Progressive collapse analysis of high-rise building with 3-D finite element modeling method

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Abstract

Using the general purpose finite element package ABAQUS, 3-D finite element a model representing 20 storey buildings were first built by the authors to perform the progressive collapse analysis. Shell elements and beam elements were used to simulate the whole building incorporating nonlinear material characteristics and non-linear geometric behavior. The modeling techniques were described in detail. Numerical results are compared with the experimental data and good agreement is obtained. Using this model, the structural behavior of the building under the sudden loss of columns for different structural systems and different scenarios of column removal were assessed in detail. The models accurately displayed the overall behavior of the 20 storey buildings under the sudden loss of columns, which provided important information for the additional design guidance on progressive collapse.

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

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1 INTRODUCTION

Progressive collapse first attracted the attention of engineers from the structural failure of a 22-story apartment building at Ronan Point, London, UK, in 1968. The terminology of progressive collapse is defined 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’’ [1]. After the event of 11 September 2001, more and more researchers have started to refocus on the causes of progressive collapse in building structures, seeking ultimately the establishment of rational methods for the assessment and enhancement of structural robustness under extreme accidental events. The UK Building Regulations [2] has led with requirements for the avoidance of disproportionate collapse. These requirements, which are refined in material-specific design codes (e.g. BS5950 [3] for structural steelwork), can be described as (i) prescriptive ‘tying force’ provisions which are deemed sufficient for the avoidance of disproportionate collapse (ii) ‘notional member removal’ provisions which need only be considered if the tying force requirements could not be satisfied, and (iii) ‘key element’ provisions applied to members whose notional removal causes damage exceeding prescribed limits. In the United States the Department of Defense (DoD) [4] and the General Services Administration (GSA) [5] provide detailed information and guidelines regarding methodologies to resist progressive collapse of building structures. Both employ the alternate path method (APM) to ensure that structural systems have adequate resistance to progressive collapse. APM is a threat independent methodology, meaning that it does not consider the type of triggering event, but rather, considers building system response after the triggering event has destroyed critical structural members. If one component fails, alternate paths are available for the load and a general collapse does not occur. 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. The technique can be used for the design of new buildings or for checking the capacity of an existing structure.

Izzuddin et al [6][7], proposed a novel simplified framework for progressive collapse assessment of multi-storey buildings, considering sudden column loss as a design scenario. It analyzed the nonlinear static response with dynamic effects evaluated in a simple method. It offers a practical method for assessing structural robustness at various levels of structural idealization, and importantly it takes the debate on the factors influencing robustness away from the generalities towards the quantifiable. Kim et al [8] studied the progressive collapse-resisting capacity of steel moment resisting frames using alternate path methods recommended in the GSA and DoD guidelines. The linear static and nonlinear dynamic analysis procedures were carried out for comparison. 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. Khandelwal et al [9] studied the progressive collapse resistance of seismically designed steel braced frames with validated two dimensional models. Two types of braced systems are considered: namely, special concentrically braced frames and eccentrically braced frames. The study is conducted on previously designed 10-story prototype buildings by applying the alternate path method. The simulation results show that while both systems benefit from placement of the seismically designed frames on the perimeter of the building, the eccentrically braced frame is less vulnerable to progressive collapse than the special concentrically braced frame. Paik et al [10] investigated the possibility of progressive collapse of a cold-formed steel framed structure. Five different analysis cases were considered. The results showed that the removal of corner wall columns appeared to cause progressive collapse of a portion of the second and third floor of the end bay directly associated with the column removal, and not the entire building. Tsai et al [11] conducted the progressive collapse analysis following the linear static analysis procedure recommended by the US General Service Administration (GSA). Using the commercial program SAP2000, the potential of an earthquake-resistant RC building for progressive collapse is evaluated. However, no validation of the accuracy of the SAP2000 model is presented.

Although there are some design guidances as shown above, some major shortcomings have been recognized by the researchers. As pointed out by Izzuddin et al [6][7], the prescriptive nature of the tying force requirements, deemed sufficient for the avoidance of disproportionate collapse yet unrelated to real structural performance, and the exclusion of ductility considerations at all levels of the provisions made the provisions unsafe. On the other hand, the alternative notional member removal provisions are more performance-based, but these are applied with conventional design checks, and hence they ignore the beneficial effects of such nonlinear phenomena as compressive arching and catenary actions. This in turn can lead to the prediction of an unrealistically large damage area exceeding the prescribed limits, thus forcing the member to be designed as a key element when this may be unnecessary. Therefore, more detailed research toward the progressive collapse of multi-storey building is timely. However, as mentioned above, the research on the behavior of the progressive collapse of high rise building is quite limited due to the limited research tools. Full scale test of this type of problem is not possible due to its high cost. A 3-D Finite element model is definitely a best option. However, due to the geometric complexity of multi-storey building and poor preprocessing functions of current general purpose finite element packages, no full scale 3-D finite element model for investigating progressive collapse has been built so far there is also little research toward the modeling of the structural behavior of multi-story buildings under sudden column loss. Most of the modeling techniques mentioned in section 1 are either the simplified models based on current design guidance or two dimensional models, which could not accurately monitor the overall structural behavior of the whole building.

In this paper, using the general purpose finite element package ABAQUS [12], a 3-D model is first developed by the authors which enables the nonlinear progressive collapse analysis of high rise building. Two 3-D finite element models representing 20 storey buildings with different structural forms were built to perform the progressive collapse analysis. The models accurately displayed the overall behavior of the 20 storey buildings under sudden loss of columns, which provided important information for additional design guidance on progressive collapse.

2 FINITE ELEMENT MODEL TECHNIQUE

2.1 *Preprocessing method*

In order to accurately monitoring the structural behaviors of the high-rise building for progressive collapse, it is useful to build up a 3-D full scale finite element model. However, the preprocessing function of all the general purpose programs such as ABAQUS and ANSYS used in the current research is quite limited. Therefore, it is difficult to set up a multi-story building model due to the geometric complexity of the structures. The geometric shape of the model becomes a key issue of the modeling approach. To solve this problem, the multi-story building analysis program ETABS [13] was used to set up the whole model geometrically. ETABS is a leading high-rise building analysis program. It is easier to set up a high-rise building and mesh the whole structure with the preprocessing function of ETABS. However, due to its shortcomings, for example it could not simulate the cracking of the concrete slab; ETABS is not suitable to perform the progressive collapse analysis. Therefore, the output file of the ETABS model is converted into ABAQUS input file using the converter program designed by the authors with Visual Basic Language. The converter program can transform exactly all the information of the ETABS model such as the beam orientation, beam size, shell thickness, shell orientation, moment release, constrain equation and the mesh of the shell etc into ABAQUS model. The typical floor layout of ETABS model is shown in Fig. 1.

