Response of a multi-storey steel composite building with concentric bracing under consecutive column removal scenarios

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Abstract

In this paper, a 3-dimensional finite element modelling technique developed by the author was used to analyse the progressive collapse of multi-storey buildings with composite steel frames. The nonlinear dynamic analysis procedure was performed to examine the behavior of the building under consecutive column removal scenarios. The response of the building was studied in detail and the measures to mitigate progressive collapse in future designs were also recommended.

Keywords*: progressive collapse, connection, finite element, modelling*

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1 INTRODUCTION

Progressive collapse has attracted more and more interest to researchers after the event of 11th September 2001. SEI/ASCE 7-05 [1] gives the accurate definition of the term progressive collapse as -- ‘‘the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it.” Currently, there are some design procedures to mitigate the potential of progressive collapse in both in the UK and US. The UK Building Regulations [2] and BS5950 [3] state requirements for the avoidance of disproportionate collapse. In the United States, the Department of Defense (DoD) [4] and the General Services Administration (GSA) [5] provide detailed guidelines regarding methodologies to resist progressive collapse of building structures. Both employ the alternate path method (APM). The methodology is generally applied in the context of a ‘missing column’ scenario to assess the potential for progressive collapse and used to check if a building can successfully absorb loss of a critical member. FEMA 2002 [6] and NIST 2005 [7] also provide some general design recommendations, which require steel-framed structural systems to have enough redundancy and resilience, allow for alternative load paths and additional capacity redistributing gravity loads when structural damage occurs. There are four procedures for alternate path method: linear elastic static (LS), linear dynamic (LD), nonlinear static (NS), and nonlinear dynamic (ND) methods. The last method is also recommended by FEMA 274 [8] for seismic analysis and design of structures.

So far, there are some analytical studies on the progressive collapse behaviors of buildings. Kaewkulchai et al [9] proposed a beam element formulation and solution procedure for dynamic progressive collapse analysis, which provide guidance for further study on the modeling of progressive collapse. Powell [10] reviewed the principles of progressive collapse analysis for the Alternate Path method. Khandelwal et al [11] studied the progressive collapse resistance of seismically designed steel braced frames with validated two dimensional models. The simulation results show that the eccentrically braced frame is less vulnerable to progressive collapse than the special concentrically braced frame. Kim et al [12] studied the progressive collapse-resisting capacity of steel moment resisting frames using alternate path methods recommended in the GSA and DoD guidelines. It was observed that the nonlinear dynamic analysis provided larger structural responses and the results varied more significantly. However the linear procedure provided a more conservative decision for progressive collapse potential of model structures. Using the commercial program SAP2000, Tsai et al [13] conducted the progressive collapse analysis following the linear static analysis procedure recommended by the US General Service Administration GSA. Liu [14] investigated the methods to prevent progressive collapse by strengthening beam-to-column connections. Shi et al proposed a new method for progressive collapse analysis of RC frames under blast loading [15]. Rather than using sudden column removal methods, Shi et al directly applied the blast load on the structure. Mohamed et al used the direct element removal method to model the progressive collapse in reinforced concrete buildings [16]. They present a novel analytical formulation of an element removal algorithm based on dynamic equilibrium and the resulting transient change in system kinematics.

As mentioned above, for the research undertaken so far, most have involved 2-D models and are based on bare steel frames without considering the contribution of the floor systems which reduces the accuracy of the model. Recent studies by the author and other researchers found the importance of accounting for three dimensional effects and that the concrete floor slabs also play a crucial role in the progressive collapse response. To solve the above problem, Fu [18], using ABAQUS [17], proposed a 3-D finite element model to investigate the progressive collapse of multi-storey buildings in different column removal scenarios. Fu then extended his study of progressive collapse of the multi-storey buildings and found that, with normal column spacing, the beams may still be in the elastic stage after one column removal on the condition that they are designed with the current design code [19]. Plasticity is normally observed in more than two column removal scenarios. As plasticity is very important in absorbing the energy caused by the columns removal, so, in this paper, two columns removal scenarios are studied in detail and the plasticity developed in the steel member and the response of the slabs are studied in detail. In the previous study, the columns were removed simultaneously, which is a conservative approach. However, in reality, the chance for two columns to be damaged at the same time is rare. When attacks like car bomb or an aeroplane impact happen, it will hit one column first, then another. The columns are normally destroyed consecutively. The structural behavior will be different. Therefore, the consecutive column removal scenarios are studied and presented here.

In this paper, using the 3-D finite element modeling techniques developed by the Fu [18], several 3-D finite element models representing 20 storey composite steel frame buildings were built to perform the progressive collapse analysis under two column removal scenarios. The lateral stability of the model is achieved by using concentric bracing. In the analysis, except for case 3, the columns were removed consecutively rather than simultaneously. Based on the analysis, the structural behavior of the multi-story buildings under consecutive column removal scenarios was investigated in detail. Throughout the study, measures to mitigate progressive collapse were also recommended.

2 3D FINITE ELEMENT MODEL

2.1 Description of the prototype structure

As shown in Fig 1, a three-dimensional finite element model was created using the method of Fu [18] with ABAQUS [17]. The model simulates the structural framing of typical high-rise buildings in the current construction industry with composite slabs. The model replicates a 20-storey steel composite frame building with the major grid spacing of 7.5m in both directions as shown in Fig 1. The floor height is 3 m for each level. The floor system is a full shear interaction metal deck with a slab thickness of 130 mm; the shear studs are evenly distributed along the steel beam. The steel rebar mesh in the slabs is A252. All the steel beams are British universal beam UB305x102x25 sections with the spacing shown in Fig 1. The columns are British universal column with UC356x406x634 from ground floor to level 6, UC356x406x467 for level 7 to level 13 and UC356x406x287 for level 14 to level 19. The main lateral stability is provided by cross bracing in the four elevations as shown in Fig.2. The cross bracings are British Circular Hollow section CHCF 273x12.5. The above structural steel member sizes are determined based on the current BS design guidance.

2.2 Modeling techniques and validation

Detailed modelling techniques were explained in Fu [18]. For the convenience of the reader, a brief introduction is given here. All the beams and columns are modelled using *BEAM elements. The slab are modelled using the four node *Shell element. Reinforcement was imbedded in each shell element using the *REBAR element as in smeared layers. The beam and shell elements are coupled together using rigid beam constraint equations to give the composite action between the beam elements and the concrete slab. The model also incorporates nonlinear material characteristics. The material properties of all the structural steel components were modelled using an elastic-plastic material model from ABAQUS. The incorporation of material nonlinearity in an ABAQUS model requires the use of the true stress (σ) versus the plastic strain (ε^{pl}) relationship; this must be determined from the engineering stress-strain relationship. The classical metal plasticity model defines the post-yield behaviour for most metals. ABAQUS approximates the smooth stress-strain behaviour of the material with a series of straight lines joining the given data points to simulate the actual material behaviour. The material will behave as a linear elastic material up to the yield stress of the material. After this stage, it goes into the strain hardening stage until reaching the ultimate stress. As ABAQUS assumes that the response is constant outside the range defined by the input data, the material will deform continuously until the stress is reduced below this value. The concrete material was modelled using a concrete damage plasticity model. The shell elements are integrated at 9 points across the section to ensure that the concrete cracking behaviour is correctly captured. The models are supported at the base of the ground floor columns. The mesh representing the model has been studied and is sufficiently fine in the areas of interest to ensure that the developed forces can be accurately determined. The steel beam to column connections is assumed to be fully pinned. The continuity across the connection is maintained by the composite slab acting across the top of the connection. A pin connection is also assumed for the brace to simulate the conventional gusset plate connection.

