Analysis of 3D Steel-Concrete Composite Buildings using Abaqus

S Jeyarajan[1*] and J Y Richard Liew [1,2]

[1]Department of Civil and Environmental Engineering, National University of Singapore, 1 Engineering Drive 2, Singapore 117576
[2]College of Civil Engineering, Nanjing Tech University, China
[1]eng.rajan@gmail.com ; [2]ceeljy@nus.edu.sg

Abstract: *Research on robustness analyses of steel-concrete composite building structures has been performed over the last two decades with few simplifications in composite building frame components. This is because the detailed modelling of the nonlinear behaviour of steel-concrete composite slabs and joints is rather tedious and involves interaction between floor beams, slab and beam-to-column joint behaviour. Past research on progressive collapse analysis of building frames has reported that full three-dimensional (3D) building frame analysis is computationally expensive and consumes substantial computational resources in order to predict the non-linear dynamic response of buildings. Although well-calibrated simplified plane frame models can be relied upon to model progressive collapse, the results obtained from plane (2D) frame analyses may not be conservative. The main objective of this research study is to develop efficient computational models using ABAQUS to capture the behaviours of steel-concrete composite building structures subject to extreme load. The proposed composite slab model avoids complex geometric modelling of metal deck profile, concrete profile and shear stud, and it requires less computational time for analysing large building framework. The proposed joint model using Eurocodes avoids detailed finite element modelling of joint components such as bolts, gap, bolt-hole size, plates, etc., to improve the computational efficiency of analysing large three-dimensional (3D) building frames. The incorporation of semi-rigid joints and composite slabs in 3D frame analysis tends to produce more realistic estimate of frame behaviour compared to model using pin or rigid joints or skeleton frame. Then using the proposed numerical models, variety of moment and simple braced frames were investigated using conventional alternate path approach and direct blast analysis to evaluate their progressive collapse resistance. It was found that simple braced frames are susceptible to progressive collapse under extreme load compared to moment resisting frames. The skeleton steel frame model should not be used for progressive collapse analyses, because they over-predict the deflections of the frame and do not predict well the internal force distributions as compared to the frame models with the slab. The slab reduces the frame deflection by as much as 50% compared to the skeleton steel frame and thus it contributes favourably to resist progressive collapse. Finally, variety of structural strengthening approaches was proposed to improve the progressive collapse resistance of simple braced building due to accidental loading.*

Keywords: *Robustness, Progressive collapse, Composite building, Composite slab, Semi-rigid joint, Moment frame, Simple braced frame.*

1. Introduction

Research on the progressive collapse of building structures has been initiated with the partial collapse of Ronan Point apartment, UK, due to a domestic gas explosion. It has been intensified after several recent high profile collapses of multi-storey buildings [(Sadek et al., 2008); (Alashker et al., 2010, 2011); (Fu, 2010); (Izzuddin et al., 2008,)]. Some of the major progressive collapse incidents that have occurred in the past have been: (i) 9-storey reinforced concrete Murrah Federal office building at Oklahoma City collapse due to a truck-bomb attack, and (ii) World Trade Centre twin towers and World Trade Centre 7 collapse due to terrorist attack. The computational time required for analysing the progressive collapse behaviour of large building frame is still intensive even with the use of powerful desktop computers [(Kwasniewski,2010); (Alashker et al.,2011)]. Fu,2010 reported that the research on the behaviour of the progressive collapse of a composite building has been limited due to (i) limited availability of analysis tools, (ii) the high cost and cumbersomeness of a full scale test, (iii) the complicated geometric models for detailed three-dimensional (3D) framework, and (iv) the fact that a two-dimensional model does not predict the overall structural behaviour accurately and thus 3D analysis is often needed. Many researches focused only on analysing small scale single storey composite building to avoid high computational cost associated with detailed geometry modelling of composite slab, joint and the complex interactive behaviour between the frame components [(Sadek et al.,2008); (Alashker et al., 2010)]. Structural engineers tend to avoid detailed modelling of composite joints in frame analysis due to complexity of geometric modelling, insufficient guidelines, high computational cost and complicated interaction behaviour between joints and other structural components. As a result, joints are generally simplified into pin or rigid. In reality these simplified assumptions may be inaccurate or unconservative. They represent the limiting cases of the joint behaviour and lead to a wrong interpretation of the structural behaviour in terms of force distribution and structural responses. Several researchers have proven that joint rigidity improves the robustness and redundancy of building frames [(Sadek et al.,2008); (Alashker et al.,2010); (Izzuddin et al., 2008); (GSA,2003)]. Eurocodes 3 and 4 [(EN1993-1-8,2005); (EN1994-1-1,2004)] provide sufficient details to realistically predict behaviour of steel and composite beam-to-column joints based on the component method. As a result, the component method is now widely adopted for modelling semi-rigid joints for frame analysis.

Alternate Path (AP) approach, which is a threat independent methodology, is commonly used as a design guide for minimising the potential for progressive collapse (GSA,2003). Nevertheless, the effectiveness of this method is still questionable for abnormal loads, because a single member is generally removed, although the method does not exclude the possibility of removing more than one member. This assumption may lead to inaccurate prediction of the building response, especially under heavy blast loads and thus nonlinear analysis is performed to investigate the building response under extreme load. Although progressive collapse analysis procedure is similar for steel and reinforced concrete (RC) building as per GSA guideline, a RC building behaves differently under a blast load compared to a steel building in terms of joint and material behaviour, frame response and floor slab reinforcement configurations. Column-to-beam and beam-to-beam connections in a RC building are rigid, whereas connection of a steel-concrete composite building shall be pin/rigid/semi-rigid and its rotation stiffness and moment resistance are much less than a similar RC column-to-beam connection. The load acting on a profiled deck composite slab is typically distributed one-way in the direction of the metal deck profile. The load distribution in a RC slab depends on the direction of the main reinforcement bars. The metal deck on a profiled slab acts as a tensile reinforcement and helps to develop catenary action when the slab undergoes large deflection.

