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The impact of gravity connections on the progressive collapse response of steel-framed and concrete composite buildings

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Abstract

The robustness of typical steel-framed and concrete composite buildings against progressive collapse is commonly assessed under the notion of a column removal scenario. This study sheds light on the response of the gravity floor systems of this type of buildings when the latter are subjected to an interior gravity column loss, focusing particularly on the most vulnerable component of the gravity system: the shear (or gravity) connections. These connections are designed to accommodate mainly shear forces, yet in a column removal scenario they are exposed to additional and significantly high axial demands. Previous work by the authors has identified and semi-analytically described the two most prominent collapse mechanisms of these buildings: the “yielding-type mode” (ductile and desired collapse, initiated by a sequence of gravity connection failures and extensive wire mesh and steel deck yielding, dominant in the upper part of the building column removal scenarios), versus the “loss-of-stability mode” (brittle and undesired collapse, initiated by a column buckling, dominant in the lower part of the building column removal scenarios). This study clearly demonstrates the prevalent role of the gravity connections on the building response, showing that depending on the connection type and geometric characteristics a switch from the yielding-type to the stability mode is possible for a column removal scenario on the same floor. Therefore, this work reveals the necessity to thoroughly investigate the correlation between the adopted gravity connections and potential instability phenomena on the building scale, to further enhance the structural robustness against progressive collapse.

1. Introduction and Scope

When a structure undergoes considerable damage due to an unexpected man-made or natural hazard, it is subjected to forces that often exist outside the realm of standard design loads. Whether this damage is caused by an explosion, fire, collision, or other event, the resulting changes to localized geometric and material properties warrant the need for alternative force paths to successfully carry the already applied gravity and lateral loads. In cases these conditions

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are not met, this localized damage propagates towards an eventual global failure. This phenomenon, termed progressive collapse, has been of heightened interest to the scientific community since the well-known Ronan Point Apartment collapse (London, 1968) and the infamous tragedy of the World Trade Center (New York, 2001), while unfortunate incidents of similar nature have been occurring until recently (Rana Plaza - Savar, Dhaka, Bangladesh, 2013 and Plasco Building - Tehran, Iran, 2017). The Department of Defense (DoD 2016) and the General Services Administration (GSA 2016) have released prescriptive guidelines aimed at analyzing the robustness of a building against progressive collapse. One of the more widely used methods presented in these documents is the Alternative Load Path Method (APM). This method suggests that the notional removal of a critical column is sufficient in providing meaningful results with respect to a building's vulnerability to progressive collapse. This vulnerability is observed through the structures ability to provide load paths around the removal and ultimately prevent global failure.

Research within the progressive collapse field has focused primarily on numerical (Alashker et al. 2011, Li and El-Tawil 2014) and experimental approaches, attempting to assess the physical processes in play during a progressive collapse scenario and the significance of their contributions to the global response. Johnson et al. (2016) conducted a half-scale experimental investigation of a three-bay by three-bay composite gravity floor system, reporting maximum capacities which were less than the extreme event load combination. Dinu et al. (2016) performed a downscaled experiment of a gravity floor subsystem where failure was governed by the rupture of the beam end converging to the removed column, observing though plastic deformations in the neighboring free edge columns and concluding that in a real scenario which accounts for the gravity load from the floors above collapse could be initiated by the buckling of these columns. Among others, Roddis and Blass (2013), Weigand and Berman (2014) and Oosterhof and Driver (2015) experimentally investigated and evaluated the behavior of steel gravity connections of several types exposed to a typical column removal scenario.

Stability can play a dominant role in the evolution of progressive collapse, particularly because the prevalent loading scheme during the column loss occurrence is the gravity load. Previous work by the authors introduced analytical methods to distinguish two characteristic types of collapse responses after a building column is completely lost. The analytical methodology proposed by Gerasimidis (2014) and expanded in Pantidis and Gerasimidis (2017a) refers to planar moment-resisting frames under any column loss, while the framework introduced by Pantidis and Gerasimidis (2018) refers to three-dimensional steel and concrete-composite gravity framed systems subjected to an interior gravity column removal scenario. In both cases, gravity forces are assumed to be the only loading scheme present, incrementally increased within a quasi-static framework from zero until specific failure criteria are met.

