UFC Progressive Collapse: Material Cost Savings

White Paper

UFC Progressive Collapse: Material Cost Savings

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Executive Summary

A brief overview of the UFC progressive collapse code requirements is provided, as well as an overview of the Applied Element Method. Two case studies taken from example problems in Appendix E of the UFC 4-23-03 progressive collapse code are reviewed using a Simplified Finite Element Method (SFEM) solution and using the Applied Element Method. SAP2000 is used for the SFEM solution for both linear and nonlinear frame analysis. The Applied Element Method (AEM) implemented in Extreme Loading® for Structures (ELS) is used for nonlinear frame and frame including the slab analysis with models including both perfectly pinned as well as partially pinned connections. Both the SFEM and AEM methods yield similar results when looking at nonlinear dynamic frame analysis. However, it is also shown that neglecting the nonlinear effects of the slabs in a structure can result in excessive overdesign. Results show that using AEM with slab models result in a savings of between 30% and 80% in structural materials while meeting UFC criteria and maintaining safety requirements.

Introduction

The attention given to progressive collapse analysis has materialized into explicit requirements for redundancy in building codes all over the world. In the US, the Department of Defense and the General Services Administration have released specific codes for progressive collapse analysis of structures (GSA 2003 and UFC 4-23-03). Although progressive collapse is a nonlinear dynamic procedure, progressive collapse codes permit the use of linear static analysis combined with some load factors.

In many cases a simplistic model which models only linear beam and column elements the neglected contribution of walls and slabs is bound to lead to uneconomic and/or unconservative results. Walls and slabs may be considered secondary members in other types of analysis but in progressive collapse analysis, walls and slabs often behave as primary members with slabs carrying load though membrane action and walls providing alternate load paths in case of loss of columns.

In this paper, results of nonlinear dynamic progressive collapse analysis are compared to simplified analysis based on code recommendations.

Load Factors in Progressive Collapse Analysis

The GSA (2003) code prescribes a load increase factor of 2.0 for both nonlinear and linear static procedures which leads to a non-uniform factor of safety. The factor of safety is not uniform because if the factor of safety of 2 is appropriate to account for the dynamic effect then a larger factor is needed to account for both nonlinearity and dynamic effects. The capacity-increase factor in the GSA (2003) code varies between 1 and 3. Combining these two factors can lead to an over-conservative or un-conservative answer, dependent on the DCR ratio.1[,5,9]

To avoid the shortcomings of GSA (2003) load factors, the UFC 4-23-03 (2009) used different load factors for linear static and nonlinear static cases. These factors were developed based on analysis of 2D and 3D frame models. These models neglected the contribution of all slabs and walls. However, in other parts of the UFC 4-23-03 code (namely, the tie force method) it is mentioned that slabs should be considered as primary members.1,5[,9] The use of simplified linear static analysis to study complex phenomena such as progressive collapse analysis without sufficient amount of experimental work can lead to a wide variation in the choice of factors of safety for different codes. This subsequently can lead to either over or under conservative design depending on the given case.

All progressive collapse codes agree that when using nonlinear dynamic 3D analysis there is no need for load factors. Hence, in order to obtain uniform factor of safety for all cases, the best way is to use nonlinear dynamic analysis. Fortunately, new numerical tools such as the Applied Element Method implemented in the Extreme Loading® for Structures software make this task much simpler and faster than traditional numerical analysis tools.

This paper examines a case study that used a linear static analysis and nonlinear dynamic analysis of a steel building based on the UFC code. Analysis is performed using SFEM and AEM analysis. Before discussing the analysis of these cases studies, a brief background about the Applied Element Method is provided.

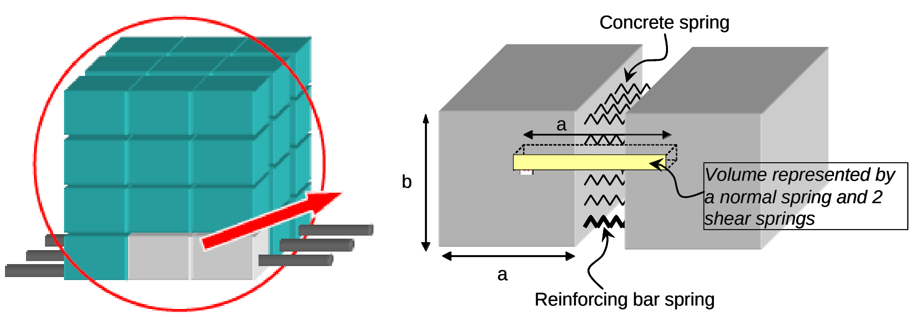
Overview of the Applied Element Method

The technique used in the analysis is the Applied Element Method. This method is implemented in the Extreme Loading® for Structures (ELS) software, which is used in this paper. The Applied Element Method6[,7,10] is an innovative modeling method adopting the concept of discrete cracking. In the Applied Element Method (AEM), the structures are modeled as an assembly of relatively small elements, made by dividing of the structure virtually, as shown in

Figure **1**(a). The elements are connected together along their surfaces through a set of normal and shear springs. The two elements shown in

Figure **1**(b) are assumed to be connected by normal and shear springs located at contact points, which are distributed on the element faces. Normal and shear springs are responsible for the transfer of normal and shear stresses, respectively, from one element to the other. Springs represent stresses and deformations of a certain volume as shown in

Figure **1**(b).



(a) Element generation for AEM (b)Area of influence of each pair of springs

Figure 1. Modeling of the Structure Using AEM.

Each single element has six degrees of freedom; three for translations and three for rotations. Relative translational or rotational motion between two neighboring elements cause stresses in the springs located at their common face as shown in Figure 2. These connecting springs represent stresses, strains, and connectivity between elements. Two neighboring elements can be separated once the springs connecting them are ruptured.

