3-D Nonlinear Dynamic progressive collapse Analysis of multi-storey steel composite frame buildings – parametric study

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Abstract

A 3-dimensional finite element model built by the author was used in this paper to analyze the progressive collapse of multi-storey steel composite frame building. The proposed model can represent the global 3-D behavior of the multi-storey building under the sudden column removal. Based on this model, parametric studies were carried out to investigate the structural behavior with variations in: strength of structural steel, strength of concrete and reinforcement mesh size. Through the parametric study, the measures to mitigate progressive collapse in the future design were recommended.

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

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1 INTRODUCTION

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 started to refocus on the causes of progressive collapse in building structures. There are design procedures to mitigate the potential for progressive collapse in the design guidance of UK and US. In the United States the Department of Defense (DoD) [2] and the General Services Administration (GSA) [3] provide detailed information and guidelines regarding methodologies to resist progressive collapse of building structures. Both employ the alternate path method (APM). 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. In U.K., The UK Building Regulations [4] and BS5950 [5] has led with requirements for the avoidance of disproportionate collapse. FEMA 2002 [6] and NIST 2005 [7] also provide some general design recommendations, which require Steel-framed structural systems have enough redundancy and resilience, such that alternative load paths and additional capacity are provided for redistributing gravity loads when structural damage occurs. Perimeter columns and floor framing in particular should have greater mass to enhance thermal and buckling resistance.

In recent study, there are some experimental and analytical studies on the progressive collapse behaviors of buildings under the missing column scenario. Khandelwal et al [8] studied the progressive collapse resistance of seismically designed steel braced frames with validated two dimensional models. Two types of braced systems are considered: special concentrically braced frames and eccentrically braced frames. The simulation results show that the eccentrically braced frame is less vulnerable to progressive collapse than the special concentrically braced frame. Izzuddin et al [9][10], proposed a simplified framework for progressive collapse assessment of multi-storey buildings with sudden column loss scenario. It analyzed the nonlinear static response with dynamic effects evaluated in a simple method. Kim et al [11] 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. Paik et al [12] investigated the possibility of progressive collapse of a cold-formed steel framed structure. 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. 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 .Yu et al [14] proposed a simplified model to perform progressive collapse analysis, parametric study were also conducted using their model.

Although there are some research has been done as mentioned above, they all based on bare steel frames without considering the contribution of the floor systems. Most of them are 2-D models. Therefore, it is unrelated to real structural performance. Without considering the contribution of the slabs, the beneficial effects of such as compressive arching and catenaries actions are not clear. This will lead to the prediction of an unrealistically large damage area exceeding the prescribed limits. Therefore, more detailed research on the progressive collapse of multi-storey building is timely. Using ABAQUS [15], Fu [16] proposed a full scale 3-D finite element model to investigate the progressive collapse of multi-storey building. Research is conducted in different column removal scenarios. Compared with two dimensional models, it provides accurate structural behavior of multi-storey buildings under different sudden columns removal scenarios

In this paper, using the 3-D finite element modeling techniques developed by the Fu [16], 3-D finite element models representing 20 storey composite steel frame buildings with bracing system were built to perform the progressive collapse analysis. Based on these models, parametric studies were carried out to investigate the structural behaviors of this type of buildings. Through the parametric study, the measures to mitigate progressive collapse in the future design were also recommended , which provided important information for additional design guidance on progressive collapse.

2 3D FINITE ELEMENT MODEL

As shown in Fig 1, a three-dimensional finite element models were created by Fu [16] using the ABAQUS[15] package to conduct the progressive collapse study of the high-rise building. The model replicates the 20 storey building with the grid space of 7.5m in both directions as it is shown in Fig 2. The floor height is 3 m for each floor. The main lateral stability is provided by cross bracing also shown in Fig.1. 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 UB305X102X25. The cross bracings are British circular Hollow section CHCF 273X12.5. This model simulated the full structural framing of the typical high-rise buildings in the current construction industry with full composite action of the composite slab.

2.1 3D finite element Modeling technique

All the beams and columns are simulated using *BEAM elements. The structural beam elements are modelled close to the centreline of the main beam elements. The slab are simulated using the four node *Shell element. 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. This reinforcement is defined in both slab directions and was assumed to act as a smeared layer. 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 material properties of all the structural steel components were modelled using an elastic-plastic material model from ABAQUS [15] which incorporates the material nonlinearity. The concrete material was modelled using a concrete damage plasticity model from ABAQUS [15]. 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. The models are supported at the bottom as shown in Fig.1. 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. 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. Detailed modelling techniques were explained in Fu [16].

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 for each of the members involved in the scenario are recorded. The column is removed over a period of 20 milliseconds with requirement of GSA[3]. The simulations are conducted with 5 % mass proportional damping.

2.2 Validation of the model

In order to valid the proposed model, in Fu [16], a two storey composite steel frame ABAQUS model was built. The model replicated the full scale testing of a steel-concrete composite frame by Wang et al [17]. Comparison between the tests result and the modelling result is made. Good agreement is achieved.

3 PARAMETRIC STUDIES

The alternate path method (APM) is applied here to perform the progressive collapse checking of the existing 20 storey buildings. The resistance 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 in accordance with the acceptance criteria outlined in Table 2.1 of the GSA guidelines [3]. The columns to be removed are forcibly removed by instantaneously deleting them. Table 1 shows the list of analysis cases considered in this study together with the different parameters which were used in each case. To facilitate the following discussion, the columns and beams are named as follows according to the grid line shown in Fig2. 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.