A brief qualitative description of the collapse mechanisms triggered in the 3D framework is provided below. The first collapse mode is a ductile and hence more desirable response. Damage is manifested in the gravity connections and the slab steel components (steel deck and wire mesh), through successive failure and progressive yielding until rupture respectively. This response prolongs the damage propagation path and provides warning of the imminent collapse. The second is a brittle response in which collapse of the structure is attributed to the buckling failure of a column, often located proximal to the removed one. Henceforth, these two responses are termed as "yielding-type" and "stability" mode.

The motivation underpinning the current paper is the intention of the authors to refine the treatment of the gravity connections in Pantidis and Gerasimidis (2018) as completely pinned, an assumption which implies that no bending stiffness and moment capacity are accommodated in the gravity connections. This is a rather conservative approach, which is anticipated to underestimate the actual response characteristics of these structural components. Shear connections are considered the weak-link of the gravity system and their collapse load largely dictates the capacity of the building gravity system. Hence, their accurate representation constitutes an essential step towards the reliable assessment of their behavior and their contribution to the collapse mechanism. The focus of the paper is therefore shifted towards the correlation between the collapse mechanism which dominates the response of the building and the structural performance of the gravity connections, attempting to quantify the impact of the gravity connections on the governing collapse mode.

2. Prototype structure

The 9-Story Pre-Northridge building by SAC FEMA (FEMA 2000) is utilized herein as the prototype structure, largely following the geometric modifications proposed by Foley et al. (2007). The plan view of a typical floor is shown in Figure 1. The structure has moment-resisting frames in the building perimeter and a gravity system which is comprised by the gravity columns, the gravity beams and the gravity (shear) connections. W-shape sections are used for the main girders (W24x68) and the secondary beams (W18x35). The building has rectangular bays of 9.144m length on each side.

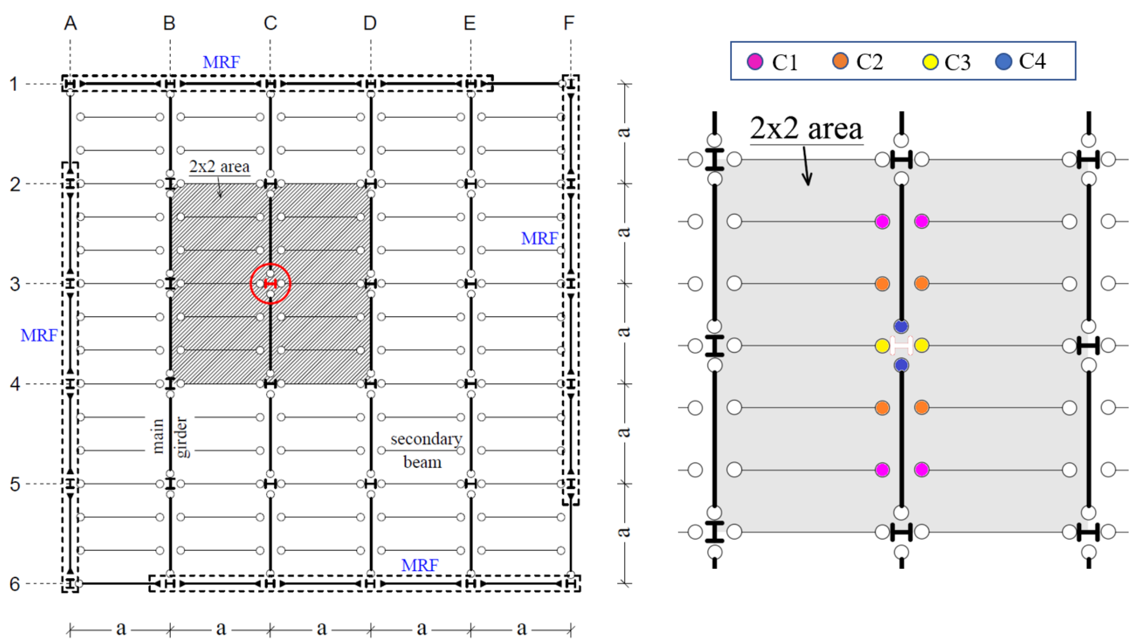

Figure 1: Plan view of the building (on the left), 2x2 area zoom-in pitcure and connections notation (on the right).