2.2 Model setup

As shown in Fig 2 and 3, two three-dimensional finite element models were created using the ABAQUS package to conduct the progressive collapse study of the high-rise building. The models replicated the 20 storey buildings with the grid space of 7.5m in both directions. The floor height is 3 m for each floor. Two different types of lateral stability systems were simulated, which are the typical lateral systems used in the current design practice. One is provided by the shear walls of 450mm thick as shown in Fig.2 and Fig.3. The gravity load is resisted by both the shear wall and steel frame. Another model, with identical grid and level height, is also built as shown in Fig 4. For this model, the main lateral stability is provided by cross bracing also shown in Fig.4. For both models, the slab thickness are 130mm, the columns are British universal column UC356X406X634 from ground floor to level 6, UC356X406X467 for level 7to level 13, UC356X406X287 for level 14 to level 19, all the beams are British universal beam UB533X210X92. The cross bracings are British circular Hollow section CHCF 273X12.5. This model simulated the full structural framing (primary and secondary beam layout), columns, core supports with full composite action of the composite slab. The dimensions and properties of above steel sections are listed in Table1.

2.3 Elements used for simulation

All the beams and columns are simulated using *BEAM elements in the ABAQUS element library. The beam properties are input by defining the relevant cross-sectional shape from the pre-defined ABAQUS cross-section library. At each increment of the analysis the stress over the cross-section is numerically integrated to define the beams response as the analysis proceeds. This allows the analysis to follow the development of the full elastic-plastic behaviour of the section at each integration point along the beam. The slab and core wall are simulated using the four node *Shell element having bending and membrane stiffness terms available from the ABAQUS library. Reinforcement was represented in each shell element by defining the area of reinforcement at the appropriate depth of the cross-section using the *REBAR element from the ABAQUS library. The main reinforcement included is the A252 mesh assumed to act 30 mm from the top of the slab and the 0.9mm thick metal deck at the bottom of the slab. This reinforcement is defined in both slab directions and was assumed to act as a smeared layer. The reinforcement was modeled using the same elastic-plastic model as the main structural steel members.

The structural beam elements are modelled close to the centreline of the main beam elements and the composite slab is modelled using shell elements at the centreline of the slab. The beam and shell elements are then coupled together using rigid beam constraint equations to give the composite action between the beam elements and the concrete slab. 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. Therefore, the beam to column connection is more or less like a semi-rigid composite connection which is to simulate the characteristic of the connections in normal construction practice.

2.4 Materials model of steel

The model also incorporates nonlinear material characteristics and non-linear geometric behaviour. 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 stress-strains relationship in compression and tension are assumed to be the same in ABAQUS. 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. Any number of points can be used. Therefore, it is possible to obtain a close approximation of 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 elastic part of the stress-strain curve is defined with the *ELASTIC option, the value 2.06×10^5 N/mm^2 for the Young's modulus and 0.3 for Poisson's ratio were used. The plastic part of the stress-strain curve is defined with the *PLASTIC option. Steel grade S355 was used for all the structural steel. The yield stress of 460 N/mm^2 is used for the material of steel rebar. Engineering stresses and strains including the yield and ultimate strength obtained from the coupon tests of Fu et al [14] were converted into true stresses and strains with appropriate input format for ABAQUS.

2.5 Materials model of concrete

The concrete was modelled using a concrete damage plasticity model from ABAQUS. The concrete compressive strength was assumed to be a nominal 30 N/mm^2. The compressive yielding curve was taken as that of a typical concrete from [15]. The tensile cracking stress was assumed to be, conservatively, approximately 5.6% of the peak compressive stress as recommended in [16]. After tensile cracking, the stress-strain relationship in tension softens as load is assumed to be transferred to the reinforcement. The tensile strength of the concrete is ignored after concrete cracking. The shell elements are integrated at 9 points across the section to ensure that the concrete cracking behaviour is correctly captured.

2.6 Boundary condition and mesh size

The models are supported at the bottom as shown in Fig.2 and Fig.4. 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.

3 VALIDATION OF THE MODEL

In order to valid the proposed model, a two-storey composite steel frame model was built up by the authors as shown in Fig. 5 and 6. The model replicated the full scale testing of a steel-concrete composite frame by Wang et al [17] as shown in Fig 7. The model was set up based on the same modelling techniques discussed in part 2 of this paper. The frame size, slab thickness and boundary conditions are exactly the same as the full sale tests [17]. The section sizes are shown in Fig.8 and the section properties are shown in Table 2. For the proposed model, the bottom of the column was defined as fixed. The same material properties of the test were defined using the *Material function of ABAQUS for both steel and concrete. As it is shown in Fig.6, the static concentrate load was applied at the same location as in the full scale tests as shown and Fig.9. The load was applied using the modified RIKS method available in ABAQUS which is to simulate the loading procedure during the full scale test. The modified RIKS method assumed that the loading is proportional—that is, that all load magnitudes vary with a single scalar parameter. In addition, it assumes that the response is reasonably smooth—that sudden bifurcations do not occur.

Fig. 10 shows the modelling results of moment rotation relationship of Frame A, Joint1, compared with Fig 11, which is the moment rotation relationship of the full scale test of [17]. It can be seen that, good agreement is achieved in the initial stiffness and yield strength. However, for the proposed model, it can predict less rotation capacity after yielding. As discussed by Fu et al [18], this is due to the accuracy of the concrete smeared model of ABAQUS to simulate the concrete behaviour after cracking. As we are more concerned with the overall behaviour of the building, so this model is accurate enough to conduct the progressive collapse analysis.

4 COLUMN REMOVAL ANALYSIS

As stated in section 1, the alternate path method (APM) which is proposed by DOD [4] and GSA [5] is applied here to perform the progressive collapse checking of the existing 20 storey buildings. As stated in DOD [4] and GSA [5], the methodology is generally applied in the context of a 'missing column' scenario to assess the potential for progressive collapse. Under extreme events, such as blast and impact, the dynamic influences are event-independent. Sudden column loss represents a more appropriate design scenario, which includes the dynamic influence yet is event-independent. Although such a scenario is not identical in dynamic effect to column damage resulting from impact or blast, it does capture the influence of column failure occurring over a short duration relatively to the response time of the structure. It can also be applied to various other extreme dynamic events via calibrated design factors. It is therefore sudden column loss is used as the principal design scenario in [4, 5]. This method is also adopted by all the researchers introduced in section 1. Moreover, for designers, the most important issue is to check if a building can successfully absorb the loss of a critical column and prevent progressive collapse. Therefore, the ability of the building under sudden column loss is assessed here using nonlinear dynamic analysis method with 3-D finite element technique.