A building frame overall lateral stiffness and rigidity is much high for a RC building compared to a steel-concrete composite building and thus steel building under a heavy blast acts differently with a RC building.

This paper attempts to fill the gap by analysing 3D building frame response under an extreme loading considering the beneficial effects of floor slab and semi-rigid joints in resisting progressive collapse. A simplified composite slab model and composite joint models are adopted in this numerical investigation to reduce the computational cost. The investigation on progressive collapse of steel-concrete composite building subject to ground blast explosion was also carried out using nonlinear analysis and the alternate path approach.

2. Numerical modelling of composite building components

The incorporation of semi-rigid joint model and floor slab model in 3D frame analysis tends to produce more realistic estimate of frame behaviour compared to frame model using pin or rigid joints or skeleton frame without the presence of slabs. Numerical analyses are performed herein using the ABAQUS explicit solver (Simulia, ABAQUS documentation 6.11,2011) with a desktop computer of one CPU with 6-processors and 12GB RAM.

2.1 Beams and columns

Steel beams and columns are modelled using B31-two-node linear beam elements. Interaction between steel beam and floor slab is defined by tie constraint through ABAQUS library (Simulia, ABAQUS documentation 6.11,2011) to capture the composite action between the concrete slab and steel beam. Partial interaction in composite beams can be modelled using the tie constraint with spring stiffness. However, it was found that partial interaction in composite beams has negligible effects on the global response of 3D frames (Zolghadr Jahromi et al.,2013). Therefore, rigid tie constraint is adopted to represent the full composite action between the concrete slab and steel beam, i.e., no slip between the two surfaces. Local buckling of steel sections can be avoided by using sections with at least Class 3 cross section. High strain rate effect may affect the section classification. In such case, a detailed modelling of critical members using shell elements is necessary to capture the local buckling phenomenon due to high strain rate.

2.2 Composite joints

The composite joint is modelled using a six degree-of-freedom (DOF) non-linear connector. The connector behaviour is represented by axial force-displacement and moment-rotation relationships. These relationships can be established using a Eurocode 3-1-8 and Eurocode 4-1-1 component model approach [(EN1993-1-8,2005);(EN1994-1-1,2004)]. Figure 1 shows that the joint components (Fig. 1a) are represented by the simplified joint model in ABAQUS (Fig. 1b). The frame analysis assumes zero joint size and neglects the effect of panel zone shear deformation in the beam to column joints (da Silva et al.,2008). The details of the proposed simplified joint model and verification studies are given by Jeyarajan et al.2015a,b.

Although the component model is well developed for end-plate connections, limited work is done on fin-plate connections [(Anderson and Najafi,1994); (Dasison et al.,1987); (Liew et al.,2004)]. The moment-rotation behaviour of the fin-plate connection is more complicated because the centre of compression is moving with the increase in rotation. When a fin-plate beam-to-column connection is subject to hogging moment, the centre of compression zone moves from the centre of the bolt group to the bottom beam flange. This means that the Eurocode's component model cannot be applied directly to calculate the joint component's stiffness and maximum moment resistance. Therefore, a new component model for fin-plate connection was proposed in Jeyarajan et al.,2015b,2016.

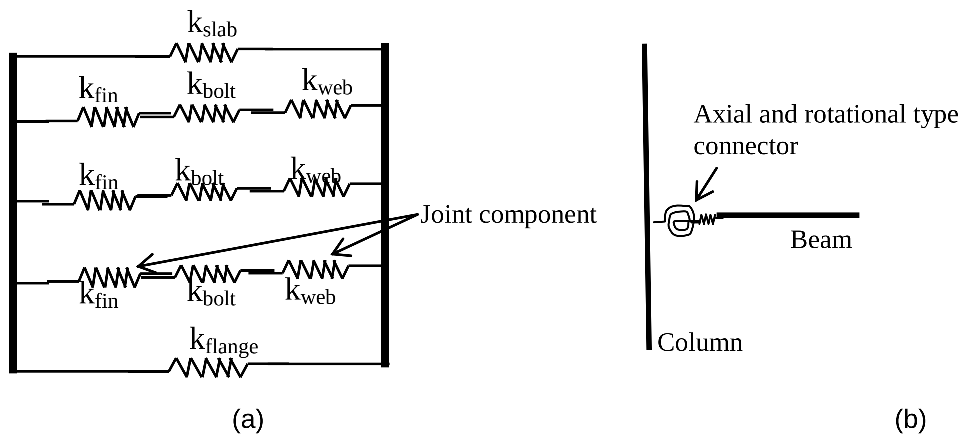


Figure 1: Model for fin-plate joint (a) Eurocode 3 component model (b) joint representation in frame analysis

2.3 Composite slab

A simplified composite slab model is used to avoid complicated geometry modelling of the profile composite slab. The profile metal deck is represented by rebars in a longitudinal direction based on equivalent area of the respective web and flange plates of the metal deck, at which rebar is assigned at the centre of each metal deck strips. Profile concrete is converted into an equivalent uniform concrete section and it is modelled using a four-node homogeneous shell element with reduced integration (S4R). The profile concrete in the composite slab, as shown in Fig. 2a, is converted into an equivalent concrete slab with uniform thickness, D_S-D_P/2, as shown in Fig. 2b. Metal deck strip areas A_1, A_2 and A_3 are calculated by multiplying the deck thickness by its strip length. The rebar area becomes, $a_1 = A_1$, $a_2 = A_2$ and $a_3 = A_3$. The composite slab will be converted into an equivalent uniform reinforced concrete section using the proposed simplified slab model and thus the equivalent concrete section could be modelled using a shell element, and rebar could be represented by the rebar option in ABAQUS. The tensile membrane effect is captured exactly since the total areas of the metal deck and concrete remain the same. Jeyarajan et al.,2015b. conducted verification studies of the proposed slab model and compared the predicted results with test results available from the literature and concluded that it is sufficiently accurate for modelling composite slab under gravity loads.

