Proceedings of the
Annual Stability Conference
Structural Stability Research Council
Orlando, Florida, April 12-15, 2016

Progressive Collapse Resistance of Composite Steel Frame Structures under Corner Column Removal

Safa S. Masajedian[1], Robert G. Driver[2]

Abstract
The significant contribution of the slab to progressive collapse resistance of composite steel frames needs to be studied to better address instability issues of the structure under extreme loading conditions. Interaction between the connection and the slab during a column removal scenario causes the ultimate response of structure to differ significantly from what is expected in conventional design philosophy. Under large deformations, the membrane forces developed in the slab, which are mainly carried by the reinforcement and metal deck, play an important role in collapse resistance of the structure, and consequently on the structural stability. On the other hand, complete tensile membrane forces (catenary action) cannot be developed without lateral restraint at the edges of the slab. Through this research, a high-fidelity finite element modelling approach is developed to predict the stability and behaviour of the gravity steel frame with composite metal deck, under corner column removal scenario. The model is reliable for predicting both linear and nonlinear performance of the structure. The model is used to investigate the influence of different factors such as metal deck thickness, slab reinforcement, and loading condition on the failure mode and instability of the system. The results show that despite all cracking and crushing that occurred in the composite slab, increasing the metal deck thickness enhanced the overall rotational and loading capacity of the system, while the loading capacity was not improved significantly by increasing the slab reinforcement.

1. Introduction
Recently, several studies have been conducted to address the behaviour of steel connections under a column removal scenario. For the progressive collapse analysis of typical steel building structures, consideration of the slab becomes extremely important when the slab experiences large displacement (Foley et al. 2009; Sadek et al. 2008; Alashker et al. 2010; Alashker and El-Tawil 2011; Masajedian and Driver 2015; Yu et al. 2010).

The results of previous studies showed that the membrane forces developed in the slab play an important role in collapse resistance of the structure. Membrane forces in the slab are mainly carried by the reinforcements provided in the slab, which are anchored to the edge or the compression ring initiated in the slab (Park and Gamble 2000). However, complete tensile membrane forces (catenary action) cannot be developed without lateral restraint at the edges of

[1] Graduate Research Assistant, University of Alberta, Edmonton, Canada <masajedi@ualberta.ca>
[2] Professor, University of Alberta, Edmonton, Canada <rdriver@ualberta.ca>

the slab. Regan (1975) performed a corner column removal on a reinforced concrete structure, and the results showed that large deflections happened along the diagonal path joining the columns, with a compression zone at the low point.

Although all the previous studies concluded that there is a significant contribution of the floor slab in progressive collapse resistance, Li and El-Tawil (2014) mentioned that it can increase the demand imposed on the connections. Also, the slab can promote the collapse by pulling on and damaging other components of the structure, once a threshold is exceeded.

Therefore, consideration of the slab remains a key aspect of investigating the real behaviour of structures under unexpected and extreme loading patterns. Through this research, a high-fidelity finite element modelling approach is developed to simulate the response of the composite steel frame under a condition when a corner column is compromised. The model is reliable for predicting both linear and nonlinear performance of the structure. The purpose of this paper is focused on determining the influence of parameters such as metal deck thickness, slab reinforcement, and the failure mode on the progressive collapse resistance of the composite steel frame structures analytically.

2. Finite Element Modelling

The numerical study consists of two phases. During the first phase, a 1x1 bay gravity steel frame consisting of beams and columns with shear tab connections is modeled without a slab. Through this model, the behaviour and failure mode of shear connections under a column removal scenario is studied. In the second phase, a composite metal deck–slab attached to the steel frame is included in the model, and analyzed under two different loading scenarios (pulling down the corner column, and incrementally increasing a distributed load). Comparison of these two models provides better insight regarding to the behaviour, failure mode, and capacity of the system. Effects of metal deck thickness and slab reinforcement on the robustness of the structure when a major load carrying component is compromised are investigated.

2.1 Description of Numerical Model

A detailed modeling approach was developed to study a 1 bay × 1 bay gravity framing system with single-plate shear connections and a composite concrete slab on metal deck under corner column removal. To investigate the effect of the composite slab on the collapse resistance of the system, two analyses were carried out. Figure 1 illustrates both models.