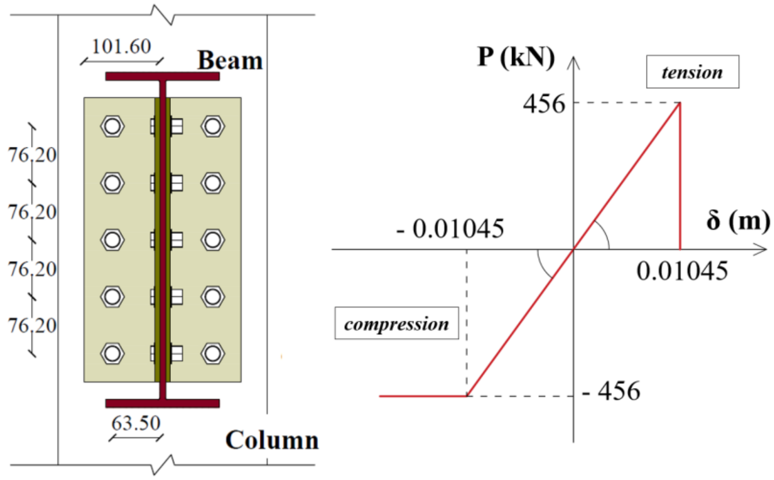

Figure 2: Geometry of the adopted gravity connection along with the axial load-displacement graph. 5 bolts are used throughout the double-angle depth

Foley et al. (2007) investigated a wide range of double-angle gravity connections which varied in their geometric characteristics. These features involved the thickness of the angle, the number of bolts and the beam that the connections were attached. As a result, the axial, moment and shear response were calculated for each connection, in the form of bilinear diagrams with a plateau post-peak behavior. Following the approach of previous studies by the authors, a specific geometry is utilized for all the building gravity connections, i.e. the angle of the thickness is 6.35mm and 5 bolts are attached on the shear connections of both the W24x68 and the W18x35 beams (Figure 2). The main deviation from the Foley et al. (2007) approach, is that a sudden, brittle failure mode is assumed to occur under tension. The adopted approach is based on experimental results of Oosterhof and Driver (2015), who observed an almost abrupt failure mode of their double-angle experiments, manifested through a tear-out mode in the angles. The plateau branch was maintained under compression, rendering therefore the connections which are subjected to tension as the most critical structural components for the gravity system.

The focus of the present study is on the response of the structure under an interior gravity column loss, removing the column which is located at coordinate C3 (Figure 1). The hatched area around the removed column is henceforth termed as “2x2 area”, because it spans 2 bays in each direction. The response of the structural members in this region is of particular interest, since the load redistribution caused by the suddenly removed column is expected to have a much more detrimental impact on the immediately adjacent components than those which are located outside of this region. The gravity connections which are color-highlighted in the right part of Figure 1 are those located closest to the removed column and therefore they are exposed to the most additional demands. These connections are subjected to tension and therefore the accurate representation of their behavior highly affects the reliability of any analytical or numerical approaches adopted to estimate the building response.

3. Modeling approach

The building was modeled using the finite element software package ABAQUS (Simulia, 2012). As it was shown in previous studies by the authors ([10] – [16]), modeling the entire structure constitutes a necessity to identify any stability-related phenomena, which are more commonly addressed when a column is removed from the bottom part of the structure. The complete model is comprised of 116840 finite elements. The key features of the modeling approach are summarized below (model shown in Figure 3):

1. Beam elements are used to model the columns, the girders and the beams throughout each floor, utilizing the B32OS element from the ABAQUS library. This element type contains 2 points of integration, as well as an additional degree of freedom to capture open section warping. 10 elements are used to model the columns, 20 elements for the beams along the x axis and 18 for the beams along the y axis.
2. The slab is modeled using the S4 elements, which is a fully integrated rectangular element. The slab ribs are not modeled, assuming a constant depth for the slab.
3. Meshing of the slab follows the meshing of the underlying beams, such as their nodes are coinciding. Interaction between beams and the slab is treated through merging of their nodes. The centerlines of the slab, the gravity and the moment beams are initially placed at identical elevations and the slab and the moment beams are then offset appropriately such as the bottom slab fibers lay directly above the upper fibers of all the beams.
4. The gravity connections are modeled using a single CONN3D2 element. This element essentially acts as a spring with available axial, shear and moment stiffness and capacity in each direction of the global coordinate system. The extent of the model necessitates the treatment of the connections as springs, since a more refined approach utilizing shell or solid elements would render the computational time intractable. The connection response characteristics are assigned using the *SLOT and the *ROTATION options from the ABAQUS library, as well as the *Elasticity and *Failure sub-options. In the aggregate, these options allow for the specification of the desired response characteristics of the connector element regarding the axial load, moment and shear across all directions. It is worth noting here that the main difference from the past studies of the authors is the assignment of bending stiffness in the gravity connections. This stiffness plays inherent a secondary role in the response of the shear connections, however it is still present; therefore, the assumption of its negligible contribution is relaxed in the present study. The beneficial action of the rotational stiffness is assumed to vanish once the gravity connection reaches its tensile capacity and fails abruptly.

