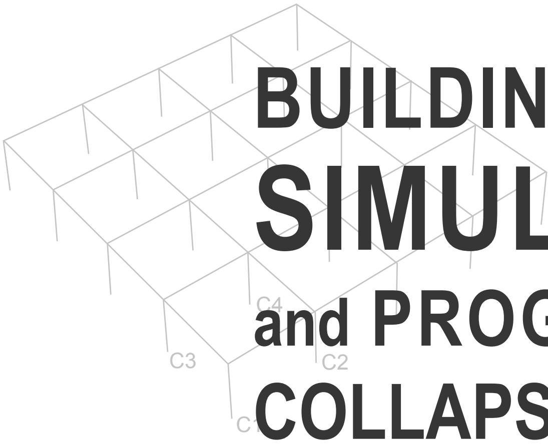
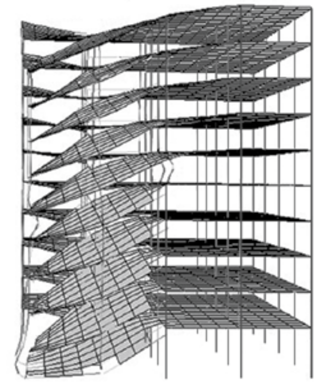


BUILDING BLAST SIMULATION and PROGRESSIVE COLLAPSE Analysis



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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. It also suggests that established structural design guidelines for severe blast loading on steel structural members contain underestimates and may need revising. In the study described below, we gained a number of significant insights through modeling the progressive collapse of a steel frame structure.

Analysis of Steel Frame Connections Under Blast Loads

Current U.S. design guidelines for steel connections in structures subjected to blast loads are based on recommendations in Department of Defense Technical Manual (TM) 5-1300TM. The approach idealizes real structures and structural elements as “equivalent” lumped-mass single degree-of-freedom systems. In addition, the guidelines are for single-storied steel frames, not subjected to any significant dead loads.

We used Abaqus/Explicit Version 6.5 to assess the behavior of steel moment connections under dead loads. We then compared the maximum rotational capacities of the connections to values derived with the TM 5-1300 approach, using four different load cases for each connection type (see Table 1).

Reference maximum blast pressures were based upon the TM 5-1300 criteria, using standalone codes SHOCK and FRANG to compute equivalent shock pressures and gas pressures (Table 1) and assuming an 18.5 lb. TNT charge in 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 and dead loads equivalent to those for a 10-story office building on the top flanges of the beams and axially on the top cross-section of the column. The numerical model was analyzed with and without dead loads to evaluate their influence on connection response.

An isotropic elasto-plastic model simulated the material property for each connection component. Yield and ultimate strengths were increased to account for strain rate effects using dynamic increase factors (DIF) as recommended in TM 5-1300. 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-node continuum brick elements with reduced integration.

When blast pressures were applied to the floor and sidewalls, the predicted global rotations of the beams were close to the TM 5-1300 results for the frangible wall cases. However, the beams near reflecting walls rotated much more than the TM 5-1300 computation predicted, transferring