In order to valid the proposed model, in Fu [18], a two-storey composite steel frame model was built using ABAQUS. The model replicated the full scale testing of a steel-concrete composite frame by Wang et al [20]. Comparison between the tests result and the modelling result were made. The comparison of the results shows that good agreement was achieved.

3 DYNAMIC RESPONSE OF THE BUILDING

The response of the building under sudden column loss is assessed here using a nonlinear dynamic analysis method with 3-D finite element technique. Rather than remove two columns simultaneously, in the analysis, one column was removed first, and then a second column was removed. This is to simulate the scenario of a large vehicle or aeroplane to impacting the building.

The loads are computed as dead loads plus 25% of the live load in accordance with the acceptance criteria outlined in Table 2.1 of the GSA [5]. The self-weight of the structure is calculated in ABAQUS, the super-imposed dead load is taken as 1 kN/m^2 and the live load is 1.5kN/m^2. Firstly the gravity load was applied to the model in the static step. After the static step, the dynamic step followed, and the columns were removed over a period of 20 milliseconds following the requirement of GSA [5]. The simulations were conducted with 5 % mass proportional damping. The maximum forces, displacements for each of the members involved in the scenario were recorded.

Table 1 shows the list of analysis cases considered in this study. To facilitate the following discussion, related to the grid line shown in Fig1, the columns and beams are named as follows: for instance, Column C1 stands for the column at the junction of grid C and grid 1. Beam E1-D1 stands for the beam on grid 1 starting from grid E to grid D.

3.1 Case 1 column A1 and A2 at ground level removed

As shown in Fig.3, in case 1, the column A1 at ground floor was first removed. It is shown in Fig.5 that, node A1 reached a peak vertical displacement of 58 mm, and then continued to vibrate. At step 2, with the removal of column A2, the vertical defection of A1 started to increase again and reached a peak vertical displacement of 118 mm. It can also be seen that, after column A2 was removed, node A2 reached a peak displacement value of 100mm and started to vibrate with the balance position of 70mm.

When the first column was removed, a redistribution of major moments in the adjacent beams was observed, as seen in Fig. 6. It can be seen that, the moment at the end B1 of beam B1-A1 reached a peak value after the removal of column A1. However, the moment change at the same location of beam A3-A2 is smaller as it is far from column A1. The force in beam A3-A2 increased dramatically after the removal of column A2, however, the peak value is smaller than with beam B1-A1.

In ABAQUS, the plastic strain is obtained by subtracting the elastic strain which is defined as the value of true stress divided by the Young's modulus, from the value of total strain. This relationship is written

$$\varepsilon^{pl} = \varepsilon^{t} - \varepsilon^{el} = \varepsilon^{t} - \sigma / E$$

ε^{pl} Is the true plastic strain

ε^{t} Is the true total strain

ε^{el} Is the true elastic strain

σ Is the true stress, and

E Is Young's modulus

Fig. 7 shows the vlaue of plastic stain due to the resultant axial force for beams B1-A1 and A3-A2. It can be seen that, when the first column was removed, no plasticity was observed in any of the beams. Beam B1-A1 went into the plastic range shortly after the removal of the second column; however, beam A3-A2 remained elastic.

Fig.4 shows the tensor distribution of plastic strain in the concrete slab. It can be seen that due to the hanger effect, large tensile plastic strain (shown in red) is observed in the slab near the region of the removed column on each floor, which indicates a crack forming in the slab. However, it is evident that for the remaining part of the structure, cracks are not observed. So this is more or less a localized behaviour. As the slab cracks are concentrated near the removed column area, it would be sensible to put more mesh in the slabs to help prevent progressive collapse.

3.2 Case 2 column A1 and A2 at ground level 14 removed

To further investigate the behaviour of the structure, as shown in Fig.8, in case 2, column A1 at level 14 was removed first. Similar to case 1, from Fig.10 it can be seen that when the first column was removed, node A1 vibrated and reached a peak vertical displacement and continued to oscillate. At step 2, with the removal of column A2, the vertical defection of A1 started to increase again and reached a peak vertical deflection. In the mean time, A2 also reached a peak value and started to oscillate. A redistribution of forces was observed to take place as shown in Fig. 11 and Fig 12. It can be seen that, in case 2, similar behaviour to case 1 was observed.

Fig.13 and Fig.14 show the comparisons between case1 and case 2. It can be seen that case 2 exhibited lower major bending moments and developed less plastic strain in the adjacent beams. This is because when the columns were removed at the higher level, only the storeys above are affected. Because the column loads at the ground level are greater than the higher levels, so, when columns are removed at ground level, more force has to redistribute into the adjacent beams. Hence, larger internal forces were observed.

Fig.9 shows the plastic strain tensor distribution in the ncrete slab. It is evident that, unlike Fig.4, large plastic strain is observed mainly on the floor above the removed column.

3.3 Case 3 column A1and A2 at ground level removed (two column removed simultaneously)

In order to clearly understand the behaviour of the building, in case 3, the columns A1 and A2 were removed simultaneously at ground level as shown in Fig.15. Compared with case 1, a different structural behaviour was observed. It can be seen from Figs 17 and 18 that, for both case 1 and case 3, the force in beam A3-A2 is smaller than the force in B1-A1. However, in case 3, both A3-A2 and B1-A1 went into the plastic stage. In case 1, only B1-A1 went into plastic stage. Fig.16 shows the plastic strain tensor distribution in the concrete slab. It can be seen that, compared with Fig.4, large tensile plastic strain is observed for the slab near the removed column however, only on the floor above the removed column.

It can be concluded that using a different column removal sequence will cause a different force redistribution path. Most researchers prefer to rely on the catenary effect to help resisting progressive collapse. However, as discussed in Fu [19] the catenary effect can only be triggered when plasticity is adequately formed in the relevant beams. Different column removal scenario will produce different plasticity forming paths, which needs to be taken into the consideration in the plastic design of the composite frame buildings in resisting progressive collapse.

3.4 Case 4 column A5, Bracing A5-A6 at ground level removal scenario

In order to investigate the effect of the bracing removal, in case 4, the column A5 is first removed at ground floor (as shown in Fig.19). It can be seen from Fig. 21 that, the internal force in beams A6-A5 and B5-A5 has increased substantially and reached a peak value. The force then started to oscillate. In step 2, Bracing A5-A6 was removed (as shown in Fig.19). The internal force started to increase again and reached a peak value. Compared with case 1, 2 and 3, the moment is quite small, and no plasticity is observed in the corresponding beam. This is because only one column is removed and the affected loading area is smaller than with the two column removal scenarios, and therefore the response is smaller.

