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Effect of Building Bay Span on the Vulnerability of RC Shear Wall Buildings to Progressive Collapse

Omer Yagob[1], Khaled Galal[1] and Nove Naumoski[2]
[1]Department of Building, Civil and Environmental Engineering, Concordia University, Montreal, Canada.
[2]Deceased, Department of Civil Engineering, University of Ottawa, Ottawa, Canada.

Abstract: The extent of initial damage to the World Trade Center Towers in New York City and the Alfred P. Murrah Federal Building in Oklahoma City during the September 11, 2001, and April 19th, 1995, terrorist attacks was far beyond what would have been practical to consider for progressive collapse resistant design. These and other extreme assaults and their tragic outcomes have initiated wide spread interest and research in progressive collapse of structures under more moderate initial damage scenarios. As the vast majority of building structures in Canada, and more generally in North America, are designed as shear wall systems with flat plate floors without particular consideration for such a threat, this paper is intended to assess the potential of progressive collapse for two RC shear wall buildings, designed with different bay spans, subjected to a ground column removal, imitating the loss or sever damage of ground columns due to extreme loadings. The reinforced concrete (RC) buildings used in this study were designed in accordance with the 2005 edition of the National Building Code of Canada (NBCC 2005). For this purpose, the buildings were analysed following the guidelines for progressive collapse analysis and design, prepared by the U.S. General Services Administration (GSA). Columns were removed at the ground level of each building. One column is removed at a time. Nonlinear dynamic analysis was conducted for each of these cases using 3-dimensional models, while applying the appropriate load combination, as required by the GSA guidelines. The results of the nonlinear dynamic analysis in this study show that both buildings have a high potential to progressive collapse when their key elements are severally damaged or removed.

1 Introduction

Progressive collapse is characterised by the loss of load-carrying capacity of a relatively small portion of a structure. This initial damage triggers a cascade of failures, affecting a major proportion of the structure. A collapse of this nature can be triggered by many abnormal forces. Any applied forces that are not specified by codes in the initial design of structures can be defined as abnormal loads. There are different kinds and examples of abnormal loads, such as bomb blasts, vehicle impacts, gas explosions, construction errors, severe storms, and malicious terrorist attacks, which are not yet quantitatively specified by codes for the initial design of building structures. Any of these abnormal loads can be the root cause of partial or total progressive collapse that might take place in many RC building structures.

The majority of RC building structures in Canada, and in general in North America, are designed as shear wall systems with flat slabs or flat plate floors, where the shear walls are designed to be the seismic and wind force resisting systems, and the slabs are usually designed to resist gravity loads only. These slabs are a very common structural element that is widely used for low-rise, mid-rise or high-rise buildings of various types, including parking garages, condominiums and office buildings. They are popular because their construction optimizes interior space and minimizes storey height by eliminating beams. In addition, these floor systems are advantageous in terms of simpler formwork, a shorter construction period, flexible room arrangement, better air circulation, and better light penetration. Though the preferred forming system in modern construction of multi-storey RC buildings is the so-called fly-forms (i.e. flat plates, where a constant thickness of slabs is normally used with neither drop panels nor column capitals), flat slabs are still widely used.

Since these categories of two-way slabs, flat plates and flat slabs, are designed just to resist gravity loads, it would be difficult for such systems to resist the local effects of any extreme lateral loading, or even to redistribute any excessive gravity loads resulting from losing one or more key supports. Given this, it is certain that flat plate and flat slab systems have a high potential for progressive collapse when their gravity load-paths are damaged or removed and they cannot redistribute the new loads to other paths. This structural vulnerability is especially important considering the increased risk of terrorist attacks on civilian RC building structures worldwide, where the first-storey columns are usually vulnerable to substantial damage that has led, in many cases, to progressive collapse of entire buildings.

2 “Sudden Column Loss” Approach

Evaluating the risk of progressive collapse is a complicated task. This complication is exacerbated by the unknown magnitude and nature of the initial impact, and the responses engendered by most extreme threats. This is because most commonly analysed threats are associated with high thermal, impact and blast loads. Given this uncertainty, the Sudden Column Loss approach has been considered the most suitable simulation technique used for progressive collapse analyses. The Sudden Column Loss approach is prescriptive and does not explicitly consider a specific threat. The extreme loading event causing the localized damage to a key structural member and triggering a partial or total progressive collapse of the entire structure is not precisely simulated, but rather its direct effects on the structure are the focus. Yet, it implicitly assumes that the initial failure corresponds to a blast or impact loading scenario or any natural disasters resulting in a full column removal. In most of the ongoing research worldwide, this approach is considered as a useful design scenario for the assessment of structural robustness.

Structurally, the ‘Sudden Column Loss’ is an event-independent approach, where the resulting failing element is removed from the initial topology and the response of the structure to this sudden removal key element is then studied. The GSA criteria include this technique in their guidelines via the alternate path (AP) method: one critical load-bearing member, typically a ground column, is unable to sustain an extreme loading and is thus damaged. The residual structure must be able to accommodate this loss by developing an alternate load path to redistribute gravity loads.

In order to simulate the dynamic response behaviour of the RC building systems using the Sudden Column Loss approach, the GSA guidelines require a separate analysis for the instantaneous loss of a single column at the ground-level of a building structure. The aim of the analysis is to explicitly determine the ability of the damaged structure to bridge across the removed column. Based on the GSA guidelines, the following four cases need to be considered for the removal of columns in all designed buildings:

- Column at the corner of the building should be removed.
- Column on the perimeter, approximately at the middle of the long side of the building, should be removed.
- Column on the perimeter, approximately at the middle of the short side of the building, should be removed.
- Interior column should be removed.

These four ground-level column removal scenarios, representing different threat locations around typical RC shear wall building structures, are analyzed in this study. Thus, one column is instantaneously removed at a time, as defined in Fig. 1.