Fully nonlinear path-dependent constitutive models for reinforced concrete are adopted in ELS. For concrete in compression, elasto-plastic and fracture model is adopted4. When concrete is subjected to tension, linear stress-strain relationship is adopted till cracking of concrete springs, where the stresses drop to zero. Since the method adopts discrete crack approach, the reinforcing bars are modeled as bare bars for the envelope while the Ristic et al (1986) model is used for the interior loops. For more details about constitutive models refer to Tagel-Din and Meguro (2000).

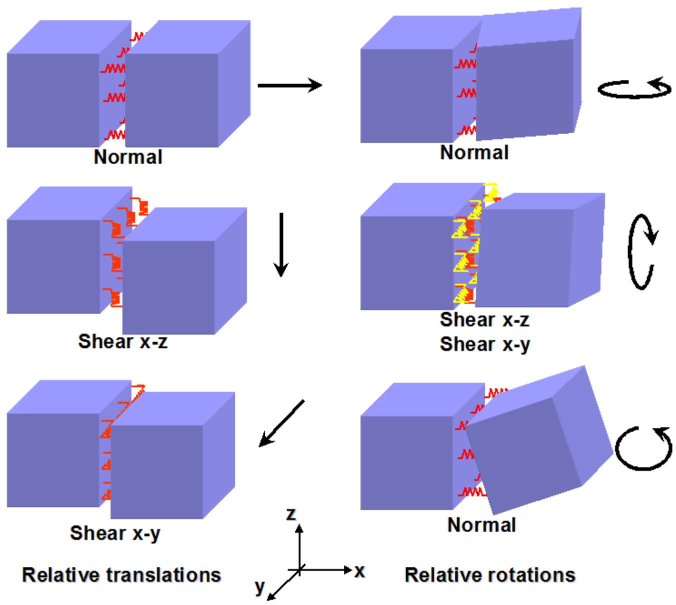


Figure 2. Stresses in Springs due to Relative Displacements.

Requirements of the UFC code

Department of Defense (DoD) Requirements for Progressive Collapse Prevention were released with major changes in 2009 with an update in 2010. These new requirements are currently applicable to all DoD buildings.

Level of progressive collapse design for a structure is correlated to the consequence based on two main factors which are *the level of occupancy* and *the building criticality.*

The code specifies four different levels of occupancy starting from Occupancy Category 1 (OC I) being the lowest level of protection (or the lowest amount of consequences) up to Occupancy Category 4 (OC IV). Based on the OC, the progressive collapse design employs one or more of three methods: Tie Forces, Alternate Path method, and Enhanced Local Resistance for Key Elements.

The Tie Forces method prescribes a tensile force capacity of the floor or roof system, to allow the transfer of load from the damaged portion of the structure to the undamaged portion. In the Alternate Path method, the building must be designed to bridge across a removed element without any local collapse (this is a conservative requirement relative to other codes which permit local damage to 15%-30% of floor area in one or two stories). In Enhanced Local Resistance method, the shear and flexural capacity of the perimeter columns and walls are increased to provide additional protection.

The Tie Force method is based on the requirements of the British Standards which were the earliest codes to have specific requirements for progressive collapse in the 1960s. Tie Forces method is difficult to implement in rehabilitation of existing buildings and design load-bearing wall construction. To remedy this, another option using the Alternate Path (AP) method was added. Thus, the use of the AP method provides some relief for existing buildings and load bearing redundant systems which may be able to resist progressive collapse but unable to satisfy the requirement of the simplified requirements of Tie Force Method.

In the AP method, the designer is required to check multiple scenarios of column removal based on the geometry of the plan of the structure. For each plan location, the designer is required to choose multiple scenarios removing first story above grade, the story directly below roof, the story at mid-height, and the story above the location of a column splice or change in column size. This removal at multiple levels is not included in most other PC codes. For a regular structure, the removal of the first story column will be the critical case unless dynamic effects and impact are taken into consideration. This is another reason to encourage doing nonlinear dynamic analysis.

In addition to requiring no collapse, the UFC code provides performance-based acceptance criteria. Separate structural models are required to verify acceptability of components and actions which are deformation controlled and force controlled. The code provides stricter acceptance criteria for linear static analysis relative to nonlinear dynamic analysis. Additionally, the code requires a sufficient amount of structural detail in the model when performing 3D analysis to allow the correct transfer of vertical loads from the floor and roof system to the primary elements.

The load combination prescribed by the UFC code for Alternate Path nonlinear dynamic analysis is as follows: (0.9 or 1.2)D + (0.5 L or 0.2 S)

This vertical load is to be combined with a lateral load equal to 0.2% of the vertical load. This lateral load is to be applied in all four main directions. In order to evaluate the procedure, the example given in the UFC code is examined here.

Case Study

In this paper, an example out of Appendix E from the UFC-4-023-03 is looked at using a frame model utilizing an SFEM program (SAP2000) for linear static and nonlinear dynamic analysis. This same structure is then modeled and analyzed using nonlinear dynamic analysis utilizing an AEM program (ELS) using a frame model, a frame and slab model using perfectly pinned connections, and a frame and slab model with partially restrained connections. Performing linear frame analysis in an AEM model, similar to the linear SFEM analysis was deemed unnecessary because it would have yielded similar results. The objective was to prove through analysis that linear static analysis is an uneconomic choice in this case.

Appendix E of the UFC 04-23-03 provides an example of progressive collapse analysis of a steel structure. The structure is a four-story steel frame health care facility design occupied by 50 or more resident patients; placing the structure in Occupancy category III. This OC requires using Alternate Path Method to select elements to demonstrate capacity to resist progressive collapse. The preliminary design, as shown in Figure 3, has been sized to meet the code requirements of IBC 2006.

