Development of 3D Models of Steel Moment-Frame Buildings for Assessment of Robustness and Progressive Collapse Vulnerability

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ABSTRACT

Several prototype steel moment-frame buildings have been designed for the purpose of assessing their vulnerability to progressive collapse. The buildings were designed for moderate and high seismic regions. This paper summarizes the development of three-dimensional finite element models of these prototype buildings, with a focus on the modeling approach used for the connections and the composite floor system. Initial simulation results under a column removal scenario are presented to illustrate the model capabilities. Ongoing assessments of reserve capacity and progressive collapse vulnerability using these models are part of a larger study aimed at quantifying and comparing the relative robustness of different structural systems.

INTRODUCTION

As part of the ongoing NIST (National Institute of Standards and Technology) research on prevention of progressive structural collapse, several prototype steel moment-frame buildings have been designed for the purpose of assessing their vulnerability to progressive collapse [Liang, Shen, and Ghosh, 2006]. The buildings were designed for Seismic Design Categories C and D (moderate and high seismic regions). This paper summarizes the development of three-dimensional finite element models of these prototype buildings, with a focus on the modeling approach used for the connections and the composite floor system.

Connections are modeled using a macromodel approach (e.g., [Khandelwal, El-Tawil, Kunnath, and Lew, 2008] and [Sadek, El-Tawil, and Lew, 2008]), in which a combination of beam and discrete spring elements is used to represent the nonlinear behavior and failure of the various connections in each building. Macromodels for the moment connections have been developed and validated against both high-fidelity finite element simulations and full-scale test data under column removal scenarios [Sadek, Main, Lew, Robert, and Chiarito, 2009], and Sadek, El-Tawil, and Lew [2008] validated a macromodel approach for a simple shear connection against high-fidelity finite element simulations. These previous studies considered the behavior of the connections in two dimensions, and in this study the macromodel approach is extended to three-dimensional connection behavior. While Sadek, El-Tawil, and Lew [2008] presented high-fidelity finite element simulations of a composite floor system under a column removal scenario, a simplified modeling approach is proposed in this paper, in which the concrete slab and metal deck are represented by a single layer of shell elements.

Initial simulation results under a column removal scenario are presented to illustrate the model capabilities. Ongoing assessments of reserve capacity and progressive collapse vulnerability using these models are part of a larger study aimed at quantifying and comparing the relative robustness of different structural systems.

PROTOTYPE STEEL MOMENT-FRAME BUILDINGS

The prototype steel frame buildings are 10-story office buildings with plan dimensions of 30.5 m by 45.7 m (100 ft by 150 ft), which utilize moment frames as the lateral load resisting system. Two plan layouts were considered: (a) 5 bays by 5 bays and (b) 5 bays by 3 bays. To examine the effectiveness of seismic design and detailing in resisting progressive collapse, buildings were designed for the following two Seismic Design Categories (SDCs):

- SDC C – (Atlanta, Georgia), which resulted in a design using Intermediate Moment Frames (IMFs) for the lateral load resisting system, and
- SDC D – (Seattle, Washington), which resulted in a design using Special Moment Frames (SMFs) for the lateral load resisting system

Connections used in the moment frames were selected from the prequalified steel connections specified in FEMA 350 [FEMA, 2000]: (a) Welded Unreinforced Flange-Bolted Web (WUF-B) connections for the IMFs required for the building in the SDC C zone, and (b) Reduced Beam Section (RBS) connections for the SMFs required for the building in the SDC D zone. Different combinations of plan layout, SDC, and moment frame configurations result in a total of five prototype steel frame building designs, as listed in Table 1.