The loads are computed as dead loads (which is the self-weight of the floor) plus 25% of the live load (which is 2.5kN/m^2).This is determined from the nonlinear dynamic analysis for comparison with the acceptance criteria outlined in Table 2.1 of the GSA guidelines [5].

Column removal is conducted by *Remove command from ABAQUS library. The dynamic response was recording use *Dynamic command from ABAQUS library. The columns to be removed are forcibly removed by instantaneously deleting them, and the subsequent response of each braced frame is then investigated. The maximum forces, displacements and rotations for each of the members/connections involved in the scenario are recorded. The fundamental frequency of a floor plate with a number of columns removed can be expected to be in the region of 1 to 5 Hz. This means a potential period of between 0.2 and 1 second. Therefore, the column is removed over a period of 20 milliseconds. The analysis is then run for a further 2 seconds to ensure that the structure has stabilised. Due to the expected material nonlinearity, the minimum time increment can be of the order of microseconds which means that potentially 1000's of increments will be needed to complete the analysis. Therefore, the model developed needs to be as efficient as possible to enable realistic runtimes.

The simulations are conducted with 5 % mass proportional damping. Table 3 shows the list of analysis cases considered in this study together with the members that are forcibly removed in each case. To facilitate the following discussion, the columns and beams are designated as follows according to the grid line shown in Fig1. For instance, Column C1 stands for the column at the junction of grid C and grid 1. Beam E1-D1 stand for the beam on grid 1 starting from grid E to grid D.

4.1 Case 1- Column C1 & D1 at ground floor removed

For case 1, when the two columns C1 and D1 as shown in Fig.12 were suddenly removed (Case 1 in Table 1), the node on the top of the removed column vibrated and substantially reached a peak vertical displacement of 25 mm. The response eventually rest at 17mm as shown in Fig. 13. The adjacent beam and column were initially overloaded and started deforming inelastically. A large redistribution of forces was observed to take place as shown in Fig. 14 the force in column E1 doubled from 2500 kN to a peak of 4500 kN before settling down at a steady value of 3700 kN. The peak force of the beam was accompanied the column force. Concurrently, as shown in Fig. 15, the force in beam B1-C1 increased to a peak value of 560 kN before settling down at a steady value of 400 kN. Other frame members remained in the elastic condition.

4.2 Case 2-Column A1 at ground floor removed

Similar to case 1, when the column A1 at the ground level as shown in Fig.16 was suddenly removed (Case 2 in Table 1), the node corresponding to the top of the removed column vibrated and reached a peak vertical displacement of 55 mm. The response eventually comes to rest at 40mm as shown in Fig. 17. The adjacent beam and column were also initially overloaded and started to deform inelastically. A large redistribution of forces was observed to take place as shown in Fig. 18. The force in column B1 doubled from 2000 kN to a peak of 3600 kN then settled down at a steady value of 3100 kN. The peak force in the beam was accompanied by the column force. As shown in Fig.19, Concurrently, the force in beam B1-A1 increased to a peak value of 400 kN with a steady value of 300 kN. Other frame members remained in the elastic region.

4.3 Case 3-Column A1 Level 14 removed

For case 3, when the column A1 at level 14 as shown in Fig.20 was suddenly removed (Case 3 in Table 1), the node of the top of the removed column vibrated substantially reaching a peak vertical displacement of 60 mm. The response eventually comes to rest at 45mm as shown in Fig. 21. The adjacent beam and column were initially overloaded and started deforming inelastically. A large redistribution of forces was observed to take place as shown in Fig. 22. The force in column B1 at the ground level increased from 2000 kN to a peak of 2700 kN before settling down at a steady value of 2500 kN. As it is shown in Fig.23, the force in column B1 at level 14 increased from 600 kN to a peak of 1100 kN with a steady value of 800 kN. The peak axial force of the beam was accompanied with the increases in the column force. Concurrently, as shown in Fig.24, the force in beam B1-A1 at level 14 increased to a peak value of 310 kN before settling down at a steady value of 200 kN. Other frame members remained elastic.

4.4 Case 4- Column A1 at ground floor removed

In case 4, when the column A1 at ground level as shown in Fig.25 was suddenly removed (Case 4 in Table 1), The node of the top of the removed column starts to vibrate and finally reached a peak vertical displacement of 55 mm. The response eventually damped out to rest at 45mm as shown in Fig. 26. The adjacent beam and column were initially overloaded and started deforming inelastically. A large redistribution of forces was observed to take place as shown in Fig. 27. The force in column B1 at the ground level increased from 2000 kN to a peak of 3500 kN before settling down at a steady value of 3100 kN. The peak force of the beam was accompanied by the increases in column force. Concurrently, as it is shown in Fig.28 the force in beam B1-A1 at ground level increased to a peak value of 400 kN before settling down at a steady value of 310 kN. As it is shown in Fig.29, the axial forces in the bracing member were also changed concurrently due to the removal of the column.

4.5 Case 5 -Column A1 Level 14 removed

For the building with the cross bracing lateral resistance system, when the column A1 at the level 14 as shown in Fig.30 was suddenly removed (Case 5 in Table 1), the node corresponding to the top of the removed column vibrates substantially reaching a peak vertical displacement of 60 mm. The response eventually rested at 45mm as shown in Fig. 31. A large redistribution of forces is observed to take place as shown in Fig. 32. The force in column B1 at the ground level increased from 2000 kN to a peak of 2700 kN before settling down at a steady value of 2500 kN. As shown in Fig.33, the force in column B1 at level 14 increased from 600 kN to a peak of 1100 kN before settling down at a steady value of 800 kN. The peak force of the beam is accompanied the peak force in the column. Concurrently, as it is shown in Fig.34 the force in beam B1-A1 on level 4 increased to a peak value of 310 kN before settling down at a steady value of 200 kN. Other frame members remained in the elastic region.

4.6 Analysis result discussion

From the comparison of case 1 and case 2, it can be seen that the building is more vulnerable to the removal of two columns. This is due to the larger affected loading area after the column removal which also determines the amount of energy needed to be absorbed by the remaining building. From the comparison of case 2 with case 4 and case 3 with case 5, it can be seen that, for the two different lateral resistance systems, the dynamic response of beams and columns are almost identical. This is because that the structural grids of these two types of building are the same, thus, the affected loading area is the same. It is can be concluded that, the response of the structure is only related to the affected loading area after the column removal, which also determines the amount of energy need to be absorbed by the remaining building.

For the structural members like the beam, column and brace, it can be seen that after the sudden removal of the column, the axial forces are more or less doubled. As the load combination used in the analysis is 1.0DL+0.25LL which is suggested in GSA guidelines [5]. So, the members at the same floor level are suggested to be designed to have the axial capacity of at least twice the static axial force of the member under the 1.0DL+0.25LL load combination to prevent potential progressive collapse. For instance, in this paper, all the sections used in the models of all the analysis cases, such as the ground floor column UC356X406X634 with the capacity of 24500 kN, are substantially more than the peek load computed so they are not overloaded. Therefore no progressive collapse occurred.