3.1 Effect of strength of Steel structural member ------ one column removal scenario

In order to evaluate the effect of strength, three grades of steel members are chosen, which are S275, S355, S460. The comparison is show in Fig.4. It can be seen that, there is no much difference between these three cases. This is because the inspection of the model shows that no plastic strain was developed in the steel beams, as is it is shown in Fig.5. That means all the beams are still in the elastic stage, so no obvious difference of the response was observed. For the research done so far, most researchers presume that after one column removed; the plastic hinge will form in the beam in their analysis model. The modelling result of this paper shows that this is not always true. The plasticity will depend on the size of the beam, the strength of the beam and the loading. The structure can remains elastic after one column removed.

3.2 Effect of strength of Steel structural member ------ two columns removal scenario

In order to further evaluate the effect of steel strength, two columns removal scenario was investigated, where, two columns A1 and A2 at ground level were suddenly removed, as it is shown in Fig.6 that the plastic strain were developed in the steel beams. The comparison of the response of the models is shown in Fig.7, 8 and 9. It can be seen that, the lower the steel grade, the larger the maximum vertical dynamic deflection was observed. It can be also seen that, the higher the steel grade the higher bending moment and axial force were observed. This is because when plasticity developed in the steel beam high grade steel exhibits higher yielding and strain hardening stress, therefore higher bending moment and axial force. From the results, it can be concluded that, increasing the grade of steel beam will increase the resistance capacity to progressive collapse as the deflection decreased.

3.3 Effect of concrete strength - one column removal scenario

In order to evaluate the effect of concrete strength, three grades of concrete are chosen, which are C30, C40, C60. They are the typical concrete grade used in the current construction practice. The response like vertical displacement, major axis moment and axial force are reordered. The comparison results are shown in Fig. 10 to 12. When the columns A1 as shown in Fig. 3 were suddenly removed, the node on the top of the removed column vibrated and reached a peak vertical displacement and eventually rest at displacement as shown in Fig. 10. The redistribution of forces was observed to take place as shown in Fig. 11 and 12. From Fig.10 it can be seen that, the weaker the concrete strength the greater the maximum vertical dynamic deflection observed. It can be noticed from the Fig11 and 12 that, the weaker the concrete strength, the greater the axial force and bending moment were observed in the steel beam, this is because for lower grade concrete strength, the concrete cracked more early than the higher grade concrete. Therefore, more force is transferred into the steel beams rather than the slabs.

It can be seen that, Increase the concrete strength will increase the resistance to the progressive collapse as the deflection reduced and the overall stiffness of the building is increased. However, the internal force of the steel beam is also increased. It can also be seen that, the tensile strength of the concrete has smaller effect on response of the structural. The reason for this is that the joints and the steel beams have provided sufficient effective tying that prevents large deformation in the floors. This means increasing of the strength in the concrete has only marginal contribution to the effective tying of the system. The similar result has been found in the research of [14] as well.

3.4 Effect of reinforcement mesh - one column removal scenario

In order to evaluate the effect of steel mesh used in the concrete, four types of steel mesh were chosen first, which are A142, A193, A252 and A393 with mesh size as 142mm^2/m, 193mm^2/m, 252mm^2/m and 393mm^2/m respectively. They are the typical mesh size used in the current composite slab design. The comparison results are shown in Fig 13, 14 and 15. It can be seen that, when one column A1 was suddenly removed, the node on the top of the removed column vibrated and substantially reached a peak vertical displacement. The response eventually rest at displacement as shown in Fig. 13. A large redistribution of forces was observed to take place as shown in Fig. 14 and 15. The comparison result shows that, for the conventional steel mesh used in current construction practice, the variation of the deformation is small. This is because for these four convention meshes used in the current construction market, the variation of rebar ratio is small, and force are mainly taken by the steel beams rather than the slab, as it is discussed in section 3.1. Therefore, the difference is not obvious.

In order to clearly investigate the effect of the steel mesh, more steel meshes are investigated which are A1930 with mesh size 1930mm^2/m and A7200 with mesh size

7200mm^2/m, although these mesh sizes are not actually used in the current construction market. From Fig.13, It can be seen that, with the increasing of the steel mesh, lager maximum dynamic deflection is caused. This is because the experiments done by Fu [18] shows that, with the increasing of the steel bars the rotation capacity the composite joints increased, therefore larger maximum dynamic deflection is observed.

From Fig.14 and Fig.15 it can be seen that, the internal force like the beam tying force and major bending moment increased as well. This is because, after removal, the point A1 is working as a roller. As the deflection increased, the axial force of the beam increased due to the increasing of elongation or compression of the beam segment. As discussed in section 3.1, the beam is still in elastic stage, no plastic hinge is formed, so from the elastic energy analysis, it can be also seen that:

$$U_e = U_i$$

$$U_e \propto P\Delta$$

$$U_i \propto \int_0^L \frac{M^2}{2EI} dx$$

Where,

U_i is the internal strain energy,

U_e is the potential work due to the loss of the column,

P is the gravity load,

Δ is the deflection.

M is the bending moment of the beam

Therefore, with increasing of deflection, U_e increase, so U_i increased as well, Therefore M increased. Which means more energy has been transferred into the system. As no plastic hinge is formed, this amount of energy is stored in the system as a strain energy, and some are dissipated through the dumping.

3.5 *Effect of reinforcement mesh - Three columns removal scenario*

In order to clearly investigate the effect of the steel mesh, in this analysis, column A1, A2 and B1 on the ground level are removed. The steel meshes investigated are A142 with mesh size 142mm^2/m, A1420 with mesh size 1420mm^2/m and A7200 with mesh size 7200mm^2/m, although the last two mesh sizes are not actually used in the current construction market.