3 Analysis Procedure and Load Combination

In the GSA guidelines, there are different levels of analysis that can be used to perform the alternate load path analysis. Yet, nonlinear dynamic analysis is highly recommended for the assessment of the progressive collapse potential of buildings, where the accuracy achieved by conducting “step-by-step integration” is the main advantage of using this procedure. In fact, this analysis approach demonstrates the safety of the studied structures over other approaches, as it determines the weak structural sections and follows their failure mechanism through the RC building structures. Given this, the progressive collapse-resisting capabilities of building structures in this study are evaluated by using nonlinear dynamic analysis. The nonlinear dynamic behaviour of the structure resulting from a sudden removal of a load-bearing structural element is conducted by considering two phases. In the first phase of the dynamic analysis, the structure is allowed to reach equilibrium under the applied load case. In the second phase, the column or wall section is removed almost instantaneously and the software tool calculates the response behaviour of the analyzed structure. The GSA guidelines specify a loading combination of

[1] $\boldsymbol{G_{ND}}$ = 1.0 DL+0.25 LL

where $\boldsymbol{G_{ND}}$ = Gravity loads,
DL = Dead load, and
LL = Live load.

to be applied in the dynamic simulation of progressive collapse for the entire structure for each of the column removal scenarios previously mentioned. It should be noted that, while the GSA guidelines allow the use of 2-dimensional models in progressive collapse analysis, the use of 3-dimensional models in such analysis is highly recommended. This is in order to account for 3-dimensional effects and to avoid overly conservative results, where more realistic evaluation of the progressive collapse phenomenon will be offered. In light of this, in this study, 3-dimensional structural models were used in all cases of column removal, for both studied RC buildings.

4 Description of the RC Buildings and Design Parameters

Two typical office building structures designed for Vancouver, BC, were used for the purpose of this study. The buildings were designed according to the 2005 edition of the National Building Code of Canada (NBCC 2005). These RC shear wall buildings are classified as mid-rise buildings (10-storey). Span ratios of 1 and 2 were considered in the design of these buildings in order to cover the category of two way slabs. RC building of a span ratio of 1 has a rectangular footprint of 882 square meters, with a 42-meter span in the East-West direction and a 21-meter span in the North-South direction. The center to center column span in both the longitudinal and transverse directions is 7 m. Thus, there are six spans in the longitudinal direction and three spans in the transverse direction, with 22 column locations in each building, as can be seen in Fig. 1(a). On the other hand, the RC building with a span ratio of 2 has a rectangular footprint of 441 square meters, with a 21-meter span in the East-West direction and a 21-meter span in the North-South direction. The center to center column spans in the longitudinal and transverse directions are 3.5m and 7 m, respectively. Thus, again, there are six spans in the longitudinal direction and three spans in the transverse direction, with 22 column locations in each building, as illustrated in Fig. 1(a). Both RC buildings in this study have a typical storey height of 3.65 m (Fig. 1(b)).

Flat plate gravity load resistance system was used in both buildings. All slab components are of a uniform thickness of 250 mm without drop panels or column capitals. The required slab thickness and reinforcing steel are the same at all floor levels and at the roof of each designed building. All structural members in this study were designed with a unit density of concrete of 2400 kg/m^3, a concrete compressive strength, f_c', of 35 MPa and a concrete modulus of elasticity, E_c, of 28 GPa. These values were chosen because they are commonly used in the structural design of RC members. A reinforcement yield strength, f_y, of 400 MPa and a steel modulus of elasticity, E_s, of 200 GPa were used in the design. The ratio between ultimate stress, f_u, and yield strength in the steel was set at 1.3. These values are also common values used for steel reinforcement.