Fig.20 shows the plastic strain tensor distribution in the concrete slab. It can be seen that large plastic strain was observed in the slab near the removed column. However, the value is dramatically smaller than with the first three cases.

3.5 Case5 column A5, Bracing A5-A6 at level 14 removal scenario

In case 5, as shown in Fig.22, the column of A5 at level 14 was first removed. In step 2, bracing A5-A6 at level 14 was removed. No plasticity was observed as well in this case. It can be also seen from Fig.24 that, case 5 exhibits a similar structural behaviour to case 4. Fig. 25 and Fig.26 are the comparison of these two cases. It can be seen that the case with columns removed from the lower level exhibit less dynamic vertical displacement but higher internal force, the reason is explained in the previous sequal.

Fig.23 shows the tensor distribution of plastic strain in the concrete slab. It can be seen that, compared with Fig.20, large plastic strain is mainly observed in the slab near the removed column several storeys above the removed column. The value is dramatically smaller than with the first three cases.

From the analysis results of case 4 and 5, it can be concluded that the building is less vulnerable to progressive collapse in the case of bracing removal unless the removal is accompanied by strong wind, earthquake or very large lateral impact loads such as that arising from an aeroplane inpact. But the chance that these loads happen together is very low. As the function of the bracing is mainly for resisting lateral force, most gravity load is transferred to the foundation through the columns. So when the bracing is removed, the gravity load can still find a direct path to the foundation.

3.6 Case6 column A2 and A3 at ground level removal scenario

In case 1, the column A1 at the corner was first removed. Different to case 1, in case 6, as shown in Fig.27, the column A2 was first removed at ground floor. At step 2, the column A3 at ground level was removed. Fig. 28 to 31 show the response of the structure. Compared to case 1, where column at A1 was first removed, Case 6 exhibits less response in the term of vertical deflection, moment and plastic strain. Therfore, the building is more vulnerable in the corner column removal scenarios.

4 CONCLUSIONS

In this paper, the behaviour of a 20 storey steel composite frame building under consecutive column removal scenarios was investigated using a 3-D finite element modelling approach.

Below are main findings:

1. The removal of the selected columns does not always produce the development of the plasticity. The formation of plasticity is also related to the different column removal scenarios. Different column removal sequences will also make different plasticity formations, which directly affect the triggering of the catenary effect. This should be taken into the consideration in the plastic design of composite frame buildings when resisting progressive collapse.

2. After the removal of the columns, the force are mainly redistributed to the adjacent beams, the beams situated more remote from the removed column were less affected.

3. To resist progressive collapse, the beams in the lower level should be designed with stronger sections than those in the upper levels. This is because the beams will withstand more force redistribution from the columns removed at a lower level than the columns removed at a higher level.

4. The building is more vulnerable in the corner column removal scenarios.

5. As the slab cracks are concentrated near the removed column area, it would be prudent to increase the steel reinforcement in the slabs to help prevent progressive collapse.

REFERENCE

[1]ASCE SEI/ASCE 7-05 Minimum Design Loads for Buildings and Other Structures. Washington, DC: American Society of Civil Engineers; 2005.

[2] Office of the Deputy Prime Minister. The building regulations 2000, Part A, Schedule 1: A3, Disproportionate collapse. London (UK), 2004.

[3] British Standards Institution. BS 5950: Structural use of steelwork in buildings, Part 1: Code of practice for design — rolled and welded sections, London (UK).

[4] Unified Facilities Criteria (UFC)-DoD. Design of Buildings to Resist Progressive Collapse, Department of Defense, 2005.

[5] GSA. Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects. The U.S. General Services Administration; 2003.

[6] Federal Emergency Management Agency (FEMA) (2002) FEMA 403, World Trade Center Building Performance Study: Data Collection, Preliminary Observations, and Recommendations. Washington, DC, USA, May.

[7] National Institute of Science and Technology (NIST) (2005) *Final Report on the Collapse of the World Trade Center Towers*. NCSTAR 1, Federal Building and Fire 318 Safety Investigation of the World Trade Center Disaster, US Department of Commerce, Gaithersburg, MD, USA.

[8] FEMA 274. NEHRP Commentary on the guidelines for the seismic rehabilitation of buildings. Washington (DC): Federal Emergency Management Agency; 1997.

[9] Griengsak Kaewkulchai, Eric B. Williamson, 'Beam element formulation and solution procedure for Dynamic progressive collapse analysis', Computers & Structures, Volume 82, Issues 7-8, March 2004, Pages 639-651

[10] G. P. Powell , 'Progressive Collapse: Case studies Using Nonlinear Analysis', Proceedings of the 2005 Structures Congress and the 2005 Forensic Engineering Symposium; New York, New York; USA; 20-24 Apr. 2005. 2005

[11] Kapil Khandelwal, Sherif El-Tawil, Fahim Sadek, Progressive collapse analysis of seismically designed steel braced frames, Journal of Constructional Steel Research, In Press, Corrected Proof, Available online 8 April 2008

[12] Jinkoo Kim, Taewan Kim, Assessment of progressive collapse-resisting capacity of steel moment frames, Journal of Constructional Steel Research, Volume 65, Issue 1, January 2009, Pages 169-179.

[13] Meng-Hao Tsai, Bing-Hui Lin, Investigation of progressive collapse resistance and inelastic response for an earthquake-resistant RC building subjected to column failure. Engineering Structures, In Press, Corrected Proof, Available online 21 July 2008

[14] J.L. Liu, 'Preventing progressive collapse through strengthening beam-to-column connection, Part 1: Theoretical analysis', *Journal of Constructional Steel Research 66 (2010) 229_237.*

[15]Yanchao Shi, Zhong-Xian Li, Hong Hao, 'A new method for progressive collapse analysis of RC frames under blast loading' Engineering Structures, Volume 32, Issue 6, June 2010, Pages 1691-1703.

[16] Mohamed Talaat, Khalid M.,' Mosalam, 'Modeling progressive collapse in reinforced concrete buildings using direct element removal' Earthquake Engineering & Structural Dynamics, Volume 38, Issue 5, pages 609–634, 25 April 2009

[17] ABAQUS theory manual, (2010) Version 6.10 Hibbitt, Karlsson and Sorensen, Inc. Pawtucket, R.I.

[18] Feng Fu, Progressive collapse analysis of high-rise building with 3-D finite element modelling method, Journal of Constructional Steel Research, Vol. 65, 2009, pp. 1269-1278.

[19] Feng Fu, 3-D Nonlinear Dynamic progressive collapse Analysis of multi-storey steel composite frame buildings - parametric study, Engineering structure, in press.

[20]Jing-Feng Wang, Guo-Qiang Li, Testing of semi-rigid steel–concrete composite frames subjected to vertical loads, Engineering Structures, Volume 29, Issue 8, August 2007, pp. 1903-1916.