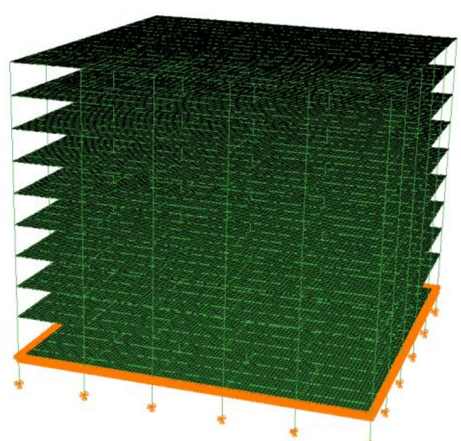

Figure 3: 3D view of the building model created in ABAQUS. Boundary conditions are highlighted with orange color (pinned nodes in the columns bases and laterally restrained nodes at the ground level)

5. The steel components of the structure are assigned typical steel properties. The modulus of elasticity is E_s = 210 GPa, Poisson's ratio is ν = 0.3, the yield stress is σ_y = 345MPa and the ultimate strength and strain are σ_u = 448 MPa and ε_u = 0.18 respectively.
6. The concrete of the slab is modeled using the *Concrete Damaged Plasticity model from the ABAQUS library. It is assigned a modulus of elasticity E_c = 20 GPa and a Poisson's ratio ν = 0.2. The compressive strength of the concrete is σ_{comp} = 20 MPa and its tensile strength is neglected. Instead, the wire mesh which acts in both slab directions is assumed to provide the first source of the slab tensile resistance and it is modeled following a similar approach as Alashker et al. (2011), assigning to the concrete an "equivalent" strength. The other source of the tensile strength stems from the steel deck, which is modeled as a rebar layer embedded in the bottom slab fiber and assumed to act only in the direction parallel to the flutes.
7. The nodes of the columns bases are located one level below the ground and they are modeled as pinned. Lateral restraint was provided to the nodes located at the perimeter of the building and at the ground level.
8. The gravity load is the only loading scheme present in the analysis. A uniform downwards pressure load is applied at the level of each floor and it is quasi-statically incremented until the analysis was terminated due to numerical convergence issues provoked by the severe damage of the structural components. According to the provisions issued by the Department of Defense (DoD2016) and General Services Administration (GSA 2016) the gravity load in the 2x2 areas above the column removal is amplified with the Dynamic Increase Factor (Ω_N) to account for the dynamic impact of the phenomenon which is apparently not accounted within a static analysis framework. In the present study the value of Ω_N is 1.16.

4. Response of single-story building

The hypothesis concerning the effects of implementing rotational stiffness was initially tested on a sub-assemblage of the building, which consists of the top story of the structure. A column removal scenario in location C3 of Figure 1 was performed, to gain some preliminary knowledge surrounding the influence of rotational stiffness. 4 different versions of the model are analyzed, with their differences lying on the presence or absence of bending stiffness and the value of the tensile capacity. In the 1^{st} and 2^{nd} model the tensile capacity is C_{max} = 456 KN and bending stiffness is absent and present respectively. The same pattern regarding the stiffness response exists for the 3^{rd} and 4^{th} model respectively, whereas the tensile capacity of the connections in those cases is C_{max} = 365 KN.

The twelve connections under investigation are organized in four groups due to symmetry. These groups are notated as C1, C2, C3 and C4. According to the notation of Figure 1, C4 and C3 refer to the four connections in the middle attached to the girders and the secondary beams respectively, C2 are the four connections adjacent to the 2x2 area center and C1 are the four tensile connections located the furthest from the 2x2x area center. Numerical analyses validated the assumption that the symmetrically arranged connections follow an almost identical behavior, therefore a single curve is representative of the group response. Additionally, since the 2x2 area has a rectangular shape and carries the gravity load equally in both directions, C3 and C4 are also grouped together. Ultimately, three curves (one for the C3-C4, one for the C2 and one for the C1