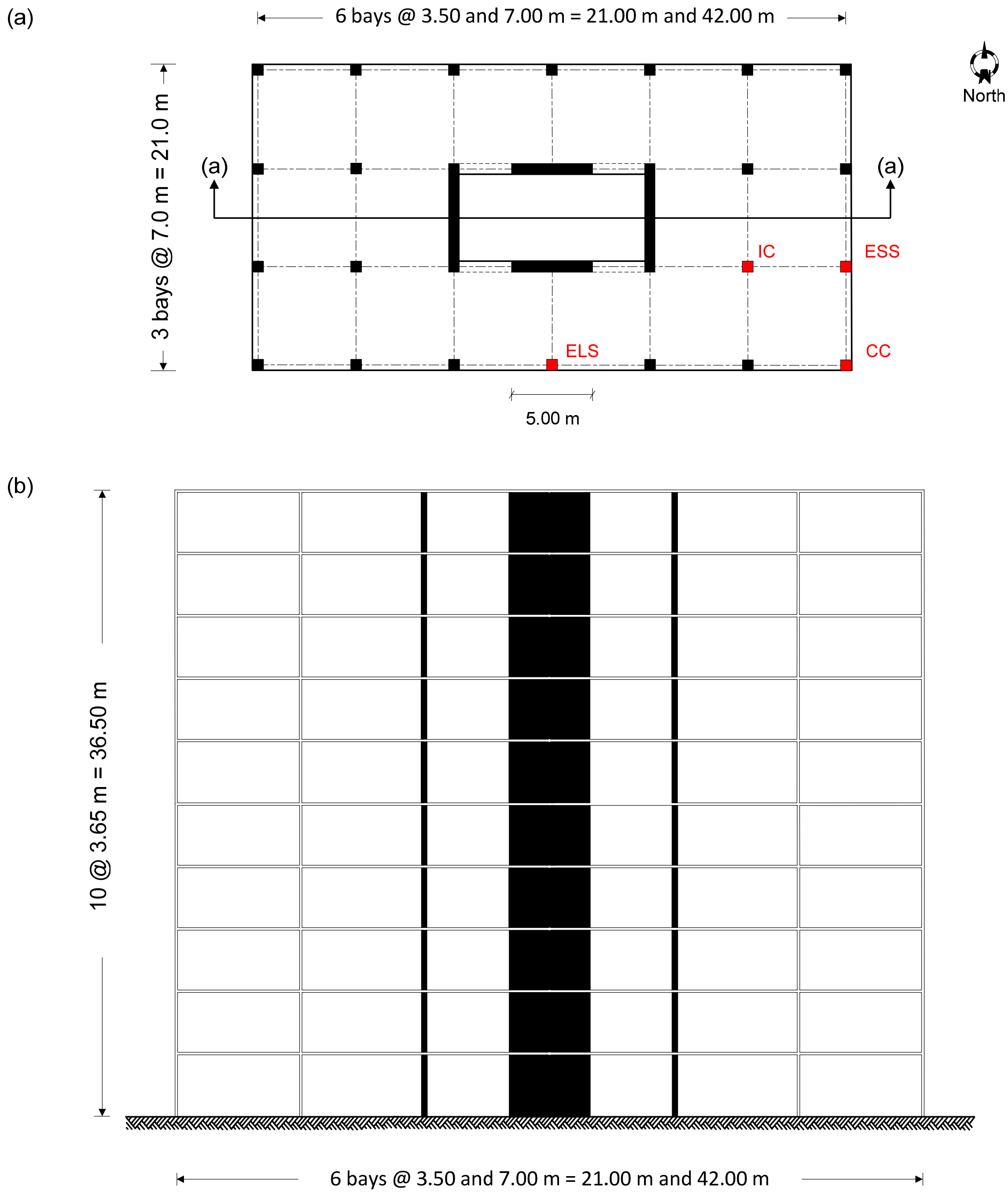

Figure 1: (a) Plan of floors and (b) cross section (a-a) of the building.

In both buildings, it was considered that each floor slab supports its own dead load plus 2.25 kPa average weight for partitions and finishes, and a live load of 2.4 kPa representing an office space use and occupancy. The PCA computer program spSlab was used to design and detail RC slabs in all both buildings, as shown in Figs. 2. For the sake of simplicity, all columns have square cross sections, and are assumed to have the same dimensions and reinforcement along the whole height, from the ground level to the roof. The column design is based on the ground floor columns of each RC building, where the PCA computer program spColumn was used to design and detail all columns of both buildings. The seismic force resisting system used in this study is a ductile shear wall system located in the center of each building, as can be seen in the plan of the RC buildings in Fig. 1(a). As the general behaviour of RC shear wall building structures in progressive collapse mainly relies on the behaviour of their slab components, only the details of the slabs are presented in this research study.

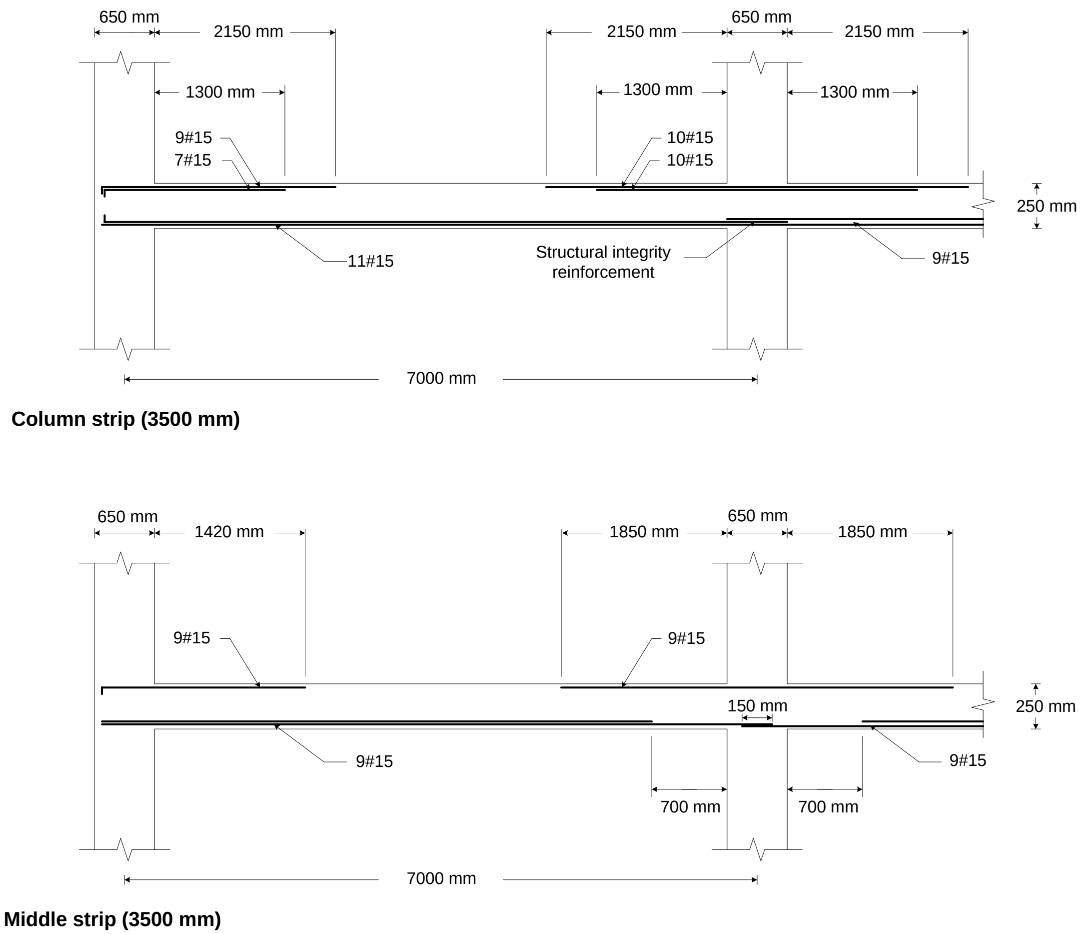

Figure 2: Interior strip of a flat plate floor system, of span ratio 1, designed and detailed for the effects of gravity loads using Canadian Standard A23.3.04 (CSA 2004).