At the mean time, also for the design of the structures to ensure resistance of the progressive collapse, the beam to column connections at the column removal level should also have enough capacity to resist the increases in axial force from the beam due to the column loss. As the peak axial force in the beams doubled from the static state after the sudden removal of the column, so the capacity of the connection should be at least twice the static axial force of the beam under 1.0DL+0.25LL load combination as well.

For both of the two different lateral stability system buildings, the comparison of the two scenarios-one corner column removal at ground level and are at level 14, suggested that the column removal in level 14 has a larger vertical displacement than the column removal at ground floor. This is because, for the column removed at the ground floor, more floors participated in absorbing the released energy than that occurred with the other scenarios.

5 CONCLUSIONS

In this paper, a 3-D finite element model was first built with the ABAQUS package to simulate the behaviour of multi-storey buildings under sudden column removal. The methodology for the modelling techniques is described in details. The model also incorporates nonlinear material characteristics and non-linear geometric behaviour.

A two storey model was built for the validation of the proposed modelling method. The numerical results are presented and compared to experimental data and good agreement is obtained. The proposed 3-D FE model can accurately represent all the main structural behaviour of the multi-story buildings. It offers a reliable and very cost-effective alternative to laboratory testing as a way of generating results. Using the proposed model, the progressive collapse analysis of two 20 storey buildings with different lateral stability systems was conducted under different damage scenario.

Below are main findings:

1. The dynamic response of the structure is mainly related to the affected loading area after the column removal, which also determines the amount of energy need to be absorbed by the building.

2. All the structural members at the possible column removal level should also be designed to at least twice the static axial force obtained when applying the 1.0DL+0.25LL load combination.

3. The beam to column connection at the column removal level should also be designed to have at least twice the static axial force acting on the connections under the 1.0DL+0.25LL load combination.

4. Under the same general conditions, a column removal at a higher level will induce larger vertical displacement than a column removal at ground level.

The research toward the progressive collapse presented is still in its infancy. Further work will be concentrated on more parametric studies to exam detailed structural behaviour, and the impact of blast forces on the dynamic structural behaviours.

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FIGURES

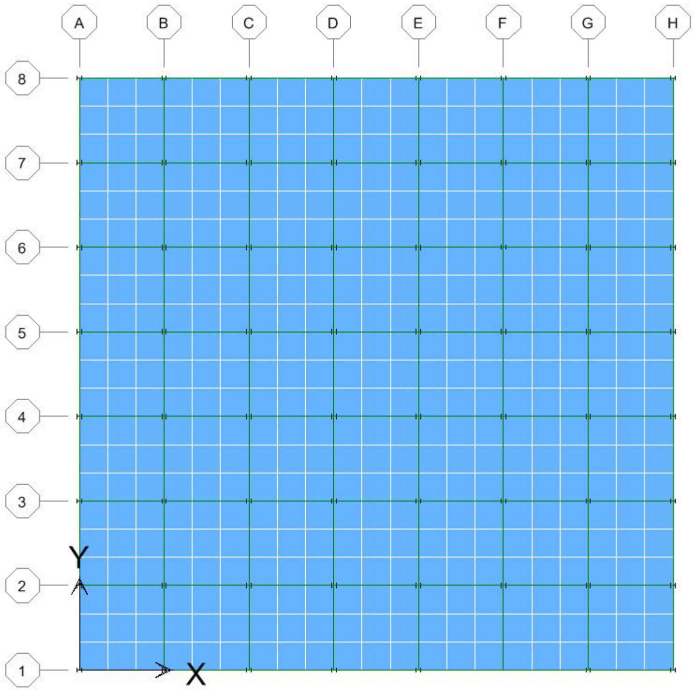


Fig 1 Typical plan layout of the ETABS model

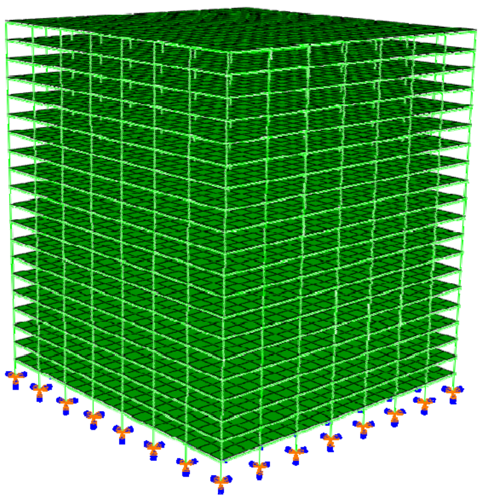


Fig 2 The analysis model using core walls as lateral bracing system

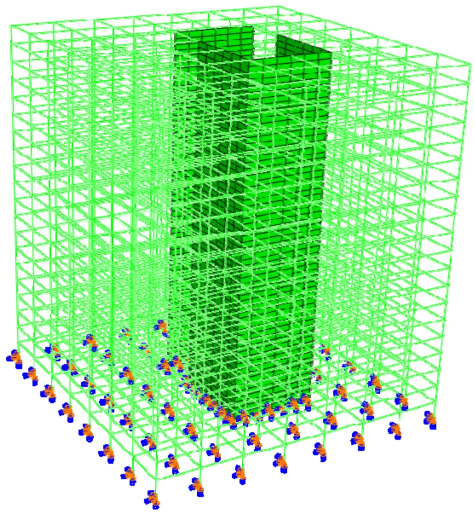


Fig 3 The analysis model using core walls as lateral bracing system without the slab showing