The floor system is composed of 3-in composite steel deck plus 4 ½-in concrete topping (total slab thickness = 7 ½-in). The roof system is composed of metal deck only (no concrete fill). The story height is 14-ft 8-in. The proposed column removal locations based on the code guidelines are shown in Figure 4. The loads are assumed as follows:

- Dead loads (D): Floor: 75 psf + 3 psf allowance for deck
- Roof: metal deck 5 psf
- Super imposed load: 15 psf
- Cladding (CL): 15 psf x 14-ft-8-in 220 plf on perimeter of the building
- Live load: Floor (LL): 80 psf + 20 psf allowance for partitions
- Roof (Lr): 20 psf
- Wind Load (W): 110 mph with exposure = B and importance factor = 1.15

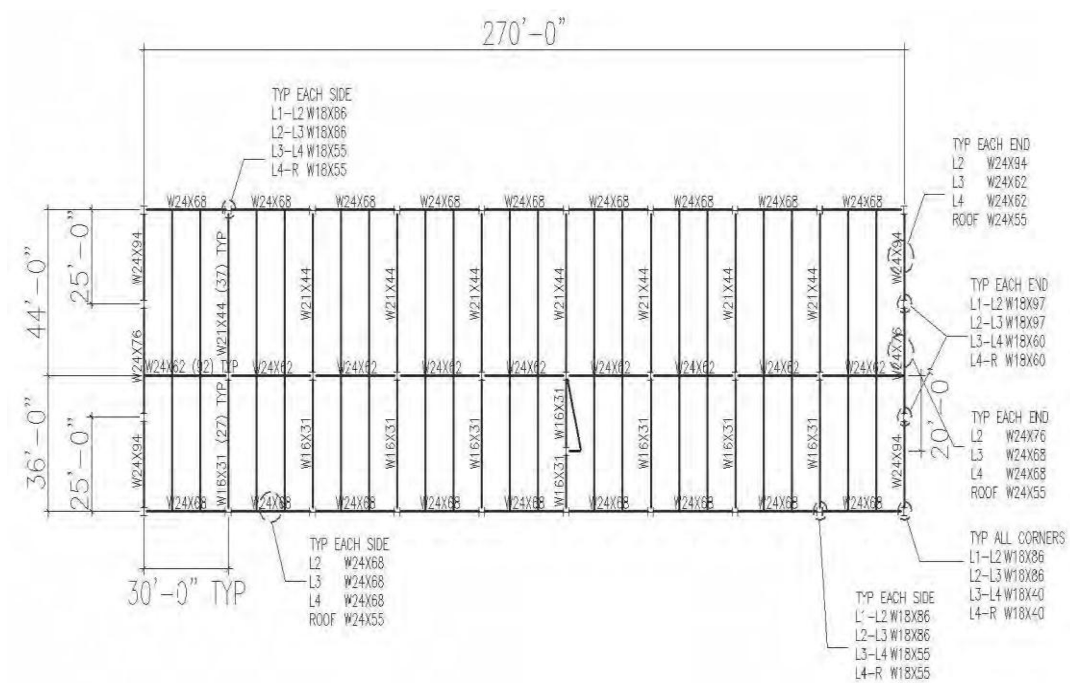


Figure 3. Plan view of the structure and member sizes [UFC 4-23-03].

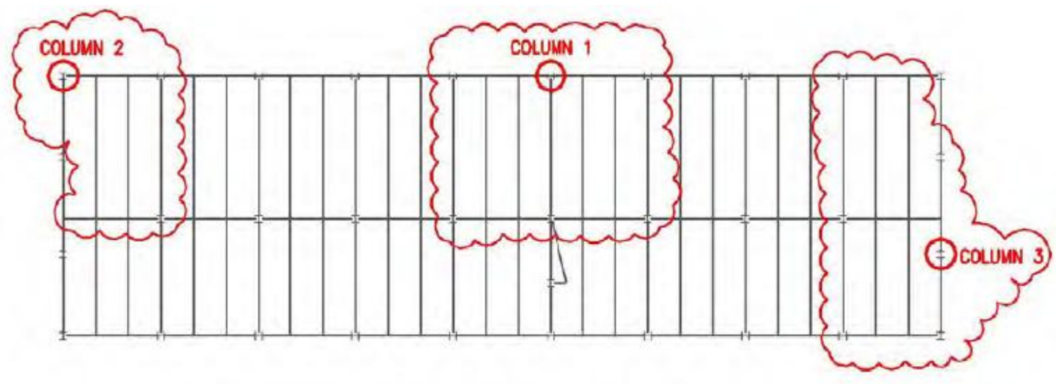


Figure 4. Column Removal Locations [UFC 4-23-03].

Linear Static Analysis

In Appendix E of the UFC code, the progressive collapse analysis was performed using linear static SFEM implemented in SAP2000 software. Due to limitations of the modeling technique, the slabs were left out from the model as seen in Figure 5. Additionally, the contribution of the secondary beams was considered insignificant because they were all assumed to have perfect hinges at both ends.

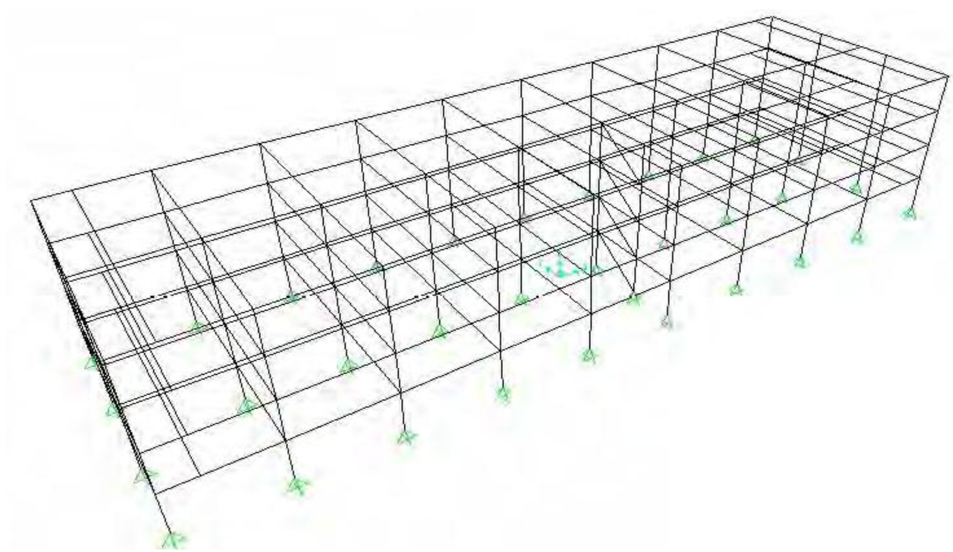


Figure 5. SAP2000 Model [UFC 4-23-03].

