

Alternate Path Progressive Collapse Analysis of Steel Stud Bearing Wall Structures¹

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ABSTRACT

The Alternate Path Method is used to examine the vulnerability of a 5-story barracks building to progressive collapse based on DoD criteria UFC-4-023-03. The building is composed of composite deck floor system and steel roof trusses, all supported by cold-formed steel stud load bearing walls. Two different wall section removal scenarios at the first story of the building were considered for the analysis. The analysis was performed using the “Extreme Loading® for Structures” software, a 3-dimensional nonlinear dynamic analysis software.

The results show the ability of the composite deck floor slabs to bridge over the removed wall sections. The slabs were able to re-distribute the gravity loads to adjacent wall components, while some of the loads were picked up in tension by the stud walls above the slabs. The study provides a good understanding of the composite deck floor-cold-formed steel stud bearing walls building system in resisting progressive collapse.

INTRODUCTION

In 2001, the Department of Defense (DoD) started to publish design guidelines for structures to resist progressive collapse resulting from an extreme loading event. The design guidelines are given in the Unified Facilities Criteria UFC 4-023-03 “Design of Buildings to Resist Progressive Collapse” (UFC, 2009). This document provides the design requirements necessary to reduce the potential of progressive collapse for new and existing DoD facilities. Since then, engineers are on the lookout for design guides that explain the implementation of DoD guidelines in conventional and new construction systems. This paper provides some guidance for implementing

the DoD progressive collapse criteria in buildings constructed with cold-formed steel stud bearing wall systems.

Cold-formed steel load bearing wall systems

The use of cold-formed steel (CFS) load bearing framing in typical mid-rise construction (three to eight stories in height) brings the value of reducing associated construction cost in comparison to other structural framing systems such as reinforced masonry, structural steel and wood. A key component of the cost savings is the lighter mass of the structure, which is the key element to the design of the lateral force resisting system (shear walls) and the foundation. CFS load bearing stud walls must be braced laterally at intermediate points to restrain the studs against lateral and torsional buckling. X-brace flat straps are commonly used in this type of construction to provide the building resistance to lateral wind and seismic forces. The X-brace system provides pre-determined load path for the lateral shear forces, and therefore requires vertical stacking of the shear walls to ensure the load transfer from all floors down to the foundation. The design of cold-formed steel framing in the US is governed by the specification published by the American Iron and Steel Institute (AISI, 2007).

It is important to note that several types of floor systems can be integrated with CFS load bearing walls. Among these types are composite steel deck-concrete slabs (composite deck), CFS joists or open bar joists with a concrete or wood floor diaphragm, and precast hollowcore concrete planks. The composite deck combines the structural advantage of a flat slab with the time saving advantage of a permanent form. The steel deck furnishes the total bottom reinforcement of the composite slab, while additional top reinforcement is required. Due to the composite action between the concrete and the steel deck, the slab can support a greater floor live load compared to a typical reinforced concrete slab of the same depth. More information about CFS load bearing wall systems are given by Rahman (2006, 2007).

DoD progressive collapse design criteria

Unified Facilities Criteria 4-010-01 (UFC, 2007) requires that all new and existing military facilities of three stories or higher be designed to avoid progressive collapse. UFC 4-023-03 (UFC, 2009) offers two general design approaches to reduce the possibility of progressive collapse; Direct Design approach and Indirect Design approach. The Direct Design approach includes the “Alternate Path” method, which is based on flexural performance of the floor system, as the building must bridge across removed vertical supporting elements. The Indirect Design approach includes the “Tie Forces” method, which is based on the catenary (membrane tension) response of the structure, and is used to enhance continuity, ductility and structural redundancy of the building. The level of progressive collapse design and the selection of the design approach and method are based on the Occupancy Category (OC) of the building. This OC is specified in UFC 3-310-01 (UFC, 2005), but also allowed to be specified by the building owner. Engineers dealing with buildings with OC II have the option of using the Alternate Path Method or the Tie Forces Method

for the progressive collapse analysis. However, design for buildings with OC III and above must include the Alternate Path Method for the analysis.