Plastic strain is also observed similar to two columns removal scenario. From Fig.16, It can be seen that, with the increasing of the steel mesh, lager maximum dynamic deflection is observed. From Fig.17 and Fig.18 it can be seen that, with the increasing of the steel mesh, the beam tying force increase however the major bending moment decreased. As it is discussed in [19], this is because when plasticity started to develop, below equation can be obtained:

$$\frac{M}{M_y} + \left(\frac{N}{N_y}\right)^2 = 1 \quad (1)$$

Where,

M_y is the plastic bending moment capacity in the absence of any axial force

N_y is the plastic axial force capacity in the absence of any bending moment.

M is the bending moment and

N is the axial force.

From Eq (1), it can be seen that, when deflection increased due to the increased mesh size, gravity loads are mainly resisted by the vertical components of axial catenary forces that develop in the beams. It is apparent from Eq. (1) that, with N approaching Ny, thus M will approximate to 0. This means that the beam bending stiffness will be greatly softened by the catenary axial force N. Consequently, when the catenary force is extremely large, the bending moment will almost disappear, the shape of vertical displacement diagram will approximate to the shape of original bending moment diagram, and the structure will change from a beam to a cable.

However, the analysis of section 3.4 and 3.5 shows that, the reduction of the M will not occur in the one column removal scenarios as the structure are still in the elastic stage, Eq (1) is not applicable, unless large deformation occurs as the three columns removal scenarios. It normally won't happen in one or even two columns removal scenarios, as long as the building is designed in normal grid and the structural members are designed according to the current code.

3.6 Measures to mitigate progressive collapse

From above parametric study it can be seen that, for the multi-storey composite steel frame buildings , the way to mitigate the progressive collapse is to increase the strength of the steel structural members and strength of concrete, however, it only has marginal effect on the resistance capacity of the building.

It can also be seen that, the building is more vulnerable to the removal of more than two columns. As it is discussed by Fu [16], 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. Therefore, another effective way to resist progressive collapse is to decrease the spacing of the grid or provide more redundancy in the structural scheme.

For one column remove scenario, increasing the steel mesh will increase the maximum dynamic deflection which is a disadvantage. However, for more columns removal scenarios, because the development of the plasticity, the behaviour of the building changed. The experiments done by Fu [18] shows that, increasing the steel rebar can increase the rotation capacity of the composite joint, which allows the plasticisation of the steel member. Therefore, increase the ductility of the joints. The increasing ductility increases the energy absorption capacity of the joints. This is because the ductile joints allow for redistribution of internal forces within the structural system by enabling large deformations so that they are suitable for progressive collapse mitigation by activating plastic system reserves by transition from flexural loading to tensile load in the members and joints and initiating of catenary action. So more steel mesh is an advantage when plasticity developed. However, it is noticed from the analysis of this paper that, this can only be achieved when more than 1 column are removed, as only large deflections can make this transfer happen.

4 CONCLUSIONS

In this paper, the behaviour of the 20 storey steel composite frame building under the sudden column removal was investigated with a 3-D finite element model using ABAQUS package. Base on this model, parametric studies were carried out to investigate the structural behaviour with variations in: strength of concrete, strength of structural steel, reinforcement mesh size. Through the parametric study, the measures to mitigate progressive collapse design were recommended.

Below are main findings:

1. The risk assessment of multi-storey building shows that, one column removal scenario is the most frequently occurred scenario. Therefore, most of recent research is focused on the one column removal of multi-storey buildings. For most research done so far, the plasticity is presumed to develop in the steel member under one column removal scenario, and plastic hinge is formed in the beam, therefore most research are based on the plasticity theory. However, for the beams size and grid used in the current design practice, this is not always true, after removal, the beam may still in the elastic stage. The elastic behaviour of the building after column removal is investigated in this paper in detail.

2. The typical multi-storey building with the cross bracing lateral resistance system used in the current design practice is less vulnerable to progressive collapse under the one column removal scenario.

3. For one column removal, for the four conventional sizes (A142, A193, A252 and A393) used in the current composite design practice, the difference mesh size have slight influence on the behaviour of the structure.

4. For one column removal scenario, increasing the steel mesh will increase the deflection, due to the increased rotation capacity. As the steel beam are still in the elastic stage, no plastic hinge are formed, therefore, the catenery effect is not significant.

5. For more than one column removal scenarios, with the increasing of the steel mesh. the ductile joint allow for redistribution of internal forces within the structural system by enabling large deformations so that they are suitable for progressive collapse mitigation by initiating of catenary action. However, it is noticed from this paper that, this can only be achieved when more than 1 column are removed as only large deflections can make this transfer happen.

