Study on non-linear static behavior of 2D low-rise RCC framed structure subjected to progressive collapse

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Abstract. In this study, the progressive collapse behavior (full load and displacement control methods) of low-rise models representing 2-bay2storey and 3-bay3storey reinforced concrete framed structures located in high seismic zone, designed by Indian codes (IS 456:2000 and IS 1893-2016) for envelope loading combination are assessed with and without U.S. General Services Administration (GSA) guidelines. For displacement-controlled mechanism, a target displacement of 2%, 4% and 5% of the height of structure are considered. Non-linear static behavior of the structure is investigated through (a) Hinge formation pattern (b) Displacement of Joints adjacent to removed column along x-axis and z-axis (c) and Pushdown capacity curves. The results indicate that the Hinge formation patterns are similar for envelope loading combination and GSA loading combination, and the accuracy of the displacement-controlled method is much remarkable compared to full load method, therefore a standard formula is obligatory for calculating the target displacement to control progressive collapse, based on structural requirements unlike the dynamic increase factor calculations based on the structural capacity. With increase in each span and height of structure consecutively, pushdown capacity curves indicate that the base shear increases approximately by two times whereas the displacement in downward direction reduces by 59% and 62.4% for corner column removal and middle column removal cases respectively.

Keywords: displacement-controlled method; envelope loading combination; full load-controlled method; nonlinear static analysis; progressive collapse; U.S. General services administration (GSA) Guidelines

1. Introduction

Progressive collapse implies disproportional global structural system failure originated by local structural damage. It is a rare event, as it necessitates an initiation of local element removal criteria either due to the inevitable forces of nature or due to manmade hazards. The gravity load of the building is now transferred to neighboring columns; these columns should resist the additional abnormal gravity loads & redistribute loads to avoid failure of the major part of the structure. Present day building design practices & lesser integral ductility and continuity, gets more prone to

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progressive collapse. However, there should be certain provisions needed for additional consideration to ascertain the safety of structure after any local failure. The concept of progressive collapse comes to image after the collapse of the 22 story Ronan Point Apartment Tower in 1968.The gas explosion occurred on the 18th floor that vigorously rapped out the exterior load bearing panels of the kitchen near the corner of the building. This results in loss of support at that story (i.e., 18th floor) & triggered above floors to collapse. The potential of this collapsing floors causes, impact load on lower stories & set up a progressive collapse. The entire exterior corner of the building collapsed from top to bottom. Recently, an interest in this topic has been increased after the destruction of Murrah Federal Office building in Oklahoma City due to terrorist attacks, and also the collapse of the unforgettable Twin tower of the World Trade Center in New York (Sept 2001). These events define the progressive collapse very well. In order to secure structural safety against progressive collapse additional considerations such as abnormal loadings must be taken. The abnormal loads arise from vast sources such as explosion of gas, vapor inferno or confined dust, malfunctioning of machines, bomb explosion, the sudden impact of vehicles, etc. Nevertheless, till date, there are no adequate tools that can analyze the progressive collapse with satisfactory reliability. Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis.

Pushover analysis can be performed either force-controlled or displacement-controlled. Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple "ASCE-41 (2006)". Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure. Using this method, structures with predictable seismic performance can be produced. In this paper a study on the behavior of two-dimensional low rise reinforced framed structures subjected to progressive collapse is made by pushdown analysis under envelope loading combination (envelope of fourteen Indian standard load combinations) and U.S. General Services Administration (GSA) loading combination using both full load method and displacement-controlled method.

"Brunesi Emanuele and Parisi Falvio (2017)" suggested new fragility models on the basis of push down analysis for reinforced concrete bare framed structures designed as per European code, their study revealed that secondary beams designed to resist self-weight proved to provide more stiffness under column loss. "Abbasnia *et al.* (2016)" calculated the sustainability of beams and their vertical displacements under accidental loading. A theoretical approach was actuated, and these results were validated with experimental results. The ability to withstand collapse against sudden loading due to element loss was also explained. "E. Brunesi *et al.* (2015)" fragility models for low rise RC bare frame structures, one resisting gravity load and other resisting earthquake load was applied. The earthquake resistant design frame was resistant to progressive collapse also when compared to frame designed only for gravity loads. "Tavakoli *et al* (2014)" performed nonlinear static analysis and investigated the response of various lateral loads under presence of infill walls against shear strength and seismic performance. They came up with vital information against seismic safety of reinforced concrete frames. "Tsai and Huang (2011)" have studied the efficacy on the level of performance and their respective response of a building equipped with reinforced concrete via three kinds of infilled walls. They have gained some profound data with
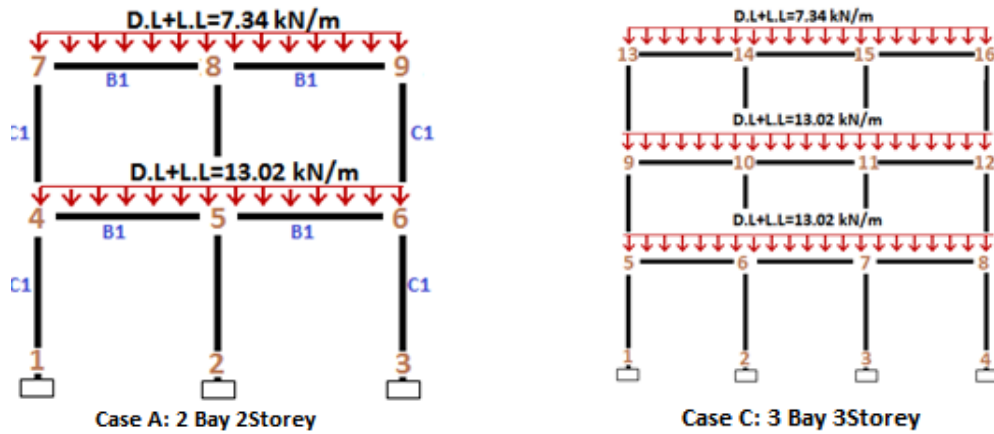
Fig. 1 Elevation of two cases with uniform bay width used for the study

the resistance of the experimental buildings against raised collapse followed by sudden column loss and increased markedly via wing-type walls. The bending potential of the structure was found to be minimized with the usage of exteriorly non-structured walls. "Yihai Bao and Shashi K Kunnath (2010)" developed a shear wall model for various seismic zones and calculated the behavior due to collapse of shear wall at first storey, their study made it evident that frames designed to resist seismic loads were more robust compared to frames designed without seismic loading. "Hayes *et al* (2005)" investigated the validity of seismic strengthening schemes to mitigate the risk of blast and progressive collapse of reinforced concrete buildings. It is obvious that preventing the progressive collapse by implementing seismic strengthening schemes adopted by "Hayes *et al* (2005)" represented increasing the strength of the building rather than enhancing the continuity and the ductility of that building.

As huge experimental set-up is needed to validate simulation results, modeling procedures are mostly preferred to understand the failure mechanism scenario. In this paper, following conclusions are made: though a large amount of the literature is accessible and many researchers have dealt with progressive collapse analysis to investigate the behavior of the structures designed as per the governing earthquake codes of respective countries, most of the researchers have used full load mechanism to study progressive collapse analysis, but very less work has been done on comparison of full load mechanism and displacement controlled mechanism with and without using "GSA guidelines"

Hence the present study aims at evaluating the progressive collapse behavior (full load and displacement control methods) of two dimensional reinforced concrete framed structures located in high seismic zone, designed seismically by Indian codes (IS 456:2000 and IS 1893-2016) for envelope loading combination and are assessed with and without U.S. General Services Administration (GSA) guidelines using "SAP2000 version 20"

1.1 Outline of proposed work

The main objectives of the study are as follows: -

1. To evaluate the Hinge formation pattern of 2D low rise framed rcc structures subjected to progressive collapse by using full load method and displacement control method.

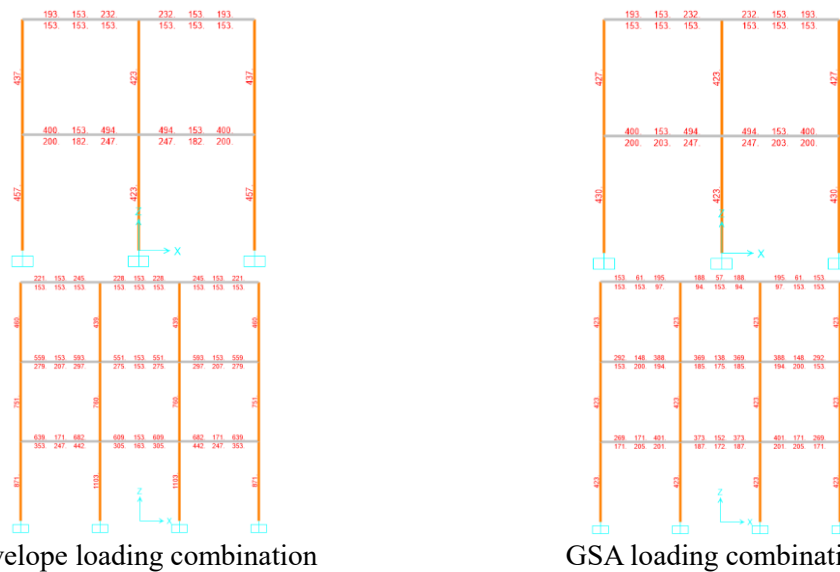| Envelope loading combination | GSA loading combination |

Fig. 2 Longitudinal Reinforcing area in columns and beams for two different loading combinations

2. To evaluate Displacement of Joints adjacent to removed column location along x-axis and z-axis direction by using both methods.