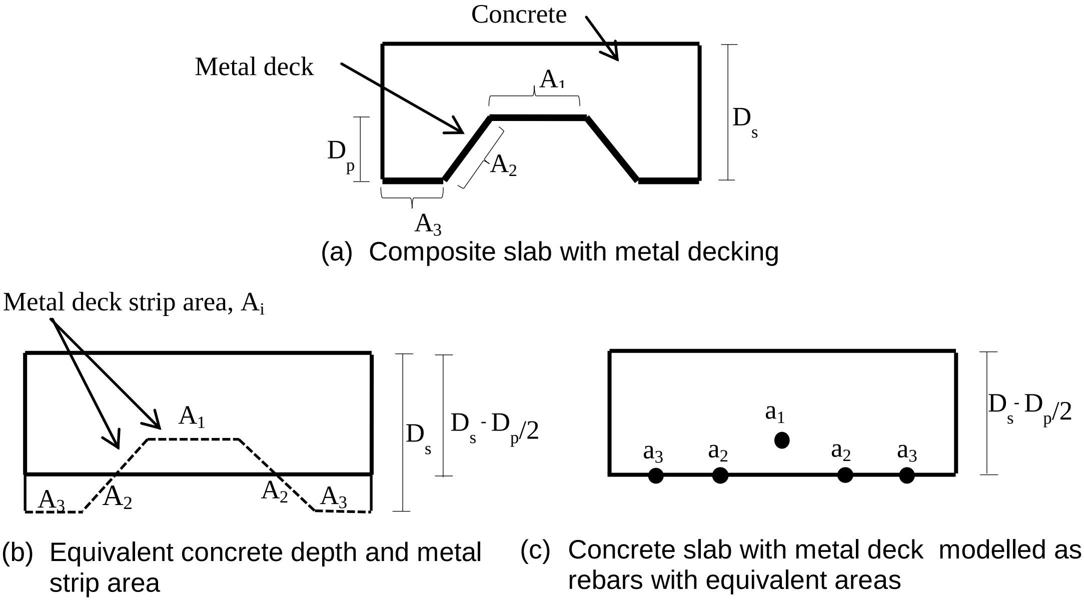


Figure 2: Proposed simplified model for composite slab

3. Numerical analysis of 3-D composite building under accidental loading

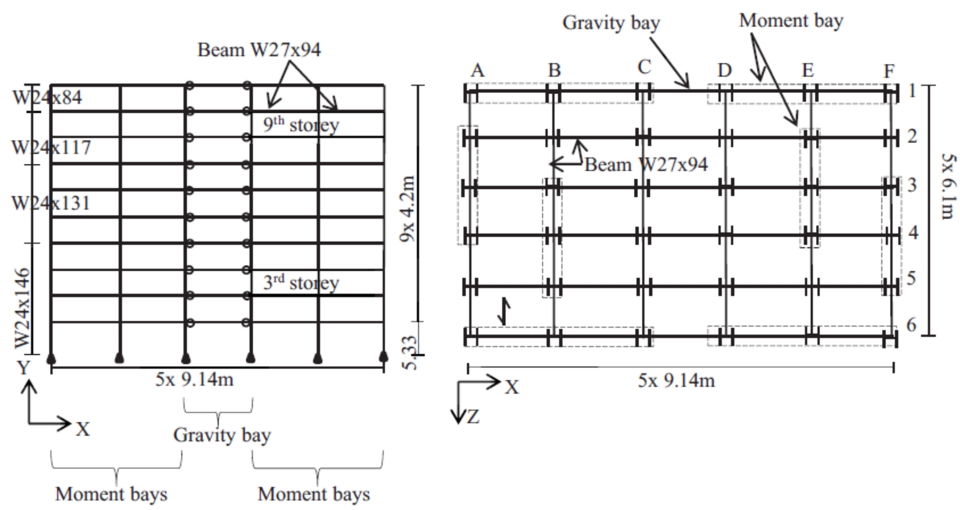
3.1 Analysis of building frame response under accidental load using Alternate Path approach

The Alternate Path (AP) approach, which is a threat independent methodology, is commonly used as a design guide for minimising the potential for progressive collapse [(GSA,2003); (UFC,2009)]. This approach would generally require the removal of single member (column or beam) regardless of threat type, although the method does not exclude the possibility of removing more than one member. The Alternate Path approach is performed to ensure the bridging capability of structure over a missing structural element under localised damage. Non-linear dynamic (ND) analysis may be required to predict accurately the response of building structures subjected to extreme load since large deflection beyond the elastic limit is expected and sudden column loss will often induce dynamic load (GSA,2003).

