Building Blast Simulation and Progressive Collapse Analysis

T. Krauthammer, University of Florida and J. Cipolla, SIMULIA, Inc.

FEA analysis of severe blast loading supports the design of survivable structures without necessarily requiring expensive physical simulations of a specific explosive or combustion event. These analyses also show that using established structural design guidelines may not be conservative for severe blast loading on steel structural members.

This article, from T. Krauthammer of University of Florida, and J. Cipolla of Simulia, describes the FEA modeling of the progressive collapse of a steel frame structure, and the qualitative insights gained. A full-length version of this paper is available in the NAFEMS 2007 World Congress proceedings.

Analysis of steel frame connections under blast loads

Currently, U.S. design guidelines for steel connections in structures subjected to blast loads are based on recommendations in the Department of Defense Technical Manual (TM) 5-1300TM [1]. The approach idealizes real structures and structural elements as "equivalent" lumped-mass single-degree freedom systems. In addition, the guidelines are for single-storied steel frames, not subjected to any significant dead loads apart from the self weight of the structure. The actual effects of blast and dead loads on real steel connections may exceed the margins predicted by TM 5-1300.

To assess the behaviours of steel moment connections under such loads [2], finite-element simulations using ABAQUS/Explicit 6.5 [4] were employed. The maximum rotational capacities of the connections were then compared against values derived with the TM 5-1300 approach [1]. Target connections were placed between beam and column at the ground floor of a multi-story building. Lengths of the beams and columns were taken from Engelhardt's, et al. [8], experimental studies (Figure 1). For each connection type, four different load cases were used (see Table 1).

Reference maximum blast pressures were calculated based upon the TM 5-1300 criteria [1].

For the finite-element calculations, the standalone codes SHOCK [9] and FRANG [10] were used to compute equivalent shock pressures and gas pressures (Table 1), assuming an 18.5 lb. TNT charge located at the centre of the room. We applied blast pressures as spatially uniform surface loads on the sidewalls that transferred to the beams and the column of the connection. Dead loads, applied on the top flanges of the beams and axially on the top cross section of the column, correspond to those for a 10-story office building. The numerical model was analyzed with and without these dead loads to evaluate their influence on connection response.

An isotropic elasto-plastic model was used as the material property for each connection component [2]. Yield and ultimate strengths were increased to account for strain rate effects using dynamic increase factors (DIF) as recommended in TM 5-1300 [1]. Since brittle fracture on the weld connections was anticipated under the blast loads, we adopted the shear failure model; ABAQUS removed elements from the mesh as they failed. The finite-element models (Figure 2) were created using predominantly 8-noded continuum brick elements with reduced integration.

The responses and failure criteria based on TM 5-1300 criteria are shown in Table 2, and indicate that the representative room could withstand the loads from the explosive charge.

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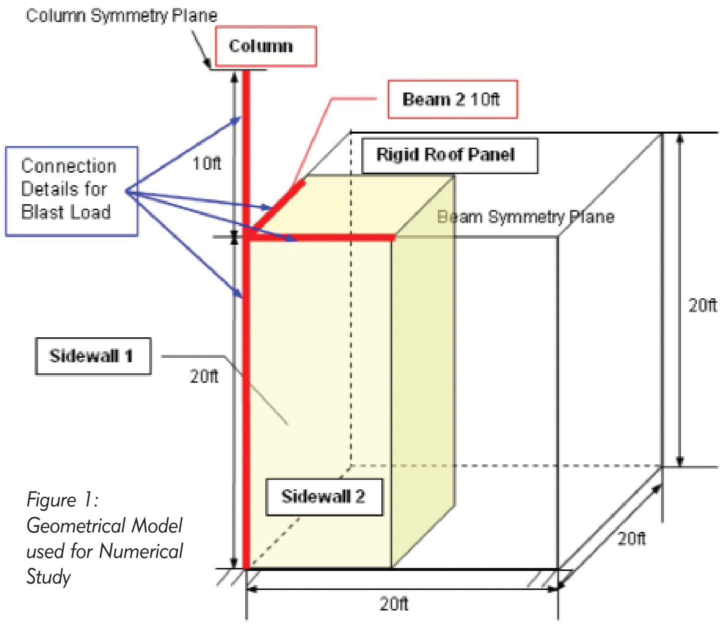