5 Expected Nonlinear Behaviour for the Designed Reinforcement by the CSA Standard

Slab structural components designed according to national design standards in general, and to the CSA design standards in particular, barely have any resistance to the propagation of progressive collapse once it is initiated. This is, simply, because there are no explicit considerations in most codes for designing these components against such a threat. Since the vast majority of national design codes and specifications have approached similar provisions in their flat plate design, though from different perspectives, much criticism has recently been aimed at current design codes. The recent evidence of propagation failure of slab structures worldwide, caused by losing one or more of their primary supports, indicates that the current slab design standard provisions used in Canada, i.e. the CSA standard A23.3-04 *Design of Concrete Structures*, need to be revised to address a wider range of design parameters if such a progressive collapse is to be avoided.

Considering the RC slabs designed in this study, the reinforcement requirement provisions of the CSA Standard are insufficient in the event of any accidental column loss that the designed RC flat plate is subjected to. In fact, these provisions would be inadequate for accidental corner column loss, as the discontinuity of the top reinforcement in the middle of the spans, where positive moments are generated, will result, for example, in the collapse of the entire corner portions of building structures when corner columns are removed or severely damaged. In addition, the outer row of bays all around the building will be vulnerable to collapse when edge columns are destroyed. This is because, in such scenarios of damaged or removed columns, the bending moments and other loading effects at all slab sections along slab supports in general, and at the bars' curtailment sections in particular, are completely different than those obtained in the initial design when slabs were sufficiently supported and subjected to distributed gravity loads. This is especially pronounced when the edge column is removed or severally damaged. Moreover, the curtailments in the top reinforcement are considered a key factor in identifying the location of the plastic hinges that may greatly contribute to the damage propagation of building structures.

Though the structural integrity reinforcement that is required in the slab system would have some contribution to limiting the spread from a local failure and preventing slab collapse when interior and edge columns are damaged, it will not play any role when corner columns are removed. Furthermore, the limited requirements of structural integrity reinforcement, only in the column section regions and not including the width of the column strips, would be insufficient in collapse prevention when interior or edge columns are removed, as the resulting dynamic-gravity load redistribution will likely lead to a catastrophic propagation of progressive collapse through the entire building. This is especially the case as about 50% of the bottom reinforcement, according to the CSA Standard, in both column and middle strips, lack continuity over the supports, i.e. in the negative moment region. Overall, it is certain that, although the structural integrity reinforcement does have some contribution to limiting the separation of slab, the dynamic analysis showed that it does not have significant impact on preventing the collapse of the slabs when the separation takes place in extreme loading events.

6 Modeling Buildings for Progressive Collapse Analysis

For the purpose of progressive collapse analysis, the building structures in this study are modeled and analysed using the nonlinear program Extreme Loading for Structures®. This structural tool is based on the Applied Element Method (Tagel-Din, H. 1998), where all structural members, i.e. slabs, columns and shear walls, are modeled as small cubic elements. The connectivity between elements in the Applied Element Method is provided through the element faces, where the elements are connected by a series of non-linear springs representing the material behaviour. These connecting springs represent both axial and shear deformations to accurately recreate the overall behaviour of the modeled structure. When the average strain value at the element face reaches the separation strain, all springs at this face are removed and the elements are separated until a collision occurs, at which point they collide as rigid bodies.

The Extreme Loading for Structures® software is, nowadays, the most frequently used software in progressive collapse research to track the structural behaviour of building structures in both elastic and inelastic modes during different stages of their stabilities and instabilities. This includes tracking the structural behaviour 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. All consequences of these failures are automatically considered and calculated during the analysis.

7 Development of the Extreme Loading for Structures® Models

A three-dimensional model of each RC building designed for the purpose of this study was developed for nonlinear dynamic analysis using the structural analysis program Extreme Loading for Structures®. The objective of this is to evaluate the potential of RC buildings with different span ratios for progressive collapse analysis during and after the free vibration phase of the analyses. While all structural components of an RC building of span ratio 1 were modeled for the analysis, i.e. structural slabs and column members as well as shear wall systems, only slab and column structural components of the RC building of a span ratio of 2 were modeled. Thus, the building with a span ratio of 2 is only a gravity load resistance system. This is because, in such a study, the shear wall system is not expected to play any role against local threats, as it is designed to globally resist any lateral loads on the building system.

For the purpose of this study, columns at the first storey level of each RC building structure were removed suddenly and in rotation, i.e. one column at a time is removed, as if it had been lost due to an explosion. However, the manner in which the column was lost is assumed to be irrelevant and no outside forces are applied to the system. This model assumption works in accordance with the alternate path method, where triggering events do not affect the system. The following four scenarios, as shown in Fig. 1, were investigated in both RC building structures:

- Corner column removed (CC),
- Exterior column removed at the long side of the building (ELS),
- Penultimate column removed at the short side of the building (ESS) and
- Interior column removed (IC).

In each of these column removal scenarios, the sudden column removal technique is employed, which in practical terms means that the removal is performed in a single time step. In these scenarios, the column is removed in 1 ms (0.001 seconds). In fact, this removal time would correspond to the duration of the impulsive phase of an explosion (Smith 1994). The load combination as required by the GSA guidelines was applied to the entire structure for each of these buildings. The loading scenario in this study consisted of two phases, Stage 1 and Stage 2. During Stage 1, the self-weight of the structure was applied over 50 increments. Stage 2 was a dynamic stage during which the element removal takes place. The total duration of Stage 2 was 10 seconds, allowing enough time for element removal and complete collapse (if it occurs). Since the duration of the progressive collapse which might take place in the building is very small, the selected time step within Stage 2 was 0.001 with 10 divisions in each time step. Thus, the total number of calculation steps was 100,000 in each analysis.