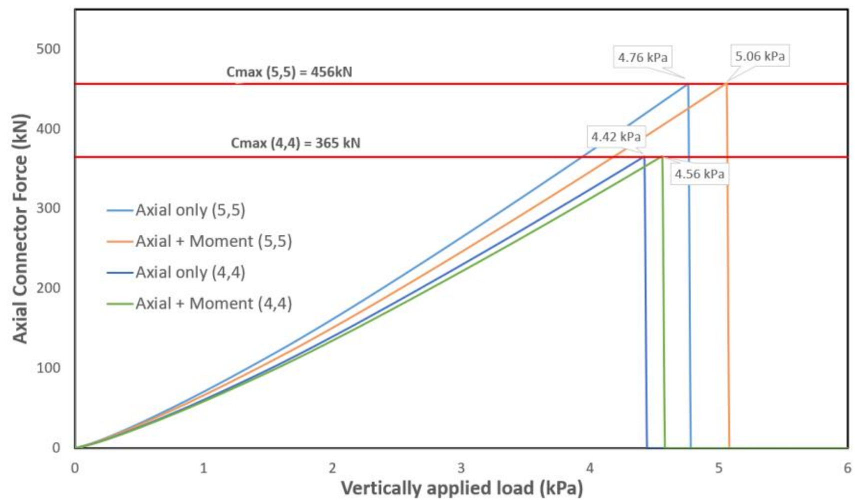

Figure 4: 9th floor removal - Axial force-applied load graphs for the C3 and C4 connections

group) are sufficient to describe the response of the twelve connections within the 2x2 area and the results for the 4 analyzed models are presented in the following figures.

Figure 4 shows the curves representing the C3 and C4 connections for the four cases. The trends confirm the assumption that accounting for the existing rotational stiffness increases the structures capability to bare load. In the case of combination 5,5 (5 bolts on both W18x35 and W24x68 connections), there is a 6.3% increase in the load at which C3 and C4 fail. This increase is about 3% for the combination of 4,4 (4 bolts on both W18x35 and W24x68 connections). These data provide a first insight of the impact that bending stiffness has in the response of the shear connections, slightly but not negligibly shifting the failure load of these components towards higher values. It should be reminded here that including this small amount of bending stiffness is closer to the actual response of the gravity connection and that the approach previously adopted in Pantidis and Gerasimidis (2018) is rather conservative.

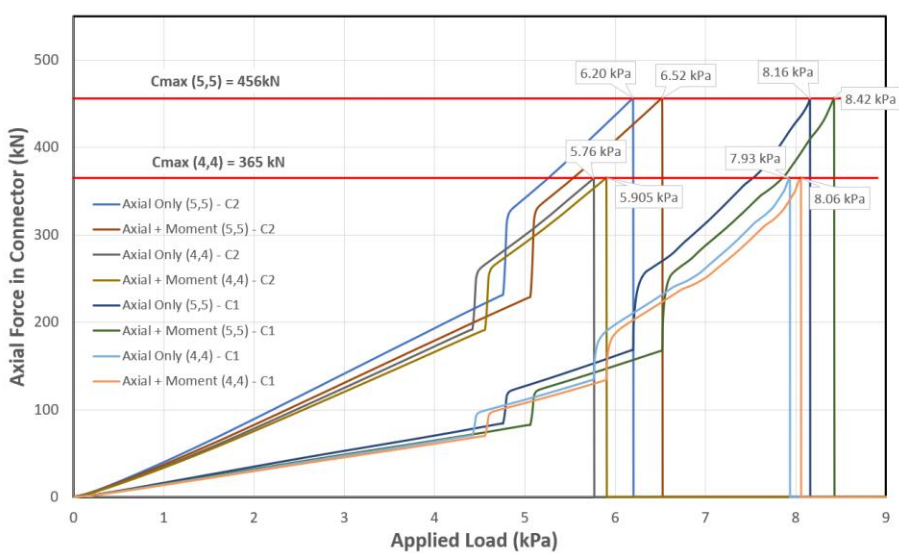

Figure 5: 9th floor removal - Axial force-applied load graphs for the C2 and C1 connections

Table 1: Impact of bending stiffness on the connections failure load

Location	Combination 5,5	Combination 4,4
C3-C4	+6.303%	+3.167%
C2	+5.161%	+2.517%
C1	+3.186%	+1.639%

As expected, the first connections to fail are C3 and C4. This causes the load to be redistributed within the 2x2 area, and C1 and C2 experience a sudden increase in their demands. Once C2 fail, another redistribution occurs, mainly affecting C1. These load redistributions are apparent in Figure 5, where a connection failure causes an abrupt increase in the demand of the remaining connections. It is observed however, that the beneficial effect of the bending stiffness occurs with reducing rate as the gravity load increases. The percent changes in the failure load of each connection are reflected in Table 1, inherently compared to the failure load of the "axial only" case. This outcome reveals that the role of the composite action deteriorates throughout the phenomenon, during which catenary action gradually emerges in order to efficiently carry the loads. Therefore, emphasis should be placed on the failure load of the last set of connections C1. Nevertheless, the results clearly indicate that there is an observable effect associated with the implementation of rotational stiffness. As the number of floors above the removal increases, more connections drift away from their purely pinned nature and the rotational stiffness which is added at each floor is anticipated to play a more pronounced role in the overall structural response.