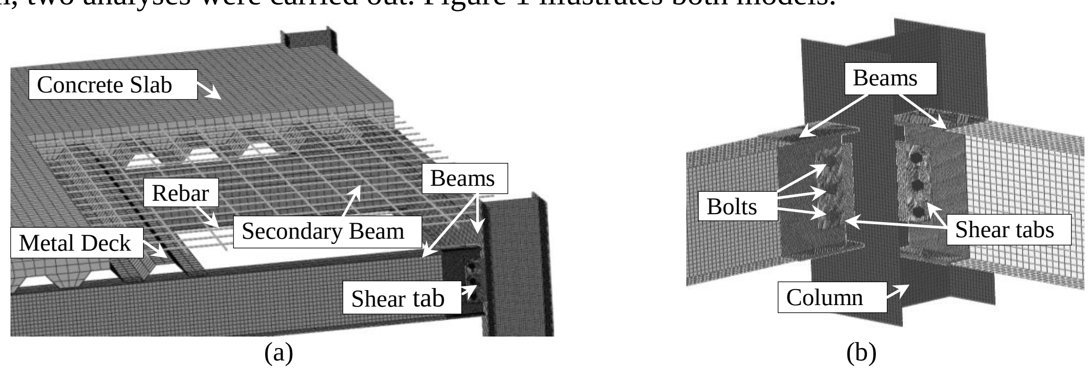

Figure 1 ABAQUS® Model: (a) steel frame with composite slab, (b) bare frame

In both cases, columns are assumed to be W310×118. W360× 33 and W410× 39 were used as the longitudinal and transverse beams, respectively. Beams are connected to columns using a 6.4 mm single-plate (shear tab) shear connection with the clear span of 6.0 m. Shear tabs are welded to the column and bolted to the beam web using 3 ASTM A325 bolts of 19 mm diameter. The center of the bolt group is placed at the vertical center of the beam with the edge distance of 35 mm.

The finite element models consist of 8-noded brick elements (C3D8R) to represent the shear tabs, bolts, column and slab, and part of the beam to study geometric and material nonlinearity. A 2-node linear 3-D truss element was utilized to model reinforcement of the slab. In order to reduce the number of elements, a 4-noded doubly curved thin or thick general purpose shell element (S4) was implemented to represent the metal deck, secondary beam, and the middle part of the beam (200 mm away from both ends), as it is shown in Figure 1. The floor system consists of a 82.5 mm-thick lightweight concrete topping on 76.2 mm metal deck. A finer mesh was confined to the critical sections to increase the accuracy and overcome the singularity issues. The mesh was structured except for the parts with holes, which was assigned as free. According to the Canadian steel design standard, Design of Steel Structures (S16-14), bolt holes are fabricated 1.6 mm larger than the bolts.

2.2 Material Properties

Material properties adopted in numerical modelling were based on previous studies (Oosterhof and Driver 2014). For the beams and columns, ASTM A992 structural steel with a yield stress of Fy = 344.8 MPa were implemented. For the shear tab connections, ASTM A36 with a yield stress of Fy = 242.8 MPa were used. ASTM A325 high-strength bolts with 19 mm diameter were modeled. The lightweight concrete with a nominal compressive strength of 20.7 MPa and a tensile strength of 1.5 MPa were simulated. The metal deck was 20 gauge (0.9 mm) and the concrete was reinforced with welded wire mesh with a bar spacing of 152 mm by 152 mm (W1.4×1.4).

In the concrete section, to predict the crack pattern and simulate the plasticity behaviour of the concrete, the “concrete brittle cracking” property was applied, and the post-failure behaviour of concrete was modeled by “tension stiffening”, which highly depends on the reinforcement provided in the concrete section, size of aggregate, the bond between the concrete and reinforcement, and the generated mesh. For standard concrete, Heger (1993) suggested that after the cracking point, the stress linearly reduces to zero where the total strain has proportionality of 10 times greater than failure strain (Figure 2).

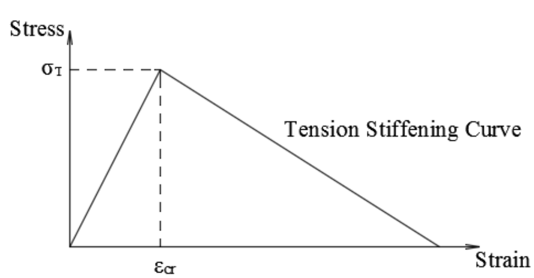

Figure 2 Tension stiffening curve

To simulate the failure of a brittle material, the "ductile damage" criterion was defined in the model, and post-failure of the steel material was modeled by damage evolution in order to simulate the degradation in the material stiffness.