3. To evaluate Pushdown capacity curve for corner column removal case and middle column removal case with increase in each span and height of storey consecutively in case of low rise rcc structures.

2. Model description

In this paper, two dimensional reinforced concrete framed structures with 2-bay2storey and 3-bay3storey buildings as shown in Fig. 1 with square plan, and the span length of 3 m were modeled using finite element method based "SAP2000 version 20" software. These models were designed and checked as per Indian standard codes.

2.1 Material and sectional property

Structures are analyzed and designed as per "Indian standard codes IS 456:2000" envelope loading combination. The dimensions of columns and beams are similar for all stories and are 230 mm x 230 mm. Material properties used for the structures are M25 for beams and M30 for columns with a unit weight of concrete as 25 N/mm^3. Fe415 grade steel is used for both beams and columns. The elasticity modules of steel considered is 200000 MPa, 25000 MPa for concrete beams and 27000 MPa for concrete columns respectively.

2.2 Reinforcing details in columns and beams

The reinforcing details in columns and beams for the given two cases i.e., 2-bay2storey and 3-

Envelope loading combination   GSA loading combination
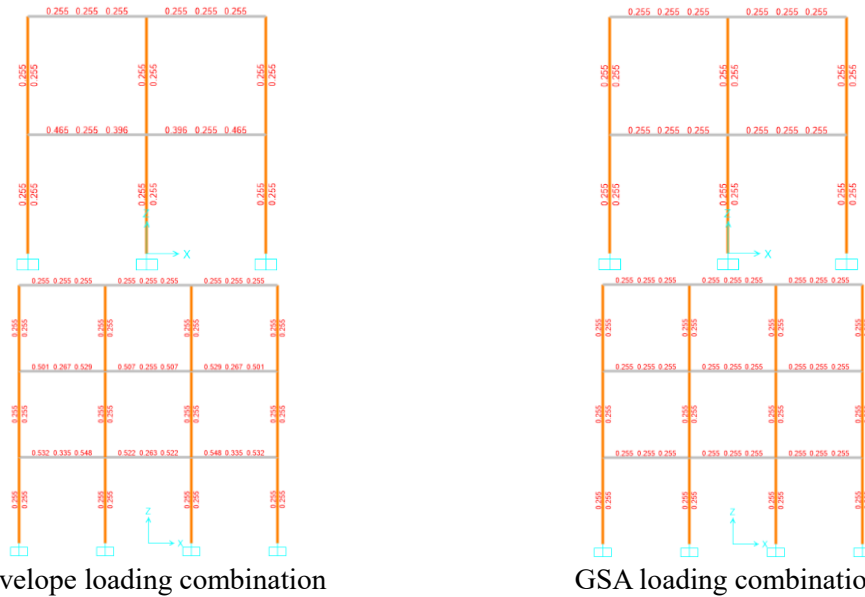
Fig. 3 Shear Reinforcing area per unit length in column and beam for two different loading combinations

bay3storey for two types of loading combinations, envelope loading combination and GSA loading combination are presented in the form of longitudinal reinforcing area and shear reinforcing area per unit length as shown in Figs. 2 and 3 respectively. These are evaluated by seismic designing of the two concrete structures using "Indian standard codes IS 456:2000". From reinforcing areas, the number of reinforcing bars at various zones can be manually determined. As the number of bays and storeys are increased, the longitudinal reinforming area in columns and beams increases from 1 to 2.5 times compared to GSA loading combination, because envelope loading combination is meant to give extreme results, can be used to assess the behavior of the structure subjected to collapse.

2.3 Intensities of loading

2.3.1 Live load
The live load is obtained from Indian standard code "IS 875(part2)".
Same Live load both on roof and floor=3kN/m^2.
After calculations, live load on each R.C frame=2.25kN/m.

2.3.2 Dead load
The dead load is obtained from Indian standard code "IS 875 (part1)". The unit weight of concrete is taken as 25kN/m^3.
Self-weight of the structural elements include: -
Floor finish (except top floor)=3 kN/m^2,
For top floor=1.5kN/m^2, assume thickness of wall 110 mm.
Thickness of parapet wall is taken as 90 mm, assume thickness of slab as 120 mm.
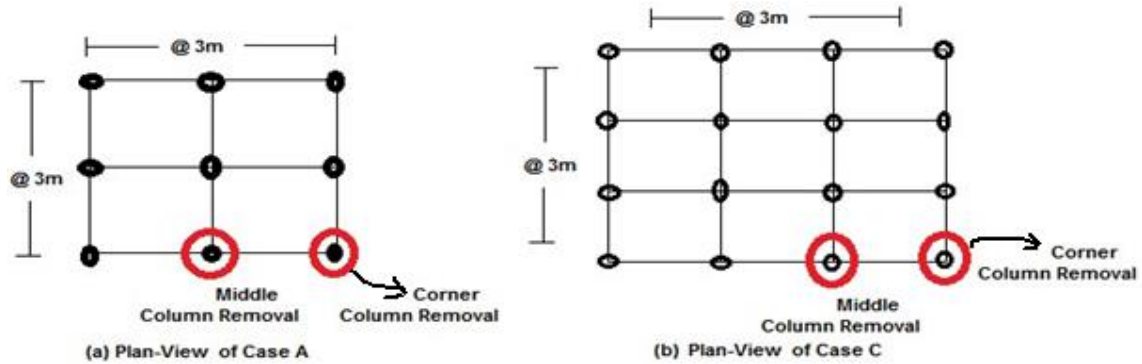
Fig. 4 Location of removed column member

Therefore, total Dead load on each frame except top floor frames=10.77 kN/m.
Total Dead load on each top floor frames=5.085 kN/m.

2.3.3 Earthquake load

The Geological Survey of India has classified the country into four seismic zones with varying seismic potential. These are classified based on the observations made and recorded after the previous earthquake and thus a zoning map of the country is designated. The seismic zone intensity is classified as zone II (low intensity zone), zone III (moderate intensity zone), zone IV (severe intensity zone) and zone V (very severe intensity zone). Structures used for the study are designed in zone V (Jammu and Kashmir) as per Indian standard code "IS 1893-2016". Zone factor which provides the valve of the peak ground acceleration for the design of the structure is taken as 0.36 as per Indian codes for zone V.

For all given structures or cases, following factors are considered as per Indian standard code
Soil type II refers to medium or stiff soil,

Response Reduction Factor=5, is the ratio of the peak lateral force that would generate in a structure if it responded totally linear elastic under the specified ground motion to the lateral force that the structure was intended to resist. It is determined by the structure's perceived seismic damage performance.

and Importance Factor=1, is a factor used to estimate design seismic force or a multiplier to increase or decrease the design base shear, depending on the occupancy or fundamental use of the structure.

3. Methodology

For the analysis, two cases are considered as shown in Fig. 1. The height of each storey is taken as 3 m. In this study, both the full load controlled and displacement-controlled pushdown analysis was carried out to understand the behavior of two-dimensional low rise reinforced concrete framed structures by visualizing the hinge formation pattern, displacement along x-axis and z-axis of horizontal and vertical joints adjacent to the removed column, and pushdown capacity curves with increase in number of spans and height of storey consecutively. Moreover, the stepwise load increase was applied through envelope loading combination and GSA loading combination, to
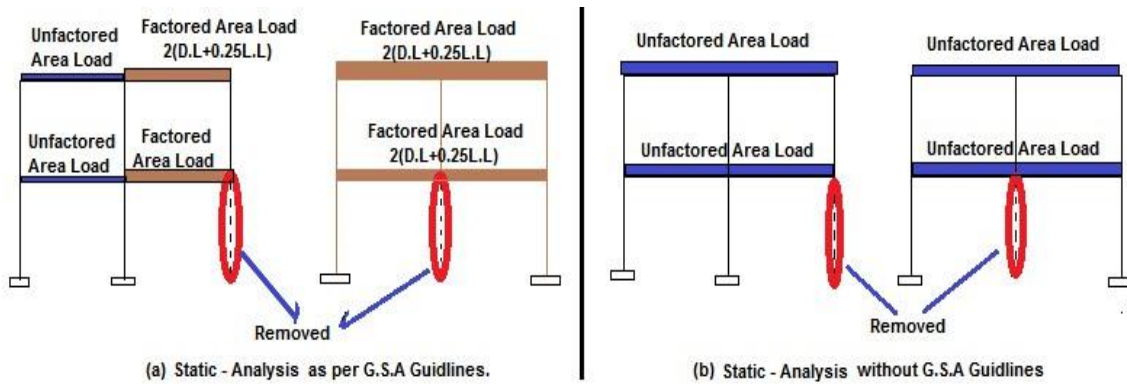
Fig. 5 Load combinations and loading approach for different column removal scenarios

ensure a more accurate prediction of the dynamic effect. To evaluate the behavior of different two-dimensional low rise reinforced framed structures subjected to progressive collapse using non-linear static analysis with and without "GSA guidelines", only two column removal conditions are considered as shown in Fig. 4

    a.   Removal of Corner Column
    b.   Removal of Middle Column

3.1 Analysis procedures used for study

3.1.1 Full load or force controlled method

When the load is known, such as gravity loading, the full load or force-controlled approach is utilised. This method involves gradually increasing the load until the structure collapses or reaches its maximum load. Maximum deformation is measured at that point. Furthermore, due to the development of mechanisms and p-delta effects, target displacement may be associated with a very modest positive or even negative lateral stiffness, causing numerical issues that impact the accuracy of the results. In nonlinear static progressive collapse analysis, a force-based method is applied and the structure is pushed down to the target force or load.