The Alternate Path Method follows the LRFD philosophy with load combination for extraordinary events and resistance factors to define design strengths. Three analysis procedures can be used; linear static, nonlinear static and nonlinear dynamic. The estimate of design strength varies depending on the classification of the action; either deformation-controlled or force-controlled action. The nonlinear dynamic analysis selected for this study requires the creation of a three-dimensional model for the primary framing members of the building. Inclusion of secondary framing members in the model is optional, but they must be checked with the results of the analysis if omitted. To perform the analysis, gravity loads and notional lateral loads are to be applied to the model until equilibrium is reached. Then, specified vertical load bearing components (columns or walls) shall be removed from the model and analysis shall continue until maximum displacement of slab is reached, or one cycle of vibration occurs at the location of the removed component. The acceptance criteria are dependent on the structural material. In this study, axial forces are checked as force-controlled action in CFS studs, and displacement is checked as deformation-controlled action in composite deck slab.

Applied Element Method for collapse analysis of structures

The Applied Element Method (AEM) is a nonlinear dynamic analysis method that is capable of predicting, to a high degree of accuracy, the continuum and discrete behavior of structures. The method is proved to track the structural collapse behavior passing through the stages of crack initiation and propagation in tension-weak material, yielding and formation of plastic hinges in ductile material, element separation, element re-contact, and collision with the ground or with adjacent structural components. In the AEM, a structure is modeled as an assembly of small elements. The two elements shown in Figure 1 are assumed to be connected by series of normal and shear springs located at contact points, which are distributed around the element edges. Normal and shear springs totally represent stresses and deformations of the elements. Steel bars in reinforced concrete components are modeled using continuous springs connecting elements together.

Each applied element has six degrees of freedom. These degrees of freedom represent the rigid body motion of the element. Although the element motion is as a rigid body, its internal deformations are represented by its springs' deformation. The global stiffness matrix is determined by summing up the stiffness matrices of springs around each element. Consequently, the developed stiffness matrix has total effects from all of the springs according to the stress status around the element. This technique can be used in both load and displacement control cases, and static and dynamic loads. For more details about the AEM, refer to publications by Meguro and Tagel-Din (2001 and 2002). The AEM is implemented in the Extreme Loading[®] for Structures (ELS) software (ASI, 2009), which is used to run the analysis in the current study.