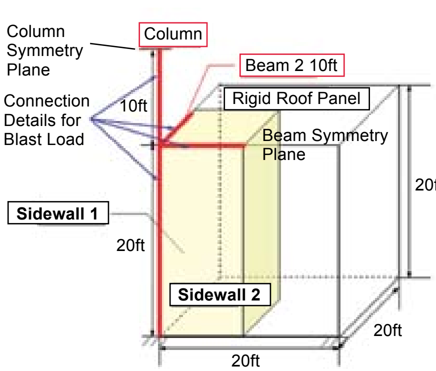

Figure 1: Geometrical Model used for Numerical Study

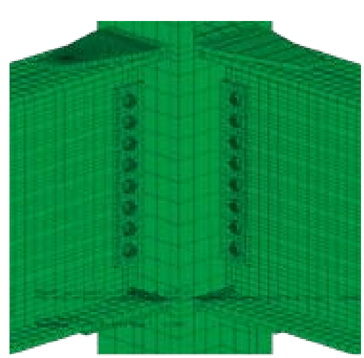

Figure 2: Representative Finite-element Model of Steel Connection

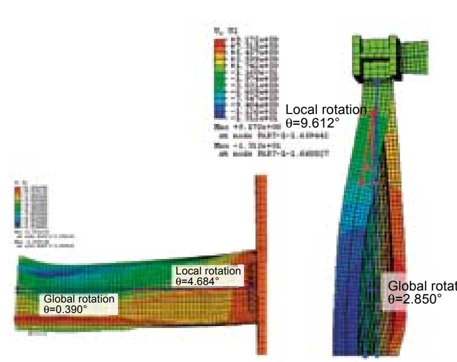

Figure 3: Global vs. Local Rotations Case 1, No DL, No DIF. Horizontal (left) and Vertical (right)

greater impulse and energy to the beam and column (Figure 3). All local rotations for the different cases exceeded the limit of 2 degrees specified in TM 5-1300.

The beams twisted severely horizontally and clearly exceeded the TM 5-1300 limit criteria. These findings indicate that extensive damage in the connections comes from the blast radiating in three dimensions as well as the vertically applied pressure. Deformation data for beams and columns 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, the TM 5-1300 criteria may need revision to reflect findings based on more complex behavior.

Progressive Collapse of Steel Frame Structures

In progressive collapse, local damage leads to large-scale structural failure—an intrinsically transient nonlinear phenomenon that is difficult to model, understand, or design against without finite element analysis.

We studied ten-story 3D moment frames with rigid and semi-rigid connections for their sensitivity to material, buckling, and connection failures of specific columns. The nonlinear moment-rotation relationship of the 10-story frame was obtained through extensive preliminary 3D finite-element simulation of steel connections.

Six initial failure cases 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 LRFD design procedure manual. Both ideal (rigid plus hinge) and semi-rigid connections were adopted for the progressive collapse analyses.

In the simulation with ideal connections, only Case 6, where three columns were removed, caused total collapse of the building. Case 6 with semi-rigid connections also collapsed, but differently, as shown in Figure 5. The first failure was initiated at a connection. As additional connections failed, the floors above the removed columns fell, causing columns to buckle in the 6th floor. These 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.

Even though the ideal and semi-rigid connection cases both caused total collapse for Case 6, nonlinear finite element results showed very different qualitative behavior. The collapse of the semi-rigid connection case was caused by a cascade of local failures, such as connection failures and

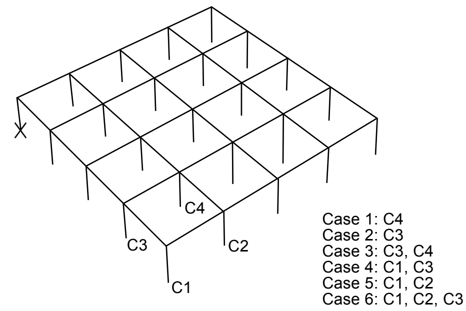

Figure 4: Initial Column Failure Cases

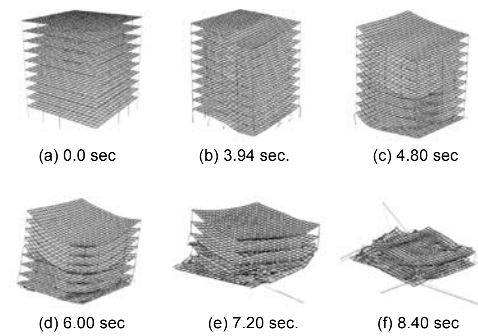

Figure 5: Case with Semi-rigid Connections

columns buckling. However, the collapse of the ideal connection case was caused by column buckling in the first floor. 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. Horizontal column buckling propagation appears to be the most critical factor to control.

Table 1: Loading Data

Sidewall Case		Member	Shock Pressure		Gas Pressure	
			Peak Pressure, Psi (MPa)	*Time, msec*	*Peak Pressure, Psi (MPa)*	*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

Conclusions

Our analyses suggest that connection behavior under blast loading varies significantly from standard design criteria, which may not be conservative enough 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 revealed sensitivity of failure mode to connections.

The complete technical paper, "Building Blast Simulations," was presented at the NAFEMS 2007 World Congress. It can also be downloaded from the SIMULIA website.

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