Building	Plan Layout	SDC	Moment Frame Configuration
1	5 bays by 5 bays	C	perimeter only
2	5 bays by 5 bays	D	perimeter only
3	5 bays by 5 bays	D	perimeter and interior
4	5 bays by 3 bays	C	perimeter only
5	5 bays by 3 bays	D	perimeter and interior

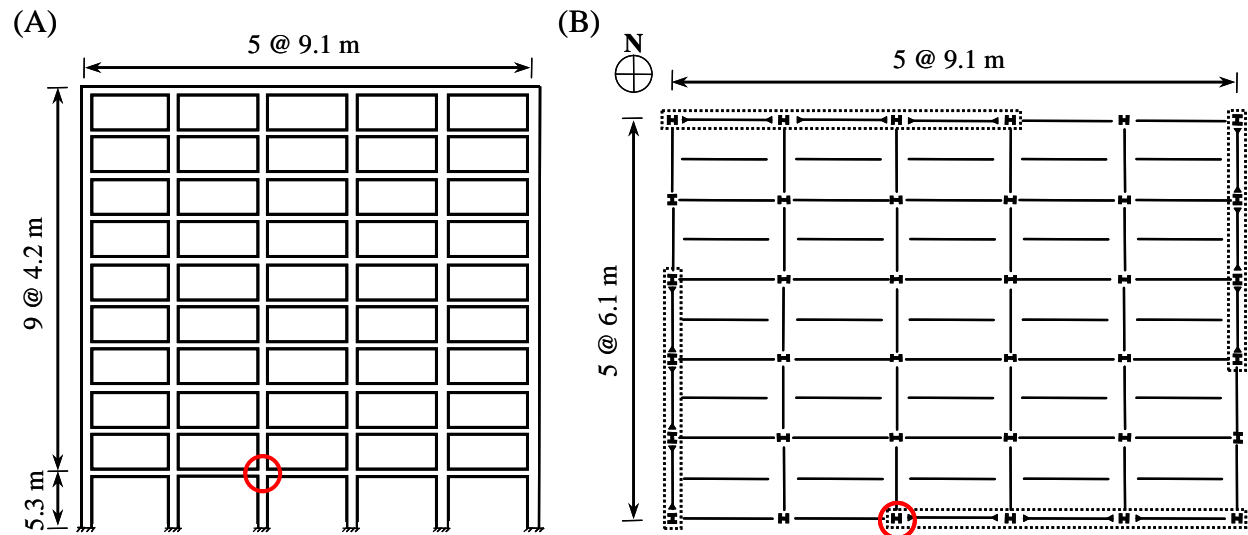TABLE 1 – SUMMARY OF PROTOTYPE STEEL MOMENT-FRAME BUILDINGS

FIGURE 1 – (A) E-W ELEVATION AND (B) PLAN VIEWS OF PROTOTYPE BUILDING 1 DESIGNED FOR SDC C. (MOMENT FRAMES OUTLINED WITH DASHED LINES; CONNECTIONS IN CIRCLED REGION SHOWN SCHEMATICALLY IN FIGURE 2.)

The design loads on the buildings were based on the ASCE (American Society of Civil Engineers) Standard 7-02 [ASCE, 2002]. The seismic and wind loads were obtained from appropriate maps in ASCE 7-02 for the two locations. The material design standards used in the design of the members and their connections were those referenced in ASCE 7-02. For typical floors, the dead load consisted of the self-weight of the floor of 2.2 kPa (46 lbf/ft^2) and a super-imposed dead load of 1.4 kPa (30 lbf/ft^2), while the design live load was assumed to be 4.8 kPa (100 lb/ft^2). For the roof, the self-weight of the slab was 2.2 kPa (46 lb/ft^2), the super-imposed dead load was 0.48 kPa (10 lb/ft^2); and the design live load, with a permitted live load reduction, was 0.96 kPa (20 lb/ft^2).

Elevation and plan views of prototype building 1 (see Table 1) are shown in Figure 1. The lateral load resisting system for this building, comprised of moment frames, was located only around the perimeter, as can be seen in Figure 1(B). Gravity frames were used for the interior of the building. The floor system consisted of a 76 mm (3 in) metal deck topped with 82.5 mm (3.25 in) of lightweight concrete with a unit weight of 17.3 kN/m^3 (110 lbf/ft^3). The slab acted compositely with the steel beams via shear studs. The steel beams in the gravity frames were connected to the columns using single plate simple shear (gravity) connections.