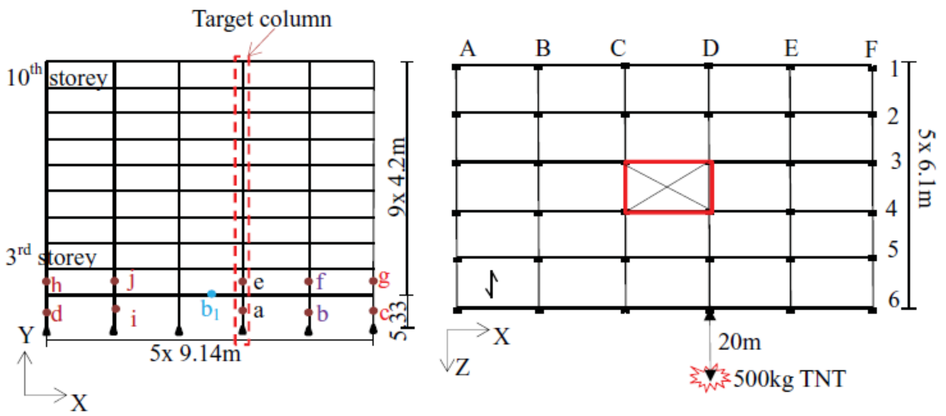
A ten-storey special moment frame (SMF) with reduced beam section (RBS) previously studied by Alashker et al.,2011 is modified herein for robustness analysis, as shown in Fig. 3a. The same ten-storey steel building with (i) diagonal braces at corners (ii) centre core wall (iii) rigid moment joints were investigated using alternate path approach (AP) for corner column (CC), perimeter column (PC) and internal column (MC) loss. Non-linear dynamic (ND) analyses were performed according to the General Service Administration guideline (GSA,2003). The ten-storey building foot print dimension is 30.5m x 45.7m. The column spacings in the longitudinal and transverse directions are 9.14m and 6.1m, respectively. Beam and column sizes are shown in Fig. 3a. A secondary beam, W14x22, is not shown in Fig. 3a for clarity. The composite floor slab consists of a 82.5mm thick lightweight concrete topping (concrete density = 17.3kN/m^3) on a 76.2mm deep metal deck. Metal deck thickness is 0.9mm and nominal concrete strength is 20.7N/mm^2. The slab is lightly reinforced with wire mesh (1.4mm diameter and 152mm spacing in both directions). Self-weight of floor is 2.2kN/m^2. Super-imposed dead load of the typical floor is 1.44kN/m^2 and the roof is 0.48kN/m^2. Live loads of the typical floor and the roof are 4.79kN/m^2 and 0.96kN/m^2 respectively. All the beam and column sections have the same steel grade with yield strength, tensile strength and modulus of elasticity equal to f_y = 345MPa, f_u = 448.2MPa, and E = 200kN/mm^2. For the simple braced frames, the beam-to-column connections are fin-plate types made by welding a 9.5mm thick single shear plate of A36 steel to the column and bolted to the beam web using 3 numbers 22mm diameter A325 high strength bolts, as shown in Fig. 4. Yield strength and tensile strength of these bolts are 634MPa and 827MPa respectively. The shear plate yield strength and tensile strengths are equal to f_y = 248MPa and f_u = 421MPa. Numerical modelling of composite joint and slab in a frame are reported in Section 2.

For the core braced simple frame, 300mm thick concrete walls are used to form the central core (between grids C-D and 3-4) to provide lateral load resistance to the simple frame surrounding the central core, as shown in Fig. 3b. The core wall is reinforced by two layers of T16-300 steel bars arranging in both way with concrete cover of 30mm. Shell elements (S4R) are used to model the core wall.

Concrete material is modelled using the concrete damage plasticity model in ABAQUS and tensile strength of concrete is neglected. Steel material is modelled using the elastic-plastic bi-linear material model with 0.5% strain hardening. The fracture strain is taken at 0.27 plastic strain as in Alashker et al.,2011. The material model for metal deck and rebar is assumed to be elastic-perfectly plastic with f_y = 248MPa and fracture strain 0.25. Structural damping of 5% is assumed. As reported, shell elements (S4R) used for slab, and beam elements (B31) used for columns and beams in the numerical modelling using ABAQUS. Full composite action is defined between slab and beam using tie constraints. Finite element mesh size of 500mm is adopted in this comparison study. The simple connections are assumed to be fin-plate bolted connections, as shown in Figure 4. The ground floor (i) perimeter column D6 (ii) corner column F6 (iii) internal column D4/E5 (D4 for mixed frame; E5 for core braced simple frame) is removed suddenly to investigate the progressive collapse resistance of building.