8 Dynamic Analyses: Column Removal Results

The nonlinear dynamic analysis for both RC buildings, span ratios of 1 and 2, shows that none of the studied buildings could resist progressive collapse under any of the column removal scenarios, as the placement and detailing of reinforcing steel within their slabs was not designed for such a threat. As discussed and anticipated in Section 5, these are the key aspects of progressive collapse resistance. In general, it was found that for the same column removal scenario in the studied buildings, the collapse in the building with a span ratio of 2 was somehow delayed, coupled with some passive resistance, compared to the building with a span ratio of 1. This could be attributed to the short span cantilevered by its column strip rebars in the building of span ratio 2, in that the moment demands on the slab's short span were much less than those obtained in the other slab span. Moreover, the short building span in the affected slab components of span ratio 2 stimulates the redistributed gravity loads finding their alternative load paths through them. Thus, the difference in moment demands caused by the span ratio, no doubt, contributed to the delayed failure in the sudden change in the equilibrium condition during progressive collapse.

It is significant to note that, on the other hand, under cases of interior column removal scenarios of both buildings, the redistribution of the loads of failed members and the impact of falling debris from higher floor slabs onto lower floor slabs caused a propagation of progressive collapse to one third of both buildings, as can be seen in Fig. 3. Likely, this is because the conventional capacity of lower floor slabs was largely exceeded by the debris loadings; thus, the slab panel components could not resist the punching failure in the vicinity of the support columns neighboring the removed column. In this scenario, a propagated brittle shear failure within the slab panels of the lower floors caused the catastrophic collapse within the structures.

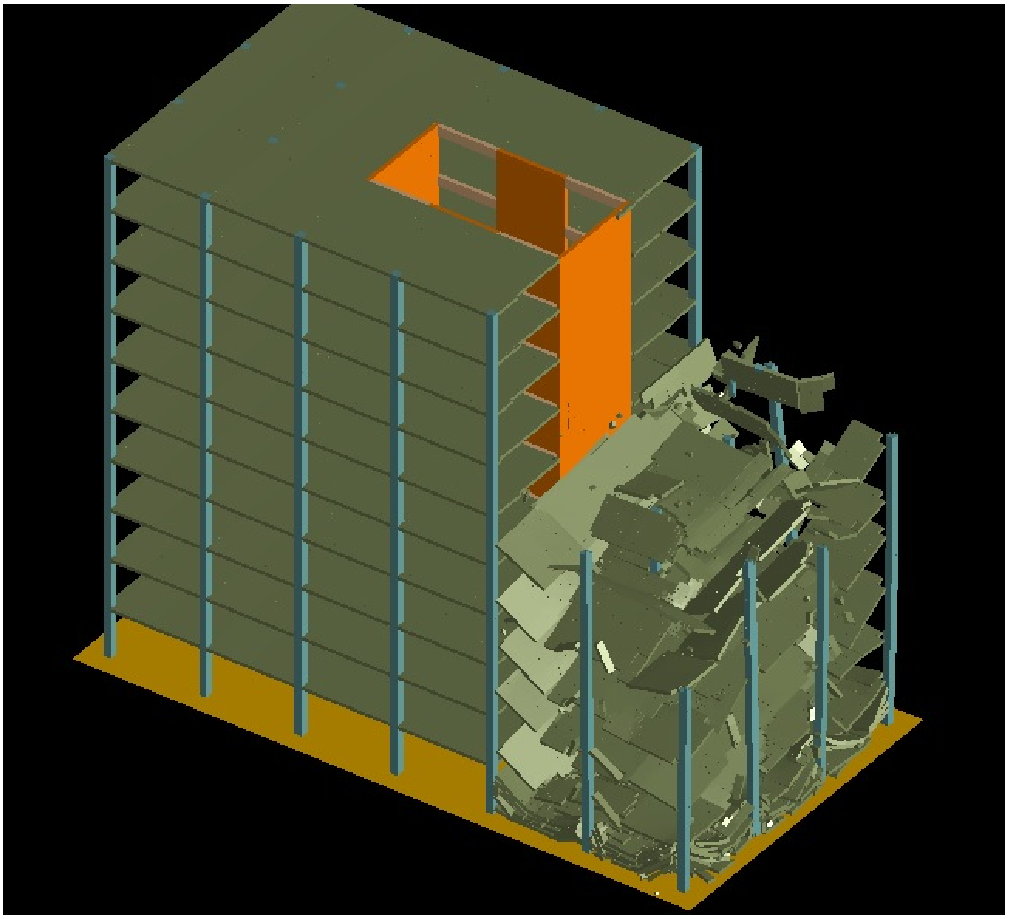

Figure 3: Interior column removed in a 10-storey building of span ratio 1.
(at 6.25 sec of the analysis)

Another observation from this study is that corner and edge column removal scenarios are more critical than those of the interior columns. This is likely because, according to the plan of the buildings in Fig. 1, the number of slab panels connected to the interior columns in each floor level is twice and four times that of the edge and corner columns, respectively. Thus, having fewer possible alternative path loads for the redistribution of gravity loads accelerates the failure mode in the affected slab panels. Another reason is that the interior column strips within the slab panels carry twice as much gravity load as the exterior column strips. Hence, the interior column strips are designed with strengthened sections, which then lead to more resistance against interior column removal scenarios. The following subsections separately discuss each of the removal scenarios conducted in this study, detailing the failure mechanisms.

8.1 Corner column removed

The corner portion of each analyzed building, along its entire height, collapsed once the IC was removed, as illustrated in Fig. 4. As expected, the first cracking started at the curtailments of the top reinforcement of the neighboring supports to the removed column, before the capacity of the slab sections at these curtailments failed under the redistributed gravity loads, as can be seen in Fig. 5. This was the case in both RC buildings considered, with slightly different performances but the same final response of partial collapse. The displacement vs. time responses of each RC building to this scenario of column removal is illustrated in Fig 6 of this study. In this scenario, the unequal moment redistribution caused by the removed column on the building of span ratio 2 slightly delayed the corner portion failure compared to the building of span ratio 1. The short column strips on slab panels of span ratio 2 functionally cantilevered the redistributed gravity loads before their insufficient detail reinforcement resulted in their collapse.