General contact was assigned to the structure to transfer the stress between the contacting surfaces, and for the contact properties hard contact and penalty properties were defined. As was discussed in the previous section, part of the beam was modeled with shell elements to reduce the size of the model. This part was constrained to the solid part using the "shell to solid" tie option in ABAQUS. "Tie contact" was implemented to tie the metal deck to beams' top flanges, and also to tie the metal deck to the concrete surface. Also, "tie contact" was used to connect the shear tab to the columns flange.

During the first phase, a push-down displacement control loading regime was applied to the simulated removed column until failure of the structure. For the second phase, two different loading cases were considered to find the failure mode and robustness of the frame under corner column removal.

2.3 Boundary Condition and Loading

In both phases, a fixed boundary condition was assigned to three columns at their ends, and a free boundary condition was assigned to the fourth column in order to simulate the column removal scenario. In order to study the response of the structure under a corner-column removal, it is worth mentioning that boundary conditions at the edge of the slab and beams play an important role. Therefore, to account for axial and rotational resistance of the adjacent slab on the structural behaviour under corner-column removal, a 2x2-span structure needed to be modeled. Therefore, to accommodate rotational and horizontal constraints coming from the adjacent panels to the corner bay, symmetry boundary conditions were assigned to the slab edge (Figure 3).

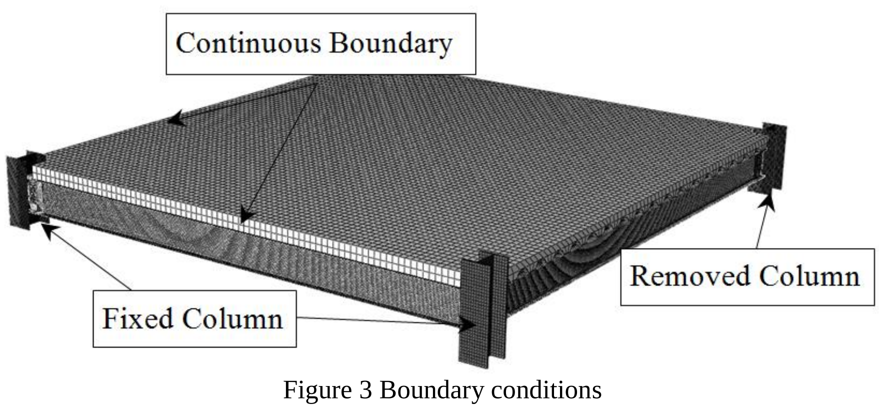

Figure 3 Boundary conditions

During the first phase, a push down displacement control loading regime was applied to the simulated removed column until the failure of the connection (this type of loading will occur if the stiffnesses of the upper levels are lower than that of the collapsing floor). In the second phase, vertical distributed load was applied to the slab to model the load produced by live and dead load. A displacement controlled loading scenario was applied in smooth steps.

3. Results and Discussion

3.1 Analysis of the bare frame

The analysis was performed using the ABAQUS explicit dynamic solver to overcome numerical convergence difficulties related to the fracture simulation. For the bare frame, although some deformation occurred in the connection far from the removed column, the main failure mode was found to be the bolt tear-out at the connection adjoining the removed column. It is worth noting that depending on the relative thickness of the beam web and shear tab connection, tear-out happened at the thinner section. The internal load development at the face of the shear tab connection is illustrated in Figure 4. It is clear that the shear force at the shear tab face is negligible compare to the axial force, and the axial force was increasing in the section until the first failure occurrence at the connection.

As shown in Figure 4, the maximum bending moment occurs at the rotation of 0.09 rad.; however, the axial (catenary) force is progressively increased until the first failure in the connection at a beam rotation of 0.13 rad. First failure in the connection causes the catenary force to be diminished dramatically. At the end, failure is by rupture due to bolt tear-out at the beam rotation of 0.15 rad.