5. Response of 9-story building

The impact of adding gravity connections bending stiffness is investigated on the 9-story building level by performing a column removal scenario at the 1st floor of the structure. Two versions of the full model are implemented: one with and one without bending stiffness (K_θ = 1531 kN-m/rad), whereas the tensile strength of the connections is C_{max} = 456 KN in both cases.

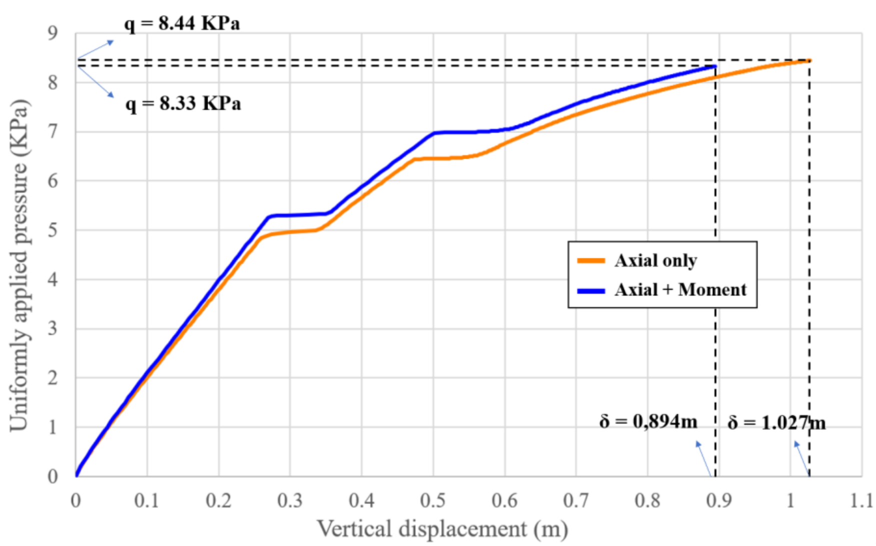

Figure 6: 1st floor column removal – Applied load-characteristic vertical displacement graph