MODEL DEVELOPMENT

In addition to the five steel moment-frame building designs discussed above, two steel braced-frame designs have also been developed and will be investigated as part of the NIST study. Three-dimensional finite element models of at least seven steel frame buildings are thus required to enable a systematic evaluation and comparison of their relative robustness. To develop these models efficiently and accurately, a script has been developed that reads the requisite information (including building layout and dimensions, beam and column section types and section properties, connection types and connection properties, and material properties) from text input files and automatically assembles a finite element model of the structure in LS-DYNA format [LSTC, 2007]. The target mesh size is a parameter in the script, allowing models with different degrees of mesh refinement to be readily generated.

The models consist primarily of (a) Hughes-Liu beam elements with cross-section integration to represent the beams and columns for the framing of each building and (b) fully integrated shell elements to represent the combined behavior of concrete slab and metal deck. A piecewise-linear plasticity model is used for the steel beams and columns, with stress-strain data and a failure strain characterizing the mechanical properties of the steel. A single layer of shell elements was used to represent the concrete slab and metal deck, using a material model that allows for different stress-strain behavior in tension and compression. The contribution of the welded wire fabric reinforcement and the metal deck to the tensile capacity of the slab was accounted for in this model. Discrete beam elements, spring elements, rigid links, and contact interfaces were also used in modeling of the connections and the composite floor system, as is further detailed in the following sections.

MODELING OF CONNECTIONS

Figure 2 illustrates a three-dimensional macromodel representation of the connections circled in Figure 1. The concrete slab is not shown in this figure, as the influence of the slab on the connection behavior is discussed in the following section. The E-W girder shown at the right of Figure 2 is part of a moment frame and is attached to the column flange by a WUF-B connection.

A rectangular panel zone is formed by horizontal continuity plates welded between the column flanges at the level of each E-W girder flange. The E-W beam at the left of the figure and the N-S beam in the upper right are attached to the column flange and web, respectively, by single plate simple shear connections.

The panel zone is modeled by rigid links on the four sides with moment releases at the corners, which permit shear forces in the panel zone to be carried by a diagonal spring. The stiffness and yield strength of this panel zone spring are calculated from expressions given by Khandelwal, El-Tawil, Kunnath, and Lew [2008], and the spring is modeled as elastic, perfectly plastic. In the WUF-B connection used in the SDC C buildings, the upper and lower flanges of the girder, which are welded to the column flange, are modeled explicitly using beam elements with a piecewise-linear plasticity model for the steel and a specified failure strain at which the elements are deleted. Rigid links connect the ends of these flange elements to the beam elements along the girder centerline.

The WUF-B connection also incorporates a shear tab bolted to the web of the girder, and this bolted connection is modeled using a single discrete beam element for each bolt. Using the discrete beam formulation, load-displacement curves and failure displacements are specified for both the axial and shear behavior of the element. This allows for modeling of vertical and/or horizontal shearing of the bolts, as well as other failure mechanisms that might govern for loading in a particular direction, such as web tearout and block shear. Distinct load-displacement curves are defined for each unique combination of bolt type and shear tab thickness. A similar modeling approach is used for the simple shear connections, with a single discrete beam element for each bolt. Rigid links connect the ends of these discrete beam elements to the beam elements along the girder or beam centerline. As shown for the N-S beam connection in Figure 2, the column elements extend partially into the panel zone region from below to allow for attachment of the simple shear connection to the column web.

For the RBS connection used in the SDC D buildings (not shown in Figure 2), the panel zone is modeled using the same approach described above for the WUF-B connection. The segment of girder with flange cutouts is modeled using a series of beam elements with varying cross-sectional dimensions. The welded connection of the girder to the column flange is modeled as perfectly rigid, since the RBS connection is designed to yield in the reduced section.
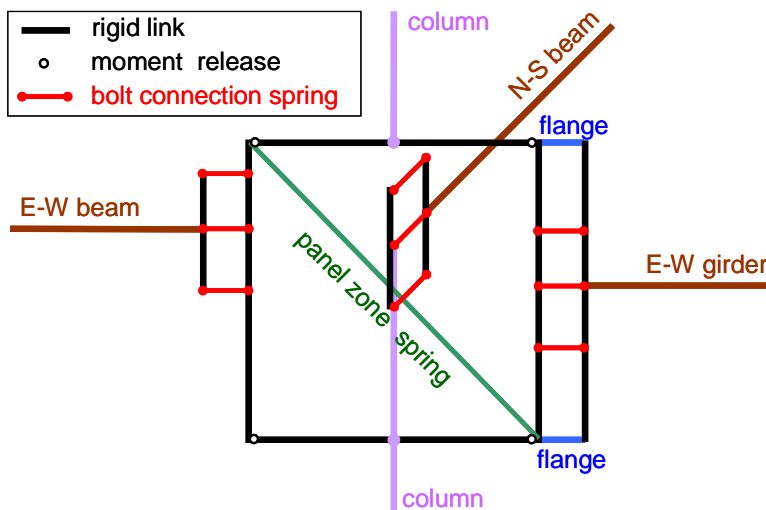