The procedure involved removing the column and checking the capacity of all the members and connections to sustain the redistributed load. As nonlinear behavior is not directly taken into consideration, it has to be accounted for using a capacity factor. Members and connections are classified into two categories based on the mode failure: deformation controlled actions (ductile failure) and force controlled actions (brittle failure). For the force controlled actions, the section capacity is used as it is without modification; while for deformation controlled actions the capacity is multiplied by an m-factor as specified in UFC code to take into consideration inherent ductility.

Based on the code requirements, the analysis needs to be done twice because the load increase factor for deformation controlled actions is different from the load increase factor for force controlled action. The load increase factor for the force controlled actions is equal to 2; while, the deformation controlled actions is a function of the lowest m-factor and is equal to 2.71 for this specific example.

Nonlinear Dynamic Frame Analysis

The UFC Appendix E also gives an example for a nonlinear analysis using an SFEM model also using SAP2000. Due to limitations of the modeling technique, the slabs were left out from the model. Only frame elements can be included in the model; and nonlinearity was modeled only by including plastic hinges at connections and center-span.

Each of the column removal scenarios is studied separately. The nonlinear dynamic analysis is an iterative procedure. Convergence of the solution must be achieved; if the solution does not converge, the reason is most probably instability of the structure due to the current loading case or due to a problem/mistake in the model. Hence, engineering judgment and experience with nonlinear analysis is required to identify the instability. Once identified, the deficient sections are strengthened appropriately and the analysis is repeated as seen in Figure 6. After eventually achieving stability, the shear check is performed manually for all sections.

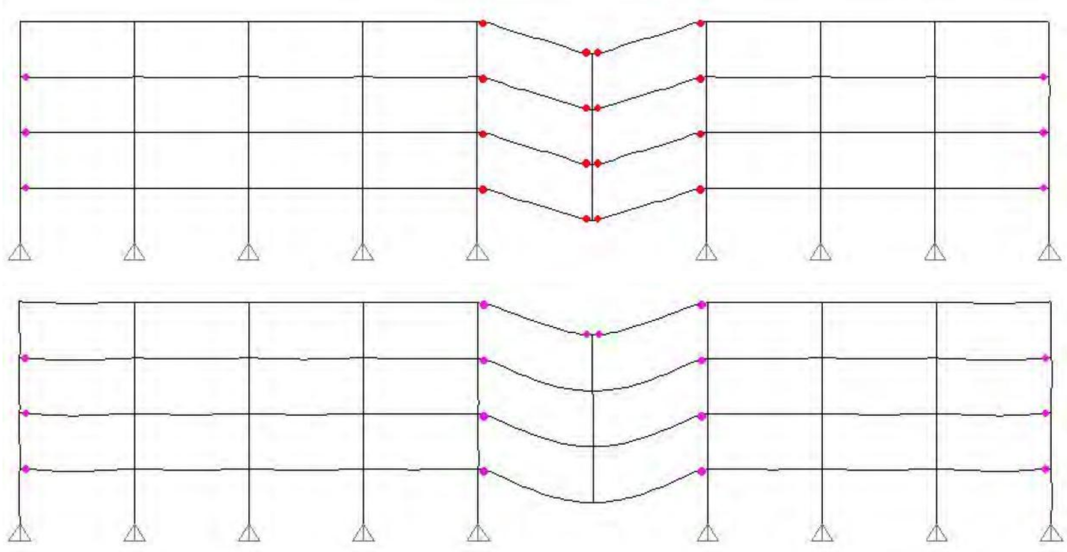


Figure 6. Iterations required to obtain a stable solution [UFC 4-23-03].

Nonlinear Dynamic Frame Analysis

To evaluate the results presented in the UFC Appendix E, the structure is modeled using an AEM model implemented in ELS. Three different models are studied: Model 1 was a frame model with all the slabs neglected (for comparison purposes to the SFEM model) in the analysis as shown in Figure 7. Models 2 and 3 included the slab as shown in Figure 8.

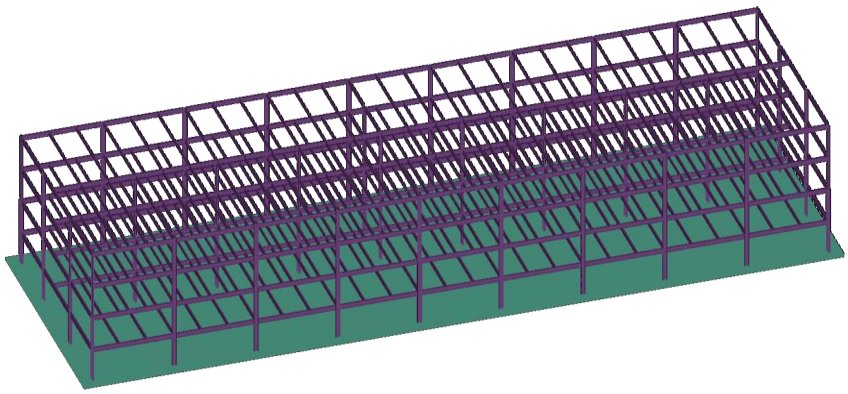


Figure 7. AEM Frame model (Model 1).