3.1.2 Displacement controlled method

Deformation controlled method is generally used for seismic or lateral pushover analysis where earthquake or wind load governs. In this method structure is pushed to undergo maximum permissible deformation under gravity load and at that time, maximum attained load is measured, in displacement-controlled procedure specified drifts are sought as in seismic loading where the magnitude of applied load is not known in advance. The structure is pushed down to target displacement rather than target force in the displacement-based method (similar to the one in seismic pushover analysis). The amount of the load combination is adjusted as needed until the control displacement reaches a predetermined valve. In most cases, the target or control displacement is the roof displacement at the structure's Centre mass. In this study target displacement is considered as 2%, 4% and 5% height of the structure. The forces and deformations calculated at the desired displacement are used to determine inelastic strength and deformation demands, which must be compared to available capabilities in order to execute a performance check.
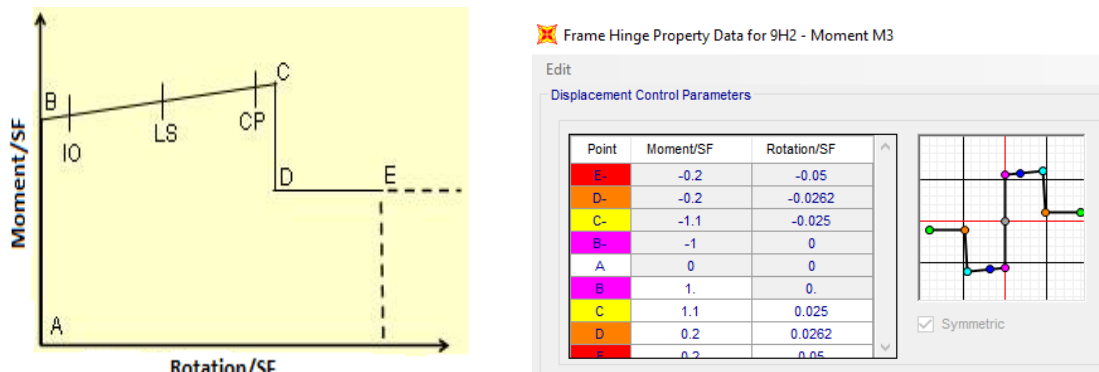
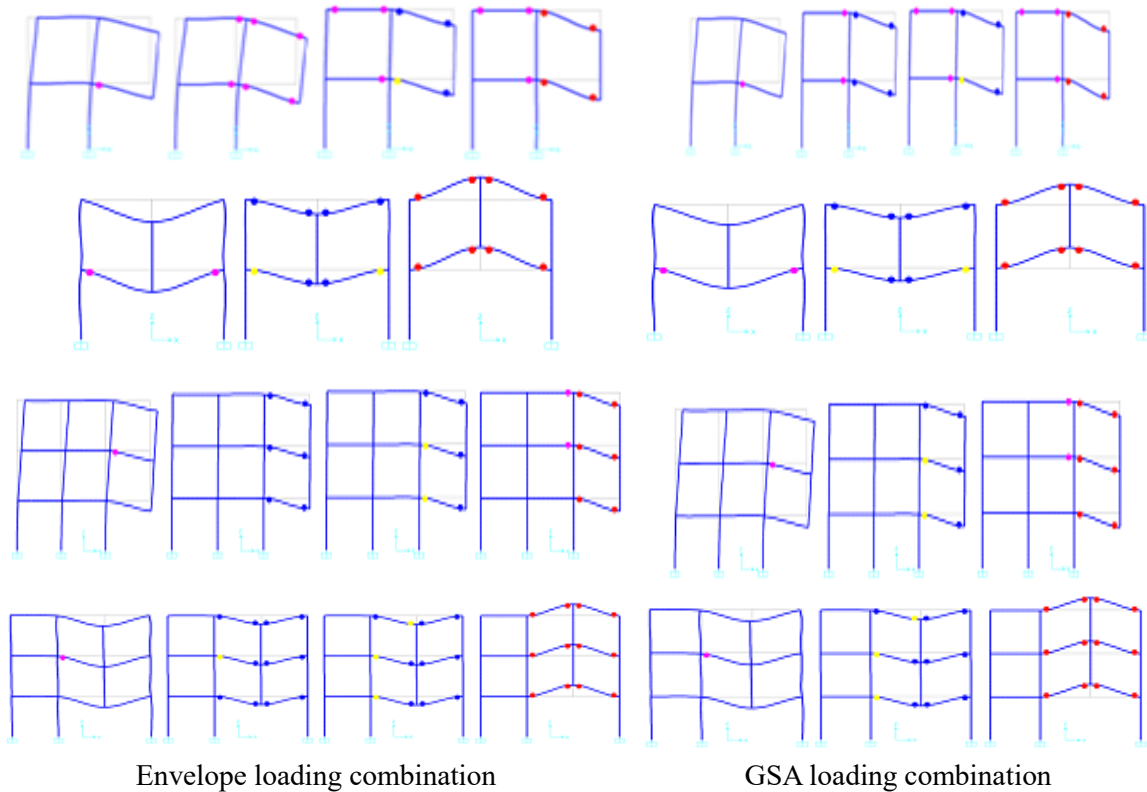Fig. 6 1f Moment (M3) plastic hinge property

Envelope loading combination   GSA loading combination

Fig. 7 Hinge formation pattern using full load mechanism for two different loading combinations

3.2 Analysis Loading

For non-linear static analysis, following vertical load can be applied in downward direction if column is removed statically, the gravity load increases as per U.S. General Services Administration (GSA) guidelines. In order to get progressive behavior, load combination for the factored area (affected area above removed column) is given by Eq. (1) and the unfactored area (remaining area)

is Eq. (2) as shown in Fig. 5

   a) Above the removed column location

$$\text{Factored Area Load} = 2(D.L+0.25L.L) \quad \text{"(GSA)guidelines"} \tag{1}$$

  b) Adjacent to the removed column location

$$\text{Un-factored Area Load} = \text{Envelope load combination "IS 875-1987(Part 1,2 and 5)"} \tag{2}$$

3.2 Settings for plastic hinges

Automatic hinge characteristics are assigned to a frame element for nonlinear analysis. The program calculates the hinge properties based on the cross-section and reinforcing details provided. SAP2000 uses auto M3 hinge property from ASCE 41-13, and Table 10-7 concrete beams-flexure items I for default moment hinges. Flexure failure mode of beam elements are only investigated since it governs the collapse of low rise reinforced concrete buildings under single column removal condition. A graphical illustration of the moment hinges property (see Fig. 6) is provided, with a force-displacement curve for each degree of freedom that gives the yield valve and the plastic deformation following yield.

This is accomplished using a five-pointed curve with valves. Fig. 6 shows the letters A-B-C-D-E. The origin is always at point A, yielding is at point B, ultimate capacity for pushover analysis is at point C, Point D denotes remaining strength and Point E denotes ultimate failure. Additional deformation measurements can be found at points IO (immediate occupancy), LS (life safety), and CP (Collapse prevention). These are data-gathering measures. At each level, IO, LS, and CP, ASCE 41-13 specifies the allowed valves for plastic hinge rotation. At both ends, auto M3 hinges are allocated to beams.

4. Analysis results and discussions

4.1 Non-linear static analysis

Nonlinear analysis is widely used to understand the behavior of structure after its elastic limit. In this analysis method, structural elements are pushed to deform beyond their elastic limit and hence it undergoes inelastic behavior. Nonlinear static analysis also known as vertical pushover analysis. Pushover analysis is mainly deformation controlled and force controlled. In non-linear analysis, the elements affected due to column removal are generally located in or nearby bay of column removal location i.e., elements far from initiating damage may not be affected hence may not yield, this can be visualized from hinge formation in the building. In non-linear analysis, results are observed in the form of hinge formation, force- deformation characteristics and collapse load. This method is widely used for detailed investigation. Here analyses of two cases Case A and Case B for only two column removal scenarios are shown i.e., removal of corner column and removal of middle column.