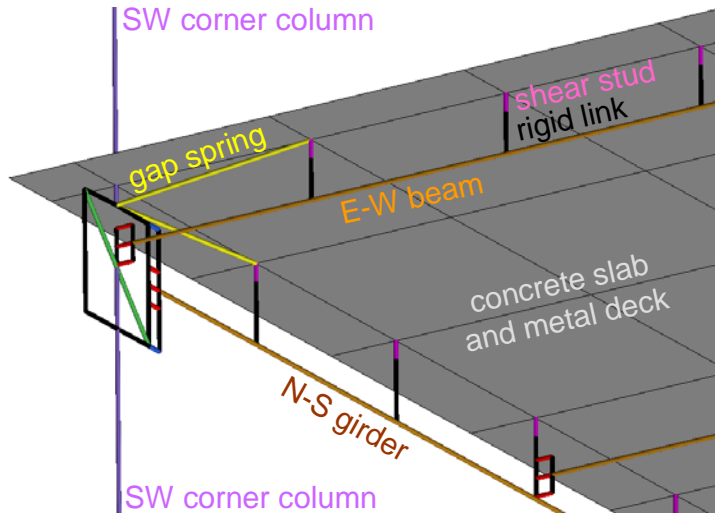FIGURE 2 – MACROMODEL REPRESENTATION OF CONNECTIONS CIRCLED IN FIGURE 1

FIGURE 3 – VIEW FROM BELOW OF CONCRETE SLAB AND METAL DECK, SHOWING MODELING
OF SHEAR STUDS AND GAP SPRINGS (FOR BEARING OF SLAB AGAINST COLUMN)

MODELING OF COMPOSITE FLOOR SYSTEM

Figure 3 shows a view of the underside of the composite floor system in the vicinity of the
southwest corner column, illustrating the approach used to model the composite behavior. The
finite element mesh shown in Figure 3 was generated with a target element size of 914 mm
(36 in). The mesh is constructed in such a way that nodes along the beams and girders are
aligned directly below nodes of the shell elements representing the concrete slab and metal deck,
so that these nodes can be interconnected by elements representing shear studs.

   As shown in Figure 3, rigid links extend vertically from the centerline of the beams and
girders to the top-of-steel elevation, and discrete beam elements representing shear studs connect
these rigid links to nodes of the shell elements. Using the discrete beam formulation, load-
displacement curves and failure displacements are specified for both the axial and shear behavior
of the shear stud elements. Rotational resistance is also specified for the shear stud elements to
provide torsional restraint along the top flange of the beams and girders. The number of shear
studs along each beam in the model depends on the specified target element size, but it is
generally less than the number specified in the design. The axial and shear resistance of the shear
studs is scaled up accordingly to account for this.

   The prototype building designs do not call for shear studs in the plastic hinge zones of girders
within moment frames. However, as shown along the N-S girder in Figure 3, rigid links and
discrete beam elements are still used to interconnect these girders with the concrete slab and
metal deck in these regions, in order to represent the torsional restraint provided by contact of the
metal deck with the top flange of the girder. To represent the absence of shear studs,
substantially smaller shear and axial resistances are specified for the discrete beam elements.

   A contact interface is also defined to prevent interpenetration of the beam and shell elements
representing the beams, girders, columns, and the concrete slab and metal deck. Gravity loads are
transferred from the floor deck to the beams and girders through the contact definition. Elements
defined with initial penetration (e.g., the shell elements immediately surrounding a column) are
excluded from the contact interface to avoid the introduction of spurious forces. To model
bearing of the concrete slab and metal deck against the columns, discrete spring elements are
defined that carry compression only after an initial gap is closed. Although these "gap springs"

are inclined, as shown in Figure 3, horizontal orientation vectors are used to specify that forces are only transmitted normal to the vertical planes of contact.