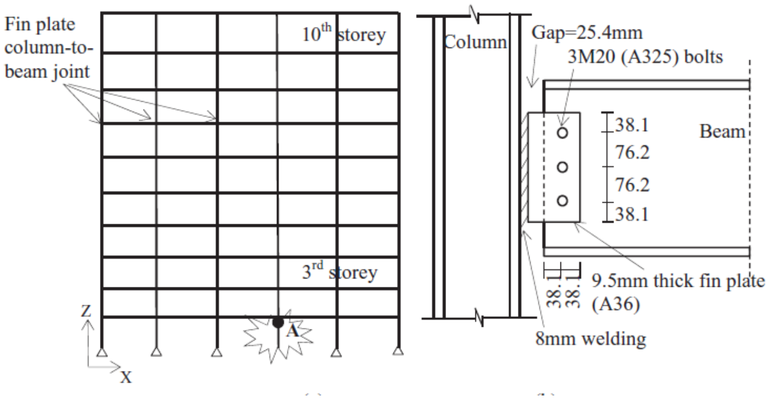


(a) Elevation and plan view of mixed frame



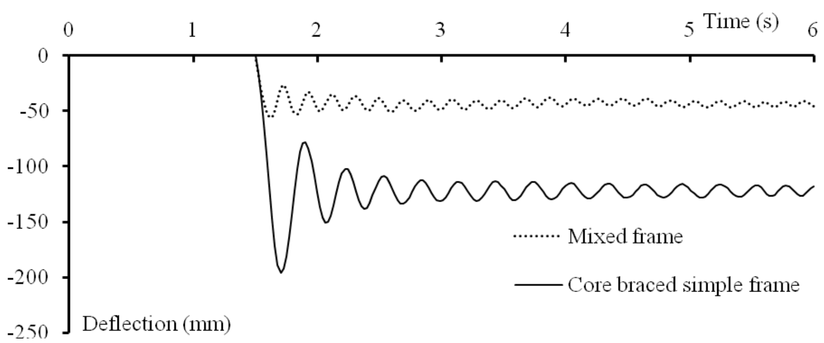
(b) Elevation and plan view of core braced simple frame

Figure 3. Elevation and plan view of ten-storey building (secondary beams are not shown)

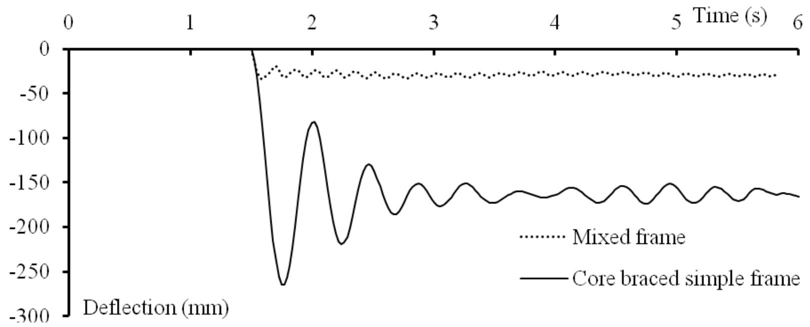


(a) (b)

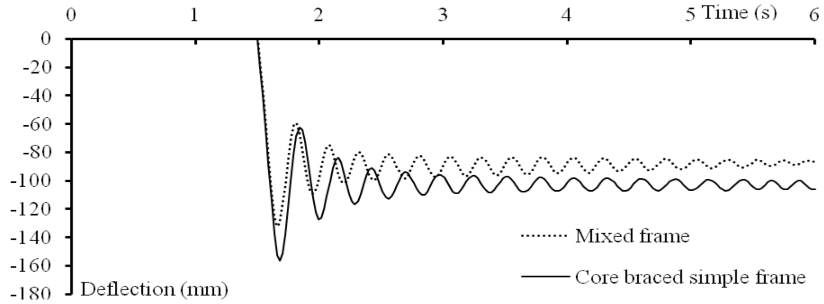
Figure 4. (a) Monitoring point A for frame vertical deflection due to perimeter column loss (b) fin-plate connection



(a) Vertical deflections at column removed position due to perimeter column loss



(b) Vertical deflections at column removed position due to corner column loss



(c) Vertical deflections at column removed position due to internal column loss

Figure 5: Vertical deflection of building frame due to a column loss

Figures 5(a) to (c) show that vertical deflection response curves of core braced simple frame and mixed frame due to column loss. The mixed frame vertical deflection is generally less than the vertical deflection of simple braced frame due to joint rigidity in the mixed frame. However, for the internal column loss, the mixed frame vertical deflection is almost same as the vertical deflection of the core braced simple frame as shown in Fig. 8c since the removed internal column was not rigidly connected with the beams in the mixed frame and therefore large deflection was observed. For unbraced frames, it is important to provide rigid connections to all the beams and columns to avoid very large displacement (exceeding the limit in GSA, 2003) due to column loss. Detailed discussion on progressive collapse analysis and structural enhancement methods against progressive collapse was reported in Jeyarajan et al.,2015 a,c.

3.2 Analysis of building frame response under blast load using direct blast analysis

An integrated nonlinear dynamic analysis involving blast analysis and fibre-element modelling of structural members including member initial imperfections would involve huge computational resources and may not be practical for the analysis of large scale multi-storey composite building frames. In the proposed two-step process, a nonlinear dynamic analysis is firstly carried out on 3D composite framework subject to blast load based on the proposed simplified joint and composite slab models, if applicable. Damaged members are then identified by checking the member forces against their maximum resistance. The damaged members are then removed for subsequent collapse analysis. The proposed two-step process is proposed for practical implementation as it involves less computational time and less effort in modelling.

Nonlinear dynamic analysis may be carried out by considering the blast to occur at any floor level within a building or at a standoff distance from the building. Critical structural elements in the building may be identified by considering the various blast scenarios. Blast pressure-time history due to an explosion may be obtained from TM-5-1300,1990 guideline and the dynamic loads are applied for blast analysis of a building. The nonlinear analysis is performed to identify the damaged elements in the building frame. The structural element is considered as damaged, if the demand on a structural element exceeds its resistance or the prescribed limits such as rotation capacity or ductility limit (GSA,2003). The structural element damage criteria may be governed by the respective resistance to axial force, bending moment, shear force, or a combination of these, or may be due to limit of deflection or rotation at accident limit states.