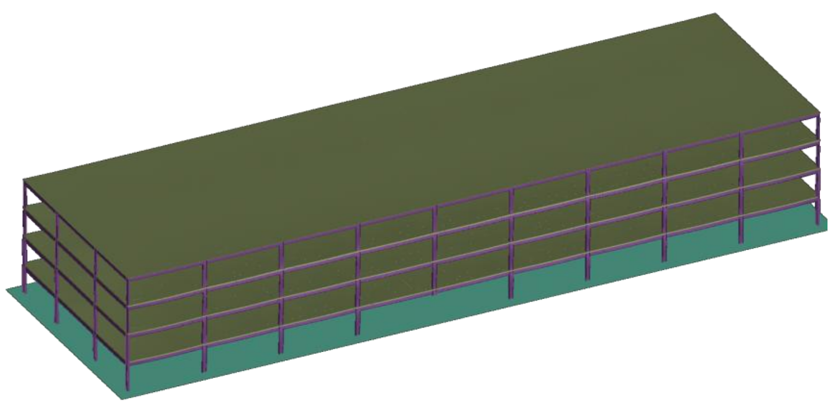


Figure 8. AEM models including the slab (Models 2 and 3).

Using 3D AEM models allows the connection to be modeled more realistically. For this case study two modeling options were considered. Model 2 used a completely hinged connection where the connection of the gravity framing beams to the columns is only through a single element that transfers shear and normal but does not transfer moment as seen in Figure 9., Model 3 used a partially welded connection where only the web of the beam is connected to the web of the column while the flanges are not as seen in Figure 10. The same concept applies for connecting the secondary beams to the main beams as shown in Figure 11. For Model 1 the type of the beam column connection was not an issue because regardless of the type of connection the model failed as will be seen later.

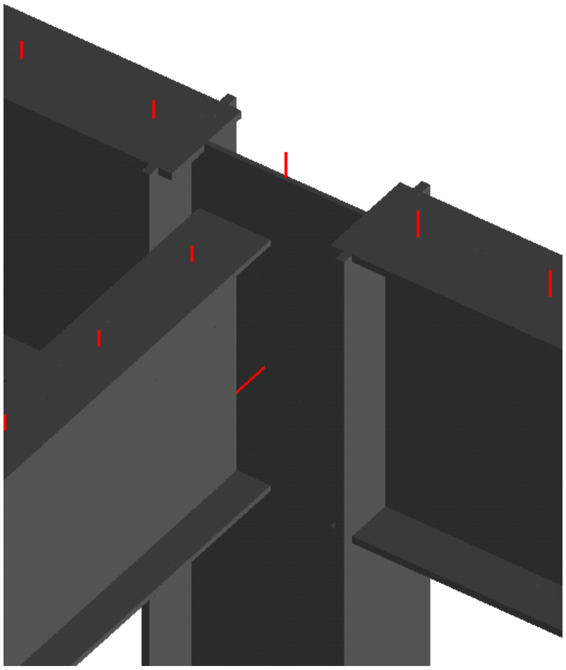


Figure 9. Perfectly-hinged connection (Model 2).

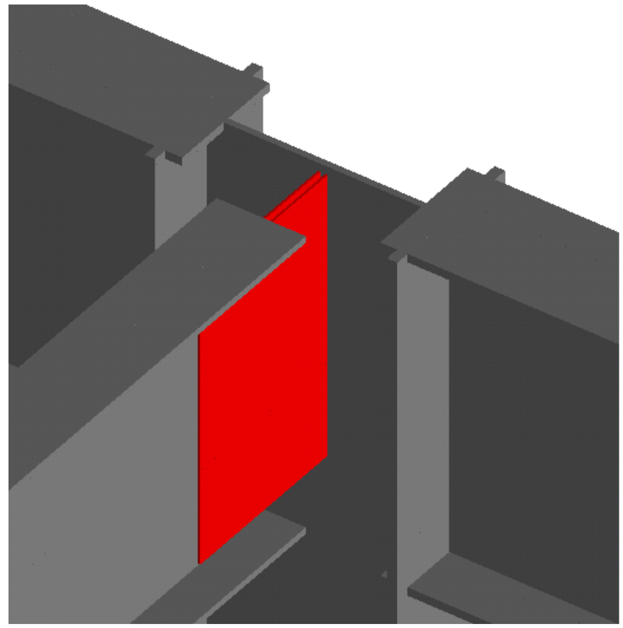


Figure 10. Partially-restrained beam-column connection (Model 3).

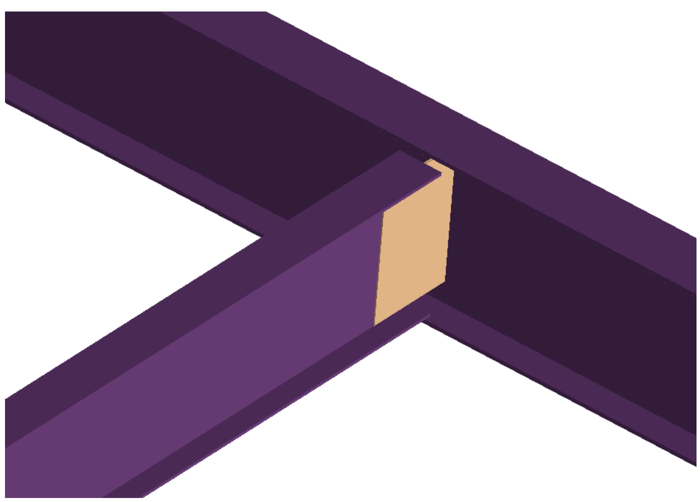


Figure 11. Partially-restrained main-to-secondary beam connection (Model 3).