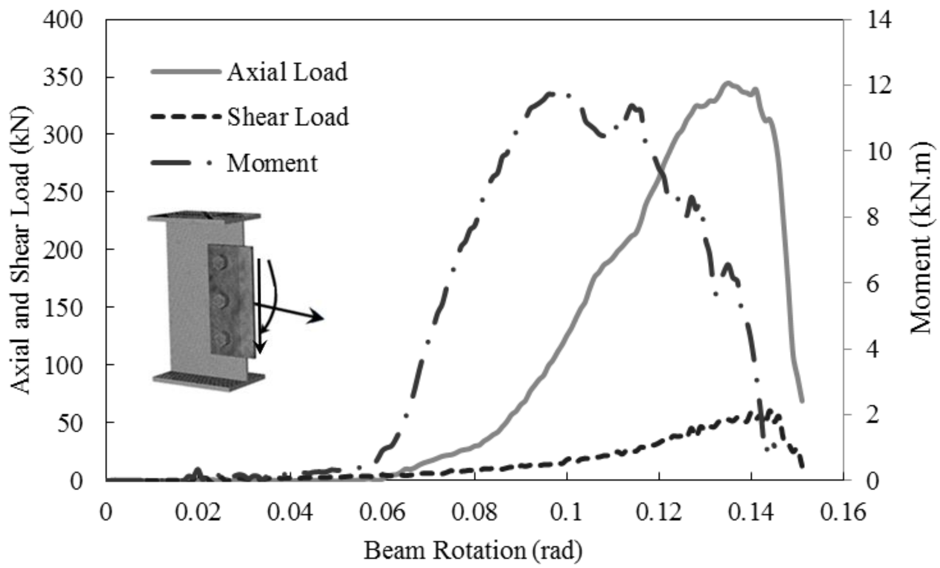

Figure 4 Internal loads vs. beam rotation (rad) for the bare frame

3.2 Composite frame with push down loading

During the next step of the analysis, the composite frame was analyzed under column removal scenario simulated as a concentrated load. Figure 5 shows the deformation and yield lines developed in the slab as a result of corner column removal simulated in the composite frame structure. Furthermore, the yield line occurred diagonally between the two fixed columns connected to the removed one, and a large deformation was observed along this line.

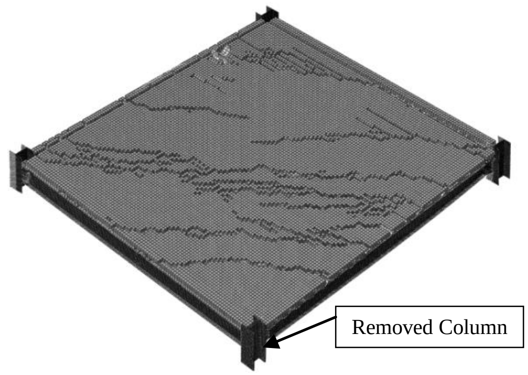

Figure 5 Deformation and failure mode of composite frame

Based on the finite element analysis results of this case (pushing down the removed column), after concrete cracking and crushing, the main failure mode of the structure was found to be the bolt tear-out and prying action of the beam section adjoining the removed column. The internal forces at the shear tab surface attached to the removed column are illustrated in Figure 6. Comparing these results with the bare frame indicates that the axial and shear load capacity of the connection are almost the same; however, the rotational capacity of the connection is less than the one for the bare frame as a result of prying action in the composite frame, and the weight of composite slab. The rupture and failure of the connection was observed at the beam rotation of 0.13 rad., while the maximum catenary action occurred at the rotation of 0.1 rad.

The failure progress of the connection adjoining the removed column for the two cases of bare frame and composite frame under concentrated load are illustrated in Figure 7. Clearly, the rotational capacity of the system has been reduced due to the prying action in the connection as a result of composite slab bearing. Figure 7 shows that despite the fact that the failure mode of the connection in the bare frame is similar to the one for the composite frame with concentrated load, the ductility of the connection in composite frame is less due to the prying action occurrence in the connection as a result of the presence of the slab.