After the removal of the damaged members, subsequent non-linear dynamic analysis is performed on the damaged building frame. The residual resistance of the removed member is ignored and it will generally lead to conservative estimate of the progressive collapse resistance of the structure. The collapse analyses are performed here using ABAQUS explicit solver with a desktop computer of one CPU (6-processors) and 12GB RAM. Computational time for typical ten-storey steel-concrete composite building frame, which has five bays in each direction, is around one to two days depending on finite element mesh size, material model, size of time increments, etc.

The ten-storey centred core building frame, which is reported in Section 3.1 and Fig. 3b is investigated for a surface bomb blast of 500kg TNT, which is detonated at 20m away from the building front column D6 (along the grid D). The US guideline TM-5-1300:1990 is used to predict the blast pressure-time history for the numerical analysis. A nonlinear dynamic analysis is firstly performed to identify the possible damaged elements in the building frame for the collapse analysis. Since the explosion occurred at 20m away from the structure, the blast load acting on the top and bottom surface of the beam and slab are almost the same and thus uplift force is neglected. However, in the case of nearby explosion, the uplifting loads on the beams and slabs must be considered as they can generate an uplifting initial velocity and displacement (Shi et al.,2010). In the present study, the blast loads are applied on the front face of the ten-storey building front columns as uniform distributed load, based on the assumption that the external claddings are fragile and only the width of column width + 200mm is effective in attracting blast pressure (SCI,2011). Similar approach was used by Fu,2013 in their numerical analyses. The building frame is subjected to a gravity load combination: 1.0 x dead load + 0.25 x live load in load step 1. A 500 kg TNT is detonated at a distance of 20m away from the target column D6 at 1.5 seconds from the initial time, which is represented in load step 2. Blast pressure-time history for each storey column is calculated using TM-5-1300 and it is applied as uniform distributed load appropriately at load step 2. The frame response is monitored for three seconds. Few critical locations on column and beams are monitored. Among them, monitoring points ‘a’ to ‘j’ are at mid-height of the 1st and 2nd storey columns and the monitoring point ‘b_1’ is at mid-point of the 1st storey beam. Building frame schematic views and monitoring points are shown in Fig. 3b (secondary beams are not shown for clarity).

Mechanical properties of the steel and concrete materials are affected due to the dynamic blast load [(TM-5-1300,1990); (SCI,2011)]. TM-5-1300 states that a structural element subjected to a blast loading exhibits a higher strength than a similar element subject to a static loading. This increase in strength, for both the concrete and steel, is attributed to the rapid strain rates that occur in dynamically loaded members. These increased stresses or dynamic strengths are used to calculate the elements of dynamic resistance to the applied blast load. Therefore, strain rate effect due to blast is considered in the numerical analysis (Fu, 2013).

Concrete material is modelled using the concrete damage plasticity model in ABAQUS and tensile strength of concrete is neglected. Steel material is modelled using the elastic-plastic bi-linear material model with 0.5% strain hardening. The mechanical properties of the steel and concrete materials are affected due to the dynamic blast load and thus dynamic design stress of steel and concrete are defined by stress-strain relationships for each elements (concrete, steel, metal deck, rebar) instead of defining their static stress-strain relationship.

The front columns of the building facing the blast are arranged in such a way that they bend about their minor axis when subject to blast pressure. Figure 6 shows the graphical views of a 3D deformed building frame under the AP approach (single column loss) and nonlinear dynamic analysis (five column damage and loss). With reference to the monitoring points depicted in Fig. 3b, the effect of standoff distance on nonlinear dynamic analysis is summarised in Figs. 7 to 10 and Tables 1 and 2. The axial force, bending moment, shear force and deflection responses of the 1st storey perimeter columns D6, E6 and F6 are summarised in Figs. 7 to 10. The numerical results clearly show that blast effect on adjacent structural member is severe compared to a structural member that is further away from the blast location. Significant axial force (AF) variation in a column is observed due to the high lateral blast pressure. Shear force induced by the direct blast is within the column shear resistance. The maximum axial force, bending moment and shear force on 1st storey columns due to blast can be three times higher than the force before the blast. Nonlinear dynamic analysis results show that many 1st storey perimeter columns reached the plastic moment resistance. Analysis results show that lateral deflection of 1st storey columns due to blast is not negligible. Large deflection may damage the column with regard to support rotation, member ductility, second order effect etc.

Table 1. Lateral deflection and force demand of building frame for 5% damping with strain rate effect

Point	Max. lateral deflection (mm) (Z-direction)	Max. minor axis bending moment (kNm)	Max. axial force (kN)	Max. shear force (kN)	Max. plastic strain
a	-329	621*	4542	496	0.031
b	-250	621*	3799	289	0.031
c	-192	621*	2448	229	0.018
d	-86.8	537	1323	143	0.001
e	-93.5	477	4029	313	0.001
f	-61.4	420	3502	260	0
g	-69.7	304	2279	233	0
h	-52.6	252	1180	229	0
i	-144	621*	2698	296	0.014
j	-43.0	386	2467	252	0
b_1	-6.49	65.4	849	150	0

* Plastic moment resistance is reached

Table 2. Maximum deflection and forces for building frame for 5% damping with strain rate effect

Point	Max. lateral deflection (mm) (X-direction)	Max. vertical deflection (mm) (Y-direction)	Max. major axis bending moment (kNm)	Max. torsional moment (kNm)	Max. major axis shear force (kN)
a	92.6	-23.1	163	35.4	88.9
b	40.8	-13.8	197	20.9	78.7
c	19.1	-8.38	276	13.2	93.8
d	6.89	-2.01	245	22.7	74.9
e	11.4	-38.0	180	27.6	152
f	8.61	-21.7	225	15.5	102
g	9.27	-11.8	155	12.4	152
h	6.29	-3.55	203	16.7	98.4
i	9.15	-5.23	173	14.5	46.7
j	3.75	-8.29	147	11.9	57.7
b_1	4.83	-50.1	861	1.57	124