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FIGURES

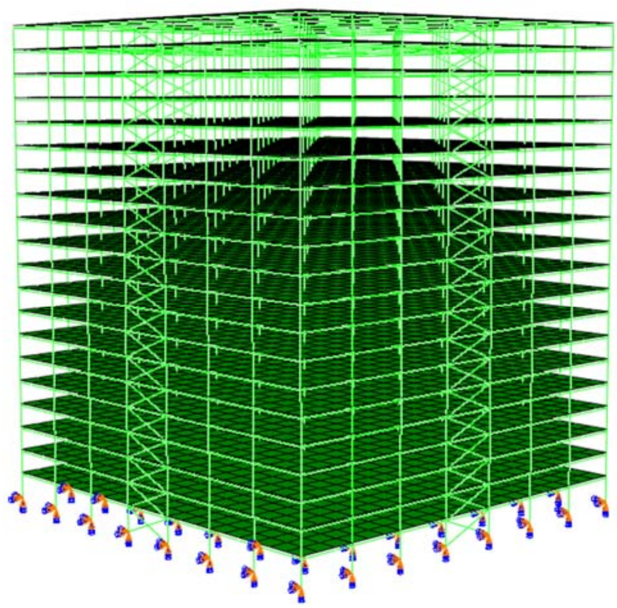

Fig 1 Analysis model with braces as lateral bracing

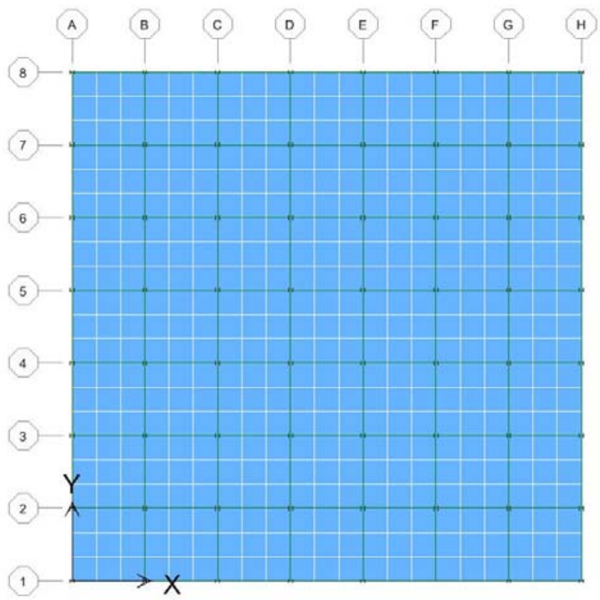

Fig 2 Typical plan layout of the ETABS model

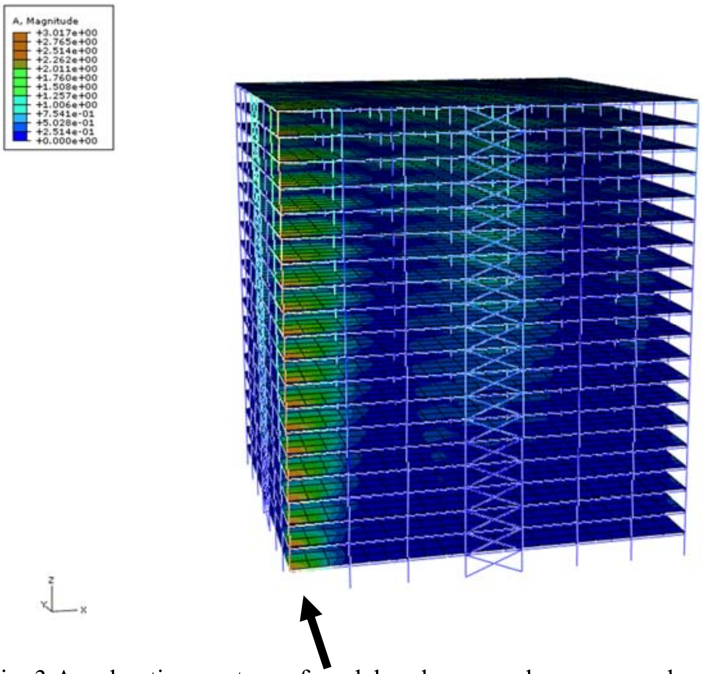

Fig .3 Acceleration contour of model under one column removal scenario

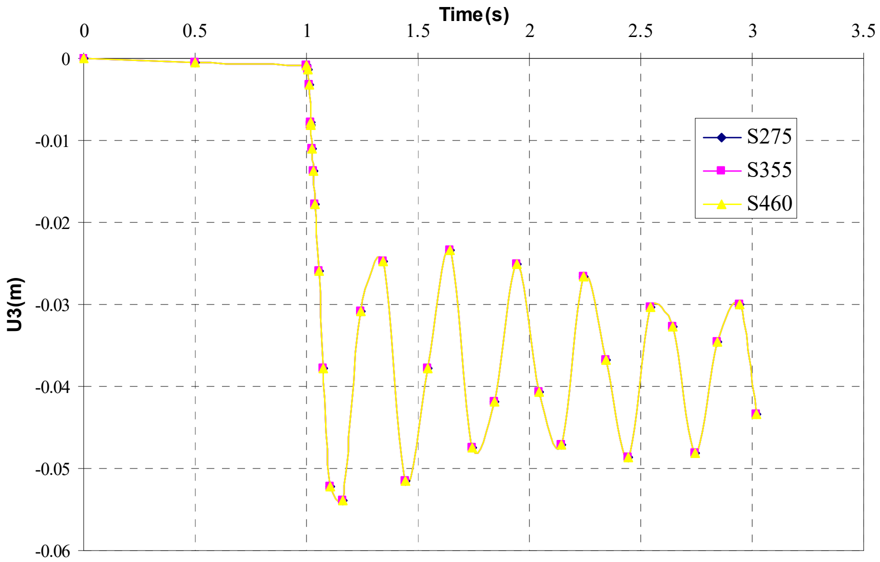

Fig.4 Displacement of the node above the removed column with different steel grade of steel members

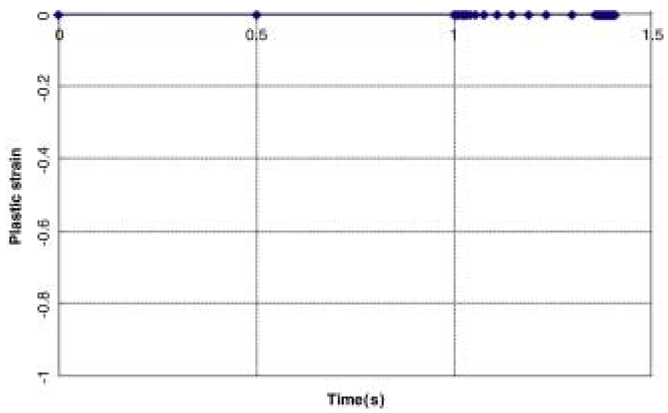

Fig.5 Axial Plastic strain of beam B1-A1 at ground level for case with S355 strength (1 column removal)