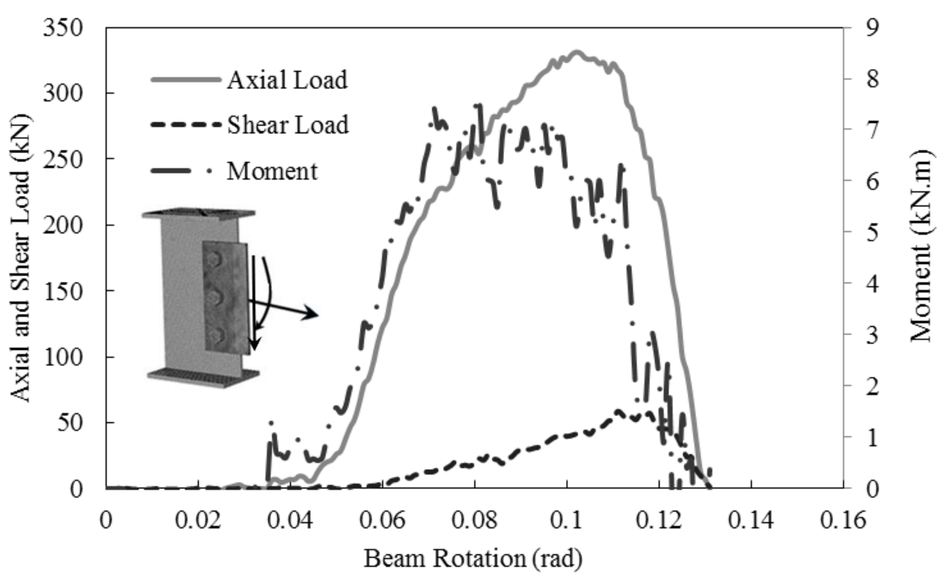

Figure 6 Internal loads vs. beam rotation (rad) for the composite frame under concentrated push-down loading

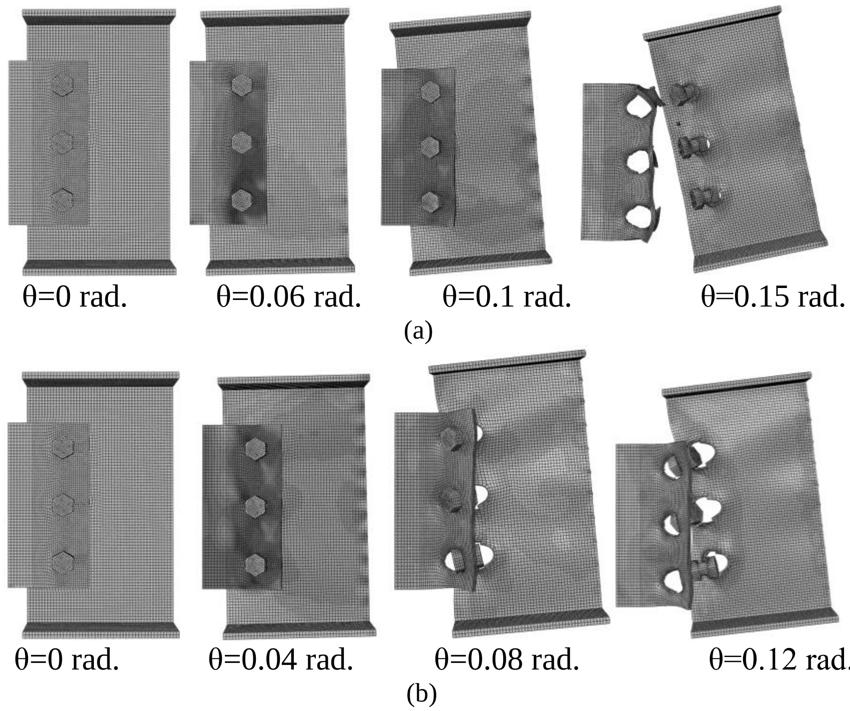

Figure 7 Failure progress and deformation of connection: (a) bare frame; (b) composite frame under concentrated load

3.3 Composite frame with distributed loading

Based on the analysis, two types of failure mode were observed for the composite frame, depending on the loading conditions. As a result of incrementally increasing the distributed load on the slab surface, the main failure happened at the connection far from the removed column. It can be concluded that the beams connected to the removed column act as a cantilever, and the connection adjoining both sides of the removed column are not contributing significantly to the load carrying mechanism. Therefore, the main failure under this type of loading is local buckling of the beam and, eventually, tearing out of the bolts at the connection far from the removed column (Figure 8). According to the deformation results, it was observed that the rotational capacity of the system was enhanced in this model compared to the model under push down concentrated loading (beam rotation at the failure is about 0.12 rad.), which indicates the importance of considering the slab, and the loading condition in analysis of progressive collapse behaviour of composite steel frame (Figure 9).

