PREVENTION OF BUILDING FAILURE AGAINST PROGRESSIVE COLLAPSING

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ABSTACT

According to the modern scenario, most of the precast structures follow progressive collapse phenomenon. Even though buildings are designed to resist all expected loads without failure, they fail occasionally due to inadequate design and construction techniques, especially for extreme and abnormal loads. The paper deals with the analysis and design of 3, 5, 7 and 12 (with 10 different cases) precast storey buildings in SAP-2000 by performing Non-linear Dynamic Alternative Path Method. The variation of deformations of the beams and axial deformation of columns over the height of the structure had evaluated and its effects on the load redistribution had discussed, identified and characterized. It is identified that following the loss of a column, premature beam bottom bar fracture can occur and also shown that with the increase of 47.86% and 63.26% of stiffness building can resist progressive collapse for 10 and more than 10 storey respectively. It is also concluded that removal of column from all four corners and all column in the middle panel of precast building leads to total collapse. The research has immense benefit in structural designing of precast buildings against progressive collapse failure.

Keywords: Progressive collapsing, Axial Deformation, Non-Liner Dynamic Alternative Path Analysis (NLD AP), SAP-2000, Stiffness, Removal of Column

INTRODUCTION

Civil structures are designed to resist all expected loadings without any failure. However, precast structural member failures do occasionally occur due to inadequate design and construction, especially for extreme and abnormal loads. This paper concerns the progressive collapse failure of structures due to abnormal loading events like removal of column at different joints and using Non-Linear Dynamic Alternative Path advanced analysis for predicting the progressive collapse behaviour of building structures in the plastic limit state. From some time now, building structures have been designed to resist normal loads such as those due to self-weight, occupancy and climatic or seismic effects. However, after the 1968 chain-reaction failure of the Ronan Point Apartment Block in London, UK, triggered by a gas explosion, abnormal loading and progressive collapse have become increasingly recognized as important phenomena to be accounted for in engineering design practice worldwide. Indeed, the complete structural collapse of the twin towers of the World Trade Centre (WTC) in New York City on September 11, 2001, has significantly increased the concern for these phenomena. Motivated by such abnormal loading events, this research addresses the topic of prevention of building against progressive loading.

Progressive collapse is defined as the spread of an initial failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it [ASCE 2002]. When local failure of primary structural members propagate to failure of adjoining members, progressive collapse will ensue unless adjoining structural members arrest further progression of failure. Progressive collapse is a dynamic and nonlinear event, as it takes place in a very short time frame and structural members undergo nonlinear deformation before failure. To analyze rigorously progressive collapse potential of a structure, nonlinear dynamic analysis should be performed to account for energy dissipation, large inelastic deformations, materials yielding, cracking and fracture. However, the nonlinear dynamic analysis requires step-by-step integration which is very time consuming.

Marjanishvili and Agnew, 2006 concluded that Nonlinear dynamic analysis is the most detailed and intricate analysis possible. It covers geometric nonlinearities, including second order effects such as P-delta. It allows material models to be specified which can define properties such as yielding, strain hardening, and strain rate effects. Mehrdad Sasani and Serkan Sagiroglu in his paper named “Gravity Load Redistribution and Progressive Collapse Resistance of 20-Story Reinforced Concrete Structure following Loss of Interior Column” is evaluated G+20 RCC structure and proposed that the structure resisted progressive collapse with a measured maximum vertical displacement of only 9.7 mm and increase in the number of story would not lead to progressive collapsing instead it helps in redistribution of gravity loads. Mehrdad Sasani and Serkan Sagiroglu in their paper “Progressive Collapse of Reinforced Concrete Structures: A Multihazard Perspective” concluded that with the smaller reduction of strength in the critical member leads to large vertical displacement. Breen and Ellingwood and Leyendecker at “Research Workshop on Progressive Collapse of Building” have made a distinction between direct and indirect design. Indirect design incorporates implicit consideration of resistance to progressive collapse through the provisions of minimum levels of strength, continuity, and ductility. Direct design incorporates explicit consideration of resistance to progressive collapse through two methods.

The US Department of Defence (DoD) released the newly revised version of the UFC 4-023-03 named Design of Buildings to Resist Progressive Collapse. The new UFC incorporates both direct and indirect design procedures. The main direct design procedure is the Alternate Path (AP) method, in which a structure is analyzed for collapse potential after the removal of a load bearing vertical components at different joints. Different analytical procedures may be used for AP, including Linear Static (LS), Nonlinear Static (NLS), and Nonlinear Dynamic (NLD). Significant changes have been made to AP methods for analysis in the new criteria which result in less conservative and more efficient structural components. Typically, when doing AP analyses, designers often choose static procedures which tend to be simpler and required less labour. However, progressive collapse is a dynamic and nonlinear event, and the load cases used for the static procedures that do not account for inertial and nonlinear effects and hence static methods tend to add conservatism to final design. The procedure changes in that paper, from NLS to NLD, offer analysis procedures of increasing complexity and time investment, but offer a significant increase in design efficiency. This paper presents a detailed example of how to properly perform an NLD AP analysis on a building following the guidelines in the new UFC-4023-03 and the advantages in the final design when compared to the results of an LS analysis.

PROCEDURE EMPLOYED

In the non-linear dynamic procedure, the un-factored load case (extreme load event case) is directly applied to a materially and geometrically nonlinear model of the structure as shown in fig. 1. In the first phase of the static and dynamic analysis, the structure is allowed to reach equilibrium under the applied load case. In the second phase, the column or wall section is removed almost instantaneously and the software tool calculates the resulting motion of the structure. The resulting maximum member deformations are then compared to the deformation in the unaffected structure or as per the guidelines. If the deformation limits are exceeded at any hinge locations, the deficient structural components are re-designed and the analysis is re-run until no deformation limits are exceeded at the hinge locations. The application of this procedure is followed step by step as

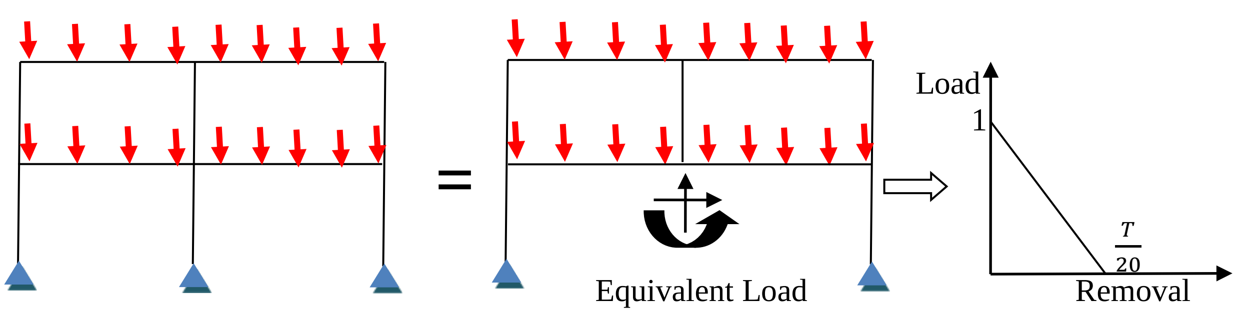

Fig. 1 Instantaneous Column Removal

a) Development of computer model
b) Load and Masses application on the model
c) Development of Non-Liner hinges by Push Over Analysis
d) Non-Linear Dynamic Analysis
e) Instantaneous removal of column

MODELLING OF BUILDINGS IN SAP-2000

The building analysed in the research is 3 to 12 storey reinforced concrete building. The description is given in table 1 and the grade of steel and concrete used in the research were estimated using the standards recommended in the IS code for Ductile Detailing of Reinforced Concrete Structure subjected to Seismic Forces (IS 13920-1993) as described in table 2

Table 1 showing building's elements description

Sr. No.	Building	Beam Size (mm)	Slab Thickness (mm)	Column Size (mm)
1	G+3	350 × 230	150	350 × 230
2	G+5	350 × 350	150	400 × 400
3	G+7	400 × 400	200	450 × 450
4	G+12	750 × 350	200	800 × 800

Table 2 Grade of Material Used

S. No.	Building Element	Grade of Concrete or Steel
1	RC beam and slab	Design Compressive Strength 30 MPa (M30)
2	RC column	Design Compressive Strength 35 MPa (M35)
3	Steel	Yield Strength 500 MPa (Fe 500)