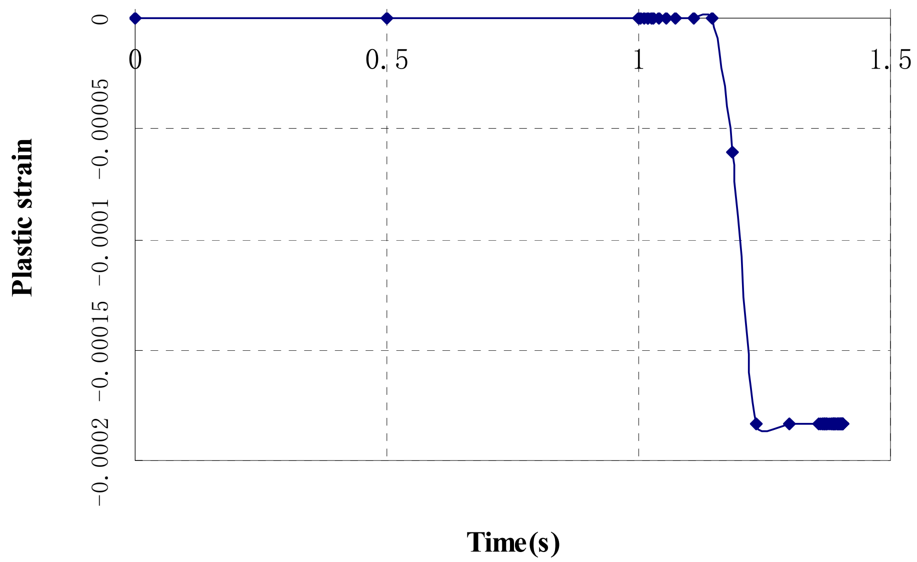

Fig.6 Axial Plastic strain of beam B1-A1 at ground level for case with S355 strength (2 columns removal)

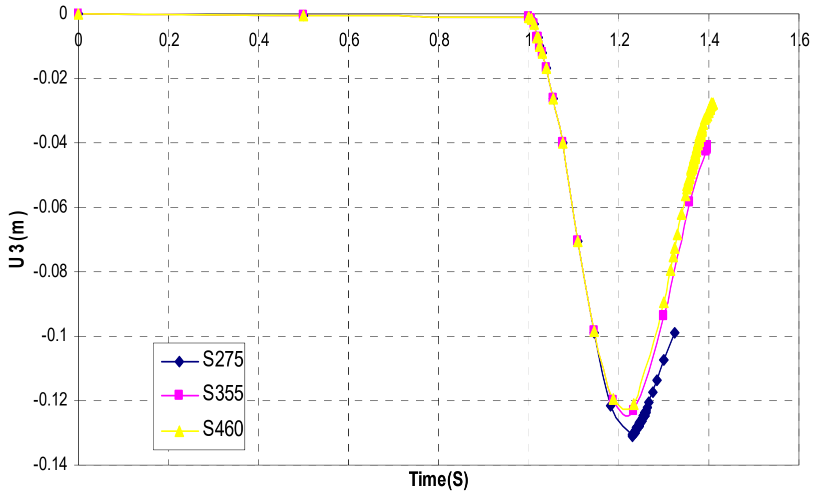

Fig.7 Displacement of the node at A1 with different steel grade of steel members (2 columns scenario)

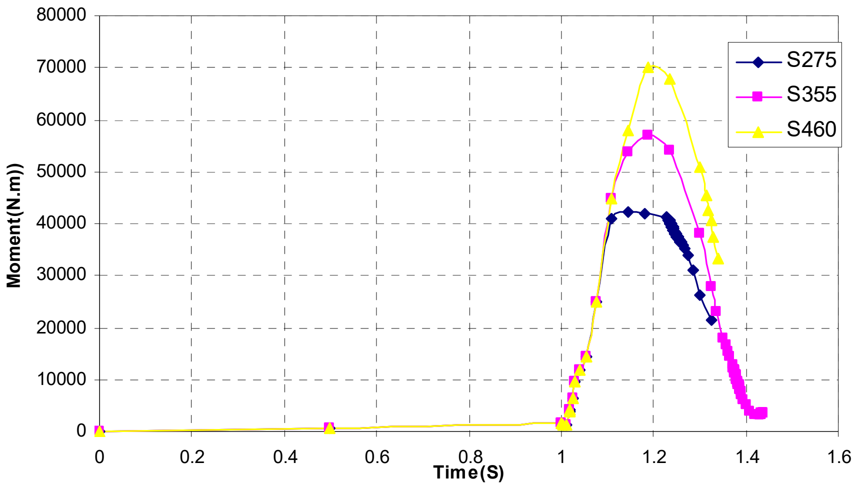

Fig.8 Major Moment of at B1 in Beam B1-A1 at ground level with different steel grade for steel member (2 columns scenario)