INITIAL SIMULATION RESULTS

To illustrate the model capabilities, initial simulation results are presented for prototype building 1 (see Table 1 and Figure 1) under a column removal scenario. The building model used in the simulations had a target element size of 914 mm (36 in) and consisted of 54 236 elements. Simulations were performed using explicit time integration. The column below the circled connections in Figure 1 was selected for removal under gravity loading corresponding to the design dead load plus 25 % of the design live load. The gravity loading was applied gradually using a smooth ramp function to avoid introduction of spurious dynamic effects. Computed deformations of the structure under gravity loading prior to column removal are shown in Figure 4 with contours of vertical displacement. Note that the displacements shown for the upper stories include the accumulated effect of column shortening over the height of the building.

After initialization of the gravity loading, the elements representing the column were deleted instantaneously with the gravity loading held constant. The connections above the deleted column elements (shown in Figure 2) were left intact when the column was removed. After column removal, the connection above the removed column dropped vertically to a peak deflection of 128 mm (5.02 in) before rebounding and oscillating about a new equilibrium position. Computed deformations of the structure at the instant of peak vertical displacement are shown in Figure 5 with contours of vertical displacement. The connections shown in Figure 2 were thus able to redistribute the loads after column removal, preventing a progressive collapse.
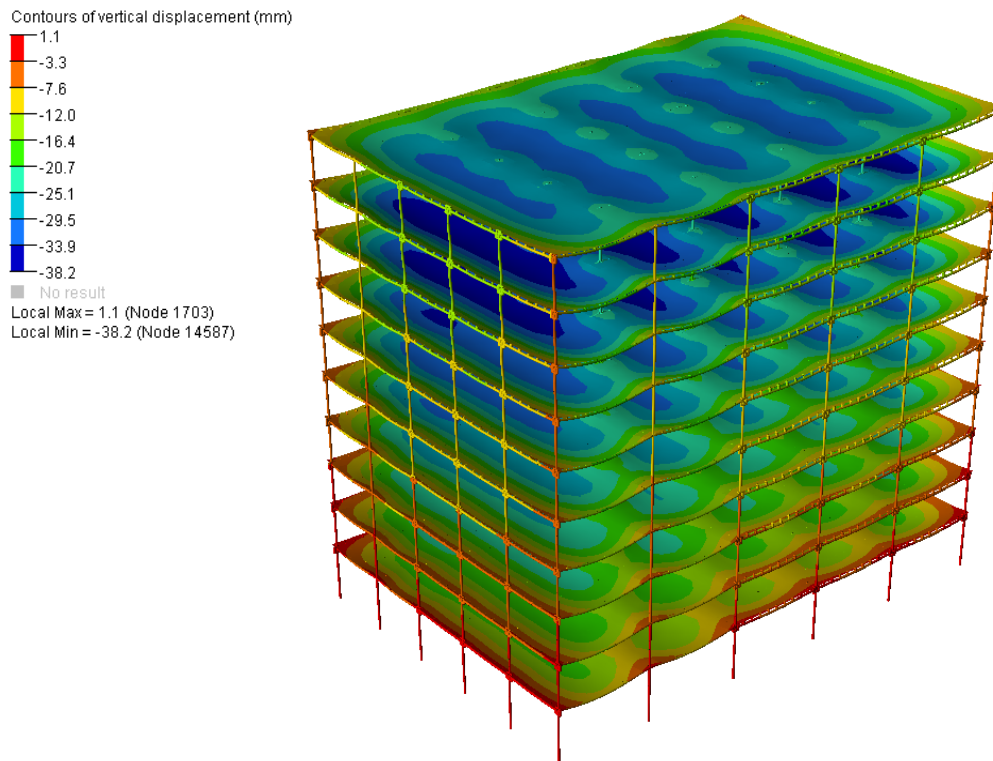