The below figure 2 shows the G+3, G+5, G+7 and G+12 building model in SAP-2000

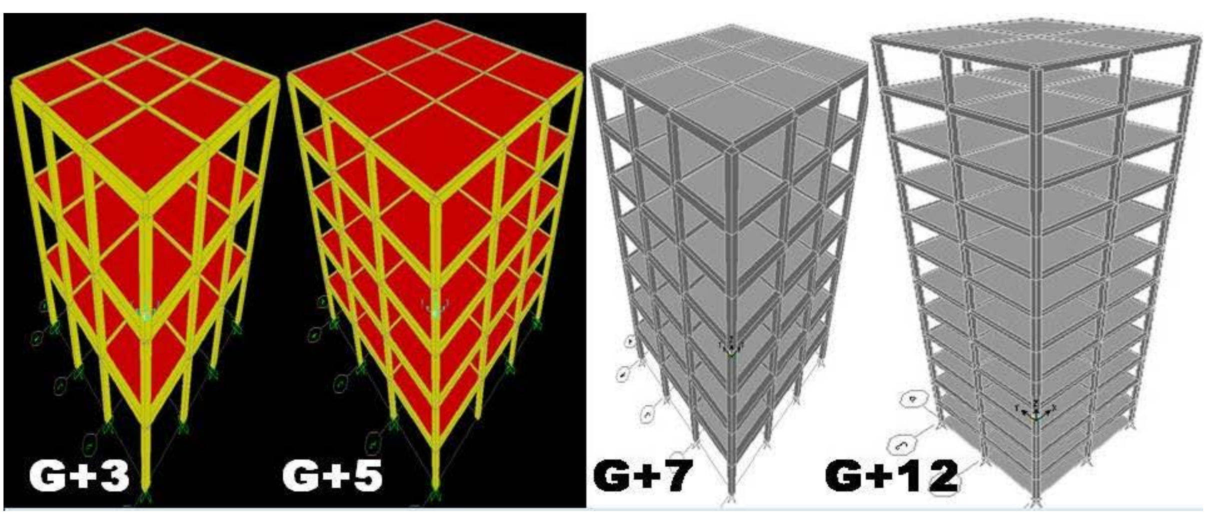

Fig. 2 SAP-2000 model of G+3, G+5, G+7 and G+12 buildings

APPIED LOADS

The analysis was performed using the extreme load combination of 1.2DL (Dead Load) + 0.5LL (Live Load) as specified in UFC 4-023-03 shown in fig 3. The loads used were as follows:
DL on the Floors = 5 KN/m^2
LL on the Floors = 2KN/m^2
Wall Load = 9 KN/m for outer wall and 4.5 KN/m for inner wall

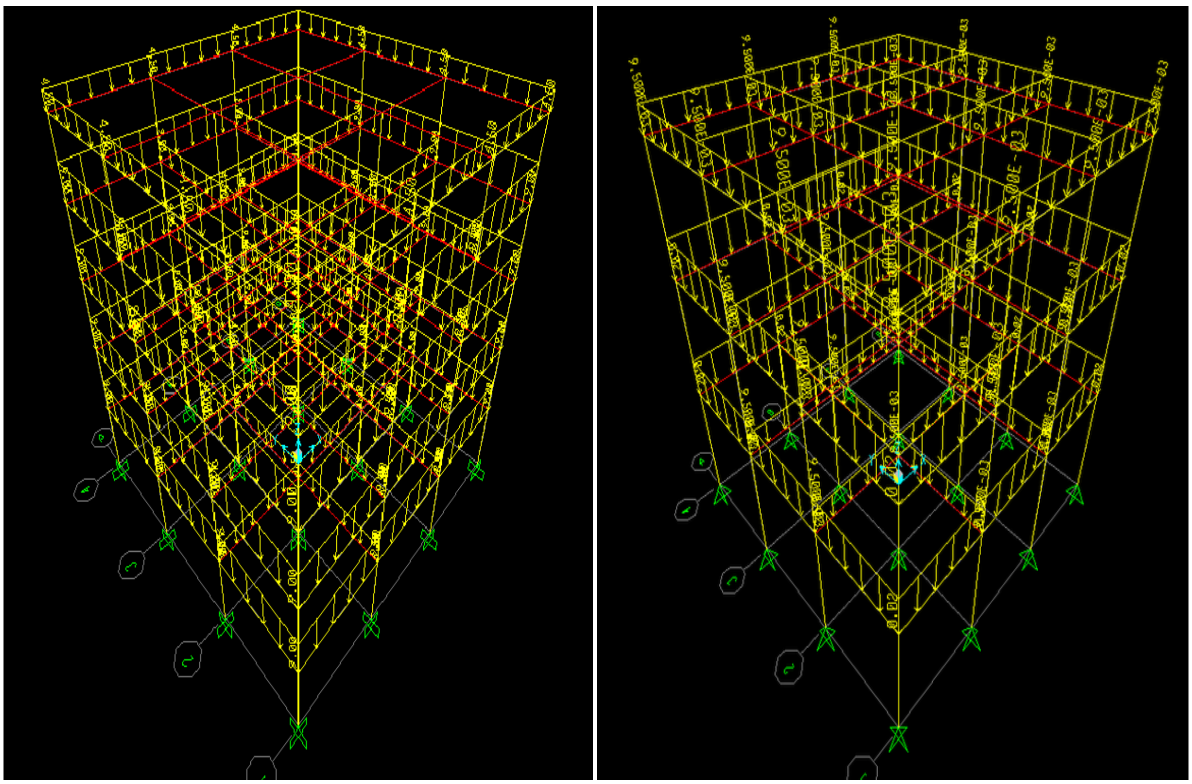

Fig. 3 Loading of the G+3 and G+5 Building

DEVELOPMENT OF HINGES

Non-linearity was incorporated into the analysis by using non-linear flexural hinges placed in the beam components. These hinges are placed at the ends of all beam elements, since these are the expected “high flexural stress” regions in the beam and each of the hinge locations shown in the figure 4.
It is important to point out that the hinge force-deformation curves allow strain hardening of 5% at the point expected to be the maximum allowed rotation. This is less than the 10% maximum hardening allowed in ASCE 41. The reason for this difference is the larger allowable rotations used in progressive collapse analyses. The yield moment capacities in the curves calculated by the model based on the reinforcing steel and material properties assigned to each component.

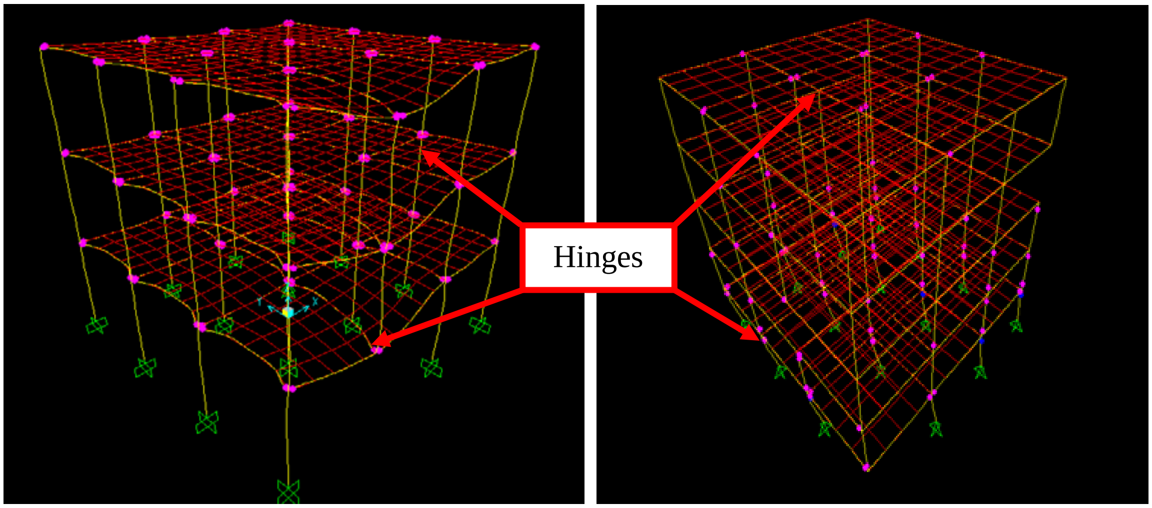

Fig. 4 showing the development of hinges on models

NON LINEAR DYNAMIC PARAMETER ANALYSIS

The dynamic analysis was performed using the "Nonlinear Direct Integration Time History" option in SAP-2000. The time integration method used was the Newmark method of integration and the Gamma and Beta parameters were used correspondingly as the default options in SAP2000 which generally provide good results and convergence times. Other analysis parameters included the damping ratio, time step, and column removal time. For this analysis, these parameters were taken as follows.