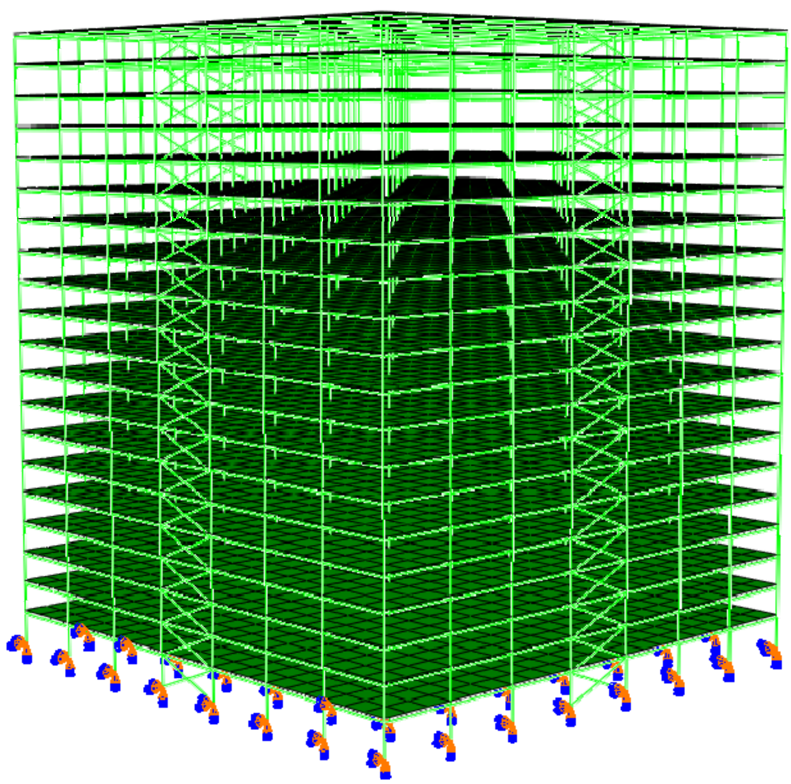


Fig 4 The analysis model using braces as lateral bracing

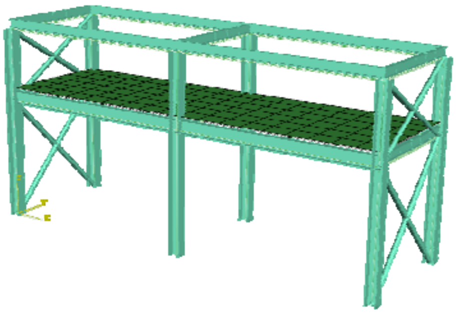


Fig 5 The numerical validation model using ABAQUS

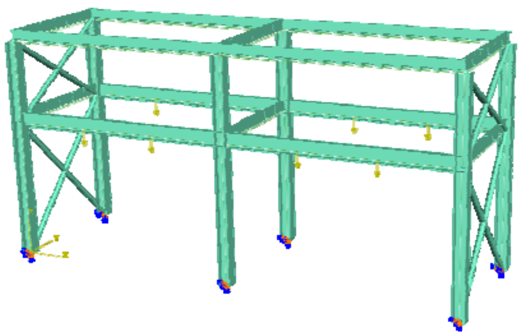


Fig 6 Load and the boundary condition of the numerical model

Fig 7 Full scale frame test by Wang et al [17]

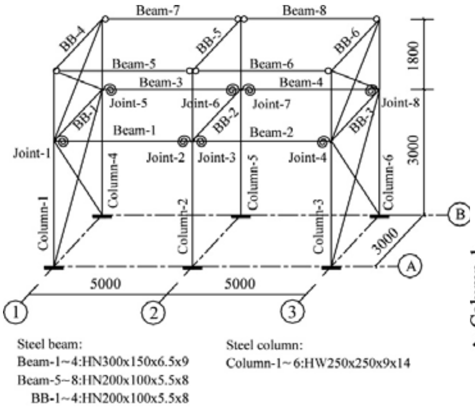


Fig. 8 Frame size and dimension of the full scale frame test by Wang et al [17]

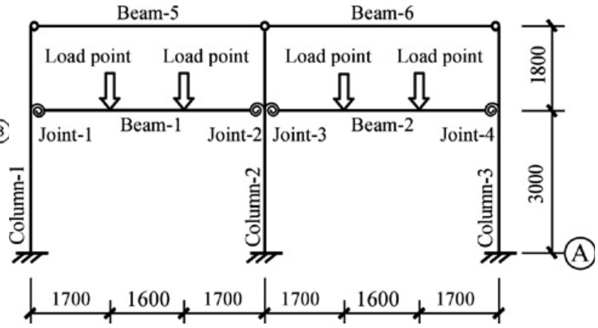


Fig. 9 Load location of the Frame A from the full scale frame test by Wang et al [17]

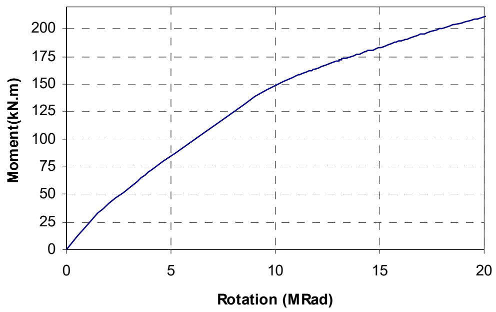


Fig 10 Moment rotation relationship of Frame A Joint1 Of proposed model

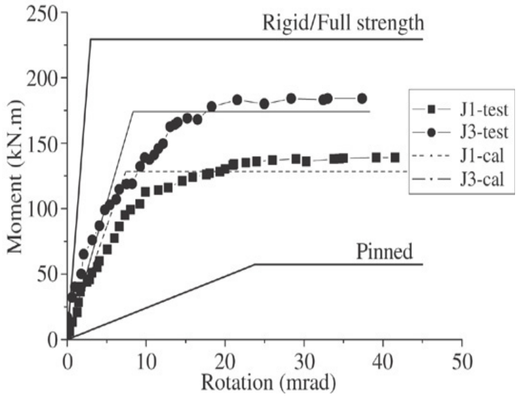


Fig. 11 Moment–rotation curves of test [17]