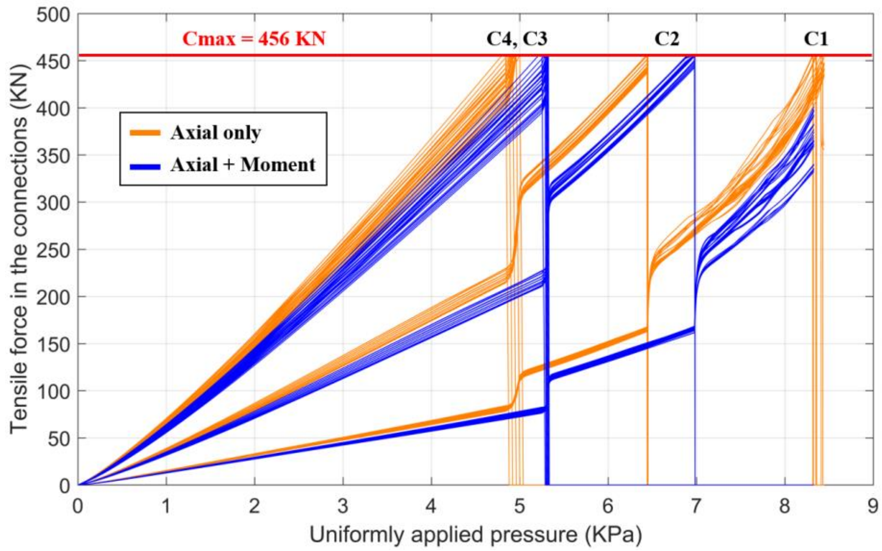

Figure 7: 1st floor column removal - Axial force-applied load diagram

Figure 6 depicts the P-δ graphs of the two models, where P is the uniformly applied load at each floor and δ is the vertical displacement of the node above the removal. These curves provide a first insight on the building response and show the increased stiffness of the structure in the case where bending stiffness is present. The two horizontal plateaus in each curve signify an abrupt deformation increase which is caused by the sudden failure of the gravity connections. They are shifted towards higher applied load values in the 2nd model, implying that the connections which accommodate bending rigidity will fail at higher loads. This is clearly verified in Figure 7, which illustrates the evolution of the axial forces of all the gravity connections in the 2x2 areas above the lost column with respect to the uniformly applied pressure. The curves with the blue color represent the axial tensile forces of the connectors from the model with bending rigidity, and the offset from the orange ones provides a quantified measure of the rotational stiffness contribution to the robustness of the building. Before proceeding with the characterization of the collapse mechanism in the model, it is interesting to observe that despite the vastly greater amount of connections (108 connections for 9 stories compared to 12 connections for a single-story building) all the curves of each group follow a very similar trend. This validates the notion that symmetrically arranged connections exhibit similar demands even at the presence of many floors above the removal, justifying that the gravity connections are essentially still operating as pinned joints.

The analysis in both models terminates due to the intelastic buckling of the column B3, which is the column next to the removed one. Evidence for this collapse mode is provided in Figures 8 and 9, which refer to the models with and without bending stiffness respectively. These figures depict the evolution of the von Mises stresses and the horizontal displacement of the buckled column. The 5 curves in the von Mises graph represent 5 different integration points across the column width (the reader is referred to Figure 1 for the orientation of the gravity column). In the case of the "Axial Only" model for example, the blue curve refers to the fiber which is closer to the removal, while the red curve indicates the fiber on the other side of the column. It is seen that in both cases yielding of half of the section renders the column incapable to obtain any additional load and causes an immediate increase in the horizontal displacement of the middle point.

Axial only

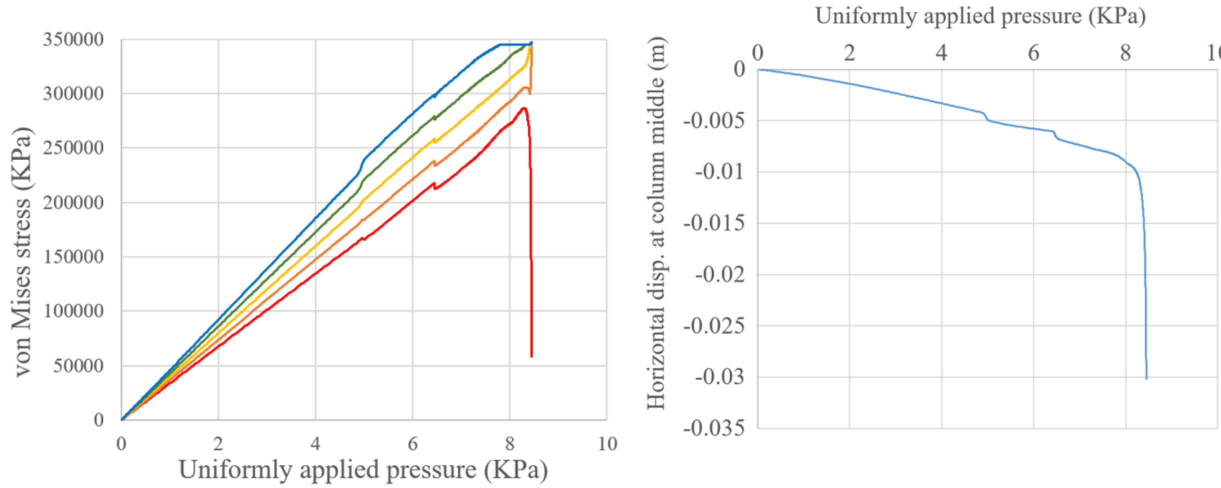

Figure 8: Axial Only 1st floor column removal – Buckled column behavior

The axial force at which both columns buckle shows an approximate 10% reduction from the purely theoretical inelastic strength. This is attributed to the following factors: a) a moment is present in the column section, observable in the von Mises stresses graph since they do not follow the exact same path and therefore the section is not compressed uniformly and b) a horizontal force is applied at the column top node, with the latter further burdening the column by causing a horizontal displacement at the top. In the aggregate, the results presented above indicate the clear inelastic buckling mode which causes the structural collapse in both models.