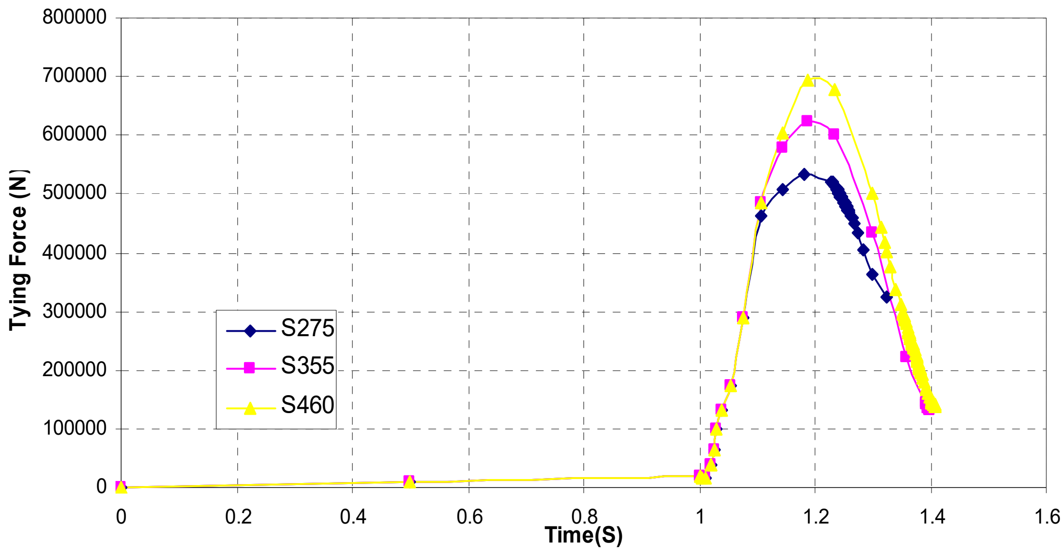

Fig.9 Tying force in the beam B1-A1 at ground level with different steel grade of steel members (2 columns scenario)

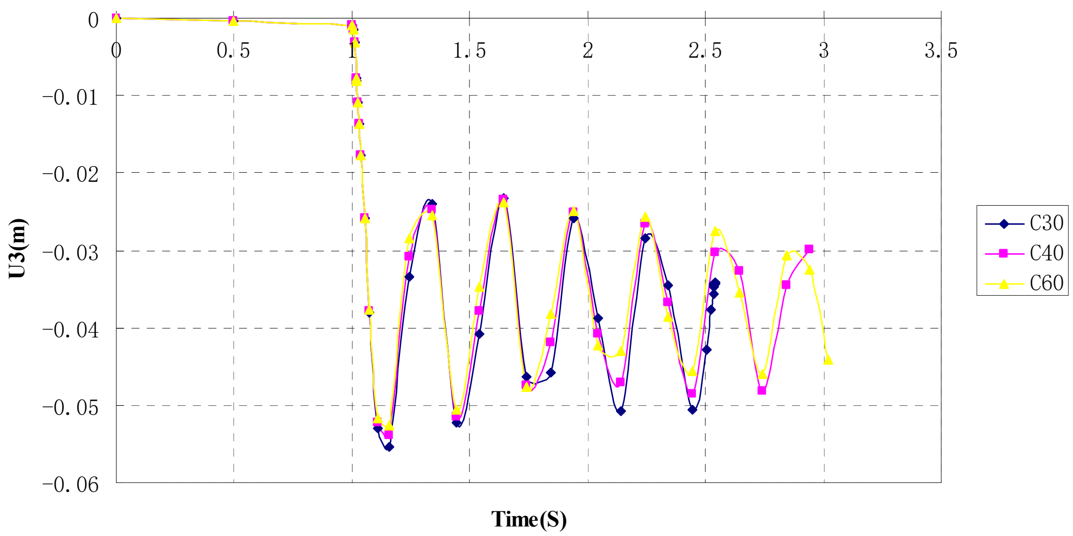

Fig.10 Displacement of the node above the removed column with different concrete strength (1 column removal)

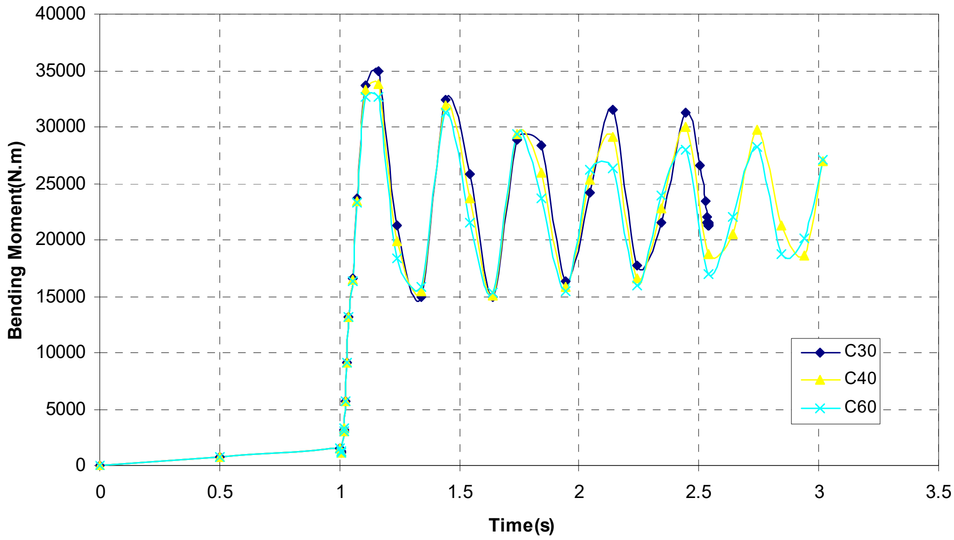

Fig.11 Major Moment of At end B1 of Beam A1-B1 at ground level with different concrete strength (1 column removal)