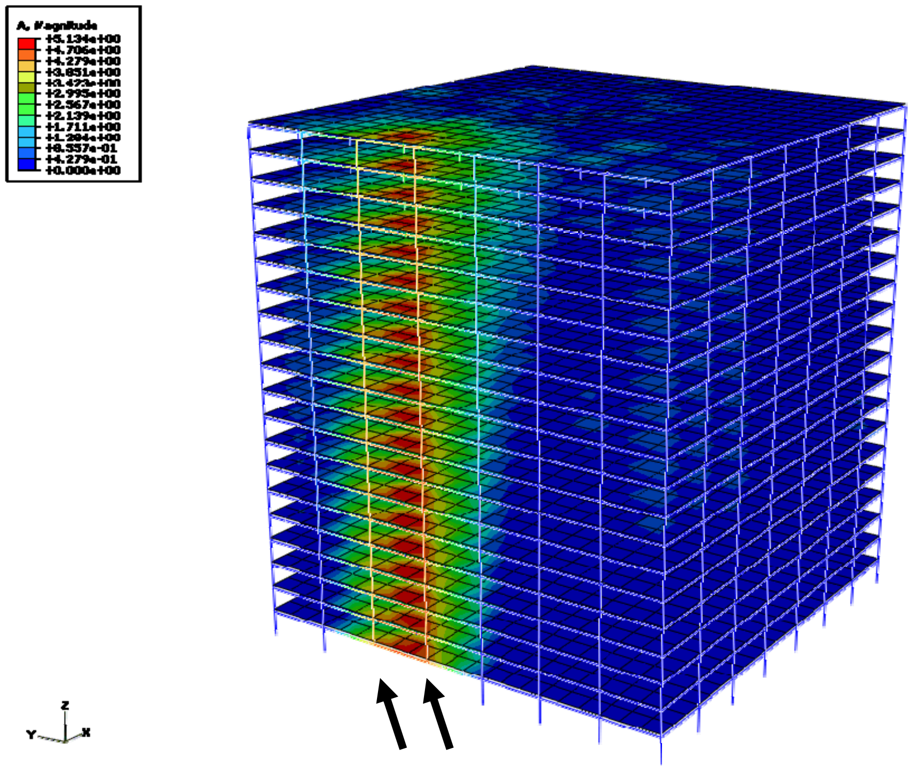


Fig. 12 Acceleration contour of Case 1

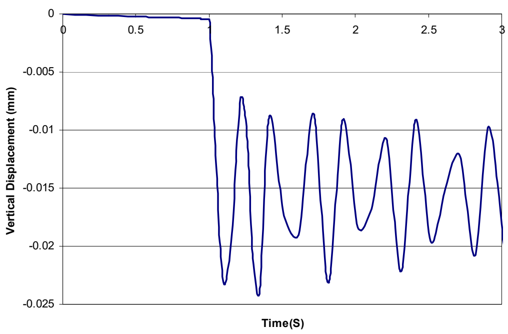


Fig .13 Displacement of the node above column C1

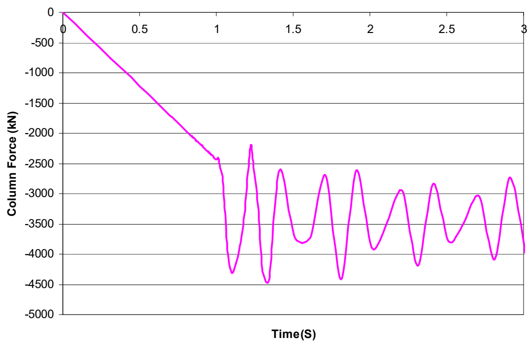


Fig .14 Axial force of column E1 in the ground floor

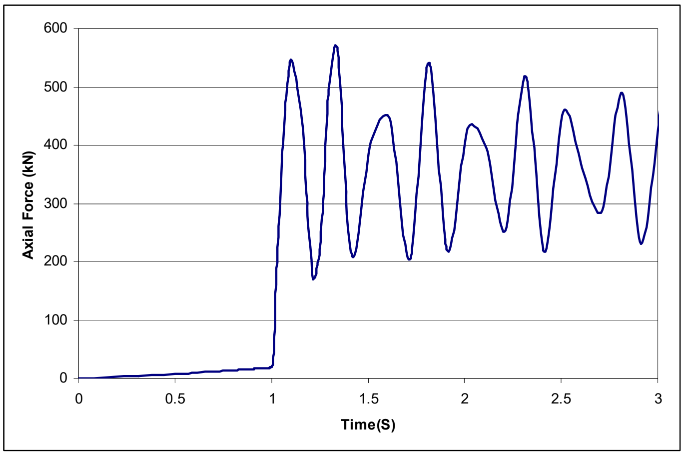


Fig.15 The tying force of the beam B1-C1 of ground floor

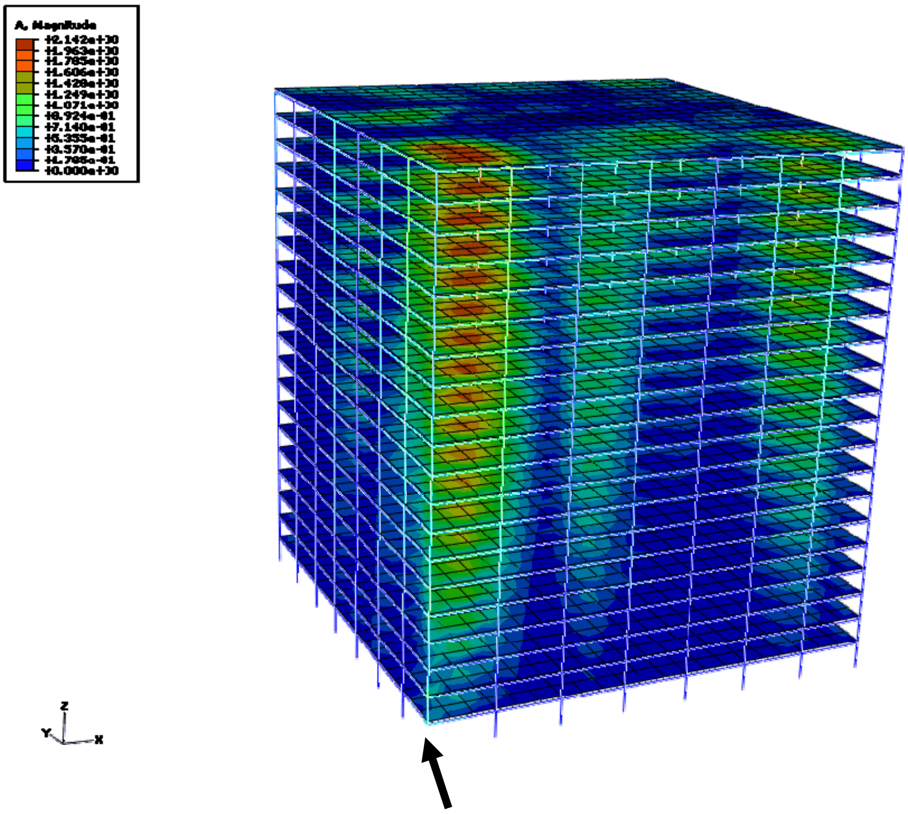
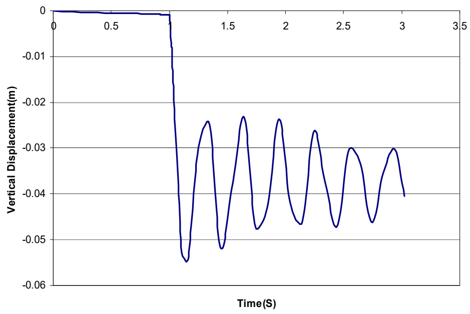


Fig. 16 Acceleration contour of Case 2



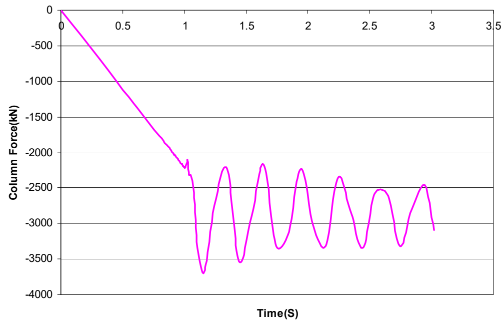


Fig.17 Displacement of the node above the removed column Fig.18 Column force in column B1 at ground level

