dead load and live load on two bay four storeyed with and without the right column and replaced by the axial load.

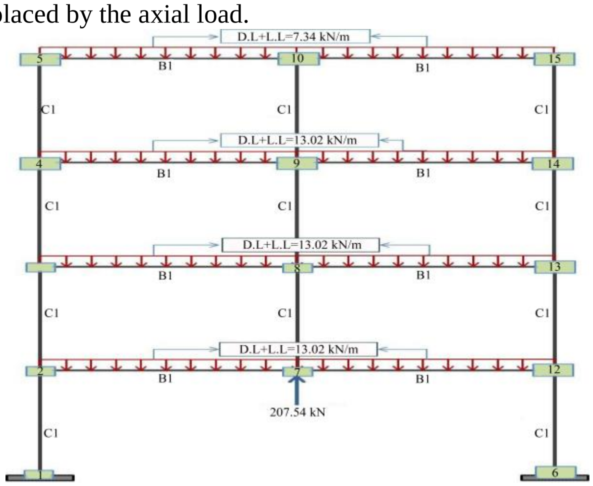

Fig. 13(a) Middle Column Removed & Replaced by Axial Load

5.6.1. Displacement Response along X-Axis at Joint 2, Joint 7, Joint 8, Joint 9, Joint 10 and Joint 12

The displacement of joint-2(0.000029m) and joint-12(0.000026m) are almost similar in magnitude but are acting in opposite directions, which indicates as the collapse progresses, the right side of the structure will displace towards the removed column at the same time, left side of the structure also tends to move towards the removed column. Thus, as it moves upward, more and more lateral force will be generated, leading to maximum displacement along the x-axis at the topmost joint-10(0.000043m).

It can be concluded from Figure 13(b) that in joint-7, displacement along the x-axis is 0.000018m; similarly, at joints (8, 9, and10), the displacements are increasing by 1.83, 1.24, and 1.05 times. On average, the displacement along the x-axis increases by 1.37 times as it moves from the ground floor to the top floor. This response can be clearly understood by visualising the deformed shape of the structure obtained after dynamic analysis, as shown in Figure 13(c).

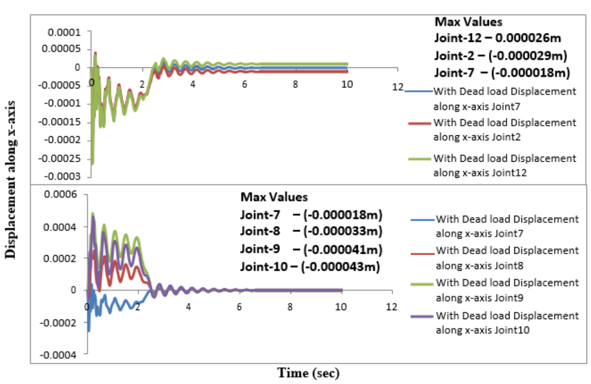

Fig. 13 (b) Displacement of joints (2, 7, 8,9,10 and 12) along x-axis

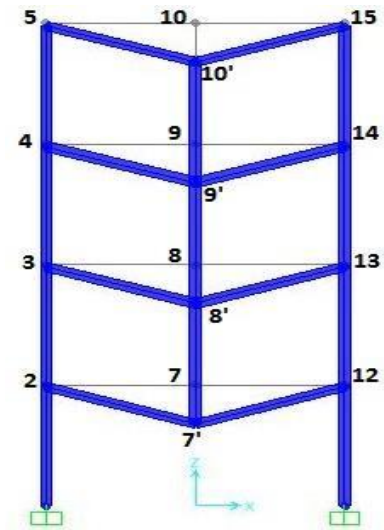

Fig. 13(c) Deformed shape after dynamic analysis

5.6.2. Displacement Response along Z-Axis at Joint 2, Joint 7, Joint 8, Joint 9, Joint10 and Joint 12

The response of joint-2(0.000169m) and joint-12(0.000169m) along the z-axis is the same both in magnitude and direction. The response of joint-7(0.000991m) is greater than its adjacent joints; thus, the collapse is first followed by the above structural members. The response of joints (7, 8, 9 and 10) in the form of vertical displacements is 58.64 times the displacement of corner joints (2 and 12), respectively. Figure 13(d) shows the graphs plotted by taking the vertical displacement of joints along the axis and time in seconds along x axis. The middle graph shows the proper visualisation of the difference in displacement of joints (2 and 12), respectively. They show the same response in both magnitude and direction.