It can be visualized from non-linear static analysis, that the hinge formation pattern is similar for both Envelope loading combination and GSA loading combination using full load mechanism and the pattern shows initialization stage to the failure stage quite clearly as shown in Fig. 7. Thus, the performance of the low-rise structures designed by Indian codes for envelope loading combination is quite excellent to behave as progressive collapse resistant structure and can resist

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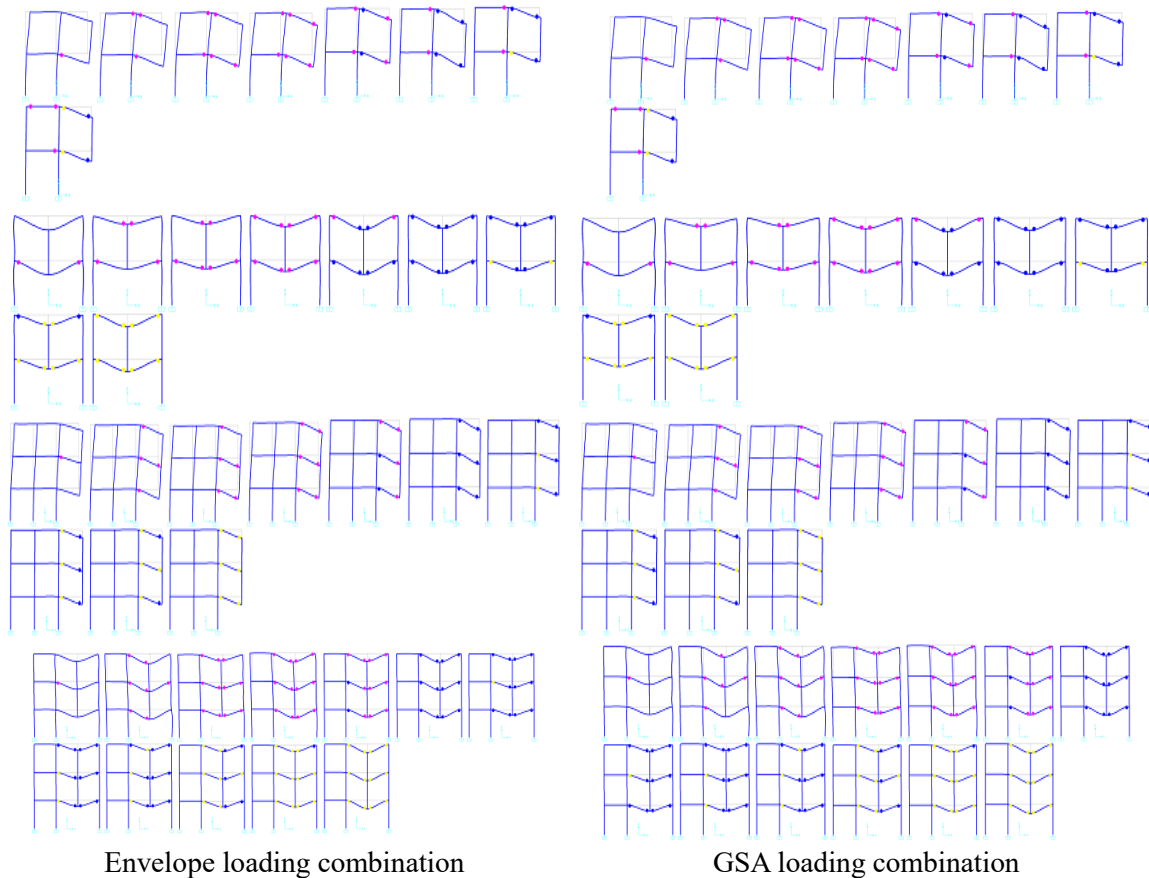</table>

Fig. 8 Hinge formation pattern using displacement-controlled mechanism with target displacement as 2% height of the structure for two different loading combinations

progressive collapse inherently which is very well in agreement with the results of the "Brunesi Emanuele and Parisi Falvio (2017)"

Secondly, Figs. 3, 4 and 5 shows the hinge formation pattern using displacement-controlled mechanism, which varies from initial stage to the failure stage depending on the target displacement considered. When 2% height of the building is taken as target displacement, nonlinear static analysis shows hinge formation pattern from initial stage to ultimate capacity whereas for 4% and 5% height of building as target displacement, hinge formation pattern ranges from initial stage to the total failure stage, therefore a well-defined target displacement formulae is obligatory with respect to GSA guidelines for the study of behavior of seismically designed structure subjected to progressive collapse.

4.2 Displacement of horizontal and vertical joints along x-axis

4.2.1 Displacement of horizontal and vertical Joints adjacent to removed column along x-axis, using full load mechanism only

Fig. 6 shows displacement of horizontal Joints (4, 5 and 6) and vertical Joints (7 and 8) for

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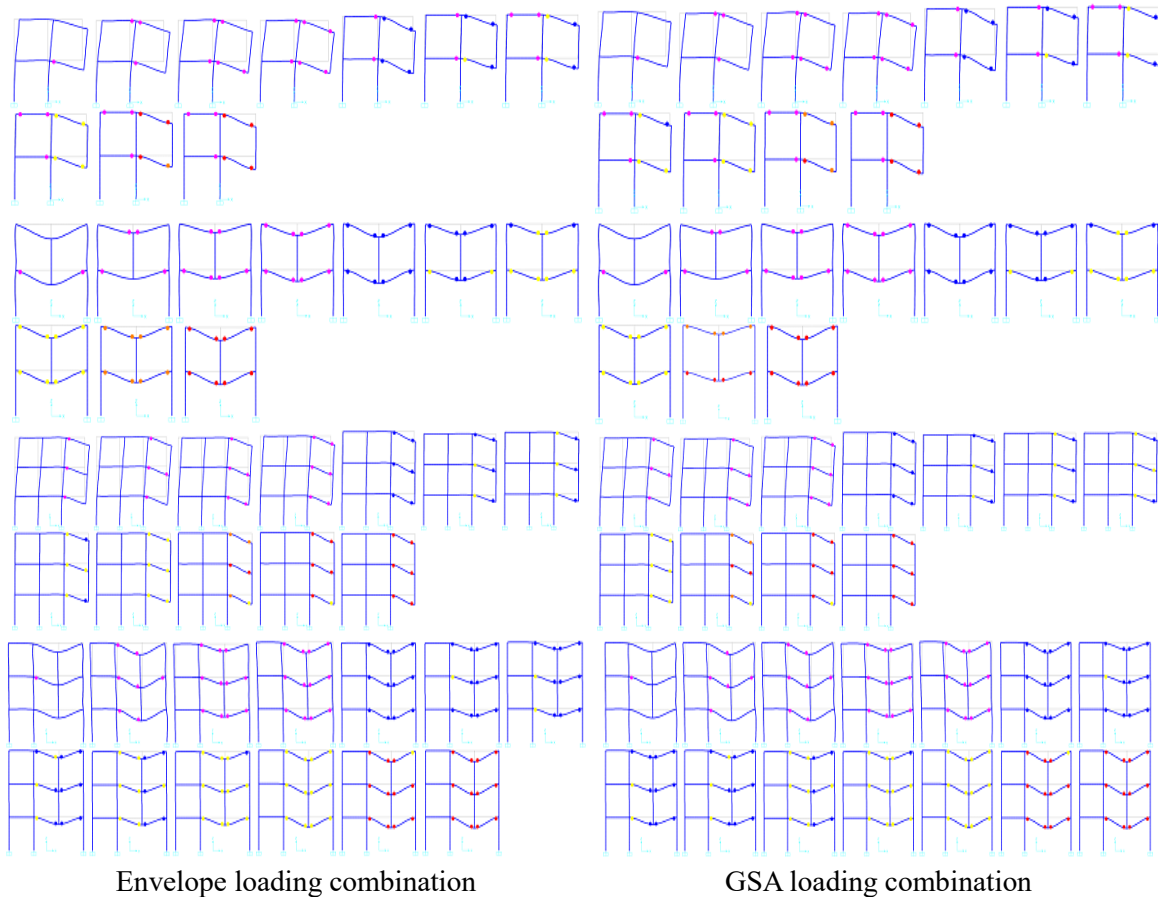<tr><td>Envelope loading combination</td><td>GSA loading combination</td></tr>
</table>

Fig. 9 Hinge formation pattern using displacement-controlled mechanism with target displacement as 5% height of the structure for two different loading combinations