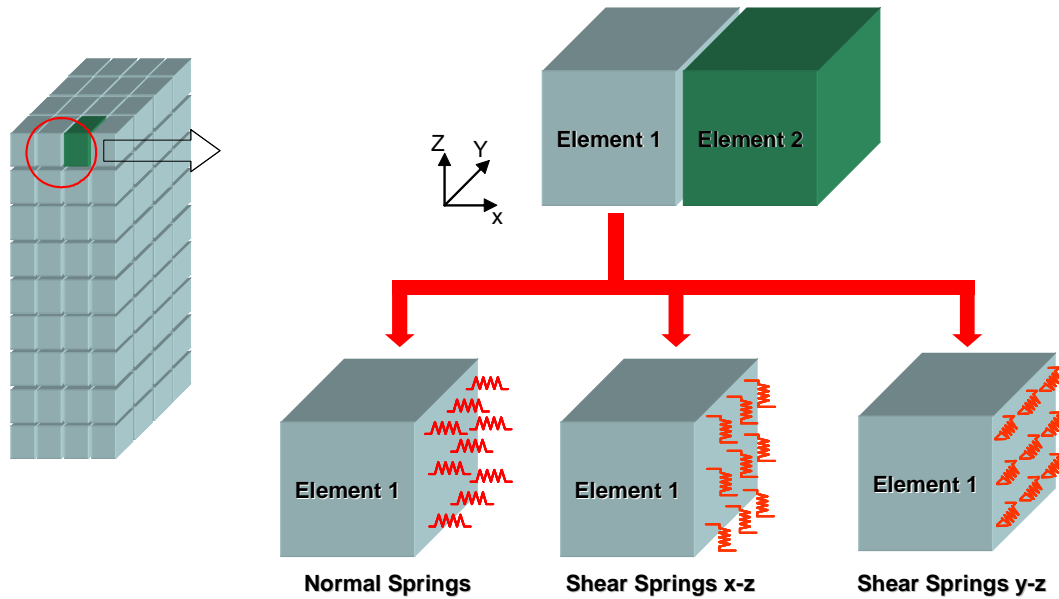


Figure 1. Connectivity springs in the Applied Element Method

BARRACKS BUILDING CASE STUDY

Building Description. Figure 2 shows the elevation, plan view and overall dimensions of the barracks building. The building is a 5-story structure composed of two wing areas and a core area with a typical slab-to-slab height of 3048 mm (120 inch) and first story height of 3200 mm (126 inch). The core area is constructed from structural steel framing, while cold-formed steel stud walls are used in the wing areas as the supporting structure for the gravity loads from the floor slabs and the roof. In the wing areas, the exterior walls on the short side of the building and every separation wall are gravity load bearing walls as shown in Figure 3. The figure also shows the locations of the X-brace strap system, with cold-formed steel stud boundary members, which is used as shear walls to resist lateral loads. The light weight roof trusses are supported on the corridor walls and the exterior walls of the long side of the building. Table 1 shows the schedule of the studs and top and bottom tracks of the main load bearing walls in standard nomenclature. The studs are distributed on typical 406 mm (16 inch) spacing, with weak axis bracing at mid-height. Table 2 shows the schedule of the X-brace shear walls in the longitudinal (east-west) and transverse (north-south) directions. All cold-formed steel (CFS) sections with nomenclature “S” and “T” have 228 MPa (33 ksi) yield strength, and all CFS sections with nomenclature “SG” and “C” have 345 MPa (50 ksi) yield strength. The diagonal straps also have 345 MPa (50 ksi) yield strength.