FIGURES

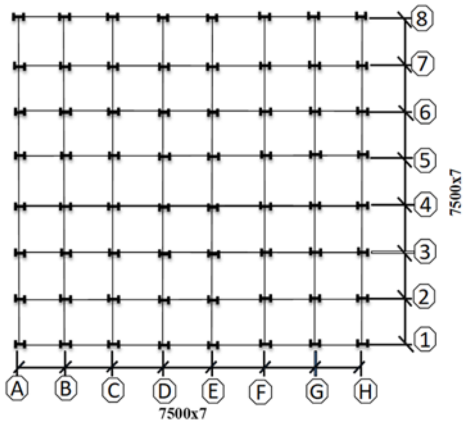

Fig. 1 Typical plan of 20-story prototype building

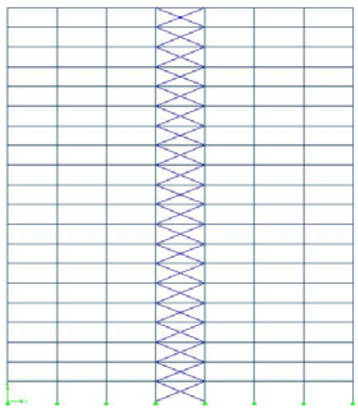

Fig 2 elevation of 20-story prototype building

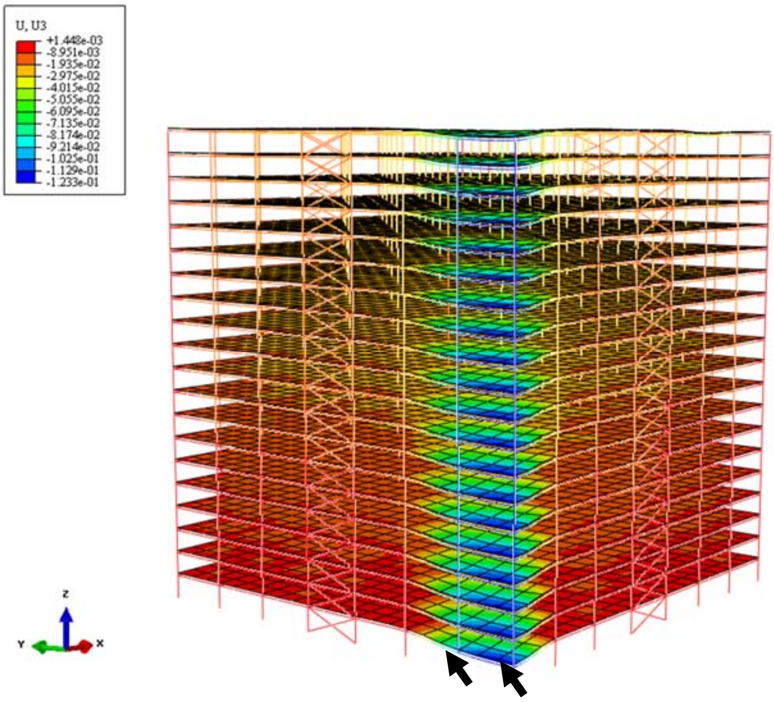

Fig .3 Vertical displacement of case 1 (deformation scale factor 10)