It is important to mention that under distributed loading, the connection experiences compression up to 0.08 rad. beam rotation, and afterward the catenary force starts to develop and reaches its maximum at a beam rotation of 0.12 rad., while the bending moment reaches its maximum around 0.08 rad., and after this point diminishes rapidly.

Moreover, in contrast to the previous cases (bare frame, and composite frame under concentrated load) where shear forces were negligible, in the current case (composite frame under distributed load) a transverse shear force due to torsion coming from composite slab plays an important role in the failure of the connection (Figure 8 and Figure 9).

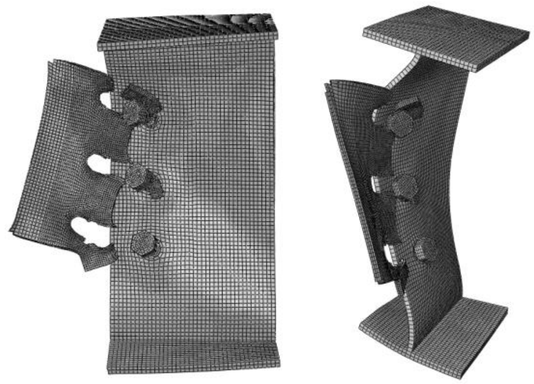

Figure 8 Deformation and failure mode of connection at far end of beam connected to removed column under distributed load

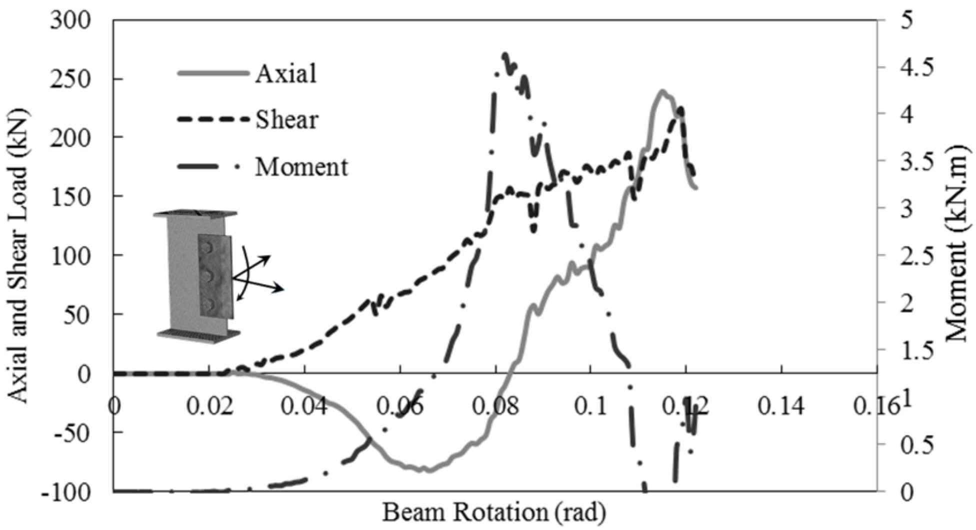

Figure 9 Internal loads vs. beam rotation for the composite frame under distributed loading

3.4 Effect of slab reinforcement

As mentioned before, after cracking occurs in the concrete slab, the main membrane resistance of the composite frame is provided by slab reinforcement and metal deck. To study the effect of reinforcement, two models were simulated including steel reinforcement with the equivalent area of 0.1 mm2/mm and 0.14 mm2/mm. The results of the simulation for the internal loading at the face of the shear tab attached to the column far from the removed column are illustrated in Figure 10.

Comparing Figure 9 and Figure 10 indicates that increasing the reinforcement in the slab does not have a marked effect on the ductility of the structure and load bearing capacity of the connection, although the bending moment enhances by 22 %.