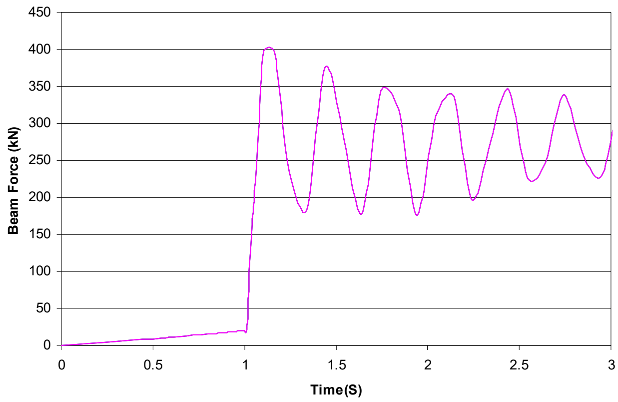


Fig.19 Axial force in the beam B1-A1at ground level

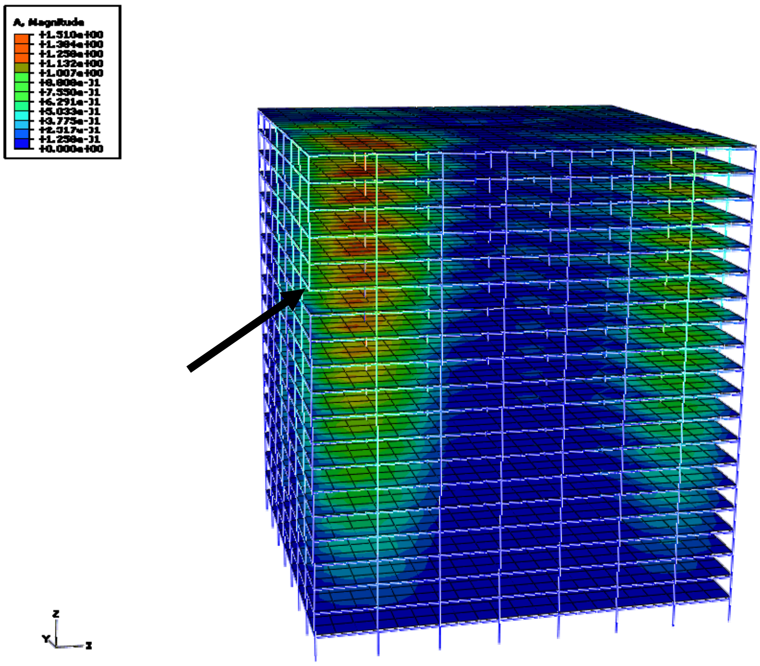


Fig 20 Acceleration contour of Case 3

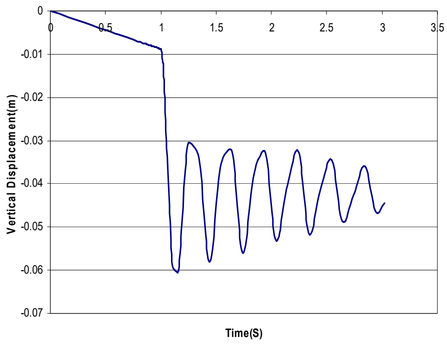


Fig.21 Displacement of the node above the removed column

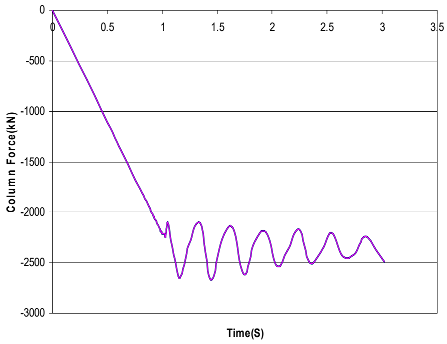


Fig.22 Axial force in column B1 at ground level

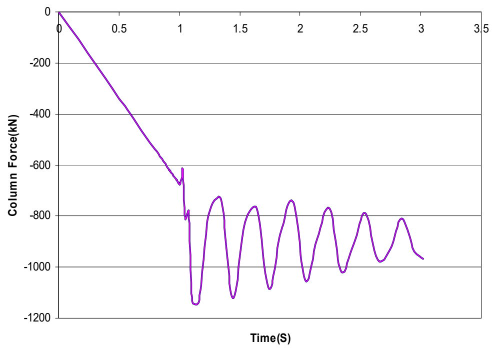


Fig.23 Axial force in column B1 at level 14

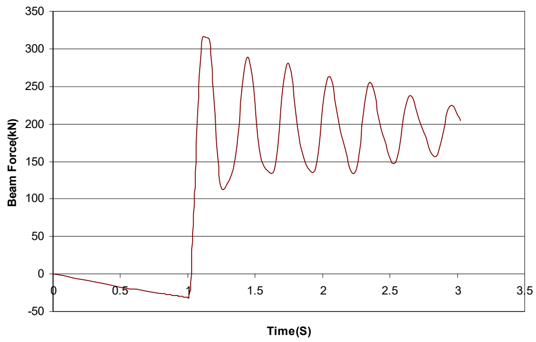


Fig.24 Axial force in beam B1-A1 at level14

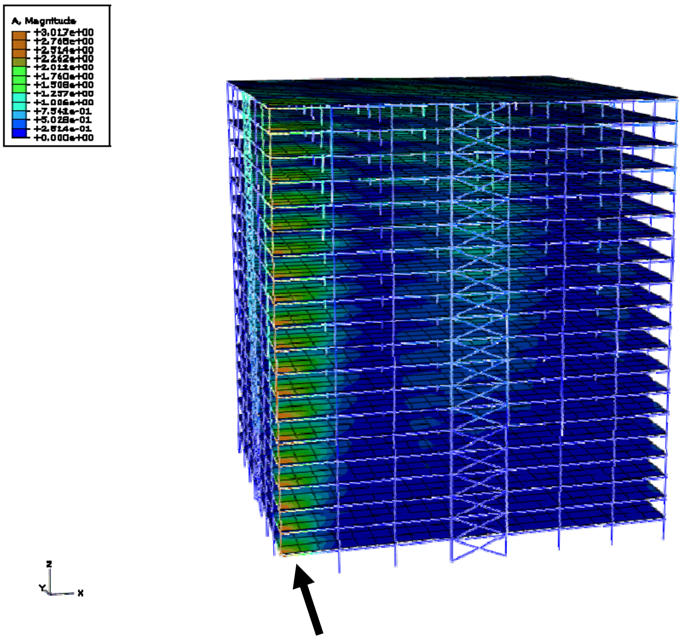
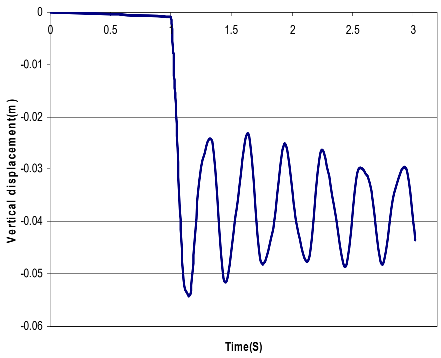