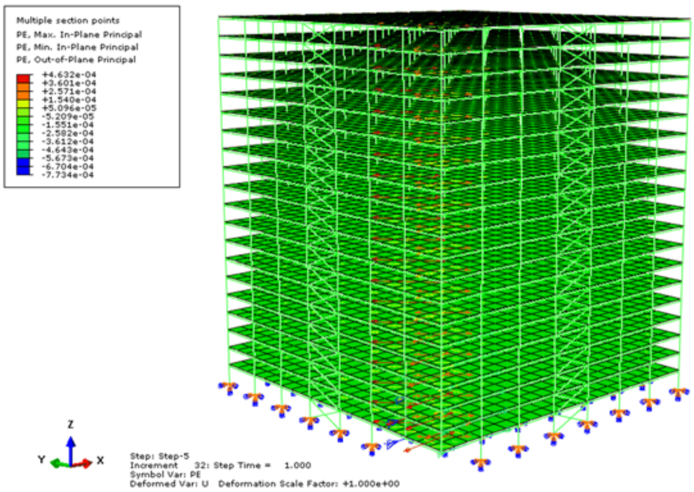

Fig.4 Tensor distribution of plastic strain of concrete slab

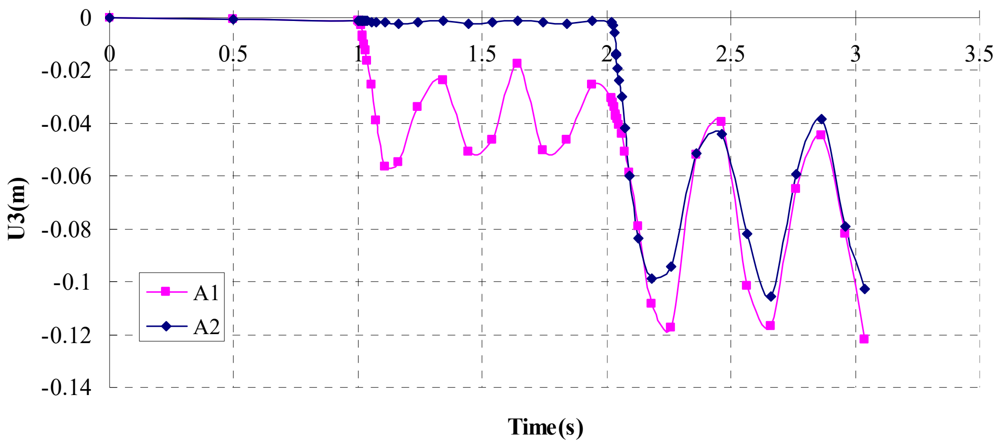

Fig.5 Displacement of the node at A1 and A2 of Case 1

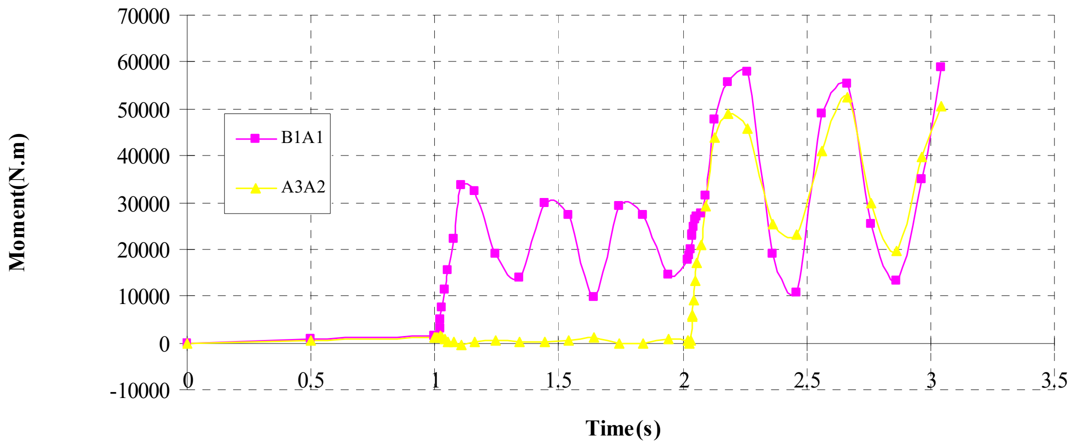

Fig.6 Major Moment of Beam B1A1 and A3A2 at ground level of Case 1

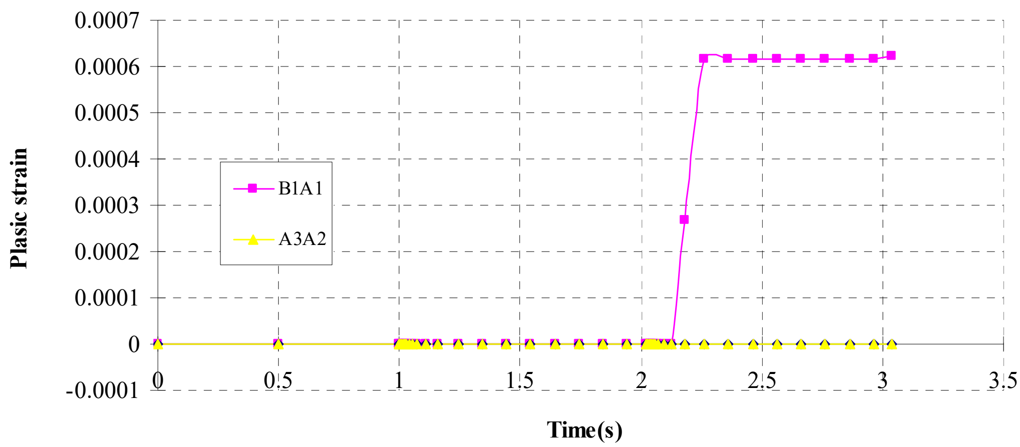

Fig.7 Plastic strain of Beam B1A1 and A3A2 at ground level of Case 1

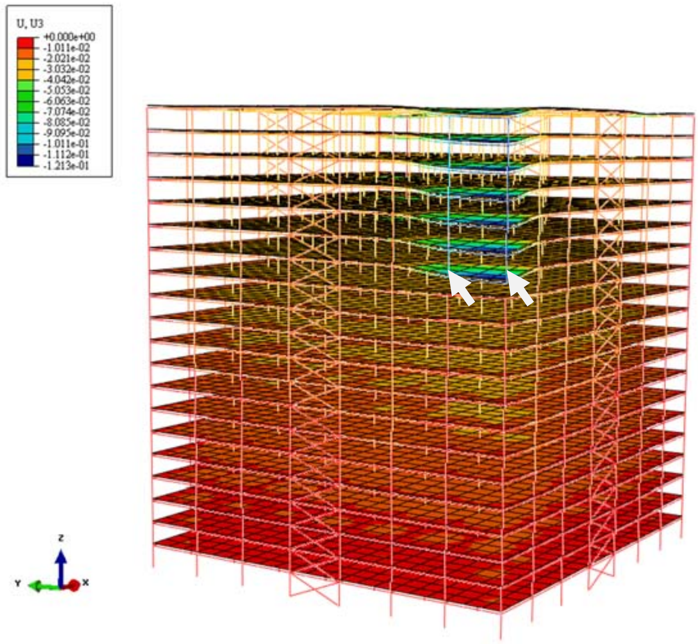

Fig.8 Vertical displacement contour of case 2

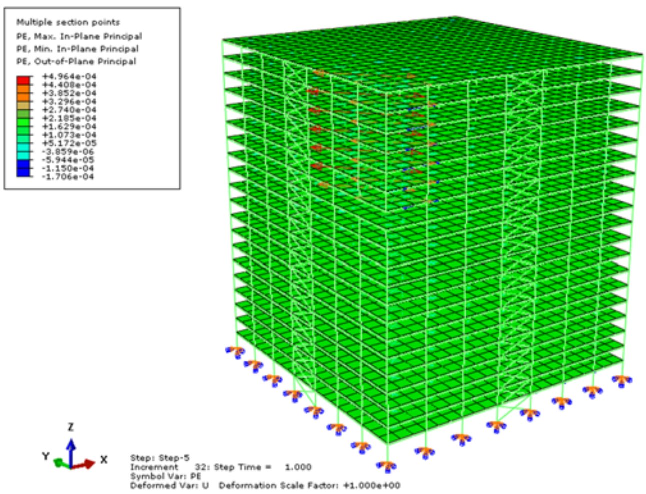

Fig.9 Tensor distribution of plastic strain of concrete slab

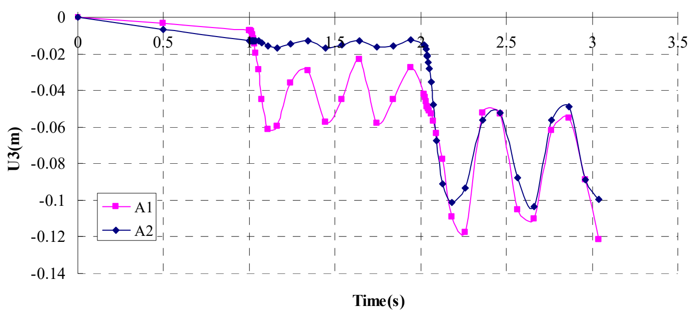

Fig.10 Displacement of the node at A1 and A2 at ground level of Case 2

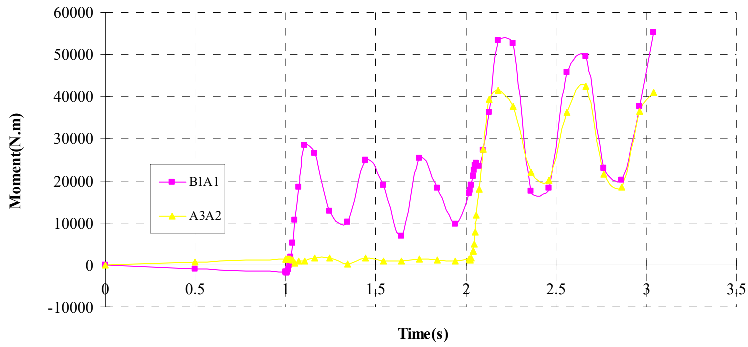

Fig.11 Major Moment of Beam B1A1 and A3A2 at level 14 of Case 2

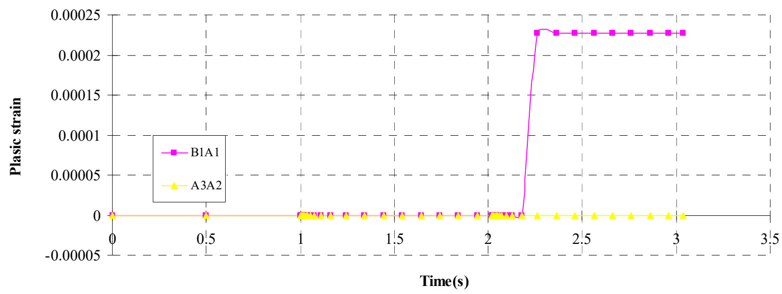

Fig.12 Plastic strain of Beam of B1A1 and A3A2 at level 14 of Case 2

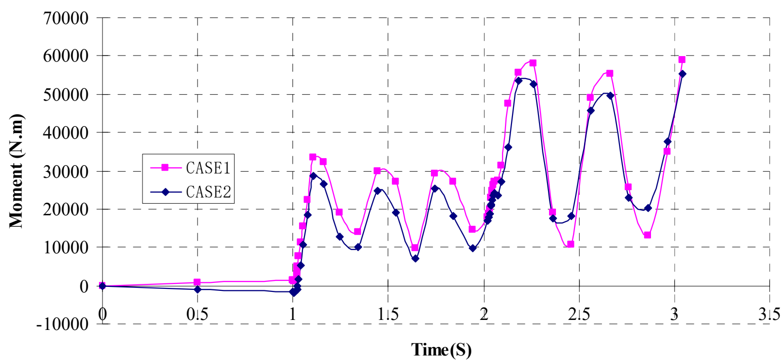

Fig.13 Comparison of Major Moment of Beam B1A1 at ground level of CASE1 and Beam B1A1 at level 14 of CASE2