- ✓ Damping ratio = 0.2%, 0.4%, 0.8% and 1%
- ✓ Column removal and time step = 1/20 of the structure's natural period
- ✓ Analysis Time Step = 1/200, 1/50 and 1/25 of the structure's natural period

The natural period of response was calculated to be 0.35 seconds. This was determined by performing a Modal Analysis, and selecting the Natural Period (T) of the dominating mode of vibration. The dominating mode of vibration was selected visually based on the location of the column removal and the motion of the structure.

INSTANTANEOUS COLOUMN REMOVAL

The final step to complete a NLD AP analysis is the instantaneous removal of the column or load bearing member. In SAP2000, the model does not permit the instantaneous removal of a structural component while performing a time history analysis. Therefore, in order to simulate the removal of the column, a series of steps are followed in which the column to be removed by equivalent superimposed forces and then removed over with time using a linear function. A more detailed explanation of each of these steps follows.

I. A linear static analysis was performed using the un-factored extreme load case to calculate the forces at the joints of the column to be removed. Then, the column was removed from the model and the calculated reactions were applied at the column joint.
II. The columns had be substituted with equivalent reaction forces, a new linear static analysis was performed, and the resulting flexural moments diagrams and deflections

were compared to the results attained from the initial linear static analysis that included the column. After that, the column was successfully replaced by equivalent superimposed reaction forces shown in fig 5.

III. Using the model with the removed column and the equivalent reactions forces applied at the column joint, a dynamic non-linear analysis case was set up using the time history option in SAP-2000. To simulate the instantaneous removal of the column, the equivalent reaction loads were removed over time using a ramping function for this analysis. The removal of the load was performed over a small period of time equal to 1/20 of the natural period of the structure.

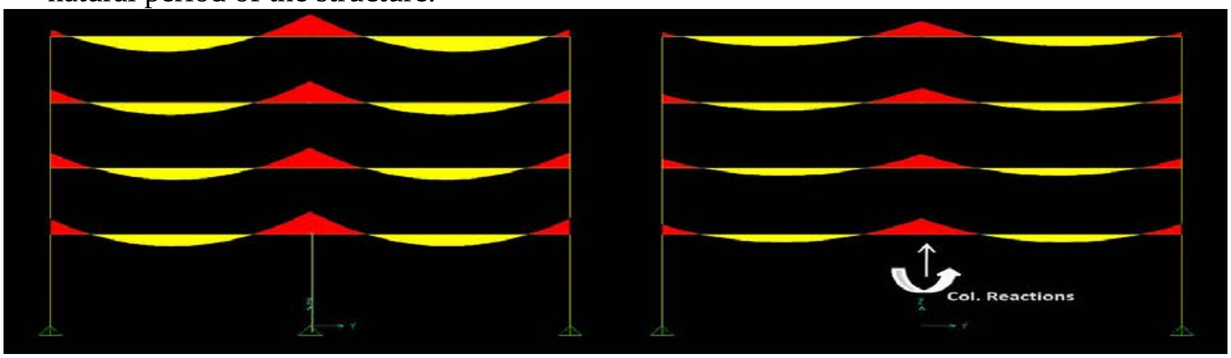

Fig. 5 superimposed reactions after removal of column

NLD AP ANALYSES OF BUILDINGS IN SAP-2000

The buildings were analysed by the removal of column at different portion and corresponding to that change in moment, axial force and shear force and deflection was calculated. The hinges so developed was analysed and redesigned the defected element according to UFC-4023-03. The analyzing of a building is shown as follow

ANALYSIS OF G+3

At the removal of intermediate column of the base the change in deflection shown in fig. 6 and the change in moment and axial force is shown in fig. 7

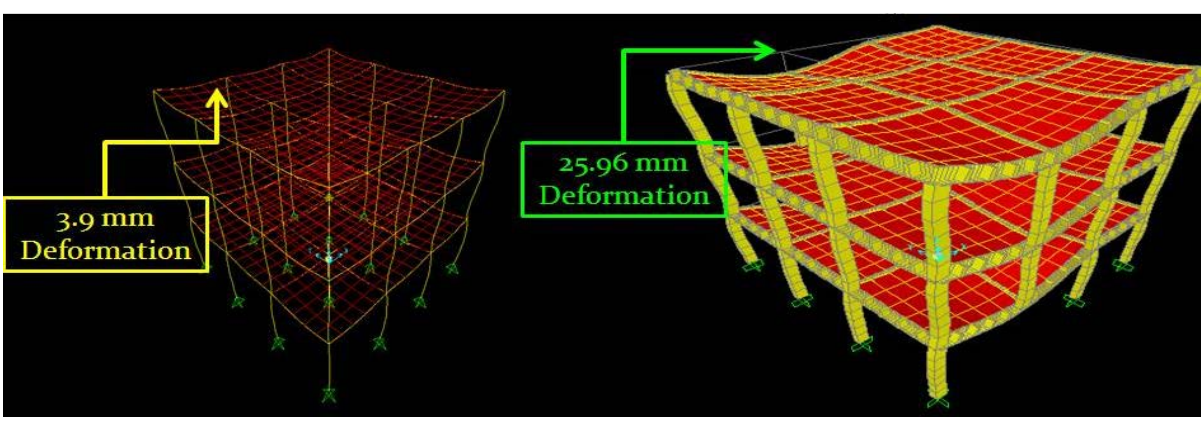

Fig. 6 change in deflection due to column removal

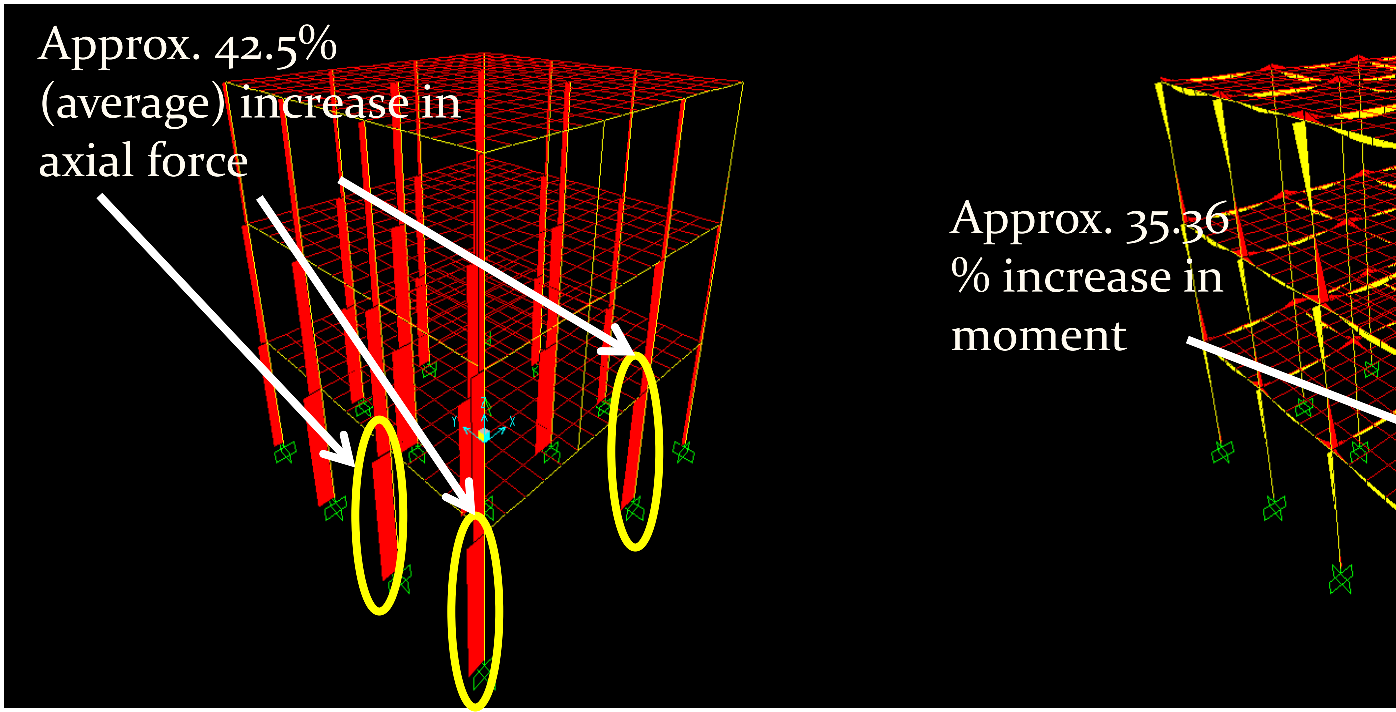

Fig. 7 change in the axial force and moment due to removal of column

From analysis of the progressive collapse failure of G+3 building, it was identified that with the removal of load bearing column the approximate change in the moment is 42.5% and the moment is 35.56% for the symmetric structure. The most abnormal case of the building case was analysed by removal of columns as shown in fig 8 and the respective axial force diagram is made and instantaneous removal of column is done for the same building and the corresponding time displacement curve and time acceleration curve was determined.