Comparison of Results

The linear static SFEM analysis suggested that the structure did not meet code. This is due to the fact that the bending moments calculated at most sections are based on the linear capacity of the section. These capacities were exceeded by factors ranging between 4.5 and 5.5 as given in Appendix E of the UFC code. The allowable value (the m-factor) is 1.8. In order to satisfy the code requirements based on this analysis, beam and column sizes had to be increased. After multiple iterations, the weight of the final steel frame which satisfied the code requirements was almost 1.8 times the weight of the steel frame in the original design.

The nonlinear frame analysis using the SFEM model given in Appendix E of the code also suggested that the original design of structure was unsafe. The nonlinear analysis failed to converge indicating that there is a structural instability due to column 1 removal as shown in Figure 12. In order to achieve convergence, the column and beam sizes need to be increased. After achieving convergence, the rotations of sections were checked against the allowable rotation values based on the UFC code requirements. The weight of the outer steel frame in the final design, reported in Appendix E of the code, was 1.34 times the weight of steel in the original design.

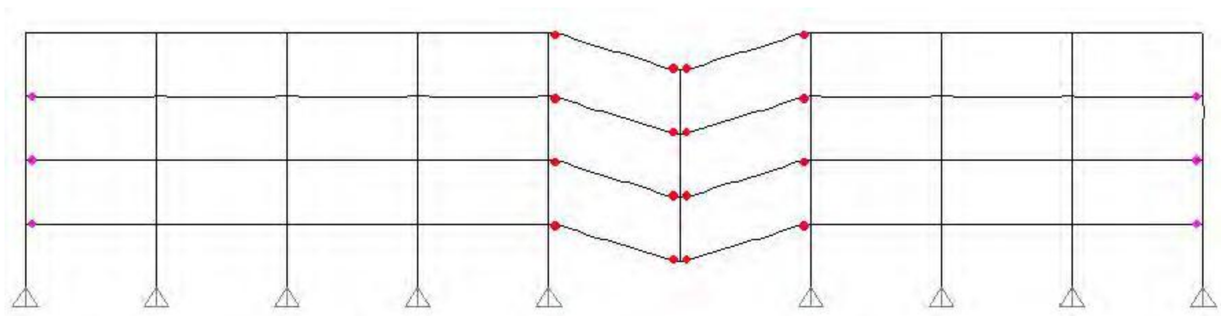


Figure 12. Results of nonlinear frame analysis using SAP 2000 [UFC 4-23-03].

The results of the nonlinear frame analysis using AEM (Model 1) correlated very closely to the nonlinear analysis using SFEM. The original design of the structure suffered local failure as shown in Figure 13. This occurred regardless of the assumption of the beam column connections: hinged, partially-restrained, or even totally restrained. However, the use of a frame model was considered uneconomic because it neglects the contribution of the composite floor slab. The floor slab is connected to the beams using screws at a spacing of 6-in which allows the development of some composite action between the roof and the roof beams.

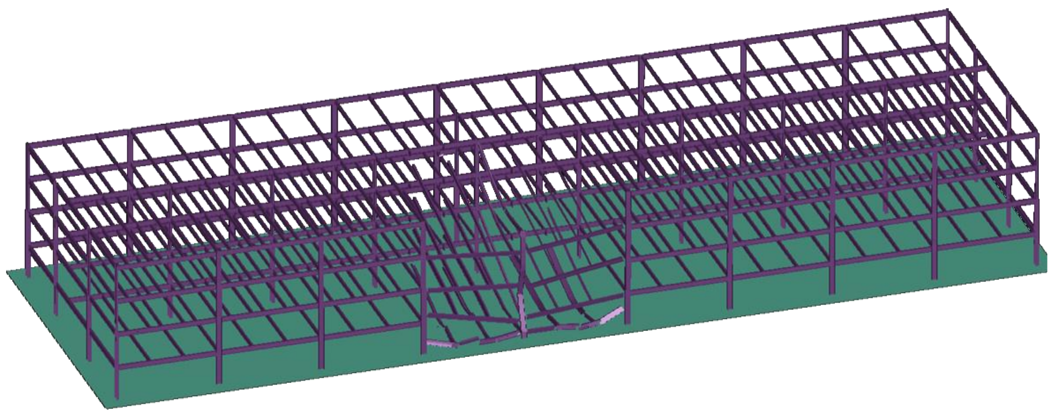


Figure 13. Results of the nonlinear frame analysis using ELS (Model 1).

The results of the nonlinear analysis using ELS and including the slab model as well as assuming all the secondary beams to be perfect hinges are shown in Figure 14. As seen the original structure design suffered excessive deformation due to removal of Column 1 but it did not fail.

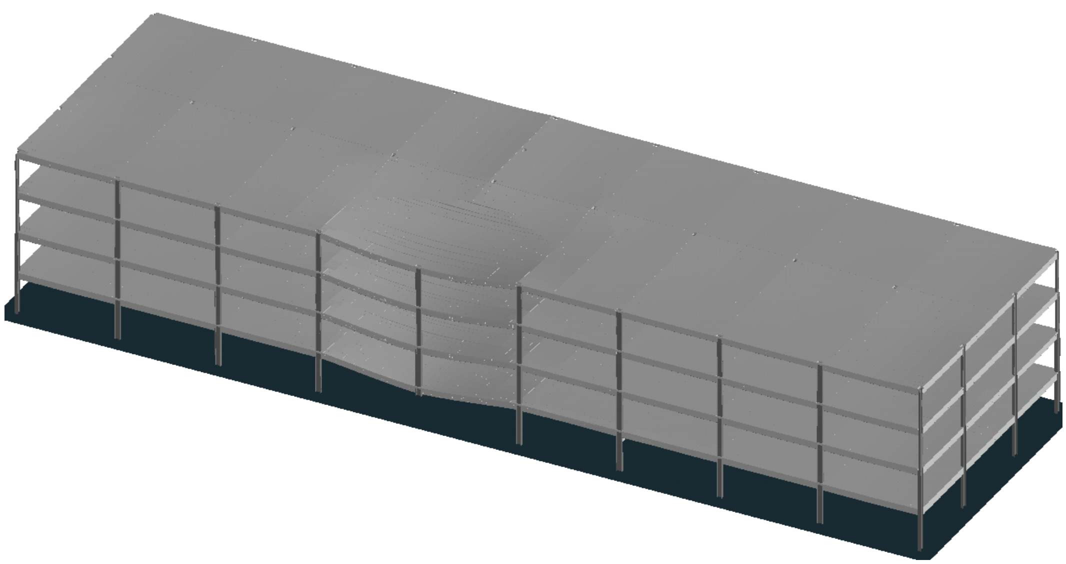


Figure 14. Results of nonlinear ELS analysis (Model 2).