Fig .25 Acceleration contour of Case 4



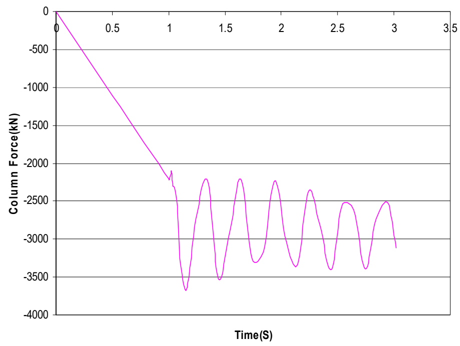
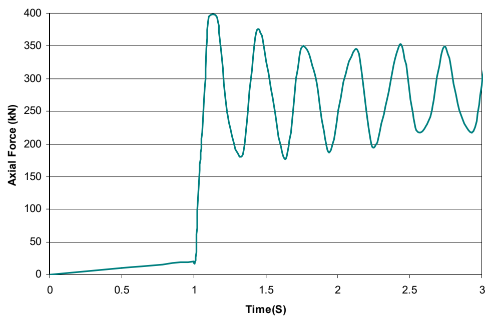


Fig.26 Displacement of the node above the removed column Fig.27 Axial force in column B1 at ground level



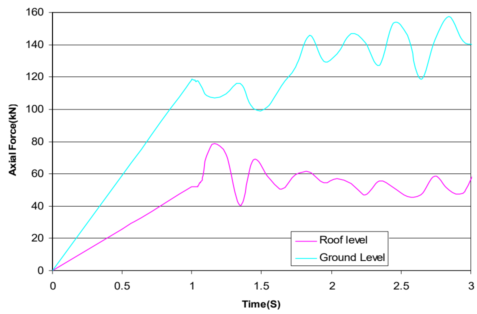


Fig.28 Axial force in the beam B1-A1 at ground level Fig.29 Axial force in the brace

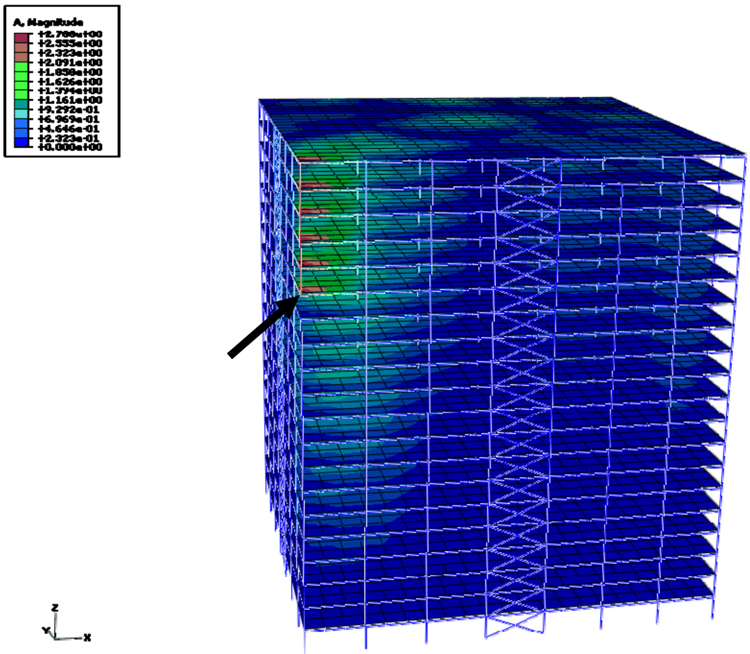


Fig. 30 Acceleration contour of Case 5

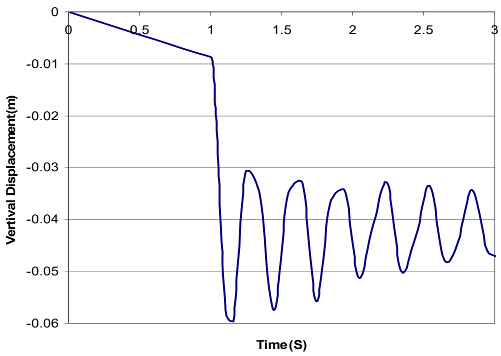


Fig.31 Displacement of the node above the removed column

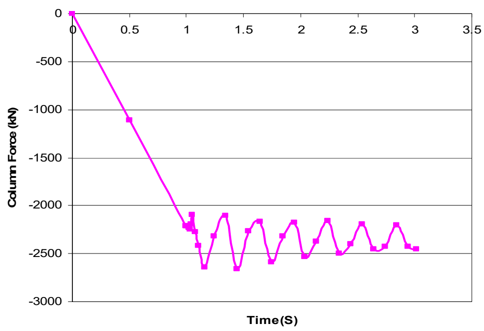


Fig.32 Column B1 force at level 14

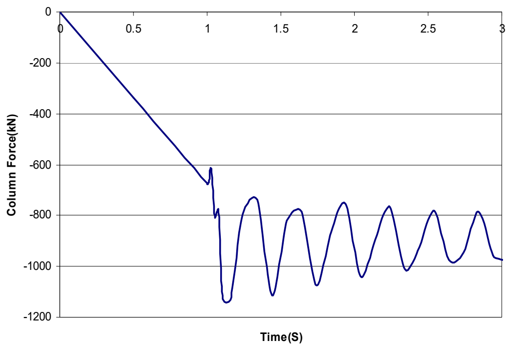


Fig. 33 Column force in column B1 at ground level

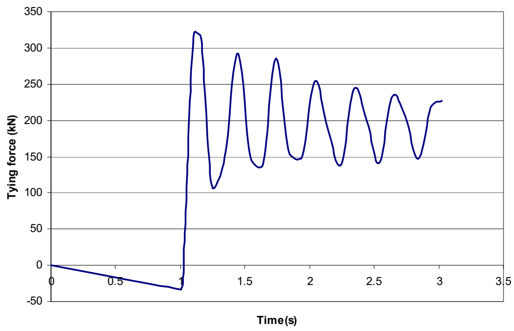


Fig.34 Axial force in beam B1-A1