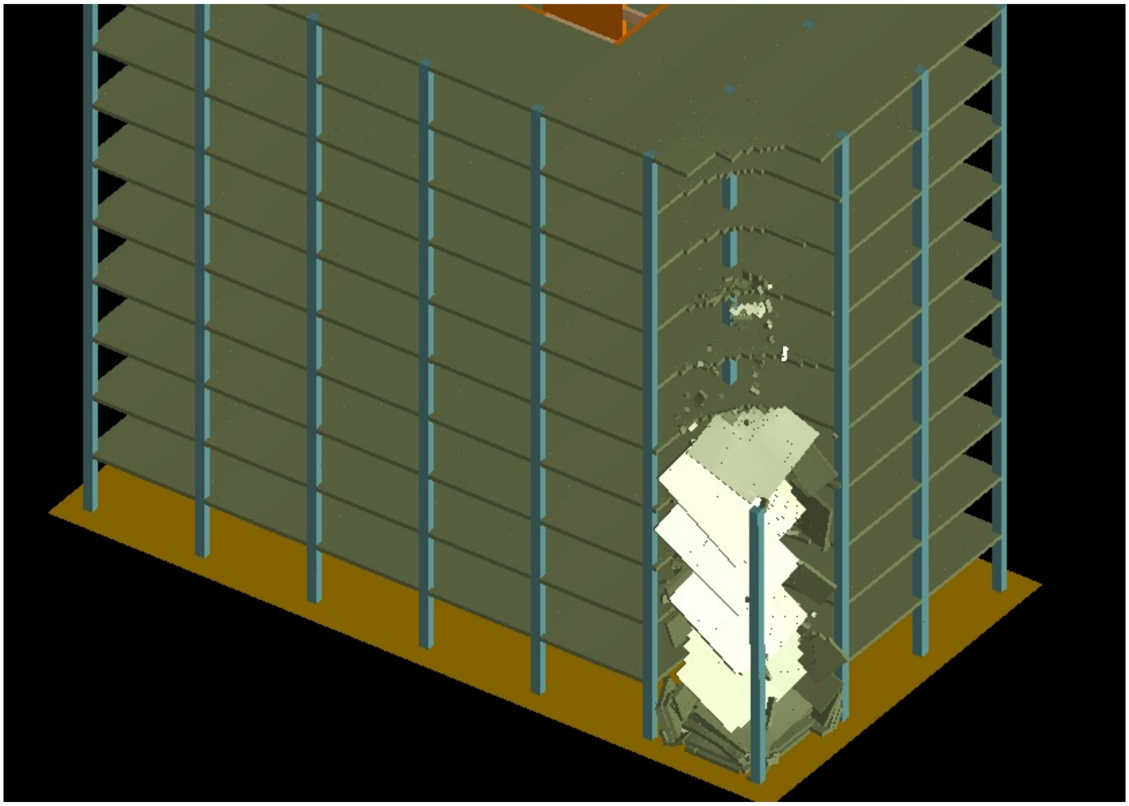

Figure 4: Corner column removed in a 10-storey building of span ratio 1.
(at second 4.20 of the analysis)

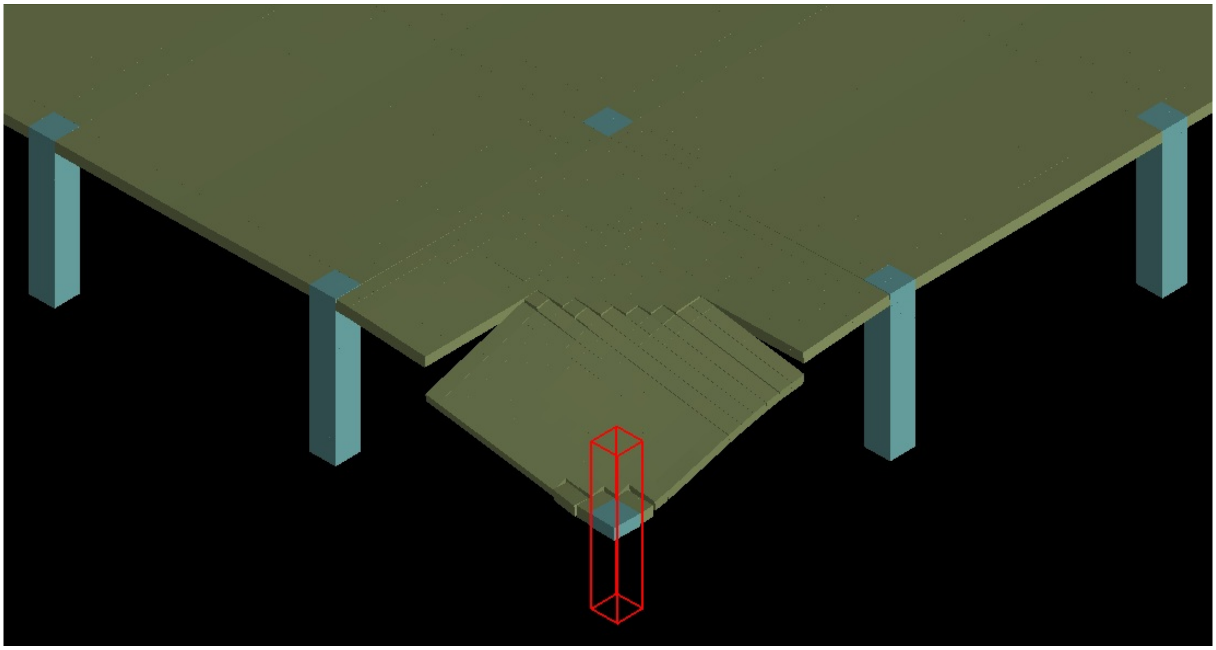

Figure 5: The failure mechanism at the top reinforcement curtailments in the first story of the building with a span ratio of 1 (at second 1.15 of the analysis).