Axial + Moment

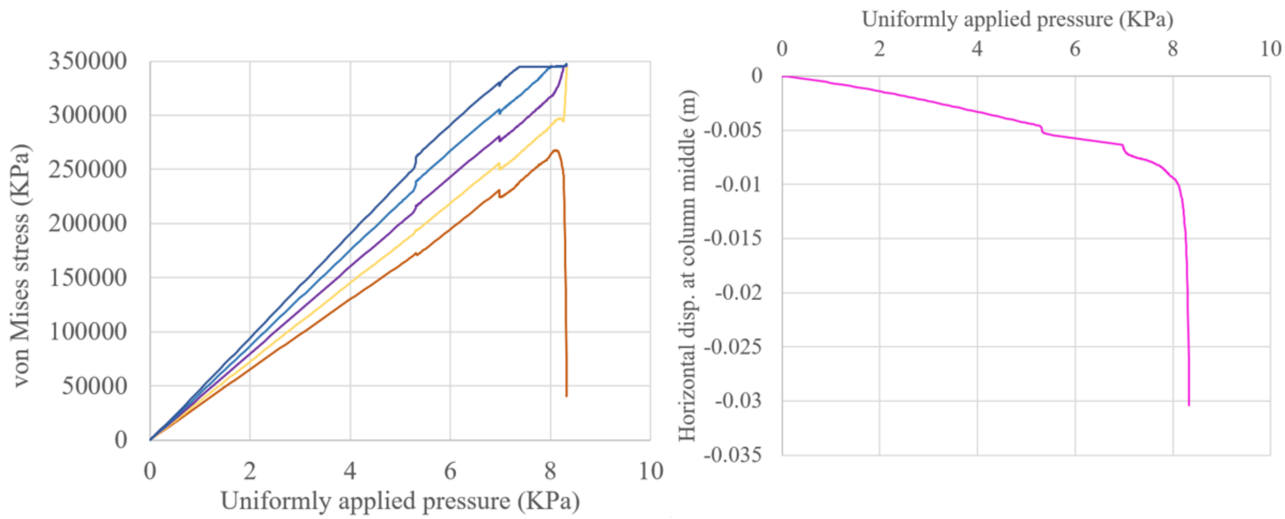

Figure 9: Axial + Moment 1st floor column removal – Buckled column behavior

6. Discussion

In order to facilitate the discussion regarding the impact of the bending stiffness contribution, the following comments can be made. It is worth noticing that when bending stiffness is present there is a slight percent increase in the failure load of C2 compared to the C3-C4 failure load, contrary to the decrease observed in the single-story building. This preliminary finding demonstrates the complex interplay of having many floors above the removal with partially-restrained gravity connections, and a parametric analysis with more floors under investigation as well as different values for the tensile capacity and the rotational stiffness is deemed necessary to gain a more thorough understanding of the phenomenon.

One of the most important observations of Figure 7 is that the connections which belong to the C1 group have not reached their maximum tensile capacity at the load at which the analysis terminates. It is apparent that the role of the composite action has not been exhausted and this mechanism still contributes to the building resistance. Column buckling is the dominant collapse mode even in the "Axial only" model, however in this case almost all the C1s are close to failure. The observed shift in the C1 forces in the "Axial and Moment" model prolongs the structural collapse due to the yielding-type mode and there is an evident attribution of the collapse purely to the stability mode. The impact of the bending contribution is therefore of crucial importance, since as the building is subjected to column removals along its height the resistance to the yielding-type mechanism is increased, driving more cases into the stability-governed scheme. Given the brittle, catastrophic and therefore highly undesired nature of the latter collapse mode, the findings of this paper open the discussion on how as well as the extent that the gravity connections should be reinforced to resist the additional demands imposed during a column removal scenario, ensuring at the same time a ductile-like collapse behavior.

7. Conclusions

The present study investigates the impact of accounting for the inherently small, nevertheless not negligible bending stiffness of the shear connections in a steel and concrete composite gravity framed system subjected to a column removal scenario. It is quantitatively shown that incorporating this stiffness in the numerical simulations may be of limited importance in the analysis of a single-story 3D gravity frame, however its impact is much more pronounced as the number of floors above the lost column increases. Treating the gravity connections as partially restrained and not strictly pinned enhances the robustness of the structure against the yielding-type mode and prolongs the collapse triggered by this mechanism, giving though at the same time rise to stability-governed cases. The results presented in this work constitute another step towards a reliable and detailed analytical and numerical robustness assessment of steel framed structures exposed to the progressive collapse loading domain. Future research will focus on the development of high-fidelity models of the gravity connections aiming to capture additional failure modes of these structural components, as well as the refinement of the analytical framework proposed by the authors to account for additional response features of the gravity connections.

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