Case A, and horizontal Joints (6, 7 and 8), vertical Joints (10, 11, 14 and 15) for Case C, flow rise reinforced framed structures nearby to removed column locations using full load method. Some of the interesting findings are observed which are very well in agreement with experimental studies, when corner column is removed statically, the horizontal and vertical joints adjacent to removed column location initially tends to displace along positive x-axis and followed by negative x-axis displacement before attaining alternate load path, moreover the vertical joints mostly displace in to and fro motion for some time in order to gain alternate load path. Whereas when middle column is removed, displacement of joints adjacent to removed column location along x-axis for symmetrical building structures (Case A 2-bay2storey) are opposite to each other with same amount of maximum displacement in either direction, Fig. 6 indicates Joint 4 displaces along negative x-axis and Joint 6 displaces along positive x-axis. On the other hand, displacement of joints adjacent to removed column location along x-axis for unsymmetrical building structures (Case C 3bay3storey) is along negative x-axis i.e., towards the building structure as shown in Fig. 6 with maximum displacement as, For symmetrical structure 2bay2storey (Joint 4= -0.000017 m, Joint 6= +0.000017 m and Joint 7= +0.000025 m). For unsymmetrical structure 3bay3storey (Joint
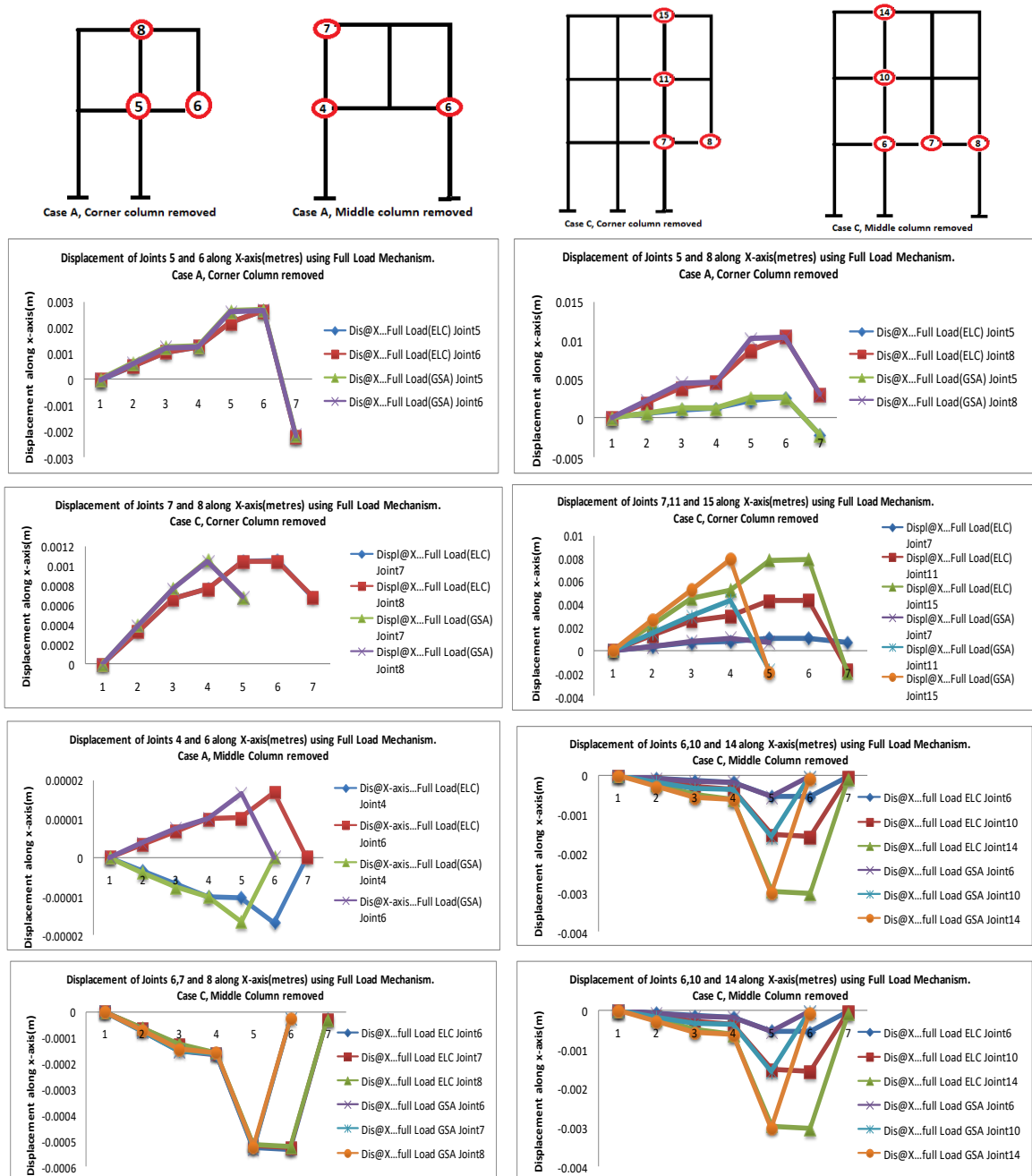
Fig. 10 6 Displacement of horizontal and vertical Joints along x-axis verses number of pushdown steps by using full load mechanism for envelope loading combination (ELC) and GSA loading combination

6=Joint 7=Joint 8= -0.00055m, Joint 10= -0.0015m and Joint 14= -0.003m) respectively.

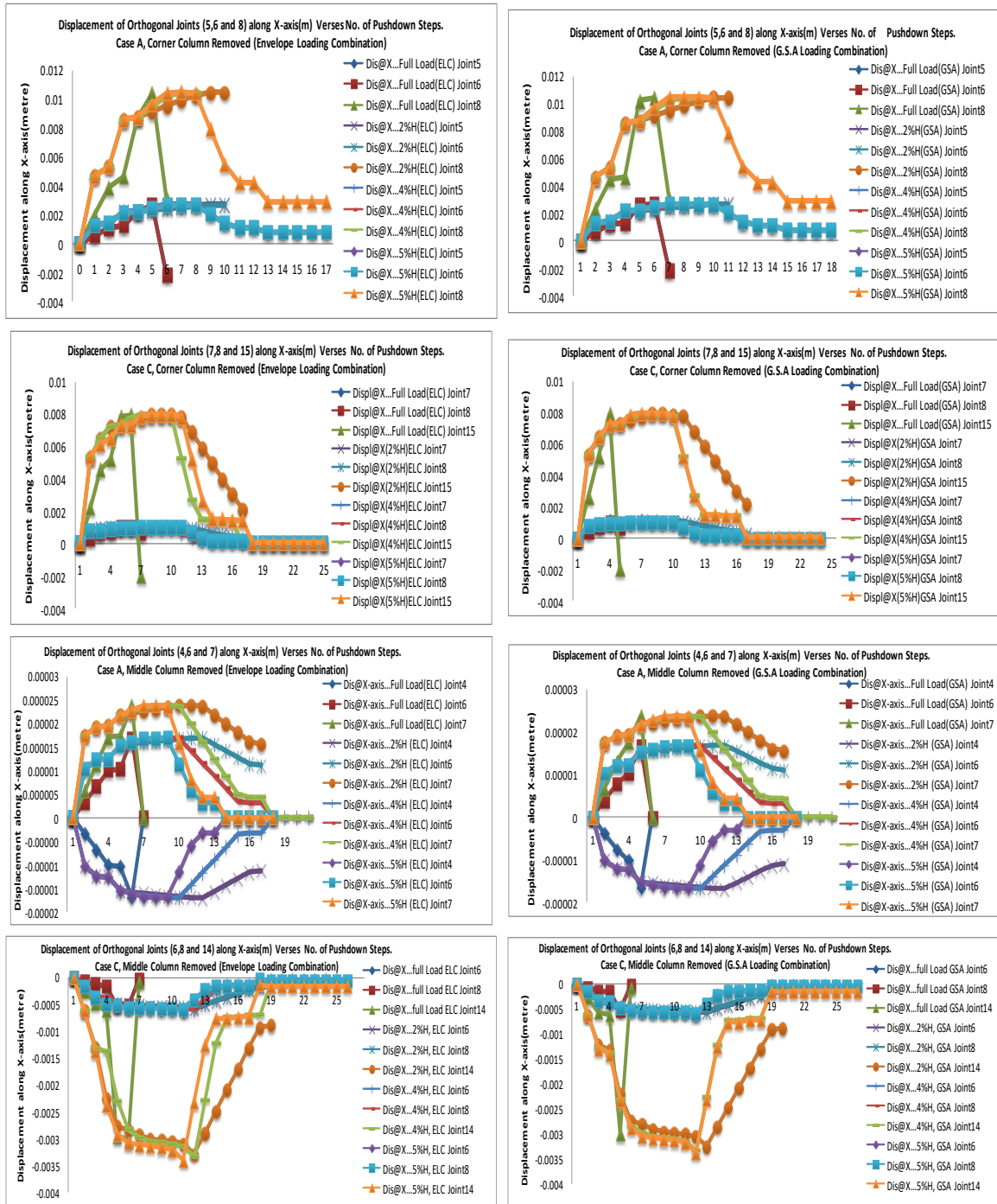Therefore, in case of symmetrical structure when middle column is removed displacement of

Fig. 11 Displacement of horizontal and vertical joints(m) adjacent to column removal location along x-axis verses number of pushdown steps (for different column removal scenario) using full load and displacement-controlled mechanism for Envelope loading combination (ELC) and GSA loading combination (GSA)

horizontal joints before undergoing progressive collapse displaces in "to and fro" motion whereas in case of unsymmetrical structures horizontal joints tend to displace towards the building structure before having complete collapse.