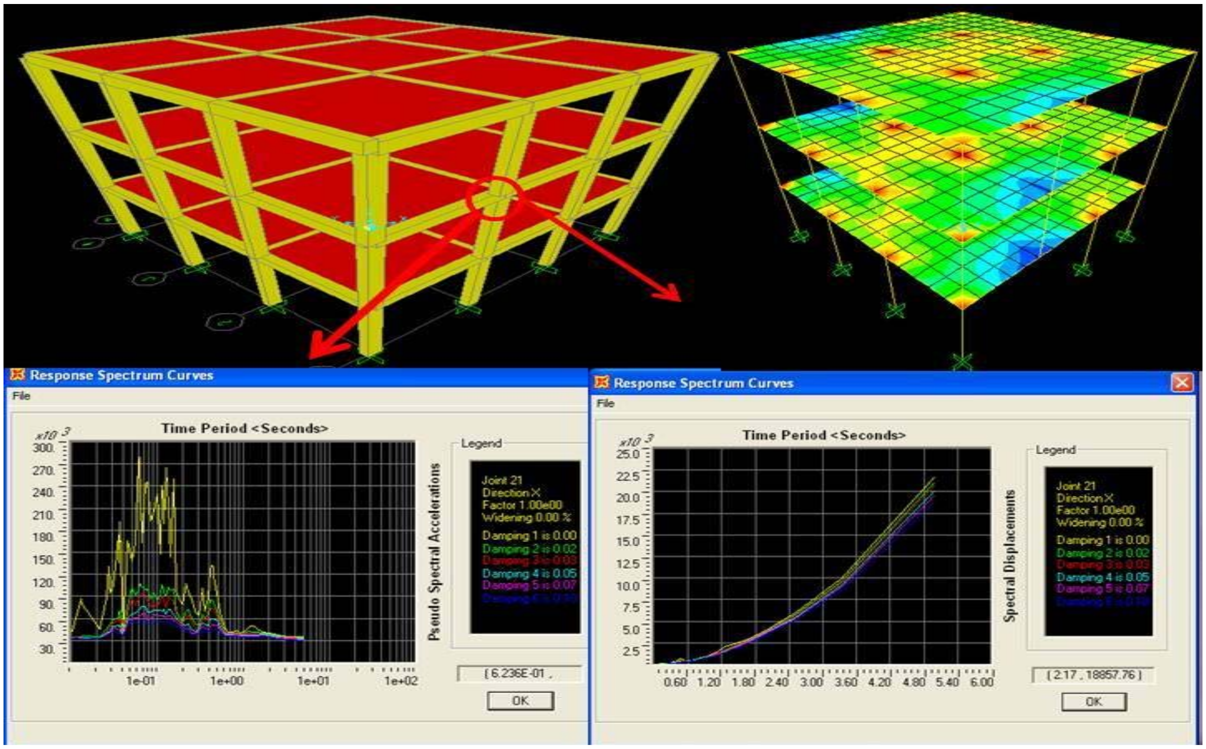

Fig. 8 showing time history analysis of building

ANALYSIS OF G+5

The instantaneous removal of column in G+5 building was analysed in SAP-2000 with both Push-over analysis and the Time History analysis and the results are shown in the fig. 9

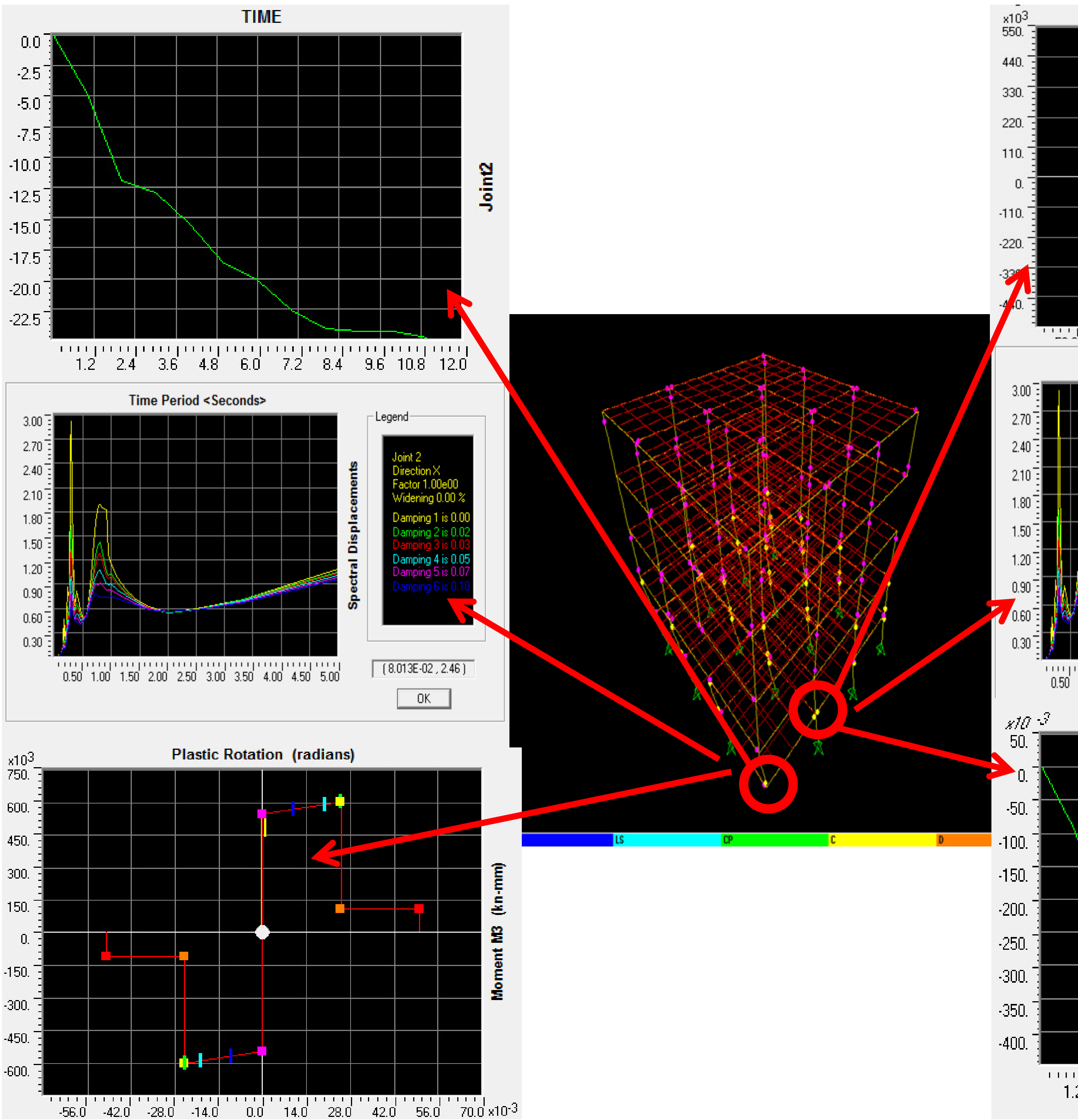

Fig. 9 NLD AP Analysis of G+5 Building

From the above figure it was concluded that if design of a building was based on the normal IS-456, it would not sustain the progressive collapsing of a building. The plastic rotation varied from 400 to 750 KN-m where as spectral displacement changed upto 25 mm when there is no damping and 13 mm when there was 0.1 damping. As far as moment and axial force was concerned it increased by 36.35% and 22.12% respectively with the removal of column as shown in fig. 9. Whereas displacement was concerned it was changed by 26.36% as compared to original structure.

ANALYSIS OF G+7 BUILDING

The NLD AP analysis of a G+7 building was performed with the random removal of column and it concluded that there was an increase of 39.63% in axial force and 35.89 % of shear force and 38.63% increase in moment of a building with the random removal of column as shown in figure 10.