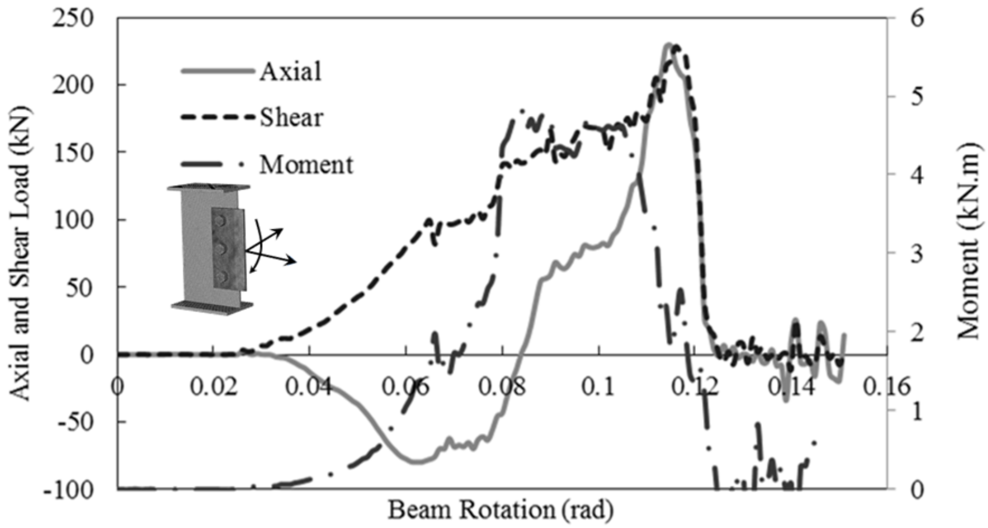

Figure 10 Internal loads vs. beam rotation for composite frame with 0.14 mm^2/mm reinforcement

3.5 Effect of metal deck thickness

As another important parameter, metal deck plays an significant role in transferring the membrane force and providing catenary resistance in the composite frame during a column removal scenario. However, due to its configuration, metal deck transmits the membrane force in one direction, and in the perpendicular direction it is quite inefficient. Therefore, to study the effect of metal deck strength, the model was analyzed based on two different deck thicknesses of 0.9, and 1.2 mm. Figure 11 depicts the internal forces and bending moment at the face of the shear tab connection at the far end of the beam connected to the simulated removed column.

Comparing Figure 9 and Figure 11 shows that increasing the metal deck thickness substantially enhances ductility of the structure. In this case, by increasing the thickness from 0.9 to 1.2 mm, the rotational capacity of the connection was increased by 25 %. Moreover, the bending moment capacity was increased significantly. This conveys the importance of centenary action provided by the metal deck in composite frames as an important parameter for gravity frames vulnerable to progressive collapse.

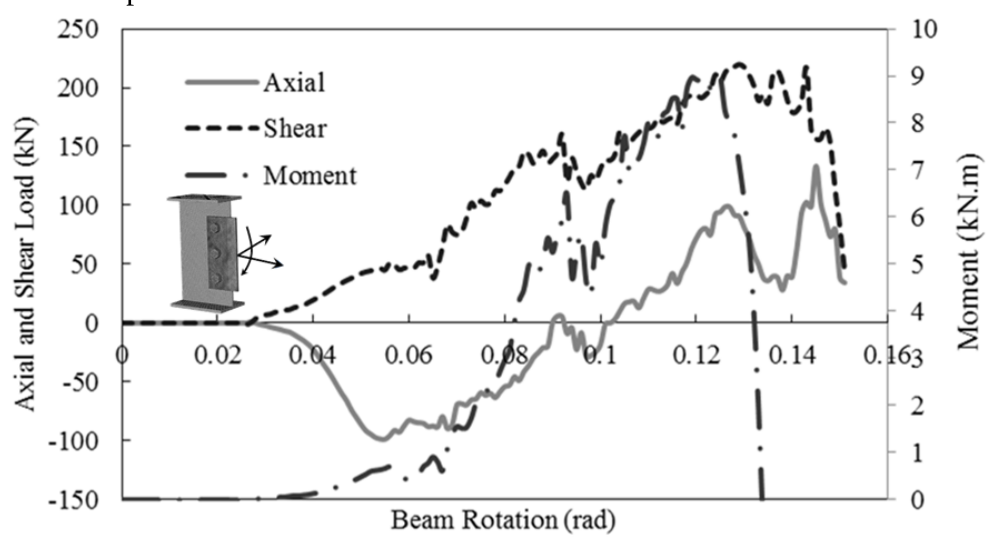

Figure 11 Internal loads vs. beam rotation (rad) for the composite frame with 1.2 mm thick metal deck