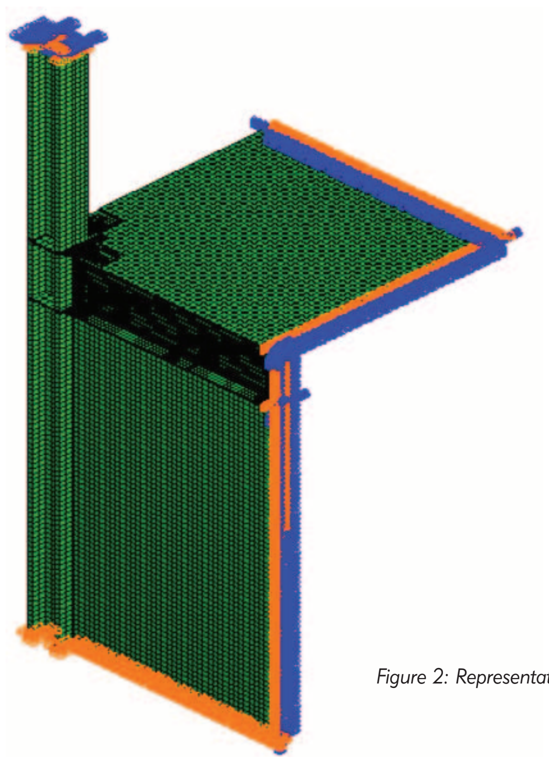
Figure 1: Geometrical Model used for Numerical Study

Table 1: Loading Data

Sidewall Case		Member	Shock Pressure: Peak Pressure, Psi (MPa)	Shock Pressure: Time, msec	Gas Pressure: Peak Pressure, Psi (MPa)	Gas Pressure: Time, msec
1	Two failed	Side Walls	154.7 (1.07)	1.81	28.1 (0.19)	29.95
		Floor	29.5 (0.2)	7.19	28.1 (0.19)	31.08
2-1	Sidewall 1 failed, sidewall 2 reflects	Side Walls	154.7 (1.07)	2.23	28.1 (0.19)	43.5
2-2	Sidewall 2 failed, sidewall 1 reflects	Floor	29.5 (0.2)	9.06	28.1 (0.19)	44.52
3	Two sidewalls reflect	Side Walls	154.7 (1.07)	2.65	28.1 (0.19)	5864.63
		Floor	29.5 (0.2)	11.17	28.1 (0.19)	5866.79

Table 2: Theoretical Response to Blast Loads, based on TM 5 -1300.

Connection Detail	Member	Max. Deflection, Xm, (in.) (cm)	Ductility Ratio	Rotation, θ (deg.) (rad.)	Rotational Limit (deg.) (rad.)
Specimen 7	Beam	0.68 (1.73)	2.2	0.32 (0.0056)	2 (0.0349)
	Column	2.75 (6.98)	1.8	1.31 (0.0228)	