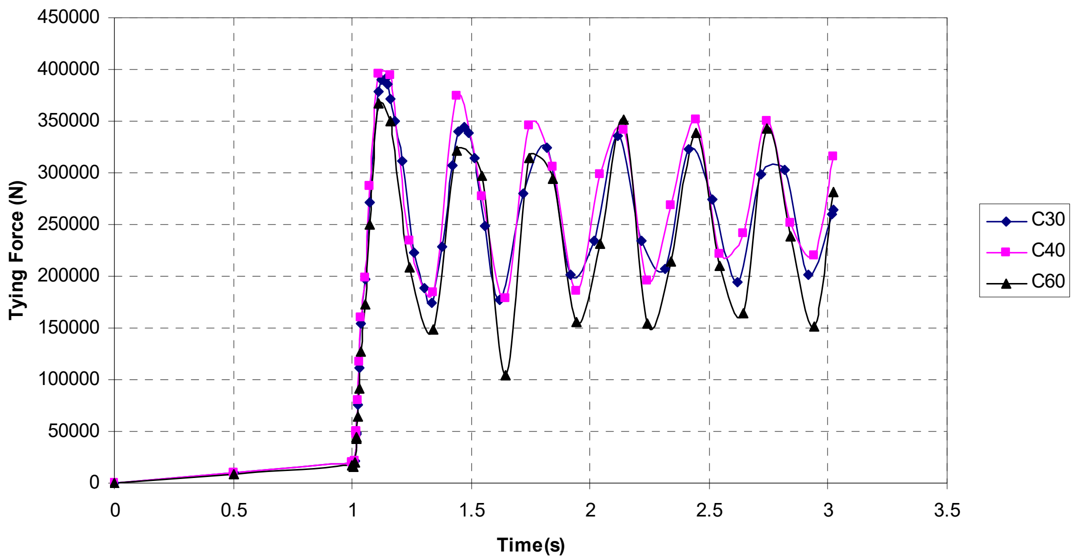

Fig.12 Tying force Beam of A1-B1 at ground level with different concrete strength (1 column removal)

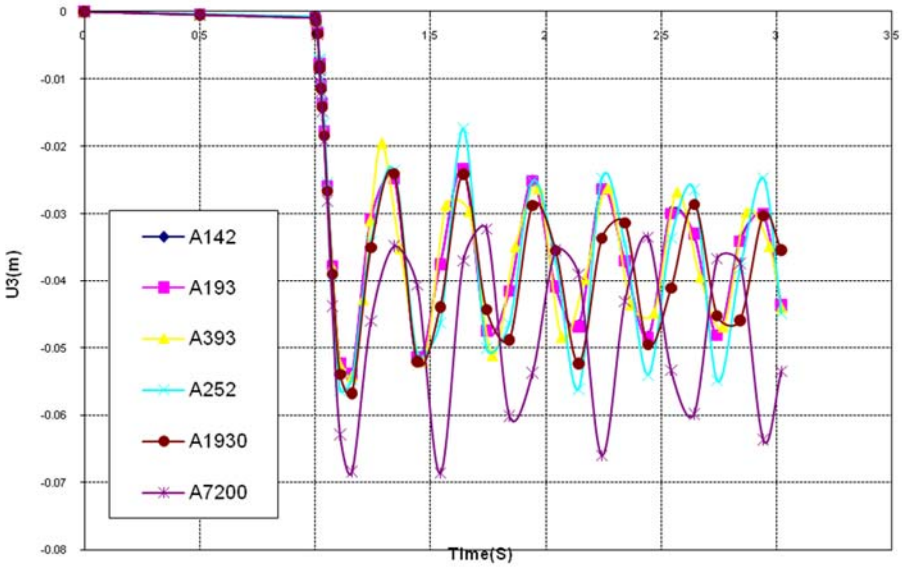

Fig. 13 Displacement of the node above the removed column with different mesh (1 column removal)

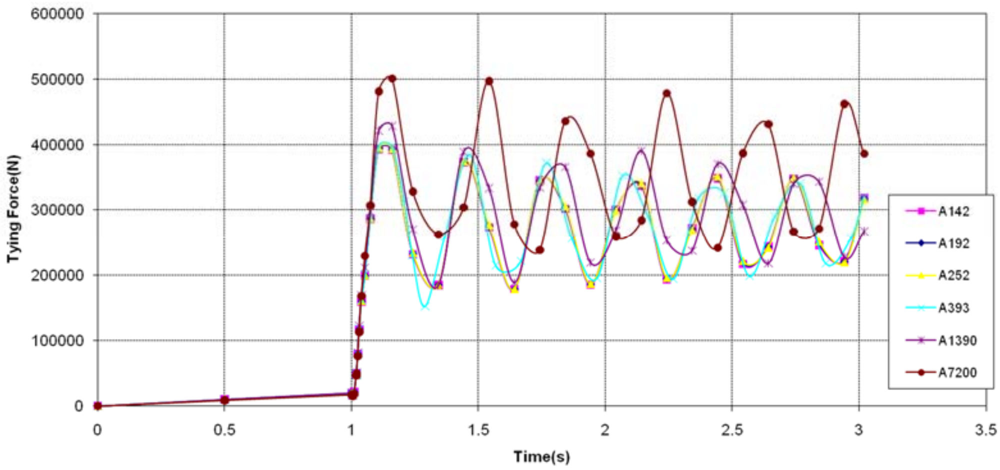

Fig.14 Axial force of At B1 of Beam A1-B1 at ground level with different mesh (1 column removal)