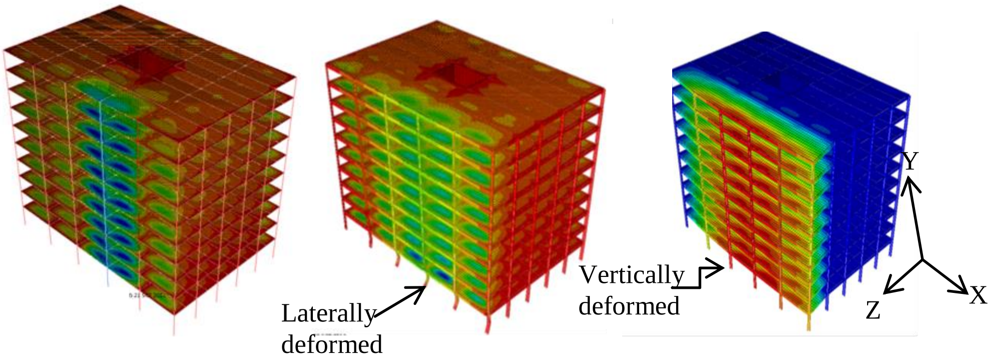


Figure 6. Deformed frame view for (a) one-column loss AP analysis (b) nonlinear dynamic analysis (c) five-column loss collapse analysis

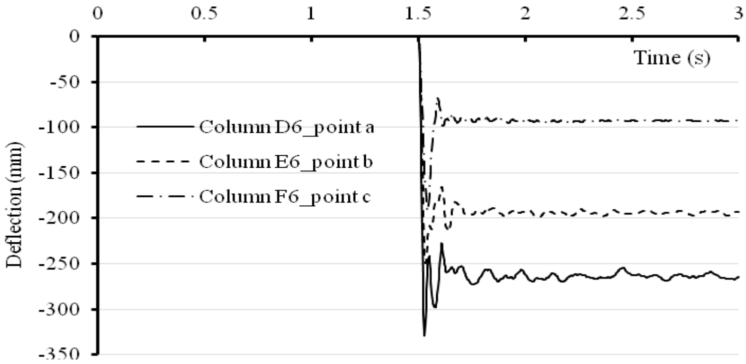


Figure 7. Column lateral deflection (dir-Z) in nonlinear dynamic analysis for 5%

damping

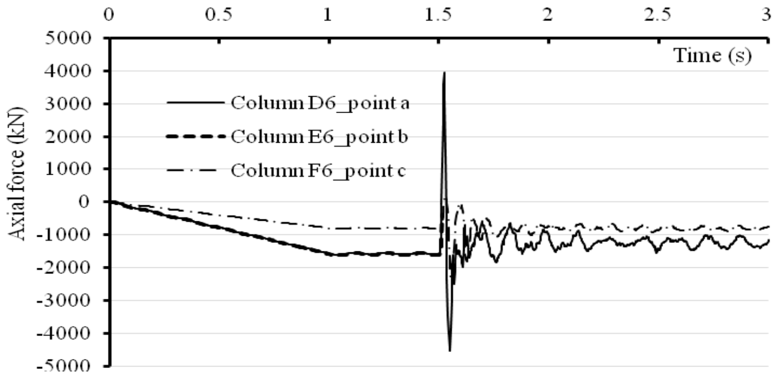


Figure 8. Column axial force in nonlinear dynamic analysis for 5% damping

Analysis results show that the moments and axial force acting on the five front perimeter columns (B6, C6, D6, E6 and F6) at the first storey due to blast load reach the member buckling resistance as shown in Table 1 and they are treated as damaged and removed in the subsequent collapse analysis. The residual resistance of the damaged member is ignored and this will generally lead to conservative estimate of the progressive collapse resistance of the structure. The plastic strain of these five columns are varies within the 0.014 to 0.031, as summarised in Table 1, and they less than the defined fracture limit of 0.27. Figure 23 shows the deflection history at column D6 (point-a). Figure 9 shows that the vertical deflection predicted by the AP approach (due to perimeter column D6 loss) is higher than from the nonlinear dynamic analysis. Subsequently, collapse analysis is required to be executed with the absence of the damaged element to complete the progressive collapse analysis. Nonlinear dynamic analysis results are summarised in Tables 3 and 4 for frame with strain rate effect.