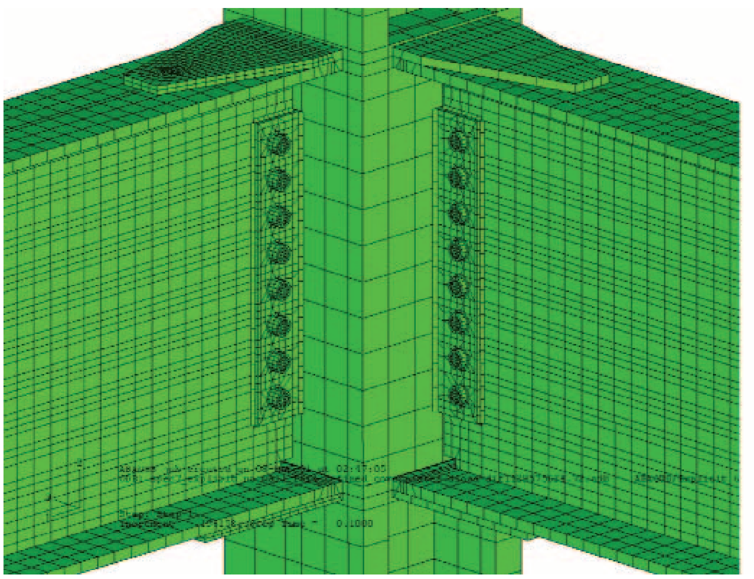


Figure 2: Representative Finite-element Model of Steel Connection

Finite-element results when blast pressures (Table 1) were applied to floor and sidewalls are summarized illustrated in Table 3. The predicted global rotations of the beams are close to the TM 5-1300 results for the frangible wall cases. However, the beams where the reflecting walls were located rotated much more than TM 5-1300 computation predicted, with the result that a greater impulse and energy are transferred to the beam and column in the room.

Sample maximum local rotations, associated with the plastic hinge that is formed in the beam, are shown in Figure 3. All local rotations for the different cases exceeded the limit of 2 degrees specified in TM 5-1300.

Note, also, that the beams twisted more severely horizontally, and clearly exceeded the TM 5-1300 limit criteria, since realistic internal blast pressures radiate outward in three dimensions. These findings indicate that severe damage in the connections comes from the blast radiating in three dimensions as well as the vertically applied pressure. Deformation data for beams and column in the various cases indicate that dead loads and DIFs enhanced structural strength, but the beam cross sections twisted additionally due to dead loads. The column rotations indicate that the columns did not significantly affect the connection damage. According to the stress and strain results, components in all connections yielded for all the cases.

These analyses show the value of investigating structural connections using high-resolution finite-element analysis. For example, a steel moment connection judged safe based on TM 5-1300 criteria failed in the finite-element simulations. Moreover, TM 5-1300 criteria may need revision to reflect findings based on more complex behaviour.

Table 3: Maximum Displacements and Rotations of Beams (Case 2-2)

			Beam 1				Beam 2			
		Response Quantity	NO DL NO DIF	NO DL DIF	DL NO DIF	DL DIF	NO DL NO DIF	NO DL DIF	DL NO DIF	DL DIF
Vertical	Global	Displacement (in)	7.308	7.033	9.521	4.070	0.527	0.519	0.509	0.504
Vertical	Global	Rotation (deg)	3.489	3.358	4.546	1.943	0.252	0.248	0.243	0.241
Vertical	Local	Displacement (in)	5.445	5.154	6.255	3.784	2.330	2.190	2.504	1.683
Vertical	Local	Rotation (deg)	13.972	13.251	15.949	9.809	6.076	5.713	6.526	4.398
Horizontal	Global	Displacement (in)	6.178	6.245	7.160	2.679	10.397	10.539	10.900	8.692
Horizontal	Global	Rotation (deg)	2.950	2.982	3.419	1.279	4.964	5.032	5.204	4.150
Horizontal	Local	Displacement (in)	4.058	3.731	4.360	2.847	6.201	5.456	6.519	4.225
Horizontal	Local	Rotation (deg)	10.504	9.675	11.266	7.412	15.818	13.998	16.588	10.926