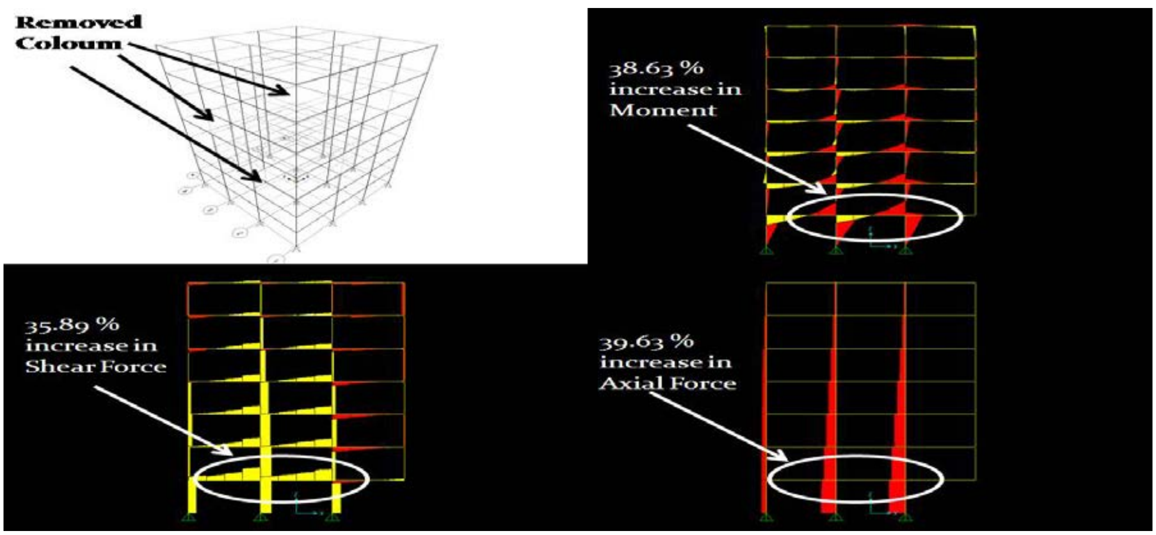

Fig. 10 Increase in Axial, Shear force and Bending Moment after column removal

The time history analysis of the building was also done on the same structure and various curves at different damping these were displacement of point with time and spectral displacement with the same removal of column as shown in fig. 11.

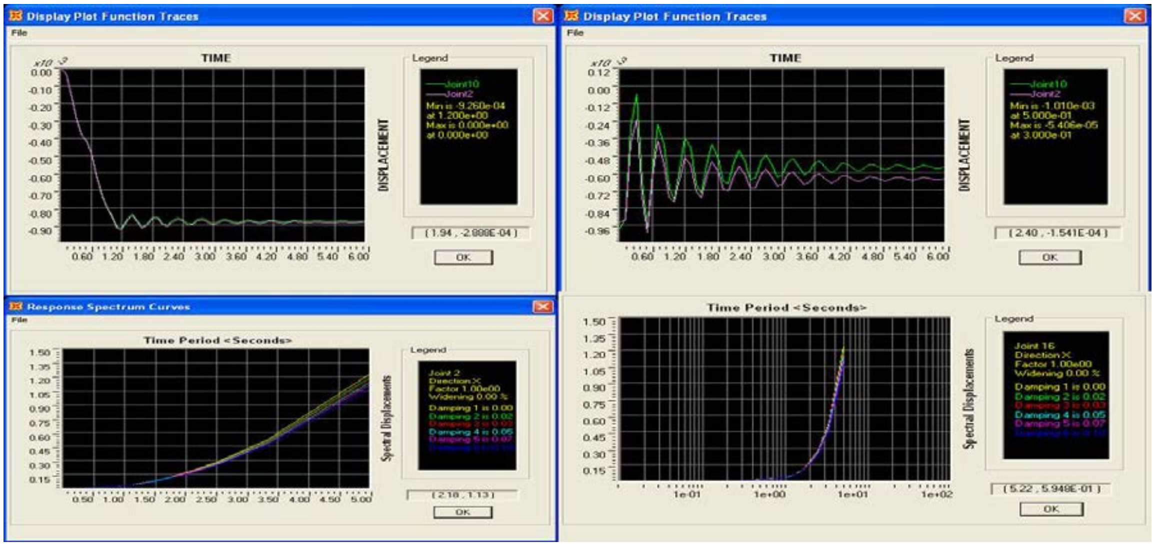

Fig. 11 Time History analyses curves

ANALYSIS OF G+12 BUILDING

The NLD AP was performed on the precast G+12 building in SAP-2000 under the given loading condition of wall load 9 KN/m^2 for two brick wall and 4.5 KN/m^2 for one brick wall that was the partition wall and the parapet wall. The beam size of 0.75 by 0.35 meter and the column size of 0.80 by 0.80 meter with M30 grade Light weight concrete and M30 grade concrete for beam and column respectively were taken for the analyses. The thickness of slab was taken as 200 mm for Non-Linear analysis for the precast building. The slab load of 5 KN/m^2 as Dead Load (DL) and 2 KN/m^2 as Live Load (LL) was used, according to IS 456 and IS 800 for wind loading. Different cases had also analysed with the removal of column at different positions and the description is given in table 3.

Table 3 Description of column removed

S. No.	Case Number	Case Description
1.	Case 1	Column was removed from the middle of the side of precast building at ground level
2.	Case 2	Column was removed from the middle of the precast building at ground level
3.	Case 3	All columns were removed from the middle of the precast building till 12th floor
4.	Case 4	Column was removed from the side of the precast building at ground level
5.	Case 5	All columns were removed from the side of the precast building till 12th floor
6.	Case 6	All columns were removed from the interior of the precast building till 12th floor
7.	Case 7	Columns were removed from all the four corners of

		the precast building at ground level
8.	Case 8	All the columns (both along length and height wise) were removed from the exterior most panel of the precast building till 12th floor
9.	Case 9	All the columns (both along length and height wise) were removed from the middle panel of the precast building till 12th floor
10.	Case 10	Displacement of 20mm in all the exterior most columns of panel 1 at ground level

The Push over analyses and Time history analyses were done in all the cases. Demand-capacity curves, Response spectrum curve, Time history curves and hinges result were plotted and compared with the structure did not having progressive collapse failure. The axial force increment and increase in displacement of the joints was also compared among all the cases as shown in fig. 12 (a, b, c & d).