4. Conclusions

In this paper, the robustness of composite gravity steel frame structures under a corner column removal scenario was investigated. The influence of different parameters on the failure mode and behaviour of composite frame with shear tab connections was studied. High fidelity finite element simulations were performed to find the effect of loading condition, composite slab, metal deck thickness, and slab reinforcement on the performance of the frame during the progressive collapse scenario. The results of the analyses presented in this paper provide insight into the behaviour of the composite floor, which will be used to conduct a full-scale experimental program to characterize the collapse resistance of composite steel frame under corner column removal scenario. The following conclusions can be drawn from the finite element simulations:

1. Results obtained from the finite element simulations determined the importance of consideration of the slab in evaluating the progressive collapse resistance of a composite steel frame. Magnification of the demand on the connection due to the presence of slab reduces the rotational capacity of the connection.
2. The main failure mode appears to be the bolt tear-out of the connection, while a large rotational capacity was observed as a result of ductile behaviour of the beams and shear plate.
3. The simulation results show that two types of failure modes were expected based on loading condition. The failure mode associated with bare and composite frames under a concentrated push-down loading scenario was observed as the bolt tear-out at the connection adjoining to the removed column, while the failure mode of the composite frame under distributed load was bolt tear-out of the connection far from the removed column. In the latter case, rotational capacity of the system was enhanced by 20% and also the connection experienced a large amount of torsion coming from the slab.
4. Tensile forces that developed in the metal deck are the primary source of catenary resistance of the floor. The results indicated that by increasing the metal deck thickness from 0.9 mm to 1.2 mm, the ductility of the system was increased and rotational capacity of the composite frame was enhanced by 25%. This is primarily due to the additional membrane force carried by metal deck. The results demonstrated that increasing the reinforcement in the slab had no significant effect on the overall collapse resistance of composite frame.

Acknowledgments

The financial support of the Natural Sciences and Engineering Research Council of Canada to complete this work is gratefully acknowledged.

References

ABAQUS, ABAQUS-User's manual, version 6.13. Pawtucket, RI: Hibbit, Karlsson & Sorenson, 2013.

Alashker, Y., El-Tawil, S., and Sadek, F. (2010). "Progressive Collapse Resistance of Steel-Concrete Composite Floors*." Journal of Structural Engineering,* 136(10), 1187–1196.

Alashker, Y. and El-Tawil, S. (2011). "A Design-oriented Model for the Collapse Resistance of Composite Floors Subjected to Column Loss," *Journal of Constructed Steel Research*, 67(1), 84-92.

CSA (2014). "Design of Steel Structures.", *Canadian Standards Association*, Toronto, ON.

Foley, C. M., Martin, K., and Schneeman, C. (2009). "Robustness in Structural Steel Framing Systems," *Report MU-CEEN-SE-07-01, Marquette University*, Milwaukee, WI.

Heger, F. J. (1993). "Concrete pipe technology handbook", *American Concrete Pipe Association*, Vienna, VA.

Masajedian, S., and Driver, R.G. (2015) “Characterization of Steel Joint Modeling Parameters for Progressive Collapse Stability Analysis.” Proc., Annual Stability Conference, Structural Stability Research Council, March 24-27, Nashville, TN, USA.

Sadek, F., El-Tawil, S. and Lew, H. S. (2008). “Robustness of Composite Floor Systems with Shear Connections: Modeling, Simulation, and Evaluation,” *Journal of Structural Engineering*, 134(11), 1717-1725.

Li, H. and El-Tawil, S. (2014). “Three-Dimensional Effects and Collapse Resistance Mechanisms in Steel Frame Buildings”. *Journal of Structural Engineering*. 140(8).

Oosterhof, S. and Driver, R. (2014), "Behavior of steel shear connections under column-removal demands." *Journal of Structural Engineering* , 10.1061/(ASCE)ST.1943-541X.0001073 , 04014126.

Park, R., and Gamble, W., (2000). “Reinforced Concrete Slabs, 2nd Edition,” *John Wiley and Sons, Inc.*

Regan, P.E., (1975). “Catenary Action in Damaged Concrete Structures”, *ACI Special Publication.* SP 48-9, 191-224.

Yu, M., Zha, X. and Ye, J. (2010). “The Influence of Joints and Composite Floor Slabs on Effective Tying of Steel Structures in Preventing Progressive Collapse,” *Journal of Constructed Steel Research*, 66(3), 442-451.