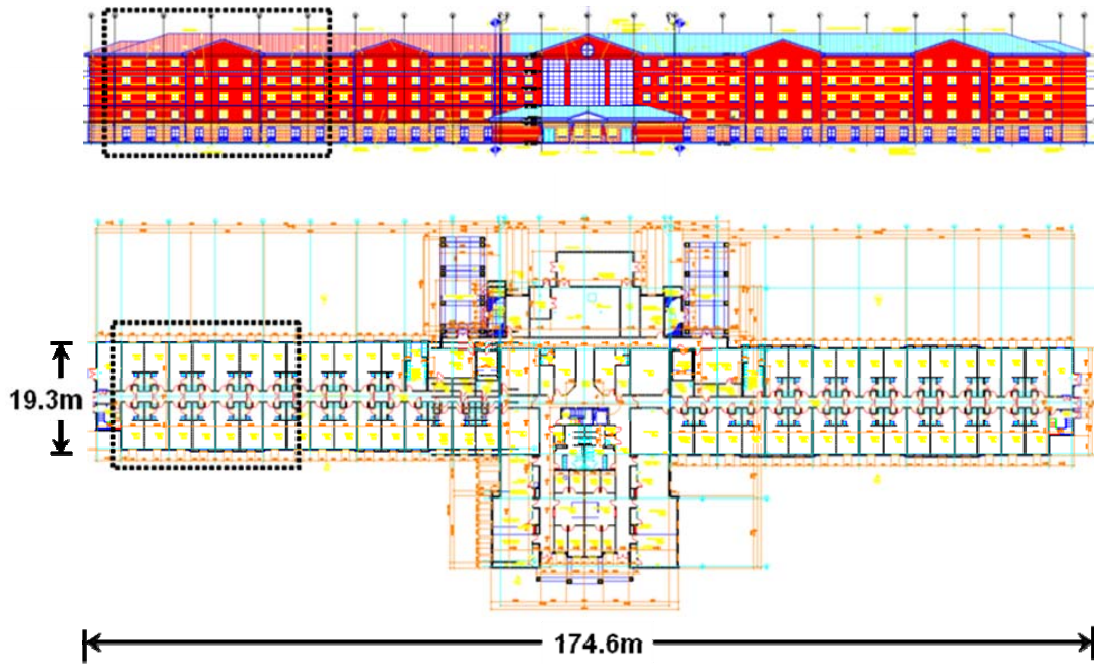


Figure 2. Layout of the barracks building

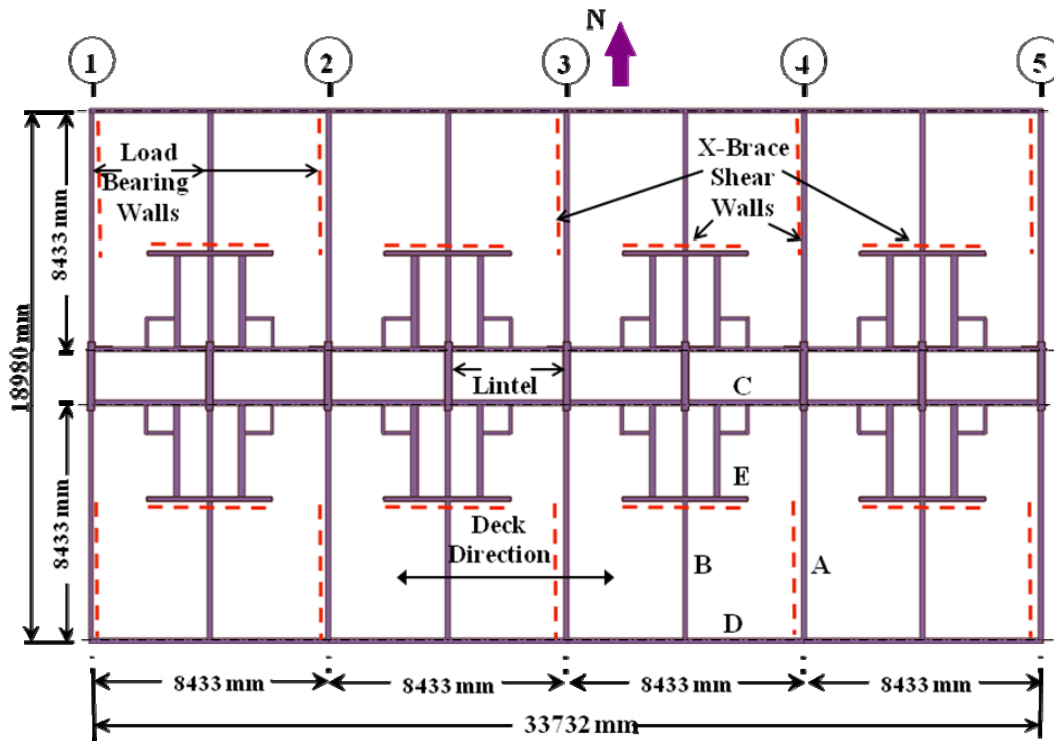


Figure 3. Cold-formed steel framing for wing areas

Table 1. Schedule of load bearing walls (standard nomenclature)

Story	Wall A & B		Wall C	
	Studs	Tracks	Studs	Tracks
5	600S162-43	600T200-43	600S162-43	Built-up section
4	600S162-43	600T200-43	600SG200-43	600T200-43
3	600SG300-43	600T200-43	600SG200-43	600T200-43
2	600SG300-43	600T200-43	600SG200-43	600T200-43
1	600SG300-54	600T200-54	600SG200-43	600T200-54

Story	Wall D		Wall E	
	Studs	Tracks	Studs	Tracks
5	600SG162-43	Built-up section	600S162-33	600T125-43
4	600SG162-43	600T200-43	600S162-33	600T125-43
3	600SG162-43	600T200-43	600S162-33	600T125-43
2	600SG162-43	600T200-43	600S162-33	600T125-43
1	600SG162-43	600T200-43	600S162-33	600T125-43

Table 2. Schedule of X-brace shear walls (standard nomenclature)

Story	Longitudinal Direction (E-W)		Transverse Direction (N-S)	
	Boundary Members	Diagonal Straps	Boundary Members	Diagonal Straps
5	600C250-43	(2) 400-54	600C250-43	(2) 400-54
4	600C250-68	(2) 600-54	600C250-54	(2) 400-54
3	600C250-118	(2) 800-54	600C250-97	(2) 600-54
2	(2)600C250-97	(2) 800-68	(2)600C250-68	(2) 800-54
1	(2)600C250-118	(2) 800-68	(2)600C250-97	(2) 800-54

The floor slab is a composite deck system supported every 4216 mm on separation walls with 50 mm (2 inch) deep, 0.9 mm (30 mil, 20 ga) thick corrugated steel sheet and 100 mm (4 inch) concrete on top of the corrugated sheet. The slab reinforcement includes welded wire mesh 152x152 MW13.3/13.3 (6x6 W2.1/2.1) on top of the deck, and top reinforcement of 16 mm diameter (#5) bars at 150 mm (6 inch) spacing in both directions. The top reinforcement is used as part of a special design requirement for the floor slabs to resist an upward blast pressure specified in a previous version of the UFC 4-023-03. Figure 4 shows the actual cross-section of the composite deck, and the simplified cross-section used in the current numerical analysis model. In the analysis model, the composite deck was simplified into a 150 mm (6 inch) thick concrete slab with a reduced unit weight of the concrete to account for the void space created by the corrugated deck. In addition, the corrugated steel sheet was modeled as an equivalent area of rebar and spacing in the longitudinal direction only, located at the centroid of the deck profile as shown in the figure.