Following corner column loss, it is clear that the discontinuity of the top reinforcement, in the middle of the spans of the corner bays, leads to the collapse of the entire corner portions of the building structures. This is because the bending moments in the slab when the corner column is removed are very different from those used during design. Thus, the curtailments in the top reinforcement in this collapse scenario are considered to be the main factors contributing to the propagation of progressive collapse in the corner portion, along the entire height of each RC building. Furthermore, it was observed that the structural integrity reinforcement that is required in the RC slab system for bonding with their corner columns does not contribute any help or even play any role in preventing the slab's collapse once these supports are damaged or totally removed. It is essential to note that, under this scenario of column loss, the failure was only partial in both studied RC buildings, where only the corner portions of these RC buildings fail and the collapse did not propagate to include other slab panels in any of the floor levels.

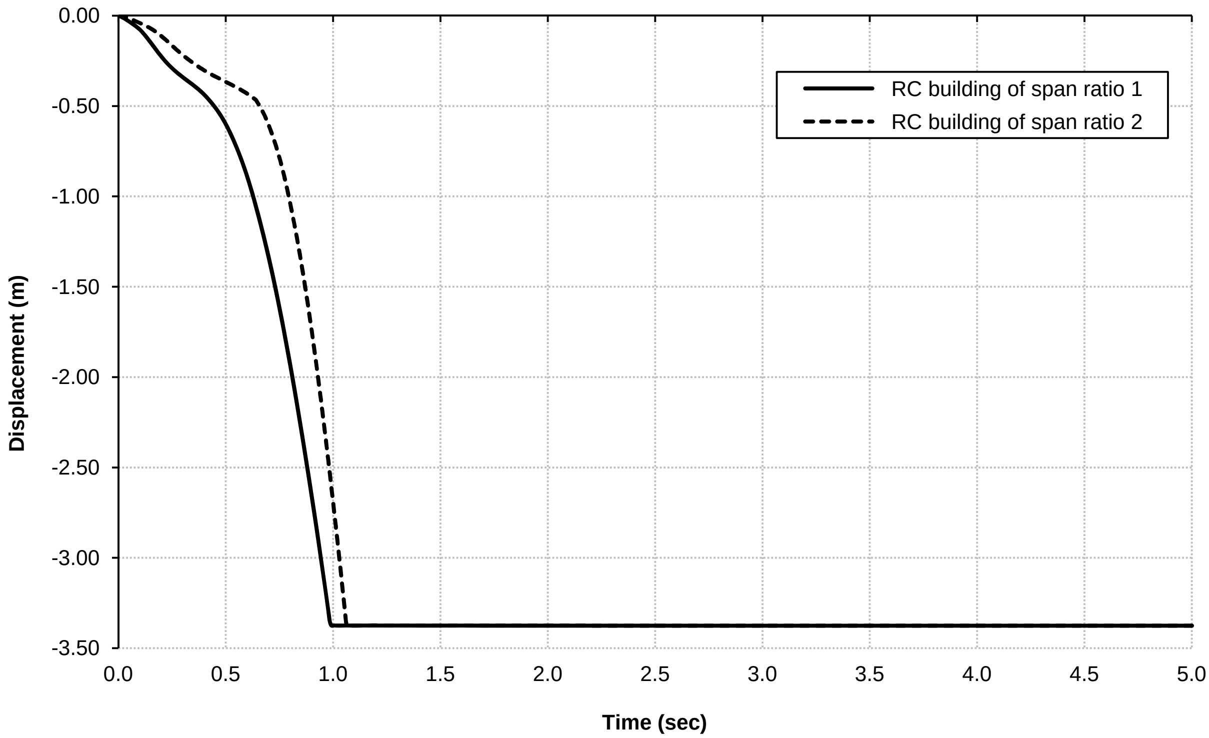

Figure 6: Response of RC shear wall buildings to the removal of CC.
(Designed according to NBCC 2005)

8.2 Periphery column removed

In this column removal scenario, whether it is ELS or ESS, both buildings could not resist the redistributed gravity loads and collapsed. The exterior two bays, which were supported by the removed edge column, along the height of all analyzed buildings completely failed. The debris falling on lower slab floors played a major role in this complete collapse. In ESS scenario, for the building of span ratio 1, this partial collapsed is illustrated in Fig. 7. While the propagated failure was identical in this scenario in ESS column scenario, where the short side spans of both buildings are the same, there was a slight difference in the failure response of the two buildings that did not affect the final collapse results once ELS column was removed, as can be seen in Fig. 8. This is attributed to the short alternative load path in the RC building of span ratio 2, which the redistributed gravity loads utilized before the failure kicks in.