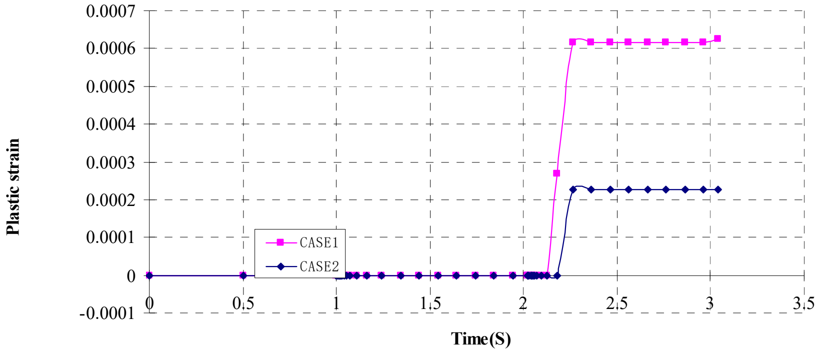

Fig.14 Comparison of Axial Plastic strain of beam B1-A1 at ground level for case 1 and at level 14 for case 2

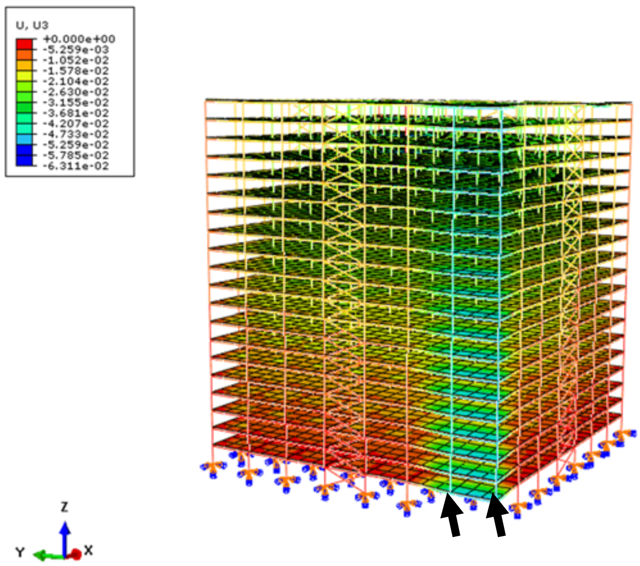

Fig.15 Vertical displacement contour of case 3

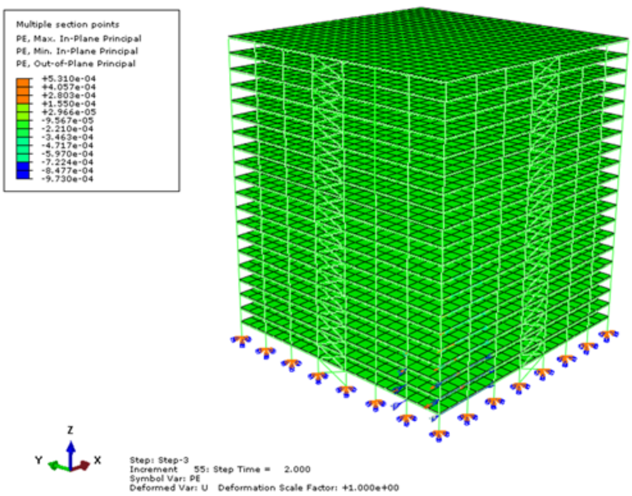

Fig.16 Tensor distribution of plastic strain of concrete slab

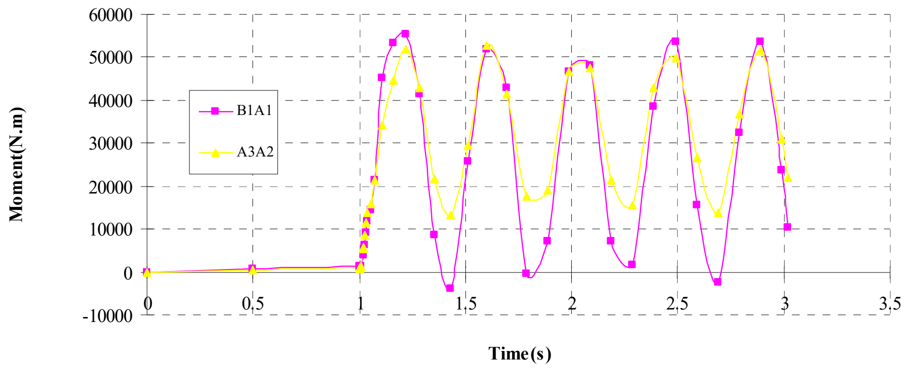

Fig.17 Major moment of Beam B1A1 and A3A2 at ground level of Case 3

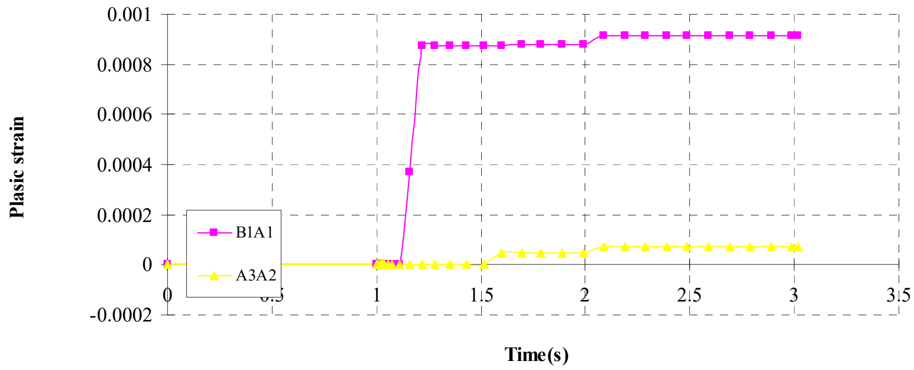

Fig.18 Plastic strain of Beam B1A1,A3-A2 at ground level of Case 3

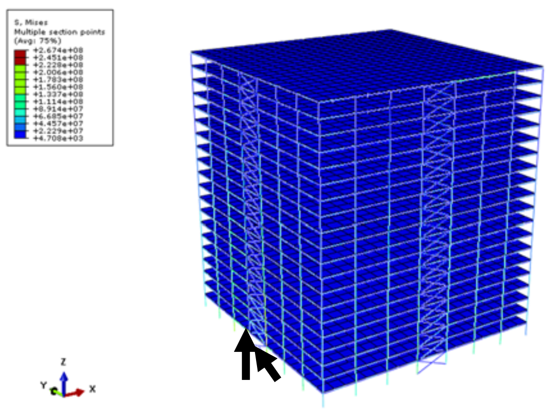

Fig .19 Von mises stress contour of case 4