FIGURE 4 – VIEW FROM SOUTHWEST SHOWING DEFORMATIONS UNDER GRAVITY LOADS WITH CONTOURS OF VERTICAL DISPLACEMENT (DEFORMATIONS SCALED UP BY 50)
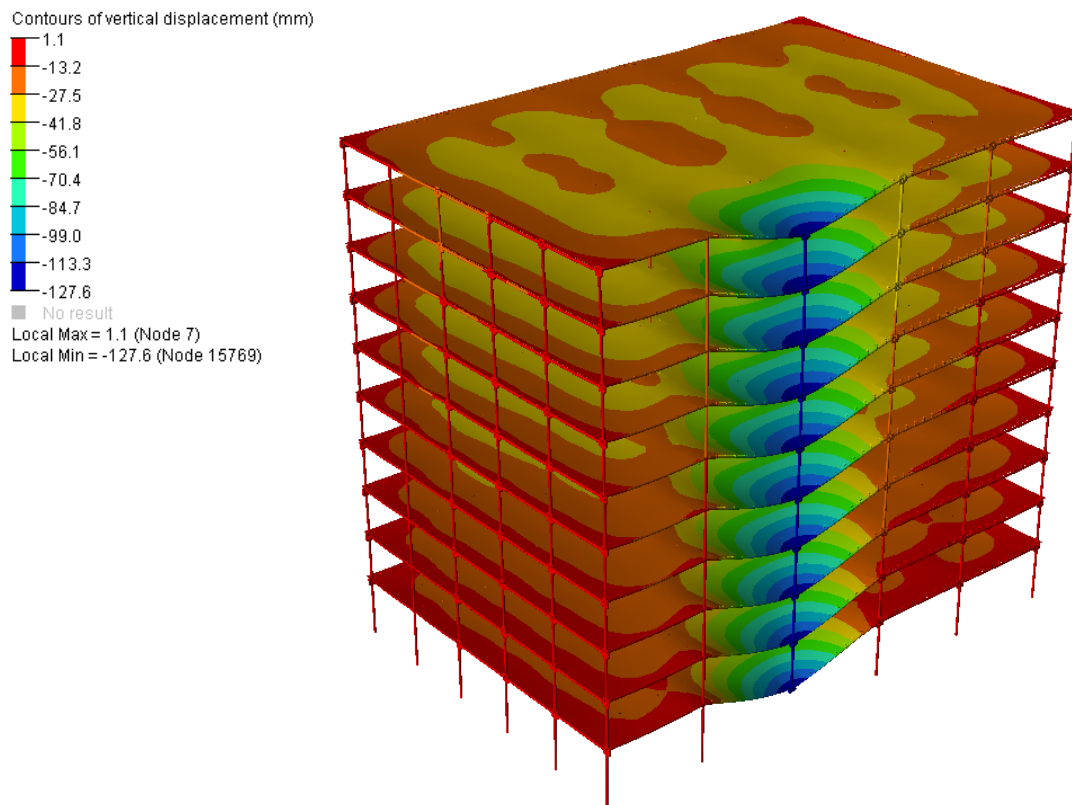
FIGURE 5 – VIEW FROM SOUTHWEST SHOWING PEAK DISPLACEMENTS AFTER COLUMN REMOVAL WITH CONTOURS OF VERTICAL DISPLACEMENT (DEFORMATIONS SCALED UP BY 20)

CONCLUDING REMARKS

This paper has summarized the development of three-dimensional finite element models of steel moment-frame buildings, with particular focus on the macromodels used to represent the connections and the modeling approach used for the composite floor system. A script has been developed to enable efficient and accurate assembly of three-dimensional models for a number of prototype steel moment-frame buildings. To illustrate the model capabilities, initial simulation results were presented for a 10-story, 5 bay by 5 bay building designed for SDC C under a column removal scenario. Future work will involve validation of the modeling approach proposed for the composite floor system against high-fidelity finite element simulations presented by Sadek, El-Tawil, and Lew [2008]. Systematic analyses of the reserve capacity and progressive collapse vulnerability of the prototype buildings will then be performed with the aim of quantifying and comparing the relative robustness of different structural systems.

DISCLAIMER

Certain trade names or company products are mentioned in the text to specify adequately the procedure used. Such identification does not imply recommendation or endorsement by NIST, nor does it imply that the product is the best available for the purpose.

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