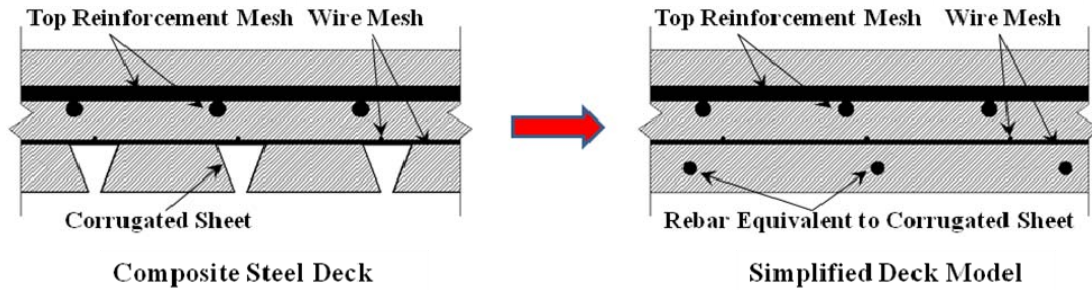


Figure 4. Cross-section of composite steel deck and the simplified model

The connection between the composite deck floor slabs and both the top track of the load bearing walls below and the bottom track of the load bearing walls above uses 4 mm (0.157 inch) diameter powder actuated fasteners. Four fasteners are used at each stud location and one fastener between studs. The steel studs are welded to the top track and the bottom track of the walls at both side of each stud.

Analysis Model. The three-dimensional model of the barracks building was created using the graphical interface and section database of the Extreme Loading[®] for Structures (ELS) software, Version 3.1. One part of the building, as outlined in Figure 2, and only the first four stories of the building have been considered in the model. This was decided to make the modeling time and analysis time of the building practical. The weight of the fifth story walls was uniformly distributed and added to the weight of the slab below. The weight of the roof trusses was added as line load to the locations of its supporting corridor walls and exterior walls of the long side of the building. Figure 5 shows the three-dimensional model for the part of the building used in the analysis.

All window and door openings on the exterior and the interior of the building were modeled with their appropriate dimensions and framing members. Full connections were used between these framing members attached together, and that between wall studs and their top and bottom tracks. The diagonal straps of the X-brace shear walls were modeled as long tension springs. Powder actuated fasteners for composite deck attachments were modeled as short tension and shear springs. The interface layer between the wall tracks and the composite deck was modeled as material with concrete compressive strength, but with very low shear and tension strengths. Table 3 gives the material properties and strength reduction factors for all components in the numerical model.