4.2.2 Comparison of full load and displacement-controlled mechanism interms of displacement of horizontal and vertical Joints(m) adjacent to removed column along x- axis

Fig. 10 compares full load and displacement control methods in terms of displacements obtained at particular joints along x-axis with respect to pushdown steps for two type of loading conditions, envelope loading combination and GSA loading combination for different scenarios of column removal locations. It can be seen that full load method generates results in a smaller number of pushdown steps (4-6 steps) compared to displacement control method (7-12 steps), both methods provide almost same results, but the behavior can be well understood through displacement control method provided a well-defined target displacement formula is designed. Almost same results are obtained for GSA loading and envelope loading in terms of displacement of horizontal joints adjacent to removed column. Displacement of top storey along x-axis is 5 to 6 times more than the displacement of bottom storey before undergoing complete collapse.

4.3 Displacement of Joints adjacent to column removed location along z-axis

4.3.1 Displacement of horizontal and vertical Joints (m) adjacent to column removed location along z-axis, using full load mechanism only

Fig. 11 shows displacement of horizontal Joints (4, 5 and 6) and vertical Joints (7 and 8) for Case A, and horizontal Joints (6, 7 and 8), vertical Joints (10, 11, 14 and 15) along z-axis for Case C, of low rise reinforced framed structures nearby to removed column locations using full load method in terms of displacements obtained at particular joints along z-axis with respect to pushdown steps for two type of loading conditions, envelope loading combination and GSA loading combination for different scenarios of column removal locations. It is quite clear that the displacement of horizontal joints adjacent to removed column differs, for symmetrical structures displacement of horizontal joints along z-axis are same in magnitude (displacement at joint 4 = joint 6) whereas for unsymmetrical structures column removed joints displaces more followed by corner column joint and then internal column joint location respectively. For example, displacement at (joint 7 > joint 8 and joint 8 > joint 6). The reason is due to structural continuity from internal column location.

4.3.2 Comparison of full load and displacement-controlled mechanism in terms of displacement of horizontal and vertical Joints(m) adjacent to removed column along z-axis

Fig. 12 compares two methods, full load and displacement control method in terms displacements obtained at particular joints along z-axis with respect to pushdown steps for two type of loading conditions, envelope loading combination and GSA loading combination for different scenarios of column removal locations. It can be seen that full load method generates results in a smaller number of pushdown steps (4-6 steps) compared to displacement control method (7-12 steps), both methods provide almost same results, but the behavior can be well understood through displacement control method provided a well-defined target displacement formula is designed. Displacement of top storey is 1.5 to 2 times more than the bottom story before having complete collapse. Moreover, GSA loading combination give almost same results but few steps earlier than envelope loading combination.
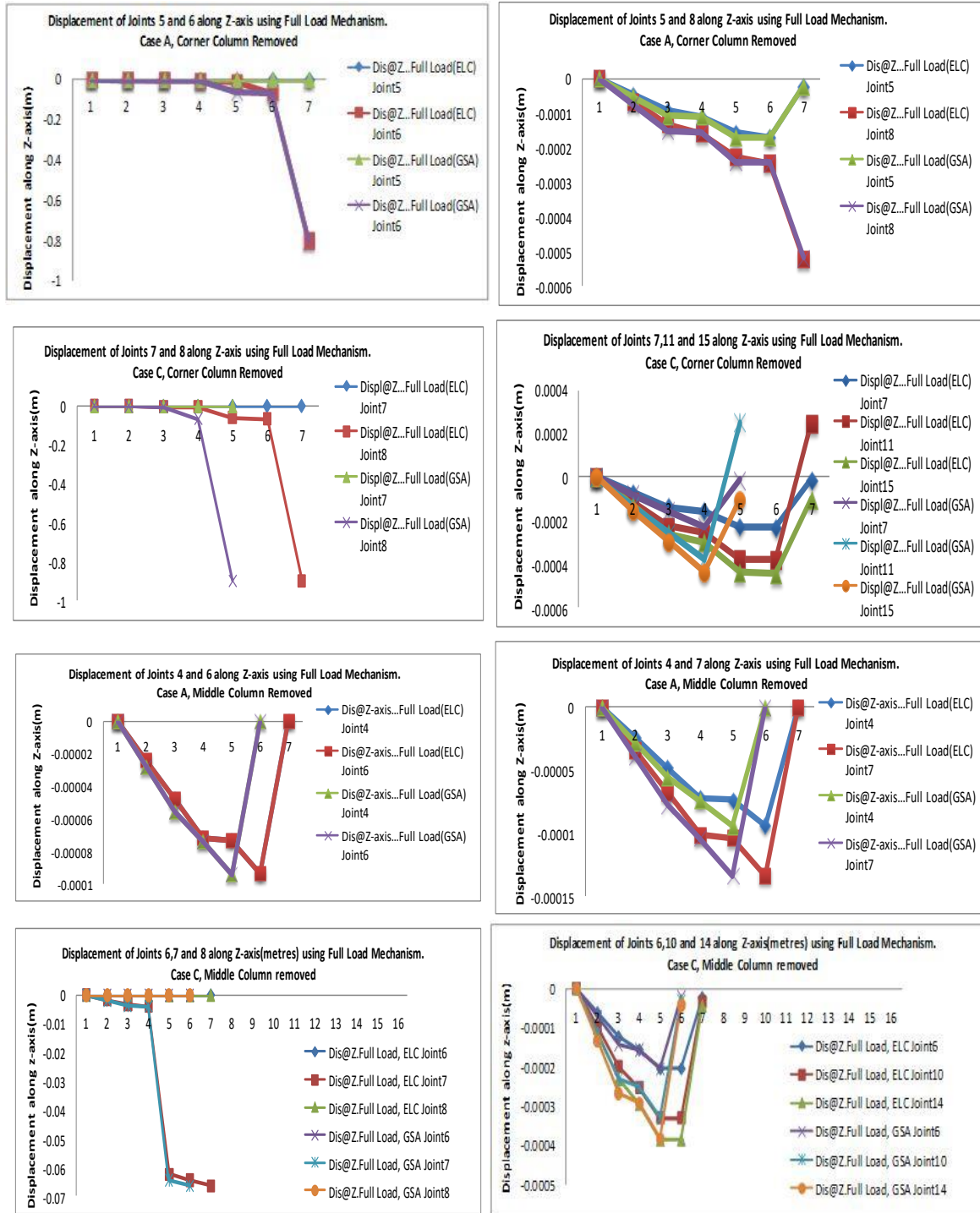
Fig. 12 Displacement of horizontal and vertical Joints(m) adjacent to column removed location along z-axis verses number of pushdown step, using full load mechanism for envelope loading combination (ELC) and GSA loading combination (GSA)
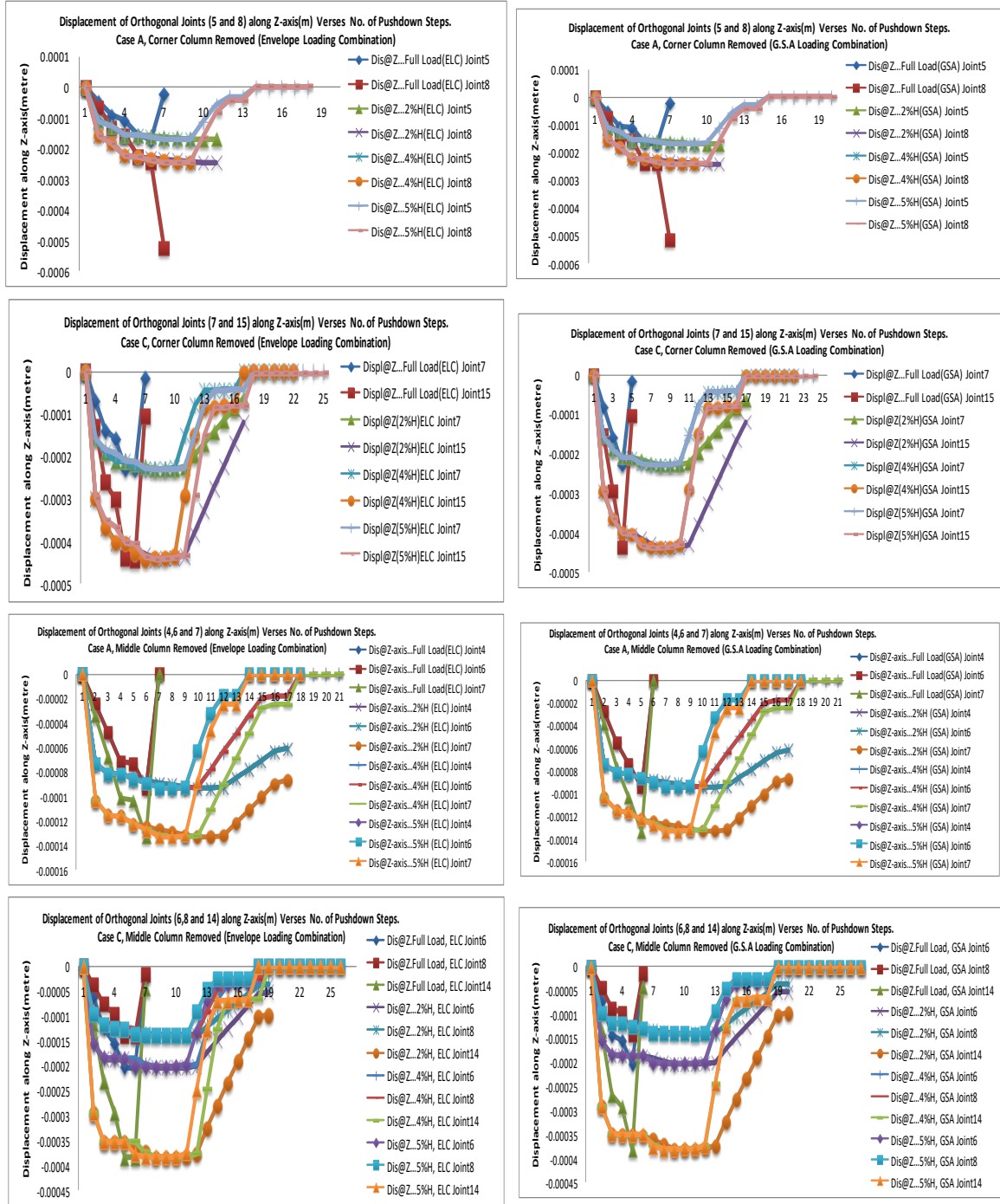
272   *Tariq Ahmad Sheikh, J.M. Banday and Mohammed A. Hussain*