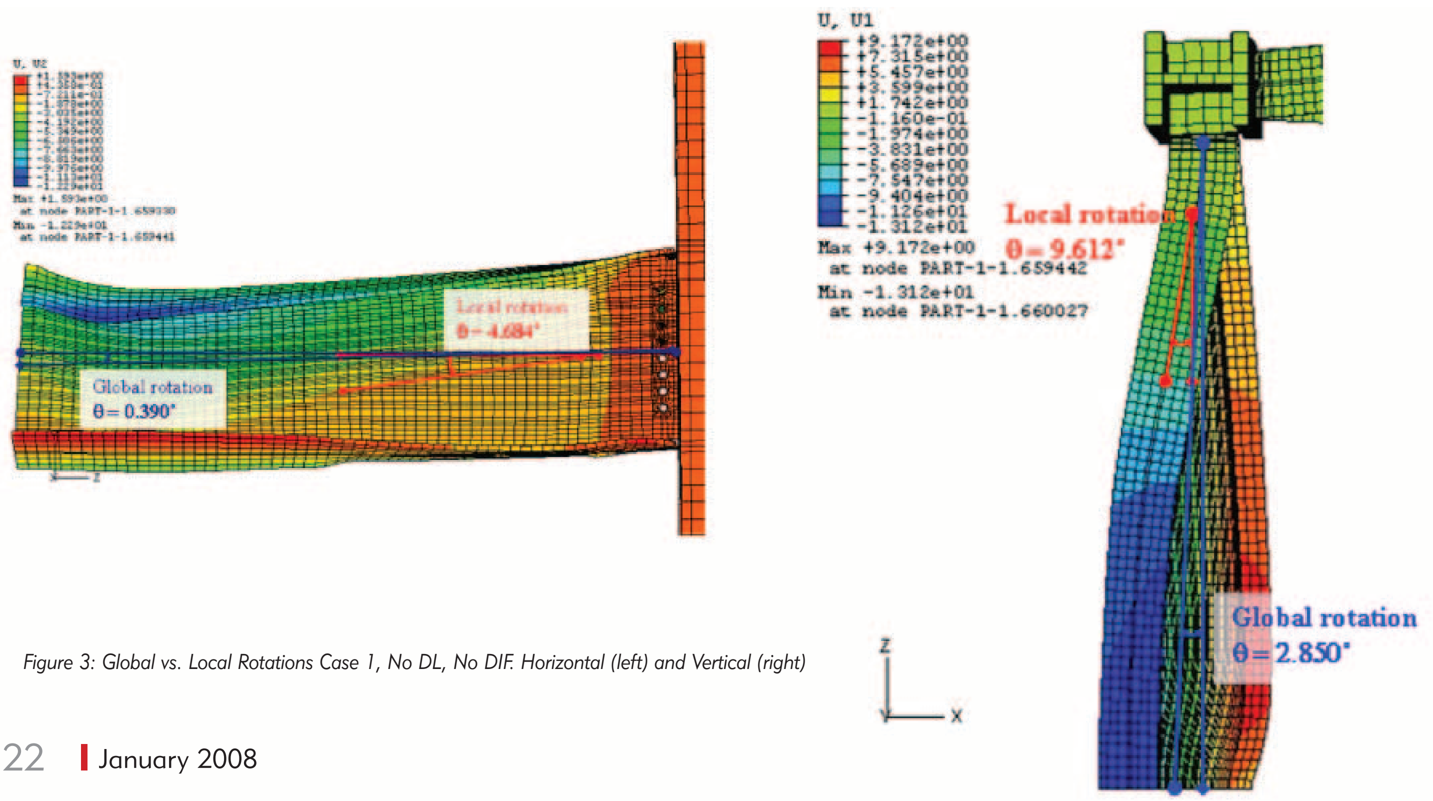


Figure 3: Global vs. Local Rotations Case 1, No DL, No DIF. Horizontal (left) and Vertical (right)

Progressive collapse of steel frame structures

Progressive collapse is a failure sequence in which local damage leads to large scale collapse in a structure. It has been an important issue in building design since the collapse of the Ronan Point apartment building in 1968 [11]. This type of intrinsically transient nonlinear phenomenon is difficult to model, understand, or design against without finite-element analysis.

Ten-story 3D moment frames with rigid and semi-rigid connections were studied for their sensitivity to failure of specific columns [3]. Three failure modes were considered: material, buckling, and connection failures. The first two have been studied extensively elsewhere [12, 13, and 14]. Experiments show that a real steel connection is neither rigid nor pinned [15]. In this study, the nonlinear moment-rotation relationship of the 10-story frame was obtained through extensive preliminary 3D finite-element simulation of steel connections.