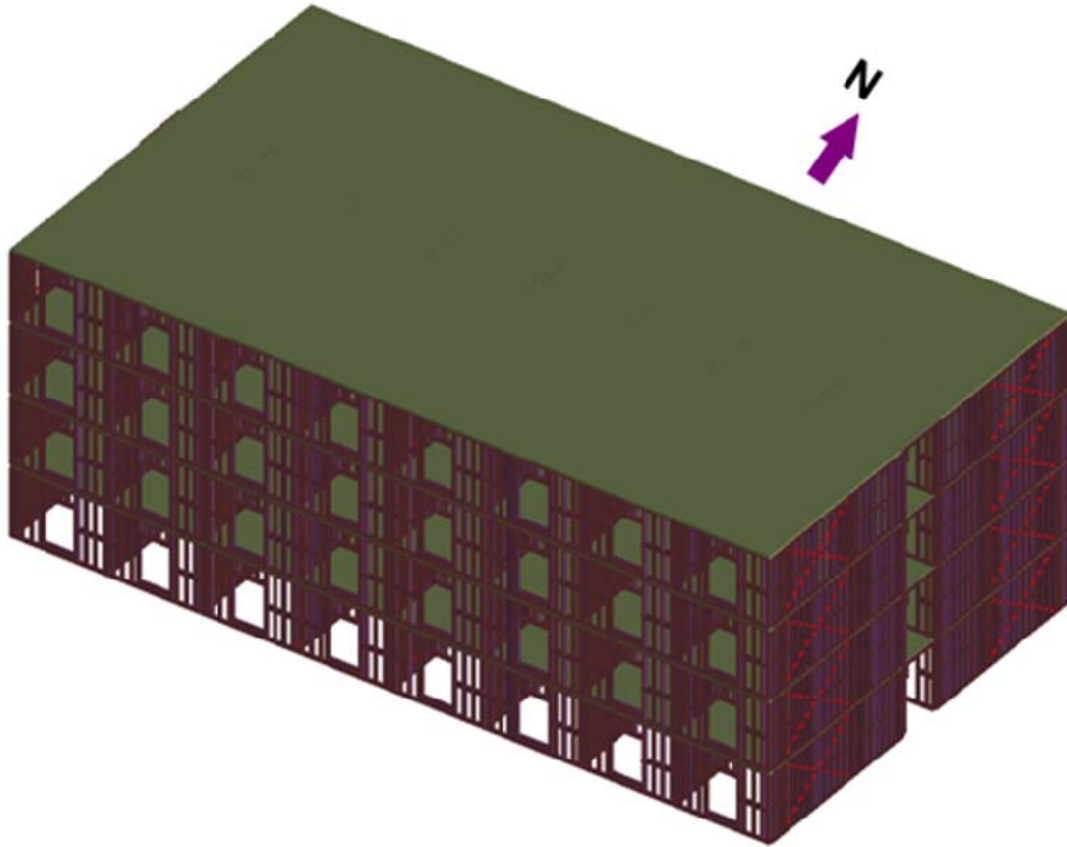


Figure 5. Three-dimensional model of the barracks building

Table 3. Lower bound material properties and strength reduction factors for components

Material	Min. Φ	Type	Young's Modulus (MPa)	Material Strength (MPa) or (kN)
Concrete (ACI 318-08)	0.90	Tension	26,000	$F_c = 27.6$ $F_t = 2.5$
	0.65	Compression and Bearing		
Reinforcement (ACI 318-08)	0.90	Wire mesh and Rebar	200,000	$F_y = 415$ $F_t = 620$
Cold-Formed Steel Framing (AISI S100-07)	0.90	Tension	200,000	$F_y = 228$ or 345 $F_t = 310$ or 450
	0.85	Compression		
Steel Deck	0.85	Composite Concrete	200,000	$F_y = 275$ $F_t = 415$
Powder Actuated Fasteners	0.5	Deck/Track Connection	200,000	Tension = 4.3 kN Shear = 4.1 kN

Loading and wall removal. The load combination applied for the analysis was “1.2D + 0.5L”, as recommend by UFC 4-023-03 for nonlinear dynamic analysis. The total load calculated for floor slabs was 6.3 kN/m² (132 psf) and for roof level was 3.7 kN/m² (78 psf). Two wall removal scenarios for the Alternate Path Method at the first story of the building were considered in this analysis. Scenario 1 was removal of an exterior wall section at the short side of the building equals to twice the first story height, or 6.4 m. Scenario 2 was removal of a corner wall section of the building equals to the first story height, or 3.2 m, in each direction. The two wall removal scenarios are illustrated in Figures 6 and 7.