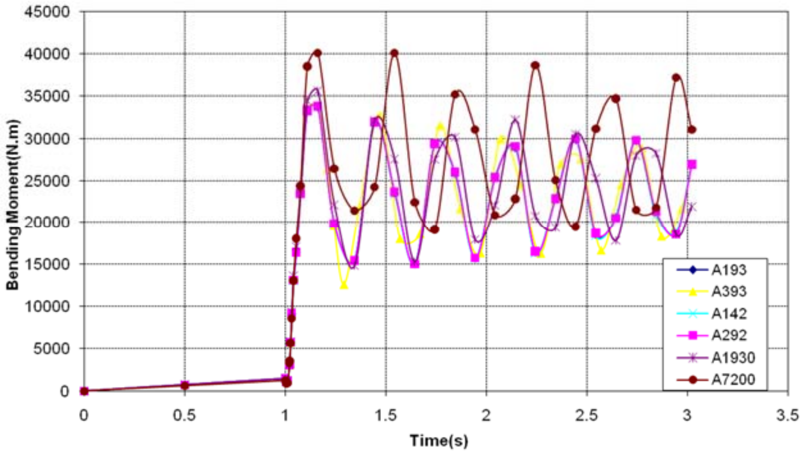

Fig.15 Major Moment at B1 of Beam A1-B1 at ground level with different mesh (1 column removal)

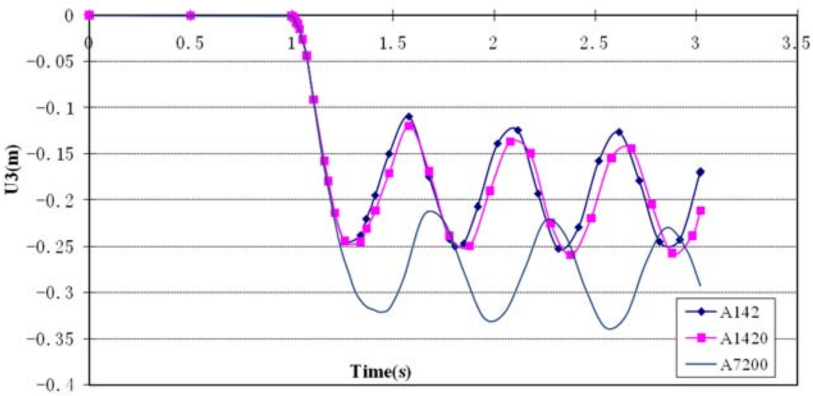

Fig. 16 Displacement of the node above the removed column with different mesh (3 columns removal)

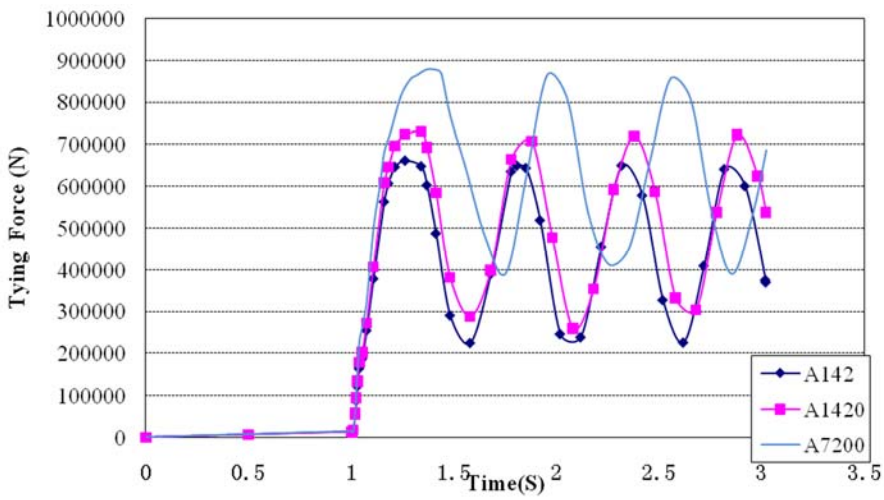

Fig.17 Axial force of At C1 of Beam B1-C1 at ground level with different mesh (3 columns removal)

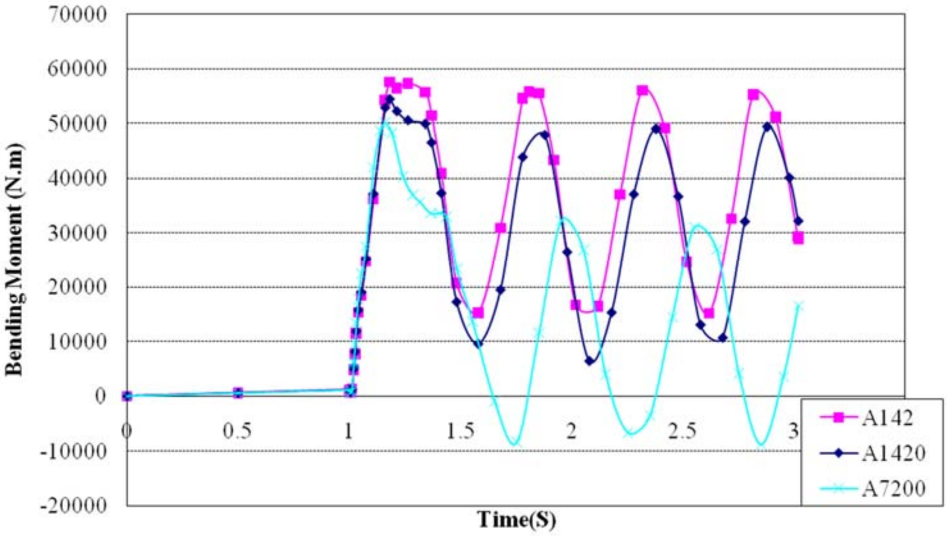

Fig.18 Major Moment at At C1 of Beam B1-C1 at ground level with different mesh (3 columns removal)