Six initial failures with rigid and semi-rigid connections were used to analyze the frames for progressive collapse of five stories. Frame columns were based on a simple Load and Resistance Factor Design (LRFD) procedure manual [16]. Both ideal (rigid plus hinge) and semi-rigid connections were adopted for the progressive collapse analyses.

Analyses were performed up to seven seconds after the "initial failure", modeled by instantaneous removal of a designated column. The failure cases are shown in Figure 4. Only Case 6, where three columns were removed, caused total collapse of the building. Figure 5 shows the result for Case 6 of the building with ideal connections. Case 6 with semi-rigid connections also collapsed, but differently, as shown in Figure 6. The first failure was initiated at a connection, as shown in Figure 6-(b). As connections failed, the floors above the removed columns started to fall to the ground, and it caused columns buckling in the 6th floor, as shown in Figure 6-(c). These

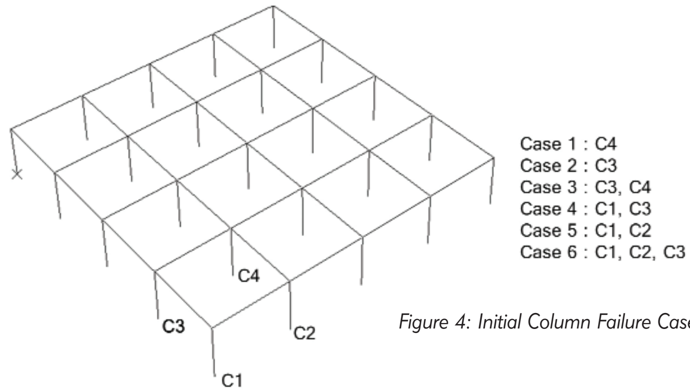


Figure 4: Initial Column Failure Cases

column buckling cases initiated horizontal failure propagation in the 6th floor, and the whole floor failed. After that, the columns in the first floor buckled because the floors collapsed, leading to the total collapse of the building.

The 10-story frame, designed for gravity and lateral loads, performed fairly well in the simulations. Even though the ideal and semi-rigid connection cases both caused total collapse for Case 6, they showed very different qualitative behaviour. The collapse of the semi-rigid connection case was caused by a cascade of local failures, such as connection failures and columns buckling. However, the collapse of the ideal connection case was caused by column buckling in the first floor. These different failure mechanisms are quite apparent in the nonlinear finite-element results.

The analyses also showed that once failure propagation initiated (i.e. horizontal column buckling), it would not stop until it caused total, or almost total, collapse. To protect a structure against progressive collapse, horizontal column buckling propagation appears to be the most critical factor to control.