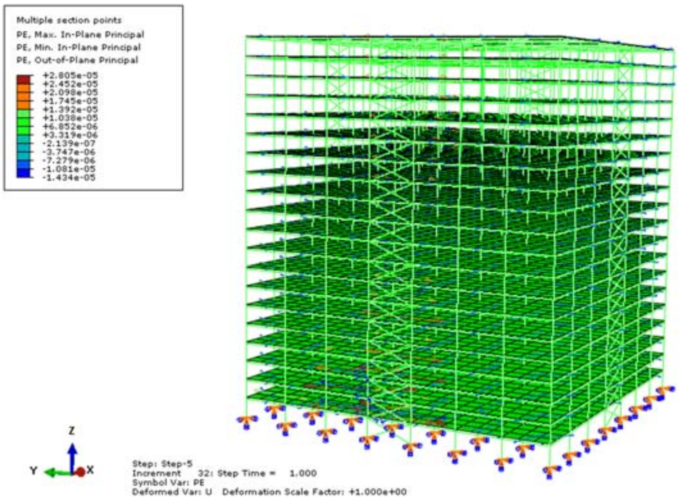

Fig.20 Tensor distribution of plastic strain of concrete slab

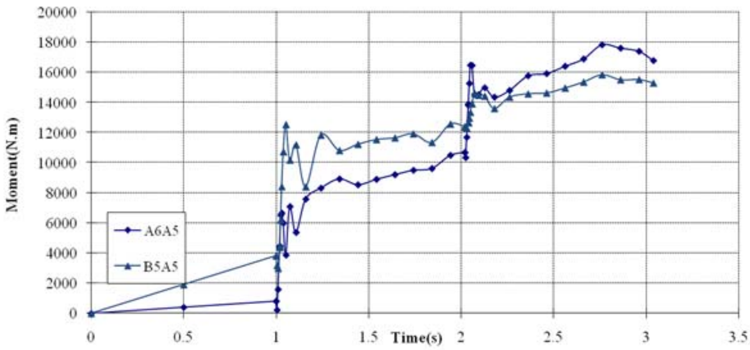

Fig.21 Major bending moment of Beam A6A5,B5A5 at Ground level of Case 4

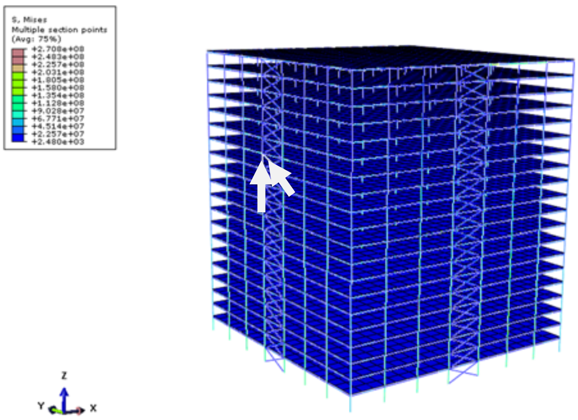

Fig .22 Von mises stress contour of case 5

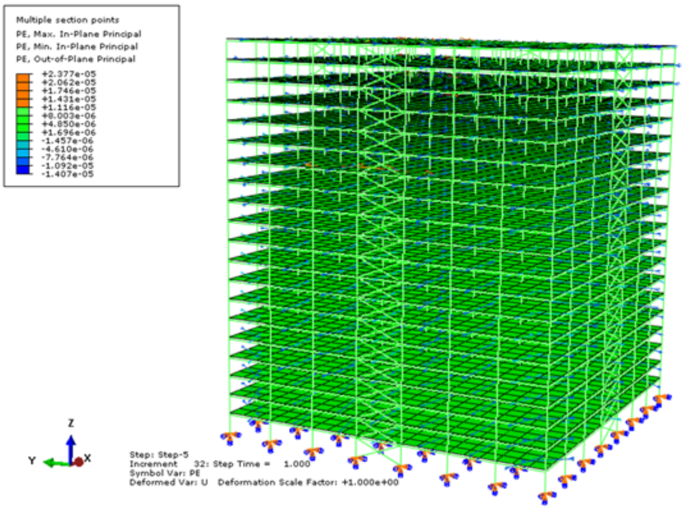

Fig.23 Tensor distribution of plastic strain of concrete slab

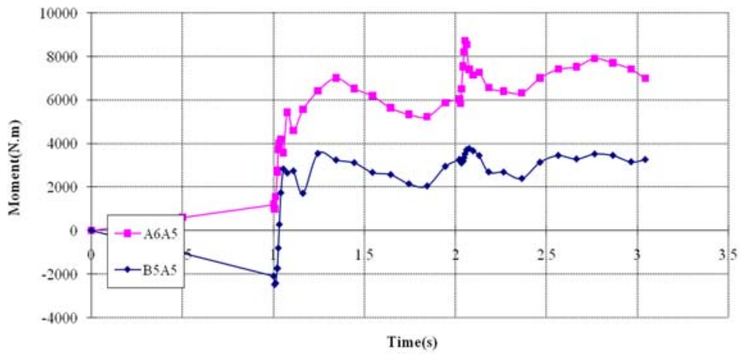

Fig.24 Major bending moment of Beam A6A5,B5A5 at level 14 of Case 5

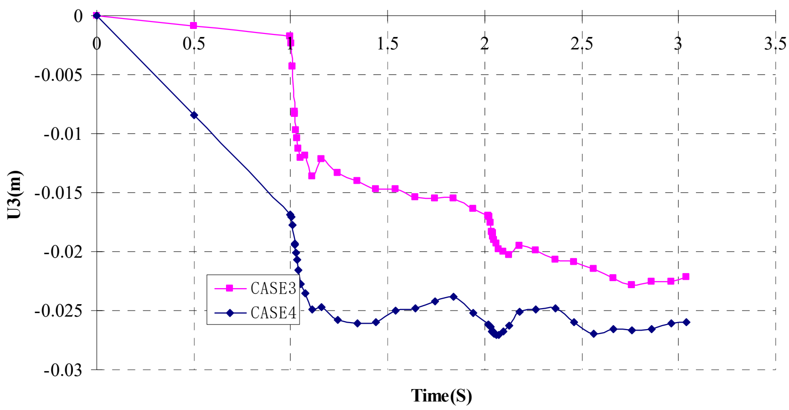

Fig.25 displacement of the node A1 at ground level of case 3 and node A1 at level 14 of case 5