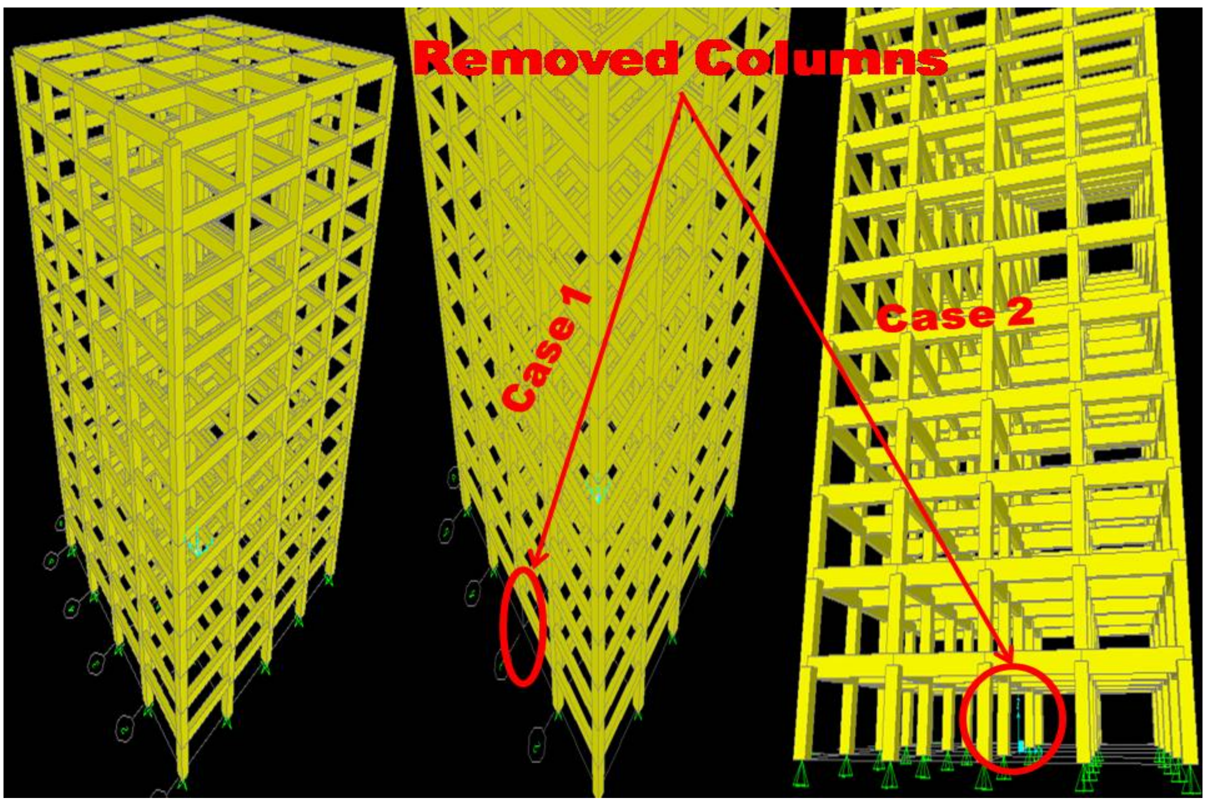

Fig. 12 (a) showing case 1 and 2

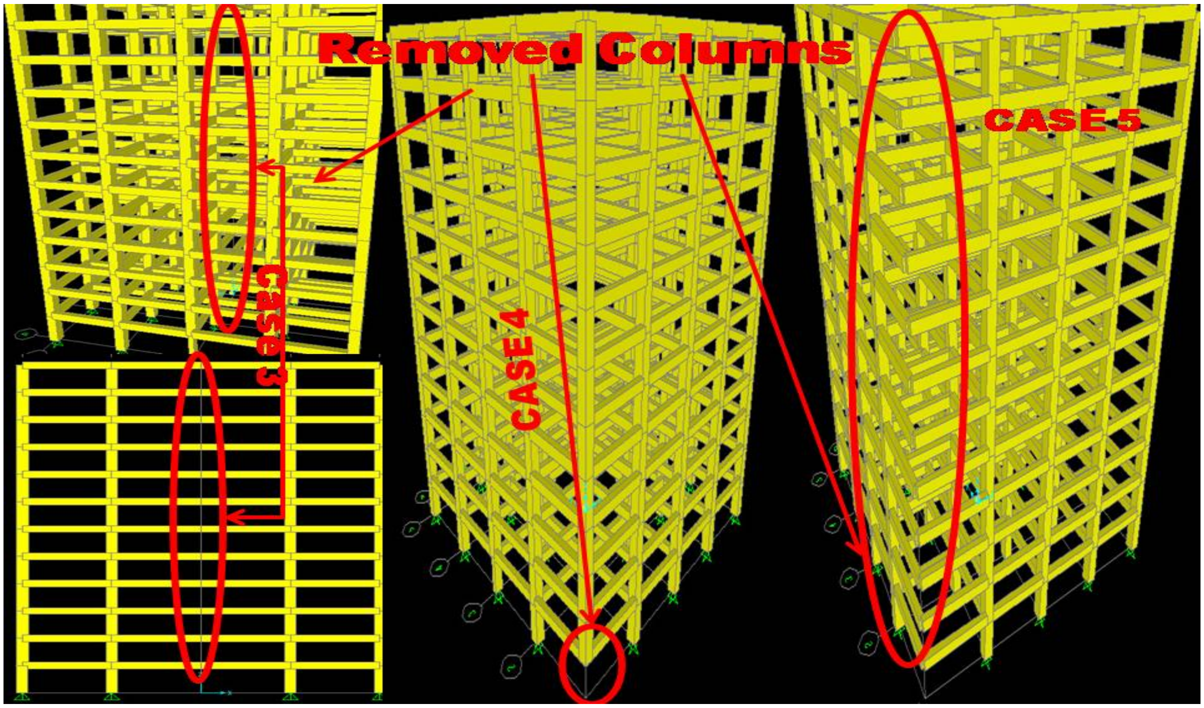

Fig. 12 (b) showing case 3, 4 and 5

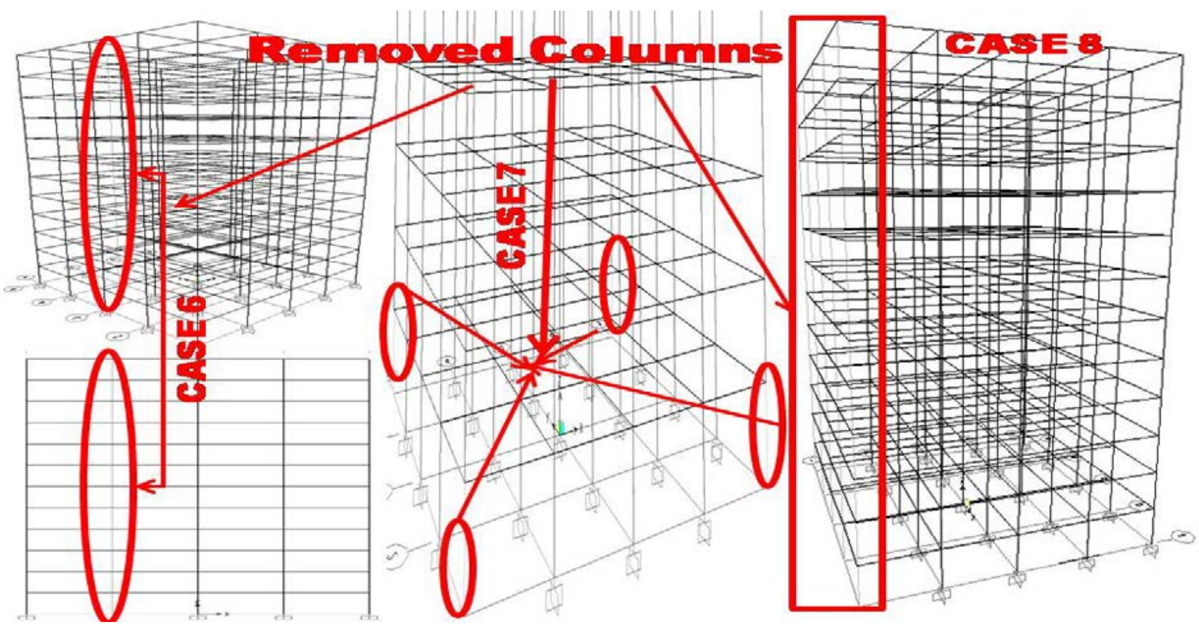

Fig. 12 (c) showing case 6, 7 and 8

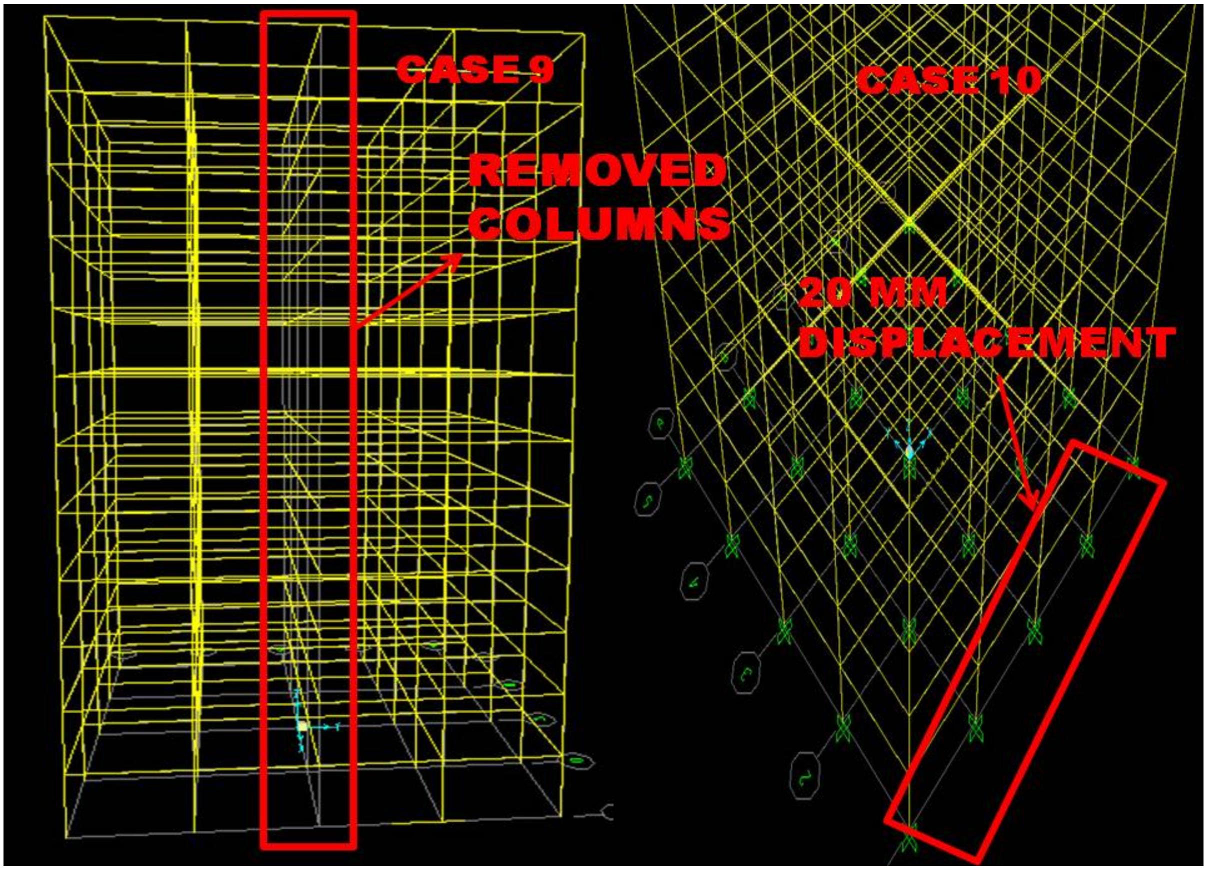

Fig. 12 (d) showing case 9 and 10

RESULT & CONCLUSION

- ✓ The principal reason for the acceptance of performing of the Non Linear Dynamic approaches on the structure for determining the capacity to the beams and slabs systems when columns are removed is satisfied.
- ✓ There is a significant amount of steel reinforcement properly detailed in the extreme areas of the slab, which helps to lessen the overall deflection of the structure while increasing the stiffness. There is the great severity of structure to collapse if proper joint moment is not considered.

- ✓ After analysing various cases of different storey precast buildings in SAP-2000, it could be conclude that building may sustain the increment of 0.428% of the design load, if the precast building was designed against progressive collapsing.
- ✓ There is 52.35% average increase in deflection in beam if columns are removed randomly. As the number of floors increases the collapsing of ground floor column increases, so after G+10, there should be increase in approx 63.26% increase in the column strength for sustaining the building against progressive collapse failure.
- ✓ There is a little bit change in the position of performance point in all the ten cases analysed for G+12 building, but it has found that case 9 is the worst situation in case of progressive collapse failure as summarized in table 4.

Table 4 Showing Performance Point of different cases in G+12 Building

Performance Point	When C_a & C_v value 0.4	When C_a & C_v value 1.0