Fig. 13 9 Displacement of horizontal and vertical joints(m) adjacent to column removed location along z-axis verses number of pushdown steps (for different Column removal scenario) using full load and displacement-controlled mechanism for envelope loading combination (ELC) and GSA loading combination (GSA)
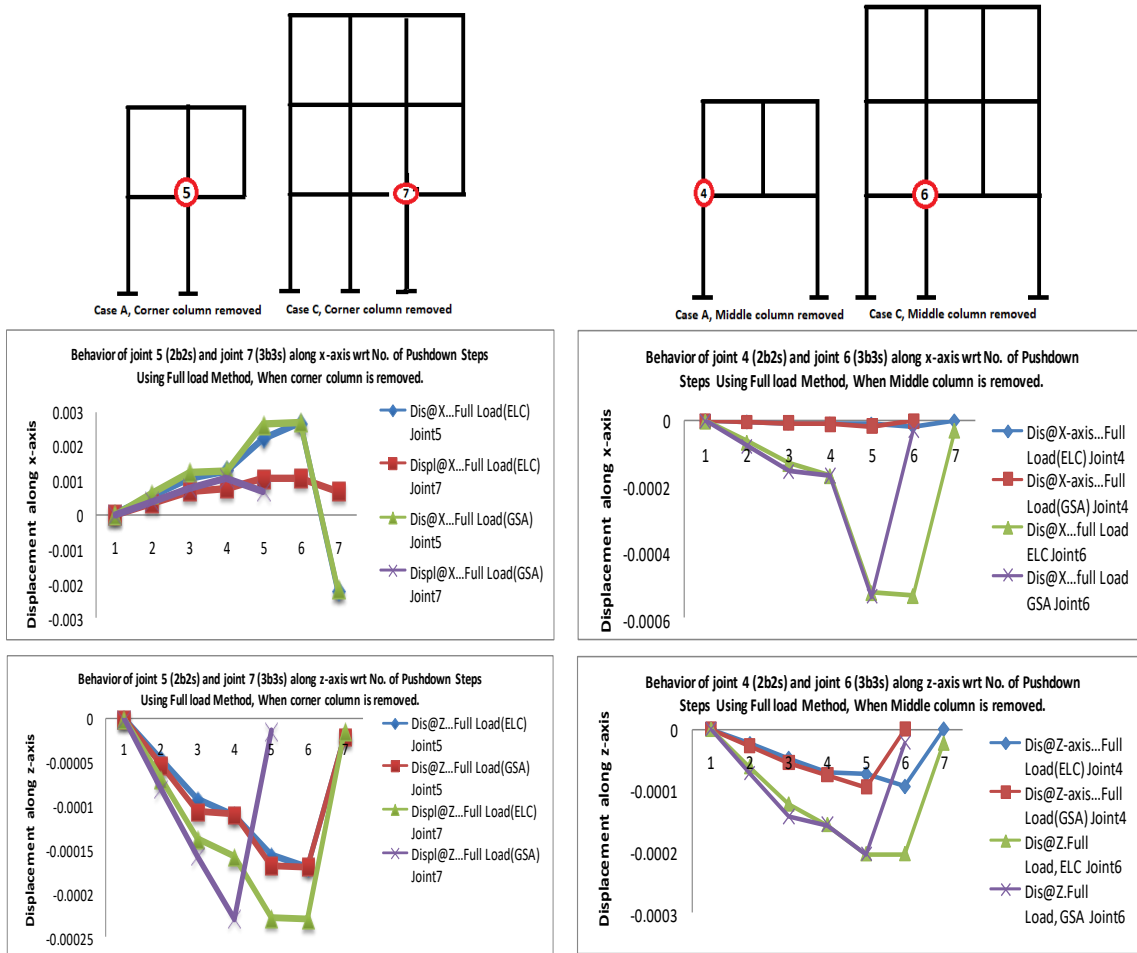
Fig. 14 Shows effect at particular joint adjacent to column removed location along x-axis and z-axis with respect to increase in one bay and one storey for different column removal scenario using full load method

4.4 Comparison of results at particular joint with increase in one bay and one storey using both analysis methods

It can be observed from Figs. 13 and 14, at a particular Joint adjacent to the removed column, the maximum displacement along x axis as well as along z-axis are same irrespective of the method and loading combination used but the only difference lies in the number of pushdown step, required to obtained the given maximum displacement, for example full load method takes 4-6 steps for corner column removal case and 5-7 steps for middle column removal case in order to get maximum displacement along x-axis as well as along z-axis. Whereas displacement-controlled method takes an average of 7-10 steps for corner column removal case and 7-12 steps for middle column removal case to get maximum displacement of a particular joint adjacent to removed column along x-axis as well as along z-axis.
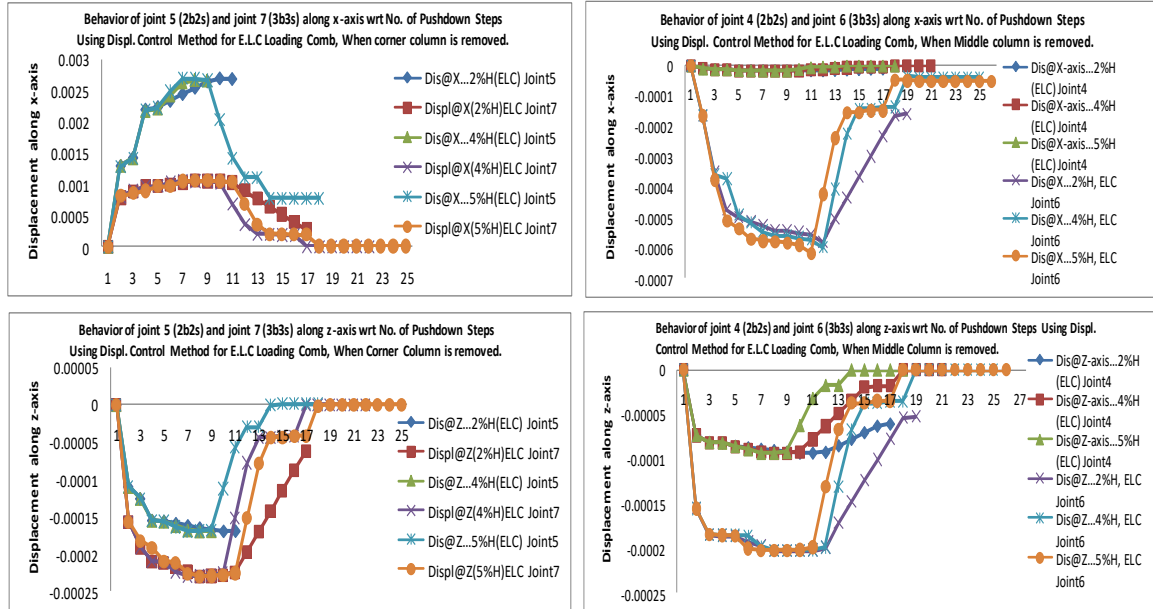
Fig. 15 Shows effect at particular joint adjacent to column removed location along x-axis and z-axis with respect to increase in one bay and one storey for different column removal scenario using displacement Control method

4.5 Pushdown capacity curves

It can be seen from Fig. 12 that when the number of spans and the height of structure is increased consecutively, difference between pushdown capacity curves is going to be almost twice and the models show less sensitivity to element removal. So, the numbers of spans and stories in the structure can have a considerable effect to resist progressive collapse in low rise structures and develop more redundant and robust structure, because the number of participating members and load redistribution paths extremely increase with the more bays and stories numbers. Fig. 12 also indicates similar type of pushdown capacity curve for both type of gravity loading i.e., envelope loading combination and GSA loading combination.