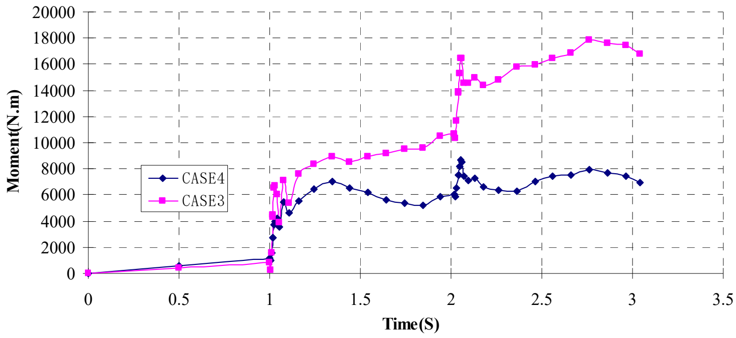

Fig.26 Major bending moment of beam B1A1 at ground level for case 3 and at level 14 for case 5

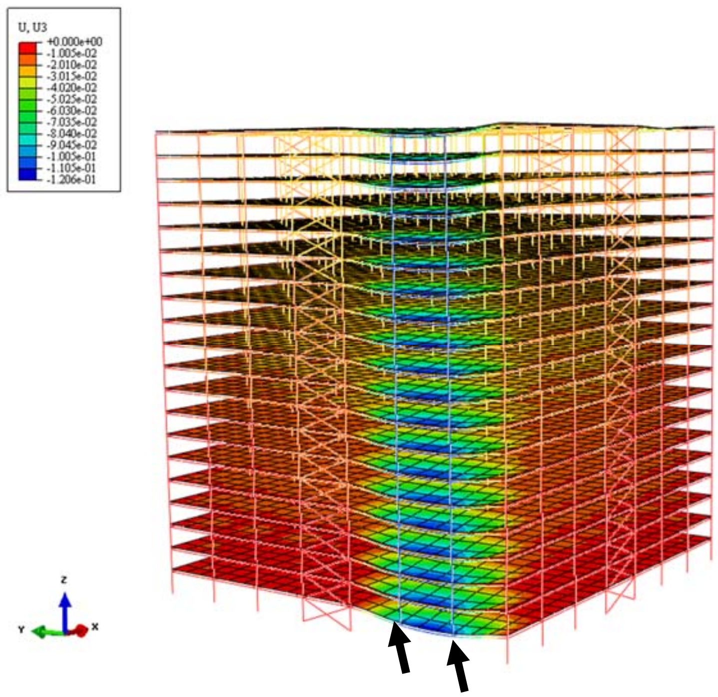

Fig .27 Vertical displacement contour of case 6

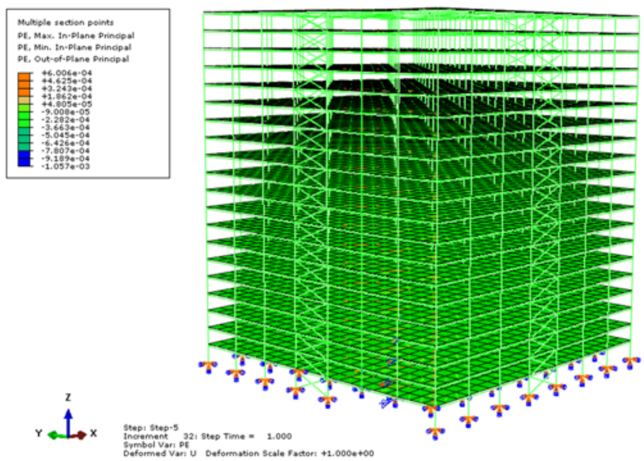

Fig.28 Tensor distribution of plastic strain of concrete slab

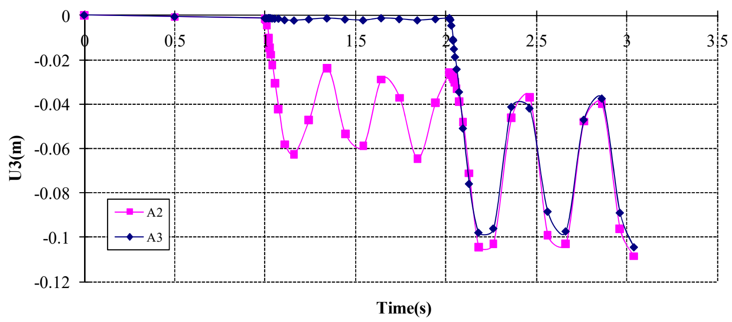

Fig.29 Displacement of the node at A2 and A3 of Case 1

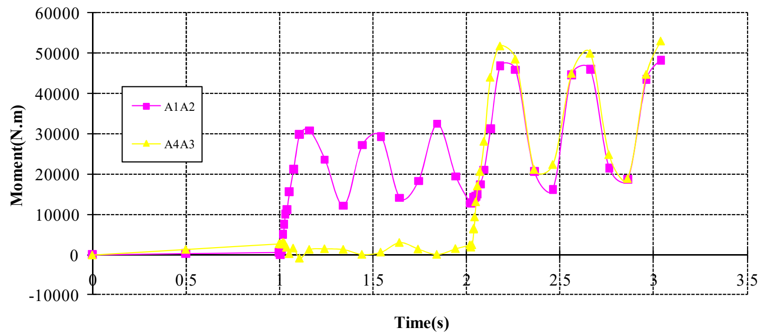

Fig.30 Major Moment of Beam A1A2 and A4A3 at ground level of Case 1

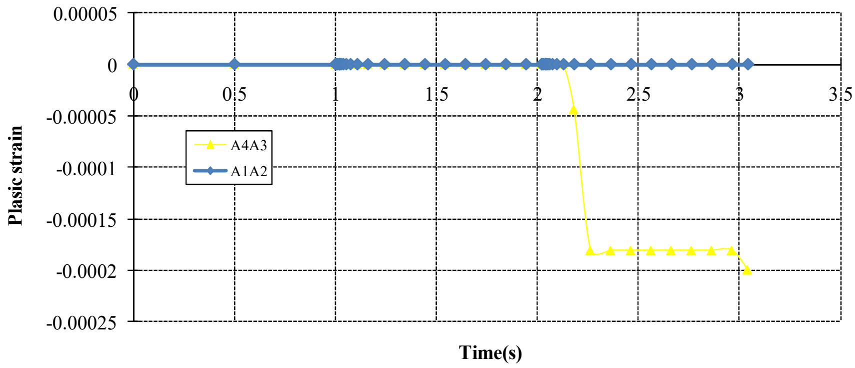

Fig.31 Plastic strain due to axial force of Beam A4A3 and A1A2 at ground level of Case 1

TABLE

Table 1 different column removal scenarios

	Level of removal	**First column**	**Second column or bracing**
CASE1	Ground level	A1	A2
CASE2	Level 14	A1	A2
CASE3	Ground level	A1	A2
CASE4	Ground level	A5	Bracing A5A4
CASE5	Level 14	A5	Bracing A5A4
CASE6	Ground Level	A2	A3