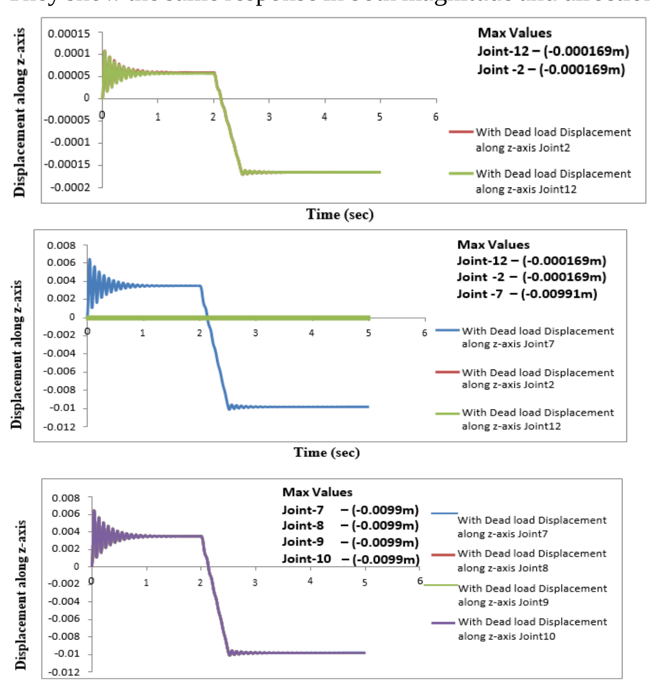

Fig. 13(d) Displacement of joints (2, 7, 8,9,10 and 12) along z-axis

6. Conclusion

6.1. Discussion in Terms of Displacement along the X-Axis of Adjacent Beams

- For single-bay 2D structures, it was observed that the displacement along the x-axis of the beam adjacent to the removed column is almost 1.74% of the length of the beam in the case of only one storey. The displacement of the same beam was altered to 5.3% for two-storey structures, and the displacement of the beam along the x-axis in the top storey is found to be 16.5% of the length of the beam, which means it has increased by 3.1 times the displacement of first storey beam.
- For two-bay 2D structures, when the corner column has been removed, the displacement along x- the axis of beams adjacent to the removed column with the linear rise in the number of stories varies in the range of 0.2% to 0.5% the length of beam and the displacement of top storey beam in 0.5% to 1.3% the length of beam.
- In the case of three-storey structures, when the corner column is removed, the displacement of top-storey beams along the x-axis varies in the range of 0.9% to 2% the length of the beam similarly for four-storey and above 2D structures, the displacement of top storey beams varies in the range of 1.2% to 3.3% the length of beam respectively.
- Thus, when the corner column of the first storey has been removed, the displacement along the x-axis of beams increases as it moves up from the first storey to the top storey by an average value of approximately 0.7% of the length of the beam. These displacements will not be uniform but vary from storey to storey depending on the resistivity offered by each storey. Moreover, the displacement of each storey beam was found to be similar, with negligible differences.

6.2. Discussion in Terms of Displacement along the Z-Axis of Adjacent Beams

- For Single-bay 2D structures, it was observed that the displacements along the z-axis of the beam adjacent to the removed column at fixed and free ends are in the range of (0.002% to 0.0055%) and (4% to 11.5%) of the length of the beam in case of only one storey.
- In the case of two-storey structures, the displacement of the beam along the z-axis in the top storey at the free end is observed to be 11.5% the length of the beam, which means the displacements at the free ends are similar and are independent of a number of storeys, as the joints connecting these beams to columns are rigid.
- For two-bay 2D structures, in the case of two-storey, when the corner column is removed, the displacement along the z-axis of the beam adjacent to the removed column at fixed and free ends is observed to be in the range of (0.01% to 0.03%) and (0.7% to 1.1%) the length of the column. It is also observed that the effect of corner column removal is greater on the nearer fixed end compared to the farther fixed end (0.001% to 0.006% the column length). This behavior is due to an increase in resistivity.
- When a middle column of the structure is removed, the displacements along the z-axis of the middle structural members are greater compared to the corner members. It is observed that the middle portion elements displace along the z-axis almost an average of (0.32% to 0.39%) of the length of the column, whereas the columns adjacent to the removed column displace by a small amount in the range of (0.005% to 0.01%) respectively.
- Thus, the displacement at a particular joint, say at fixed ends about the z-axis, is negligible compared to the displacement about the x-axis.
- The alternate load path approach ensures structural stability by creating backup load-transfer mechanisms in case a key component fails, thereby preventing widespread or disproportionate collapse.
- To safely redistribute forces from a compromised area to surrounding undamaged elements, structural components must maintain robustness, interconnectedness, and the ability to absorb and dissipate energy effectively.

Continuous bottom reinforcement in beams makes the frame robust enough to enhance alternate load paths, and these horizontal displacements reduce vertical displacements by 30%. Maintaining window openings to less than 40% of the wall area showed considerable stiffness to resist collapse. Further studies should include nonlinear behavior to understand post-yield behavior.

This research utilizes computational modelling techniques in full compliance with academic guidelines, utilizing only non-proprietary analytical methods derived from established engineering principles as per GSA guidelines. Findings incorporate public safety by proposing collapse-resistant design frames for RC structures. All assumptions and limitations are transparently documented for proper understanding.

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