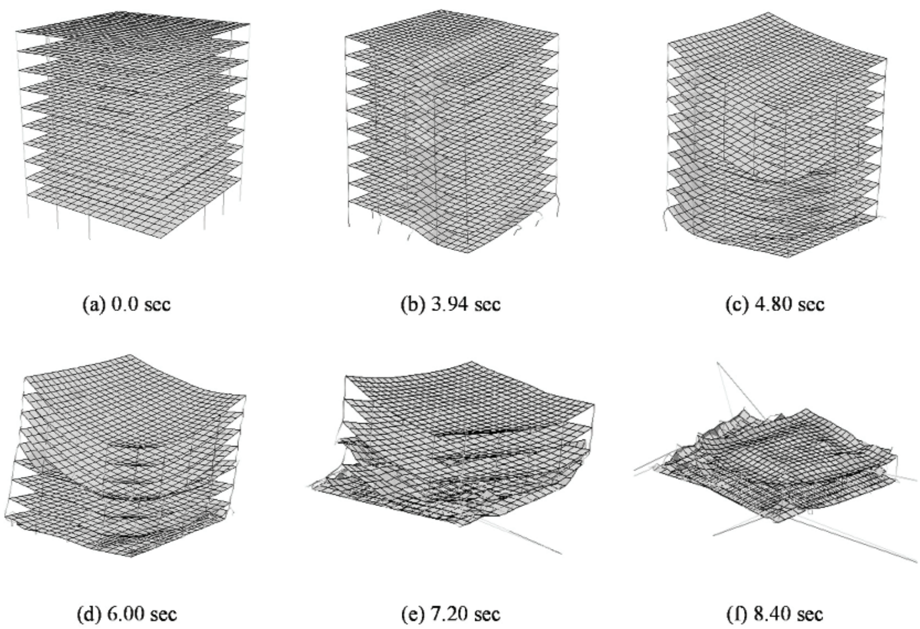


Figure 5: Case 6 with Ideal Connections

"…standard design criteria may not be conservative enough…"

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Conclusions

FEA analyses reliably simulate critical aspects of structural behaviour, such as the details of steel connection designs and failure modes.

Our analyses suggest that connection behaviour under blast loading can vary significantly from standard design criteria. Importantly, the simulations also suggest that standard design criteria may not be conservative enough for the cases modeled here and may require refinement and revision in light of nonlinear transient effects, such as progressive collapse. Finite-element analysis of progressive collapse due to blast effects also reveals qualitative information about structural failure such as, in these cases, sensitivity of failure mode to connections.

Contact

Jeffrey Cipolla
Simulia
jeffrey.cipolla@simulia.com

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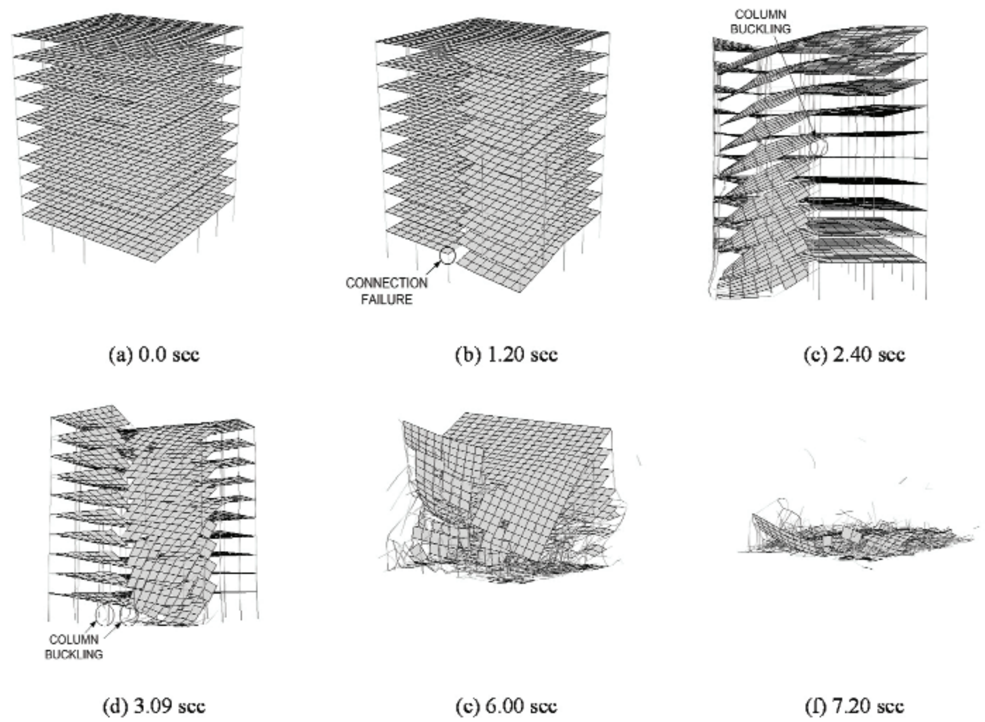

Figure 6: Case 6 with Semi-rigid Connections

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