	(Spectral Acceleration, Spectral Displacement)	(Spectral Acceleration, Spectral Displacement)
Original Structure	(0.492, 0.053)	(0.671, 0.143)
1	(0.484, 0.055)	(0.651, 0.143)
2	(0.481, 0.056)	(0.644, 0.142)
3	(0.477, 0.057)	(0.650, 0.147)
4	(0.481, 0.056)	(0.631, 0.144)
5	(0.483, 0.058)	(0.668, 0.144)
6	(0.480, 0.058)	(0.665, 0.145)
7	(0.467, 0.069)	(0.659, 0.149)
8	(0.479, 0.055)	(0.651, 0.144)
9	**(0.465, 0.065)**	**(0.628, 0.143)**
10	(0.475, 0.053)	(0.632, 0.143)

- ✓ There is a huge variation in axial force in the adjustment columns as compared in all the proposed cases of G+12 building. It has been seemed that building should be designed and analysed according to Case 5 and Case 9 as there is maximum increment in to make it resistive against progressive collapse failure as shown in axial force increases in all cases in G+12 building in table 5.

Table 5 Showing increment in Axial Force of different cases of G+12 building

Overall Axial Force of Column	Percentile increment with respect to original structure
Case 1	9.32
Case 2	10.25
Case 3	13.56
Case 4	5.76
Case 5	**30.26**
Case 6	11.67
Case 7	12.56
Case 8	13.26
Case 9	**29.25**
Case 10	10.36

- ✓ In Capacity spectrum curve of Case 8 and Case 9 having fewer slopes as compared to other cases, so it is resulted that a huge damage of structural element has occurred and large replacement is required as shown in figure 13.

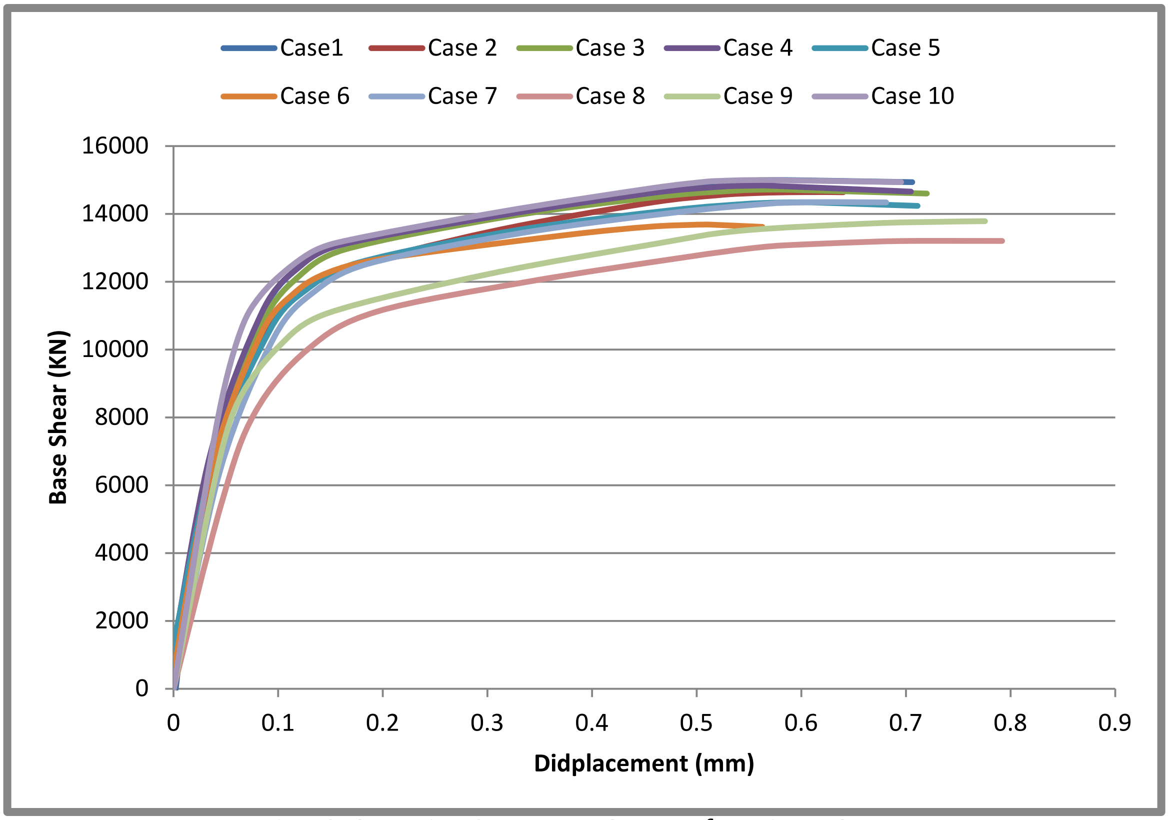

Fig 13 Capacity Spectrum Curve of Various Cases

- ✓ In case 5 of G+12 building the maximum numbers of collapse hinges has developed and it means huge collapsing can occurred at this level. Whereas in Case 9, approximately 995 basic safety level hinges are produced which means less damage also occurred at many joints in that case as concluded in table 6 below for different cases.