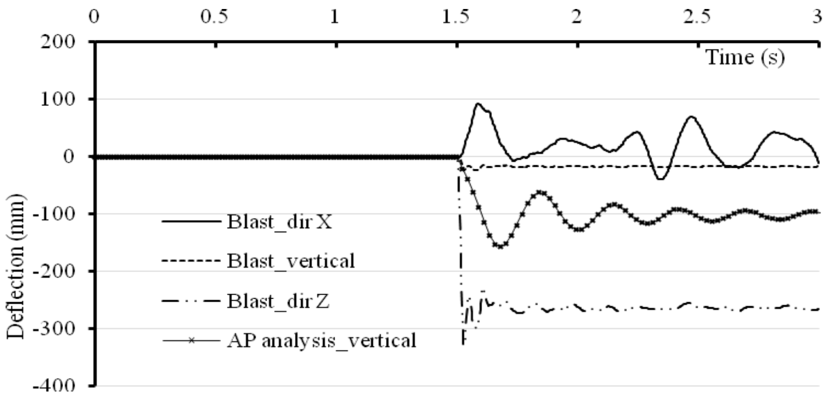


Figure 9. Deflection at mid height of the column D (point-a) with time

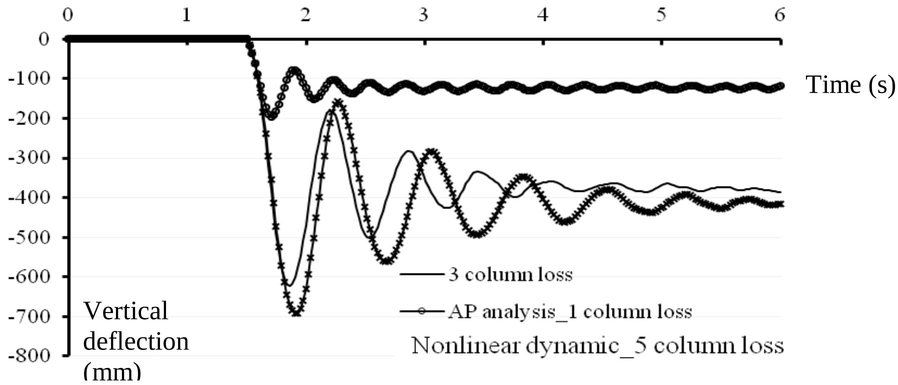


Figure 10. Column D6 (point a) force variation with time due to blast for 5% damping

Collapse analysis

In the second stage of the two-step analysis, the identified damaged perimeter columns (B6, C6, D6, E6, and F6) are suddenly removed at 1.5 seconds from the building frame in subsequent collapse analysis. The five 1st storey columns' supports are suddenly removed to simulate the five column loss due to the blast. An alternate path approach is performed with the removal of a single column (D6). For the parametric study, a three-column loss scenario (C6, D6, and E6) is investigated. Analysis results for single column loss (AP), three column loss (for parametric study) and five column loss (according to the nonlinear dynamic analysis) are compared. Lateral deflection due to a three column removal was observed less than the lateral deflection in a nonlinear dynamic analysis, since lateral load is not taken into consideration in an alternate path (AP) analysis (GSA,2003) (i.e. dead load + 0.25 live load is applied according to GSA guideline). This will under predict the building response compared to the real response of the building frame under a blast. Numerical results for vertical deflection due to columns loss are summarised in Fig. 10. Deformed view of building frame due to five columns loss is shown in Fig. 6c. It is known that a simple braced frame may be susceptible to progressive collapse for a single column loss (AP) due to weak column-to-beam joint (Jeyarajan et al. [49]). Three and five column loss, causing larger deflection in addition to large end rotation and significant force demand on members and thus frame under 500kg TNT explosion, is susceptible to progressive collapse. The 10-storey centre core braced simple frame is subjected to partial collapse due to a surface blast load of 500 kg TNT.

The analyses results show that simple braced frames are vulnerable to the progressive collapse compared with the moment resisting frames, which has higher redundancy to redistribute the loads arising from sudden column loss. Various strengthening approaches were investigated to improve the collapse resistance of the simple braced frames. Vierendeel truss was proposed at selected floor level in a building to redistribute the load due to column loss. End-plate or modified fin-plate connections were proposed for column-to-beam connections to enhance the progressive collapse resistance of simple braced frame. The resistance against progressive collapse was also investigated for building with outrigger-belt truss.

4. Conclusions

A simplified composite slab model in which the profile metal deck is represented by rows of rebars and the profile concrete is converted into an equivalent uniform concrete section has been proposed to analyse the collapse behaviour of three-dimensional composite building. Semi-rigid composite joints in the building framework are modelled using the Eurocode's component model represented by linear and rotation spring connectors. The connectors' behaviour is represented by an axial force-displacement relationship and moment-rotation relationship. The proposed slab and joint models, which are validated against experimental results, avoid the detailed finite element modelling of the metal deck profile and joint components and thus improve the efficiency of analysing large building framework subject to accidental load. The proposed models also provide a more realistic estimate of 3-D frame behaviour compared to other frame models assuming pin or rigid joint behaviour.

Using the proposed numerical modelling approach, moment and simple braced frames were analysed under difference column loss scenarios. The numerical models consider various lateral load resistance systems, and the effect of composite slab and semi-rigid beam-to-column joints in resisting progressive collapse due to column loss. It was found that simple braced frames are susceptible to progressive collapse under column loss compared to moment resisting frames. The skeleton steel frame model should not be used for progressive collapse analyses, because they over-predict the deflections of the frame and do not predict well the internal force distributions as compared to the frame models with the slab. The slab reduces the frame deflection by as much as 50% compared to the skeleton steel frame and thus it contributes favourably to resist progressive collapse. The loss of a corner column in a building is more critical than the loss of a perimeter column because of less adjacent members connecting to the corner column to redistribute its load.

A ten-storey centred core braced simple frame is also investigated for a surface blast, which is detonated at a distance away from the front surface of the building. Material strain rate effect due to dynamic blast loading has significant influence on the responses of the composite building. Nonlinear dynamic analysis shows that high blast load may wipe out several columns at the ground floor and induce high shear force on the first storey columns. High blast pressure also caused large lateral drift and significant axial force on the ground floor columns. The study summarises that scenario-base nonlinear dynamic analyses should be performed to capture the true behaviour of buildings subject to high blast load. This approach is more sensible than the alternate path approach checking the robustness of buildings based on the column removal concept. The alternate path approach can be used in preliminary design to check robustness of a building, but nonlinear dynamic analysis based on the proposed simplified models is still preferred for threat-scenario analyses.

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