Further examination of the results of Model 2 showed that although the original structure did not fail, yet it suffered excessive rotations of the joints as evident from the colored contours shown in Figure 15 where the red color indicates the location of maximum beam rotation. The maximum rotation occurred in beams G1 and G3 where the rotation was about 0.05 radian which exceeds the UFC code limits. However, assuming the connections of the beams to be perfect hinges (as shown earlier in Figure 9) is an overly conservative assumption because the number of bolts required to achieve enough shear resistance will undoubtedly lead the connection to be partially restrained rather than perfectly hinged.

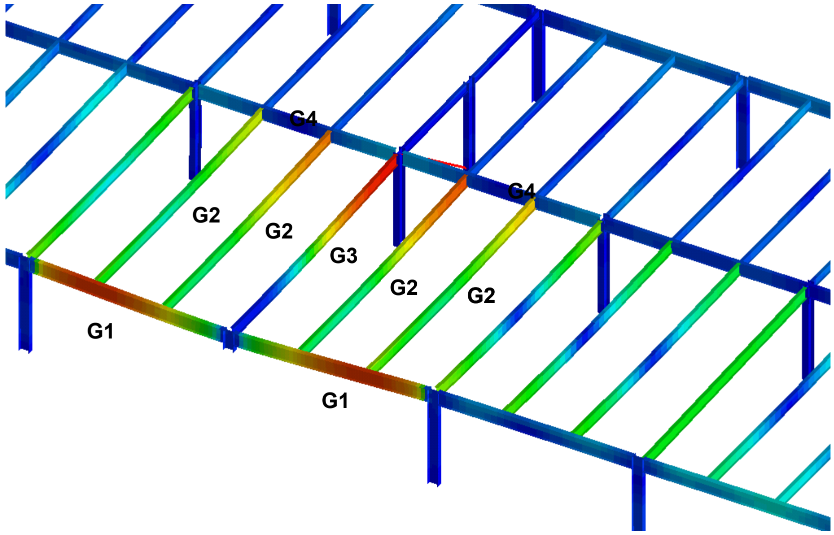


Figure 15. Locations of maximum rotation for column 1 removal (Model 2).

The results of Model 3, where the secondary beam connections are considered partially-restrained, are given in Figure 16. The results of this analysis showed that the deformation is much smaller than estimated from the model using perfect hinges. As shown in Figure 17, the vertical deflection is close to 1.5 inches. As shown earlier in Figure 10, the choice of the area of the connection plates equal to the area of the web ensured that the connection was able to resist the maximum shear force developed in the secondary beam and the fact that the flanges are not welded caused the connection to be partially restrained. Hence, the rotations of the beams were significantly reduced; as shown in Figure 18, the maximum rotation is 0.018 radian. This rotation is marginally larger than the allowable code rotation (the difference is only 0.004 radian) and the cost required to adjust the original design to satisfy the code requirements should be minimal.

The rotation values obtained from assuming a realistic partially restrained connection is significantly smaller than the rotation value obtained from assuming a perfect hinge. Hence, using a partially restrained connection is not only a more realistic assumption as explained earlier but also results in a much more economic design than using a perfect hinge. The reason a perfect hinge is most commonly used in SFEM is typically for ease of modeling and to avoid having to calculate the equivalent rotational stiffness for a partially restrained connection. This is not a problem in AEM as the stiffness is automatically calculated based on the section properties and the connection detailing as seen in Figure 10.

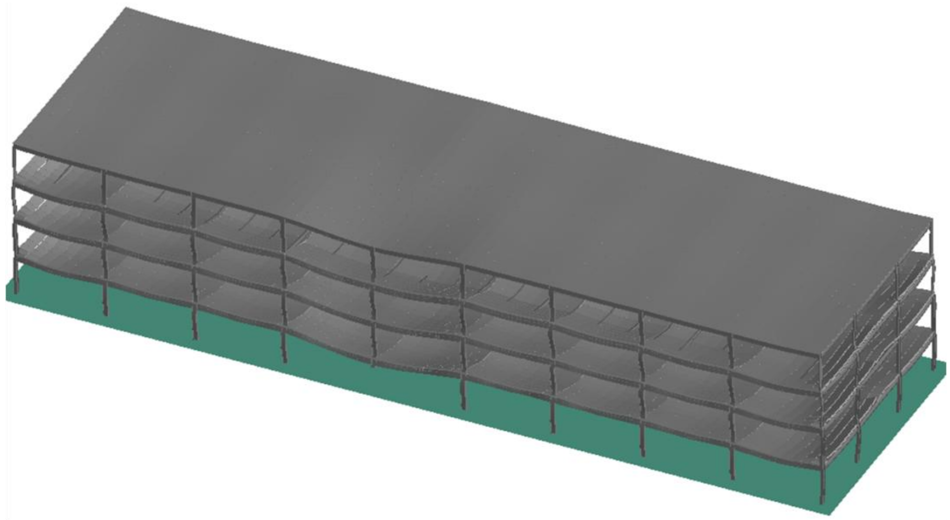


Figure 16. Overall deformed shape of Model 3 (Deformation scaled by a factor of 40).