Table 6 Production of various Hinges

Cases	Number of Hinges reaches Collapse Level	Number of Hinges reaches Basic Safety Level	Number of Hinges reaches Occupational Level
Original Structure	169	983	328
Case 1	161	982	337
Case 2	178	905	397
Case 3	182	964	334
Case 4	173	993	314

Case 5	191	915	374
Case 6	160	975	345
Case 7	149	873	458
Case 8	155	973	352
Case 9	156	995	359
Case 10	169	1008	303

- ✓ In case 9 of G+12 building maximum increase in displacement in UX direction which is of 48.06% where as in case 7 there is also 35.36% increment in the displacement in the UX direction whereas in other cases this increment varies from 8-20% as shown in fig. 14

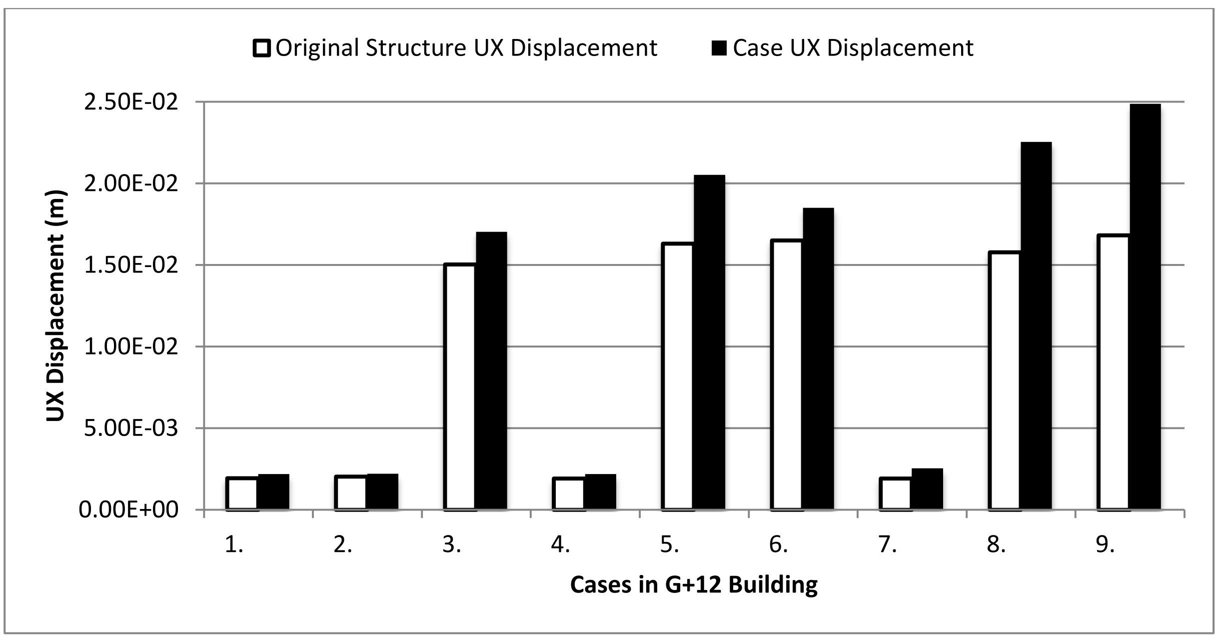

Fig.14 Change in displacement in UX Direction in G+12 Building

- ✓ In case 7 of G+12 building maximum increase in displacement in UZ direction which is of 85.72% where as in case 9 there is also 52.57% increment in the displacement in the UZ direction but in case 2 and case 3 there is not any change in the displacement in UZ direction whereas in other cases this increment varies from 18-32% as shown in fig. 15

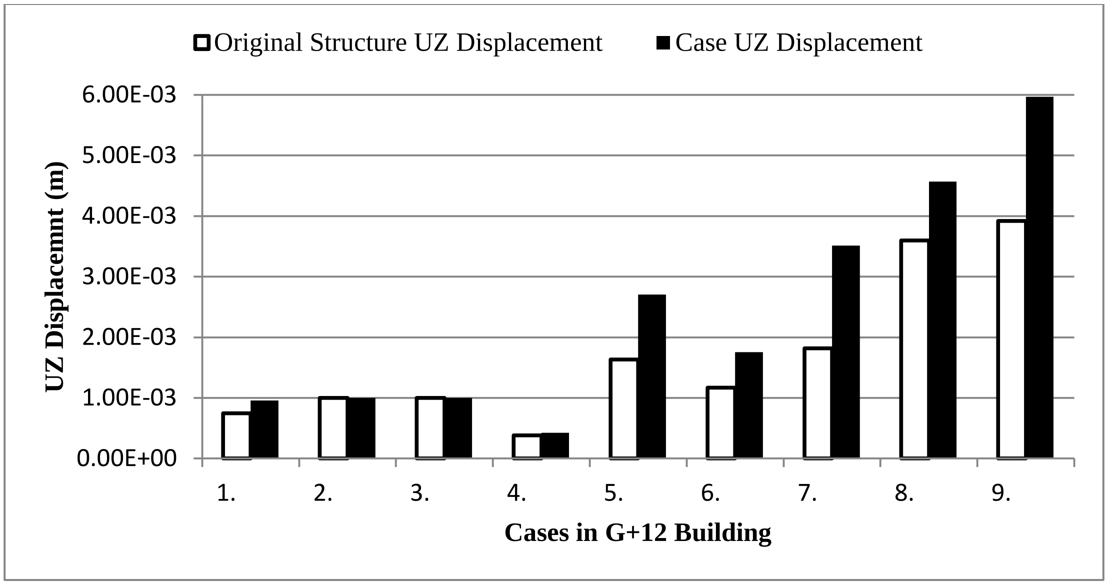

Fig.15 Change in displacement in UZ Direction in G+12 Building

- ✓ After considering and analyzing all the ten cases in G+12 precast building it can be concluded that
  1) CASES 1 to 4 is more strong and better against Progressive collapse case as in this there is less change in axial force and less number of hinges crosses collapse level and less displacement occurred in that case. It can also be stated that Case 4 should be taken into consideration when designing for residential building.
  2) CASES 9, 7 and 5 buildings analyses should be preferred in war zones mostly or strategic points as there is more collapse hinges are formed and huge increment in displacement when analyze with time history analyses.

- ✓ Taking into account the problems faced during research including obtaining relevant data, different case studies and previous work, it could be resulted that this is one of the most important field requiring insight, it lacks research and contributions.

- ✓ It has also been observed that, the research in line with the previous researches as shown in literature review.

REFERENCES

1. Marjanishvili, S.M., (2004), "Progressive Analysis Procedure for Progressive Collapse," *Journal of Performance of Constructed Facilities*, Vol. 18, No. 2, May.
2. Sasani, M., Bazan, M., and Sagiroglu, S. (2007). "Experimental and Analytical Progressive Collapse Evaluation of Actual Reinforced Concrete Structure", *ACI Journal*, 104(6), 731-739.
3. ACI Standard ACI 318-05, (2005), "Building Code Requirements for Structural Concrete," *American Concrete Institute*, Farmington Hills, Michigan.
4. Department of Defense, (DOD), (2005), "Design of Buildings to Resist Progressive Collapse," *Unified Facilities Criteria (UFC)* 4-023-03, 25 January, 2005.
5. McGuire, W., 1974, "Prevention of Progressive Collapse," Proceedings of the Regional Conference on Tall Buildings, Bangkok, Asian Institute of Technology, Bangkok, Thailand.
6. Moore, D. B., (2002), "The U.K. and European Regulations for Accidental Actions," NIST/NIBS Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse, Chicago, Ill. July. http://www.nibs.org/MMC/ProgCollapse presentations/Moore paper.pdf.
7. Marchand, K.A. and Alfawakhiri, F., (2004), Blast and Progressive Collapse, Facts for Steel Buildings Number 2, *American Institute of Steel Construction*, Chicago, Il., 67 p.
8. ASCE. (2005). "Minimum Design Loads for Buildings and Other Structures". American Society of Civil Engineers, ASCE 7-05, Reston, VA.
9. ASCE. (2002). "Minimum Design Loads for Buildings and Other Structures". *American Society of Civil Engineers*, ASCE 7-02, Reston, VA.
10. ASCE. (1997). "Design of Blast Resistant Buildings in Petrochemical Facilities". *American Society of Civil Engineers*, New York, New York.
11. FEMA-273. (1997), "Guidelines for the seismic rehabilitation of buildings". *Federal Emergency Management Agency (FEMA)*, United State of America.
12. FEMA-267. (1995). Interim Guidelines: Evaluation, repair, modification and design of steel moment frames, Report No. SAC-95-02, USA.

13. FEMA-403. (2002). "World Trade Centre building performance study: Data collection, preliminary observations and recommendation." *Federal Emergency Management Agency (FEMA)* Reign II, New York, N.Y. USA.
14. GSA. (2003), "Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects". Office of Chief Architect, General Services Administration, Washington, D. C.
15. UBC. (1994), "Structural Engineering Design Provisions", *Uniform Building Code*, Vol. **2**, International Building Officials.
16. DoD. (2005). "Design of buildings to resist progressive collapse." *Unified Facilities Criteria UFC* 4- 023-03, Department of Defence, USA.