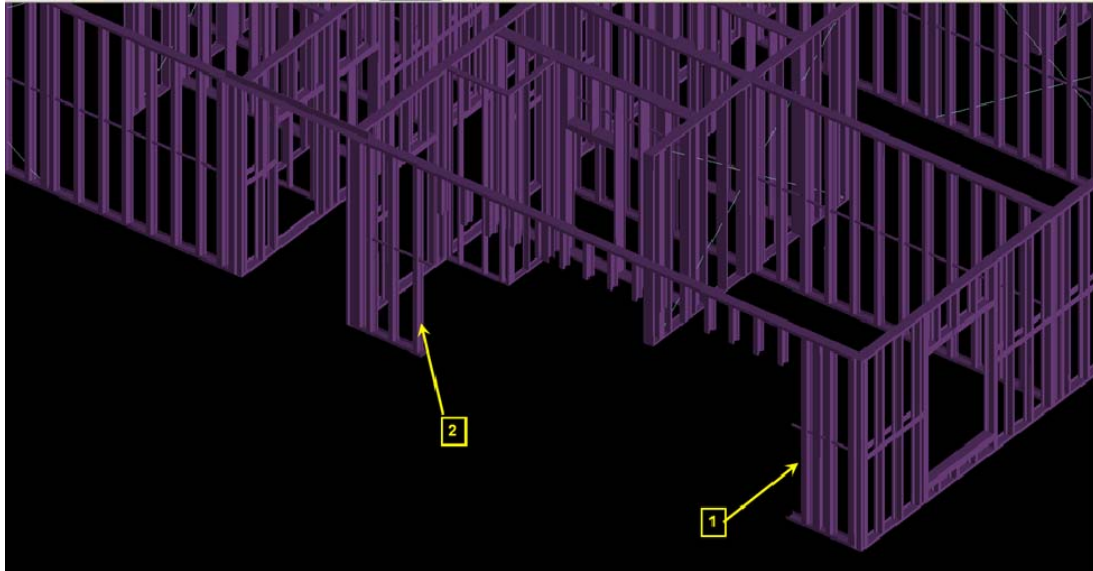


Figure 6. Removal of an exterior wall section of the building

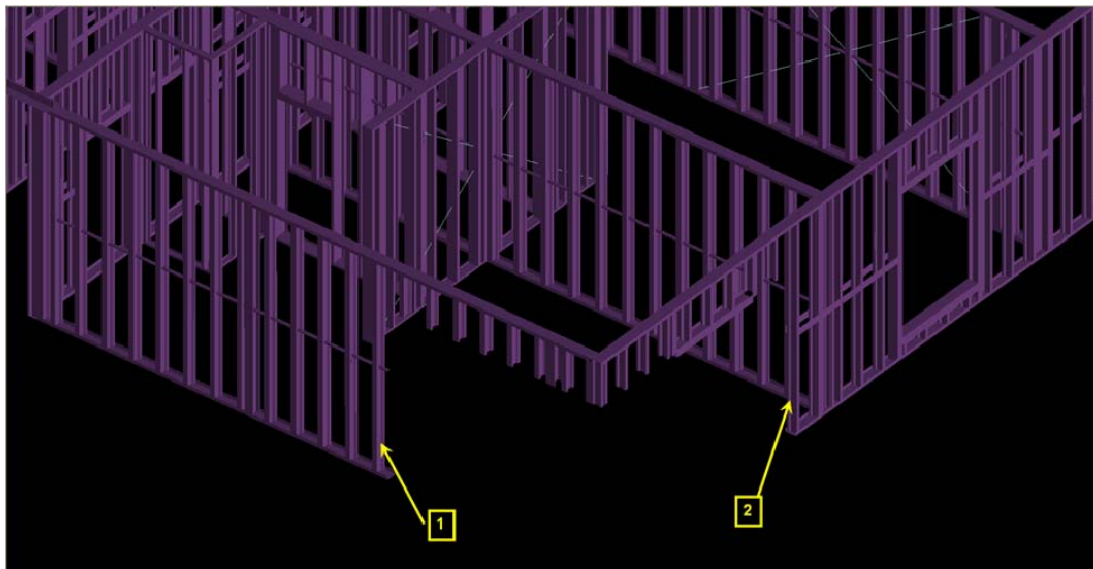


Figure 7. Removal of a corner wall section of the building

RESULTS AND DISCUSSION

There was no progressive collapse or component failure observed in the two wall removal scenarios analyzed in this study. Figure 8 show the deflection history for the floor slabs after wall removal in Scenario 1. Upon removal of the wall section in the model at this location, all the floor slabs started to deflect downward until reaching a maximum deflection of about 3 mm. The deflection stabilized for all floor slabs around an average value of 2.25 mm. Similar deflection behavior was observed for Scenario 2 wall removal with a maximum deflection of 3.5 mm. The limited deflection observed in both scenarios can be attributed to the ability of the composite deck floor slabs to distribute the load in both directions due to the existence of the relatively heavy top reinforcement mesh (16 mm bars every 150 mm). If the special upward blast pressure design requirement did not exist, the top reinforcement could be easily reduced while maintaining the integrity of the floor slabs for the progressive collapse design.