5. Conclusions

5.1 Discussion of the full load method verses displacement control method with respect to envelope loading combination and GSA loading combination by hinge formation pattern

It can be concluded from non-linear static analysis, that the hinge formation pattern is similar for both envelope loading combination and GSA loading combination using full load mechanism and the pattern shows initialization stage to the collapse failure stage quite clearly and directly as shown in Fig. 6. However, the hinge formation pattern obtained using displacement-controlled mechanism varies from initial stage to the failure stage, the intervene stages including Immediate
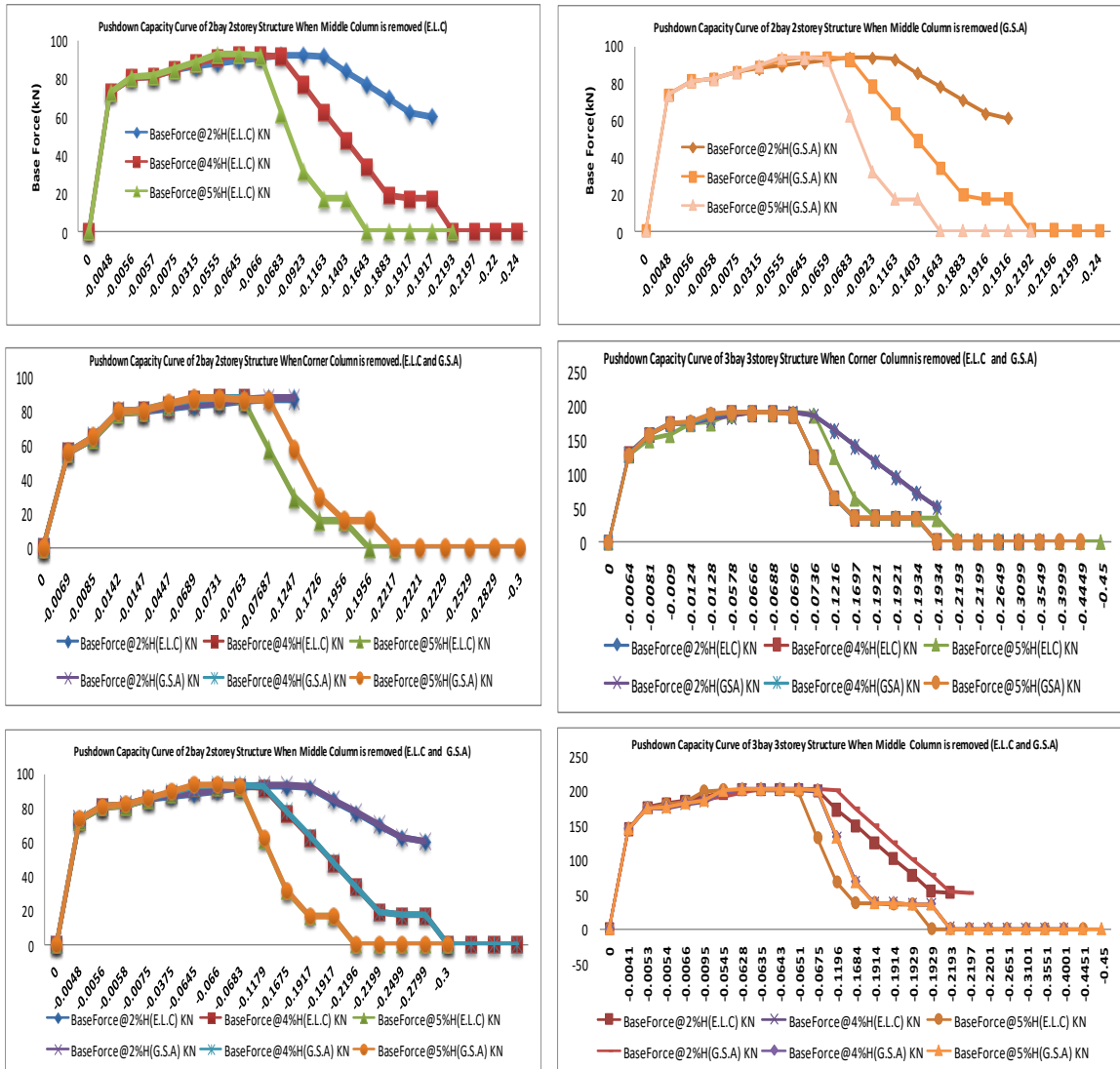
Fig. 16 Shows pushdown capacity curves of 2bay 2storey and 3bay3storey for different column removal scenario using envelope loading combination (ELC) and GSA loading combination (GSA)

occupancy, life safety and collapse prevention are quite clearly visualized depending on the target displacement considered. When 2% height of the building is taken as target displacement, nonlinear static analysis shows hinge formation pattern from initial stage to ultimate capacity whereas for 4% and 5% height of building as target displacement, hinge formation pattern ranges from initial stage to the total failure stage, as seen from Figs. 7 and 8, since there is not any particular formulae for setting target displacement, therefore a well-defined target displacement formula is obligatory with respect to GSA guidelines for the study of behavior of seismically designed structure subjected to progressive collapse.

5.2 Discussion of the full load method verses displacement control method with respect to envelope loading combination and GSA loading combination

Almost similar final results are obtained for envelope loading combination and GSA loading combination using full load method as seen from Fig. 5 and displacement control method as seen from Figs. 6 to Fig. 8, but the number of pushdown steps varies, envelope loading combination takes few more steps to give same results compared to GSA loading combination as can be seen from displacement v/s number of pushdown steps graphs and pushdown capacity curves.

5.3 Discussion of the full load method verses displacement control method with respect to envelope loading combination and GSA loading combination by knowing displacement of joints along x-axis and z-axis

When middle column is removed, displacement of joints adjacent to removed column location along x-axis for symmetrical building structure (2bay2storey) are opposite to each other i.e., in "to and fro" motion with same amount of maximum displacement before having complete collapse, Fig. 9 indicates Joint 4 displaces along negative x-axis and Joint 6 displaces along positive x-axis. On the other hand, displacement of adjacent joints along x-axis for unsymmetrical building structure (3bay3storey) is along negative x-axis i.e., towards the building structure as shown in Fig. 6 with maximum displacement as, For 2bay2storey (Joint 4= - 0.000017 m, Joint 6= +0.000017 m, and Joint 7= +0.000025 m). For 3bay3storey (Joint 6=Joint 7=Joint 8= -0.00055 m, Joint 10= -0.0015 m and Joint 14= -0.003 m).

When corner column is removed, maximum displacement of horizontal joints along x-axis is same irrespective of the method used. For example, Joint 5 and Joint 6 (2bay2storey) and Joint 7 and Joint 8 (3bay3storey) have maximum displacement along x-axis as 0.0026m and 0.001 m respectively. The only difference is the number of pushdown steps; full load method gives same maximum displacement in few steps compared to displacement control method which generates the same result in a greater number of steps, depending on the target displacement.

Displacement along x-axis of top storey of a given building structure is almost an average of 5 to 6 times more than the displacement of bottom storey whereas displacement along z-axis of top storey is an average of 1.5 to 2 times more than the bottom storey.

5.4 Discussion of the full load method verses displacement control method with respect to envelope loading combination and GSA loading combination at a particular joint adjacent to removed column

Figs. 13 and 14 indicates, at a particular Joint adjacent to the removed column, the maximum displacement along x axis (Joint 5= +0.003 m and Joint 7= +0.001 m) as well as along z-axis (Joint 5= -0.00017 m and Joint 7= -0.00023 m) are same irrespective of the analysis method and loading combination used but the only difference lies in the number of pushdown step, to obtain the given maximum displacement, for example full load method takes 4-6 steps for corner column removal case and 5-7 steps for middle column removal case in order to get maximum displacement along x-axis as well as along z-axis. Whereas displacement-controlled method takes an average of 7-10 steps for corner column removal case and 7-12 steps for middle column removal case to *get al*most similar maximum displacement at a particular joint adjacent to removed column along x-axis as well as along z-axis.

5.5 Discussion of the full load method verses displacement control method with respect to envelope loading combination and GSA loading combination by pushdown capacity curves

Fig. 15 indicates, with increase in number of spans and storey height consecutively pushdown capacity curves for envelope loading combination and GSA loading combination are similar, the base force or base shear increases approximately by 2-3 times irrespective of the column removal scenario whereas the corresponding displacement in downward direction reduces by 59% and 62.4% for corner column removal and middle column removal case respectively. Hence when number of bays and storey's are increased, larger capacity to resist progressive collapse under envelope loading combination and GSA loading are obtained because additional elements participated to resist progressive collapse.

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