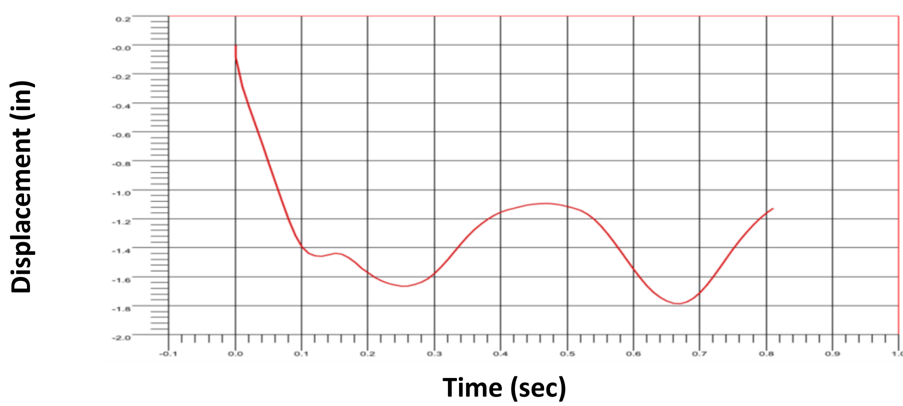


Figure 17. Vertical displacement of the point just above the removed column (Model 3).

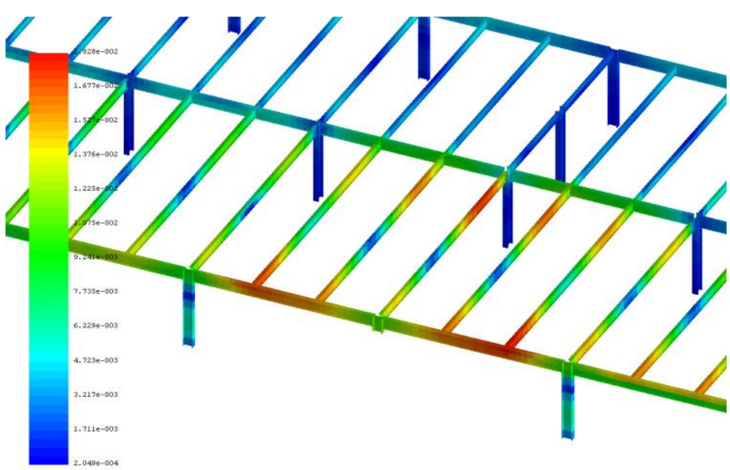


Figure 18. Vertical displacement of the point just above the removed column (Model 3).

The results of the AEM model show a more economic design of the moment-resisting steel frame when comparing the SFEM linear model and the SFEM nonlinear model as shown in Figure 19. This is due to the fact that the SFEM linear model neglects the contribution of secondary beams and slabs and the SFEM nonlinear model neglects the contribution of slabs and partial moment resistance of the gravity framing connections. The AEM model includes secondary beams and slabs and takes into account the nonlinear behavior of the structure and the partial-moment restraint of the connections.

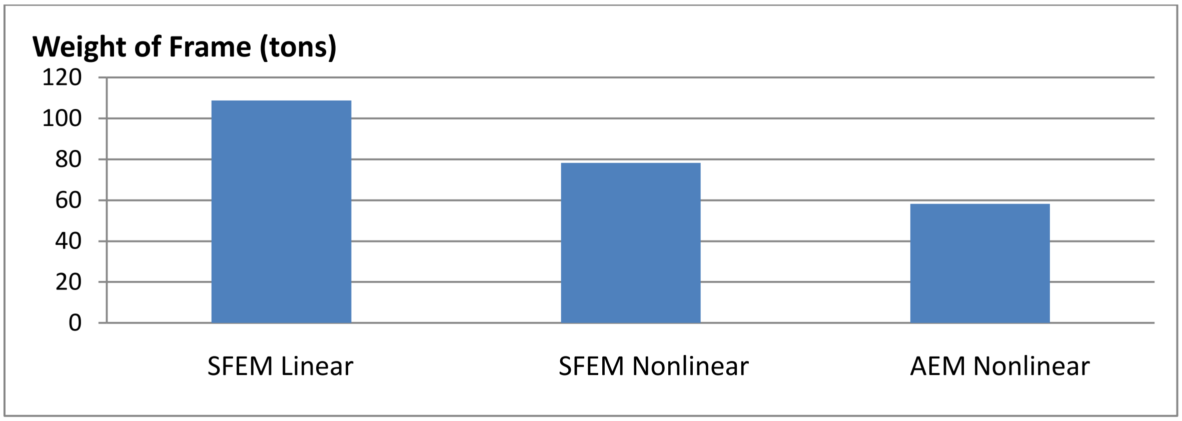


Figure 19. Weight of longitudinal steel moment-resisting frame resulting from design using different numerical analysis methods.

Conclusion

This paper has looked at progressive collapse analysis using different numerical structural tools. Analysis using simplified finite element linear and nonlinear analysis suggested a significant increase (34% and 80% respectively) in the weight of the steel frame is required to satisfy the progressive collapse code requirements. Using more advanced analysis, like AEM method shows that the original design is safe and there is no increase in weight required to satisfy the code requirements.

Additionally, AEM leads to a significant reduction in the analysis time. Using linear static methods doubles the number of required analyses due to studying force-controlled and deformation controlled actions separately. Simplified nonlinear frame analysis requires multiple iterations in each analysis case until the structure is stable: the number of analysis cases will vary depending on the problem.

Use of linear methods may result in overly conservative or un-conservative design due to “penalty factors” required by progressive collapse for linear static procedures do not provide a uniform factor of safety. This is due to lack of data on progressive collapse and the use of overly simplified assumptions and neglecting complex phenomena such as impact and collision.

Using AEM to satisfy the UFC code requirements leads to more realistic and more economic design of the structure and includes the contribution of the slabs into consideration as well as the partial fixity of some connections which are assumed pinned in simplified analyses.

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