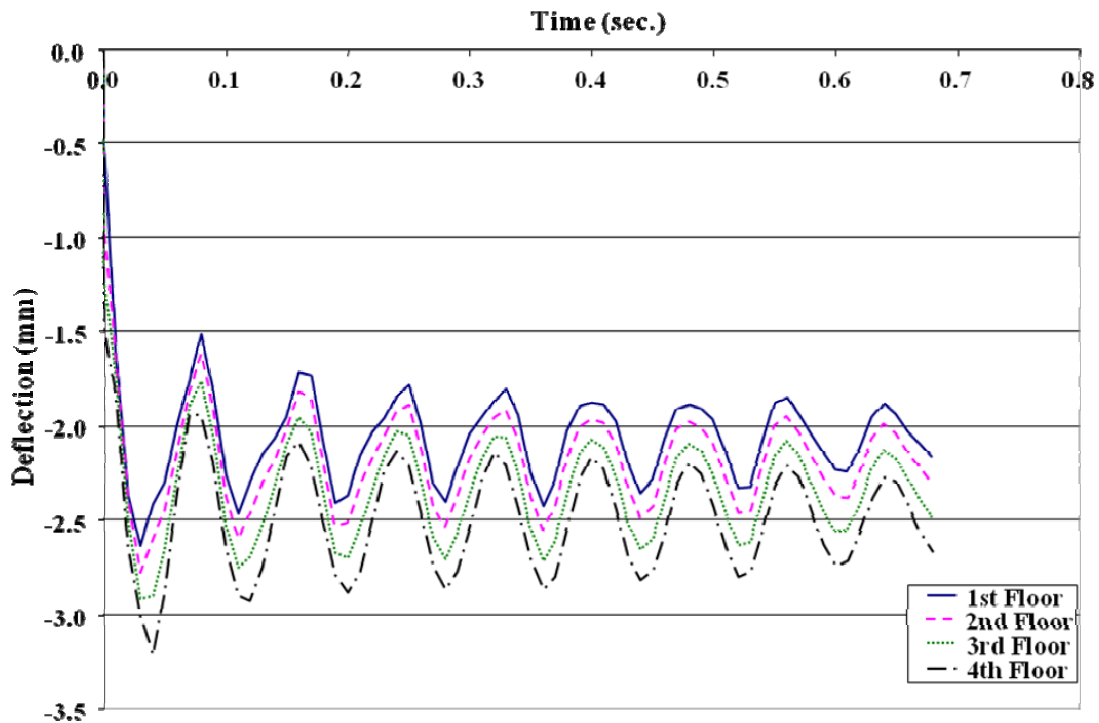


Figure 8. Deflection of floor slabs after wall removal in scenario 1

The axial forces in the adjacent studs to the removed wall sections were monitored before and after wall removal. Table 4 shows the cross sections of the numbered studs, the axial compression forces in the studs prior to wall removal, the sustained axial compression forces in the studs after wall removal, and the LRFD compression strengths of the studs calculated based on the AISI S100-07. It can be noticed that the loss of wall support of the first floor slab of the building caused the load acting on the wall to re-distribute through the slab to the nearest stud support.

The axial forces in the studs increased significantly, but still met the force-controlled acceptance criteria for compression members (ΦP_n) as shown in the table.

Table 4. Axial forces in adjacent studs for wall removals

Scenario and Stud Number	Stud Cross-section	Axial Force Before Wall Removal (kN)	Axial Force After Wall Removal (kN)	LRFD Compression, ΦP_n (kN)
Sc. 1- Stud 1	600SG300-54	8.5	33.7	88.8
Sc. 1- Stud 2	600SG300-54	5.4	24.5	88.8
Sc. 2- Stud 1	600SG300-54	16.5	49.8	88.8
Sc. 2- Stud 2	600SG162-43	5.9	20.2	27.8

The tension forces in the powder actuated fasteners connecting the composite deck floor slabs to the stud walls were also monitored. No failure was observed in these fasteners. The maximum tension force recorded after wall removal was about 0.3 kN, which is less than the acceptance criteria of the fasteners in tension. The tension strength of fasteners is controlled by the pullover strength of the steel sheet around the head of the fastener, ΦP_{nov} , which is calculated as $0.5 \times 4.3 = 2.15$ kN.

CONCLUSION

The current study provides a good understanding of the composite deck floor-cold-formed steel stud bearing walls building system in resisting progressive collapse. The results of the study show that the Alternate Path Method coupled with the nonlinear dynamic analysis procedure can be efficiently used to examine the vulnerability of multi-story buildings to progressive collapse based on DoD criteria UFC-4-023-03. The Extreme Loading[®] for Structures (ELS) software, with its Applied Element Method, assisted in the application of the nonlinear dynamic analysis procedure. The study considered removal of an exterior wall section and a corner wall section at the first story of a 5-story barracks building. The results show the ability of the composite deck floor slabs to bridge over the removed wall sections at the first story of the building. The slabs were able to re-distribute the gravity loads to adjacent wall components, while some of the loads were picked up in tension by the studs above the slabs through the fasteners connecting the slabs to the walls. Although the axial compression forces in the studs adjacent to the removed wall sections increased significantly, the redundancy in the cold-formed steel wall system helped preventing overstressing these studs and exceeding the limit acceptance criteria for compression members.

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