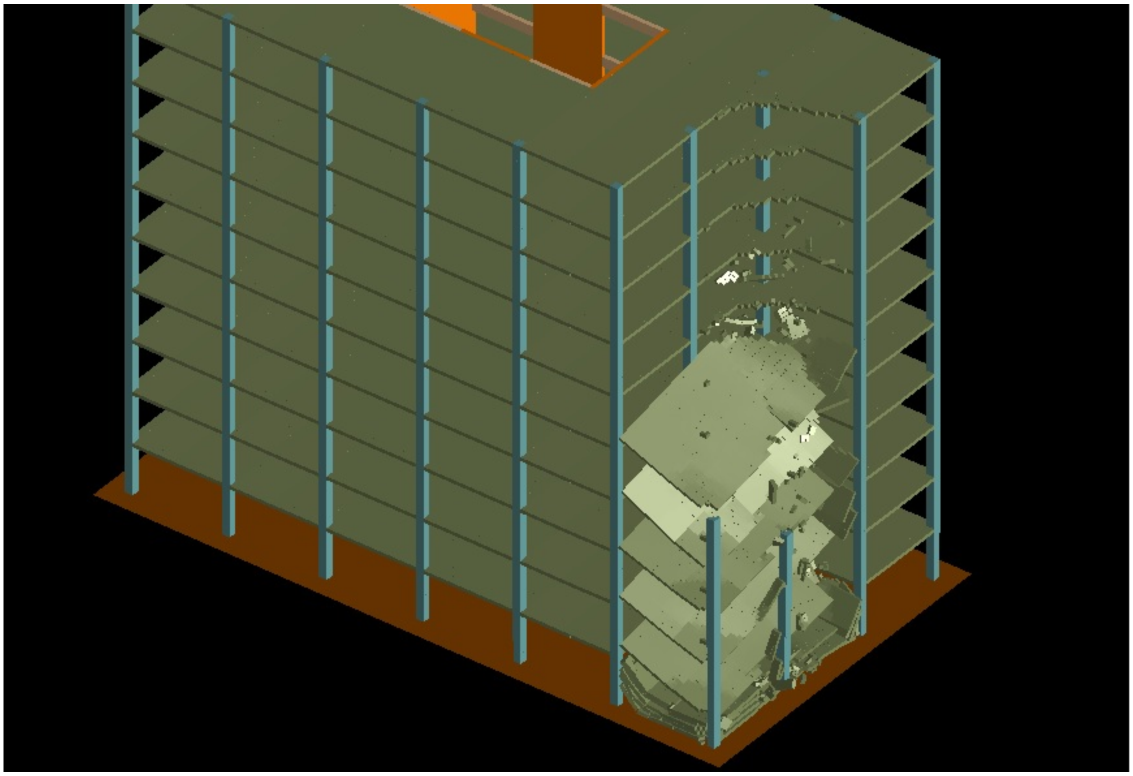

Figure 7: Partial collapse of the building with a span ratio of 1 (at second 5.25 of the analysis).

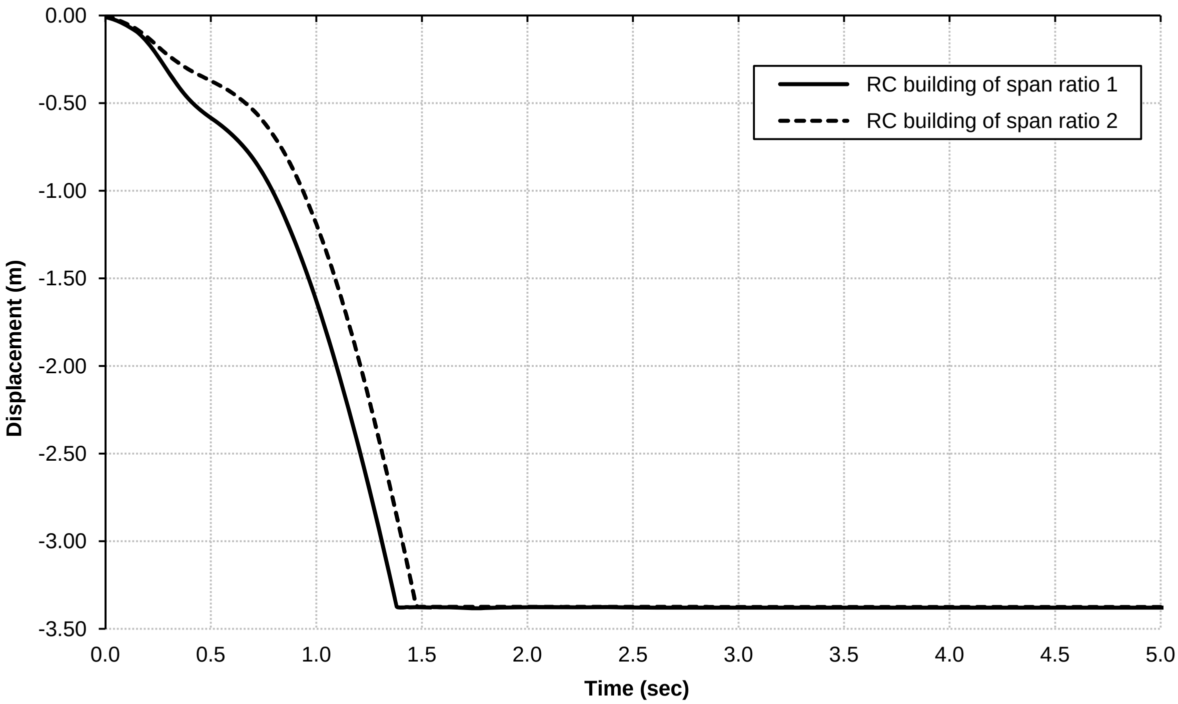

Figure 8: Response of RC shear wall buildings to the removal of ELS.
(Designed according to NBCC 2005)

Tracking the failure mechanism in both RC buildings considered in this study under this scenario of column loss (ESS or ELS), it is clear that the propagated collapse was due to the redistribution of gravity loads that the slab's double span could not resist. The redistributed gravity loads caused a failure of the slab segments at the removed column location, and at the same time, at each of the neighbouring peripheral supports, where there are curtailments of top reinforcement, along the height of each building. None of the neighboring supports, whether in the long or short side of the RC buildings, had any contribution to preventing the collapse of the exterior slab panels, as the excessive gravity load could not be functionally transferred, whether through short or long alternative load paths, i.e. short and long column strips, to these supports.

While providing continuity to the top reinforcement in the middle of the spans in the interior column strips that connect the removed columns to the interior columns, as the negative bending moment demand is shifted towards the removed column, is expected to increase the building's resistance, it is certain that there are other factors that contribute to the failure. The first is the insufficient positive reinforcement passing through the removed column area, within the column strips, at the bottom of the slab level, where about 50% of the reinforcement lacks continuity. Moreover, the improper length of top reinforcement within the same column strips connecting the removed column to the neighbouring columns is an additional reason for the failure, as the negative moments are carried over wider ranges compared to those considered in the ordinary design of the column and middle slab strips.

8.3 Interior column removed

Under the IC loss scenario considered in this study, the slabs of both buildings could not sustain the double span in each direction, and progressive collapse occurred, as seen in Fig. 3. Though the higher span ratio building of 2 shows some passive resistance that translated to a slight delay in the failure mechanism compared with the other building, the dramatic result of complete collapse of both buildings was the same (Fig. 9). The short column strip of the designed slab panels in the building of span ratio 2 acted as a girder to the long column strip that crosses this girder at the middle, which provides some passive resistance to the affected slab panels before the propagated progressive collapse begins.

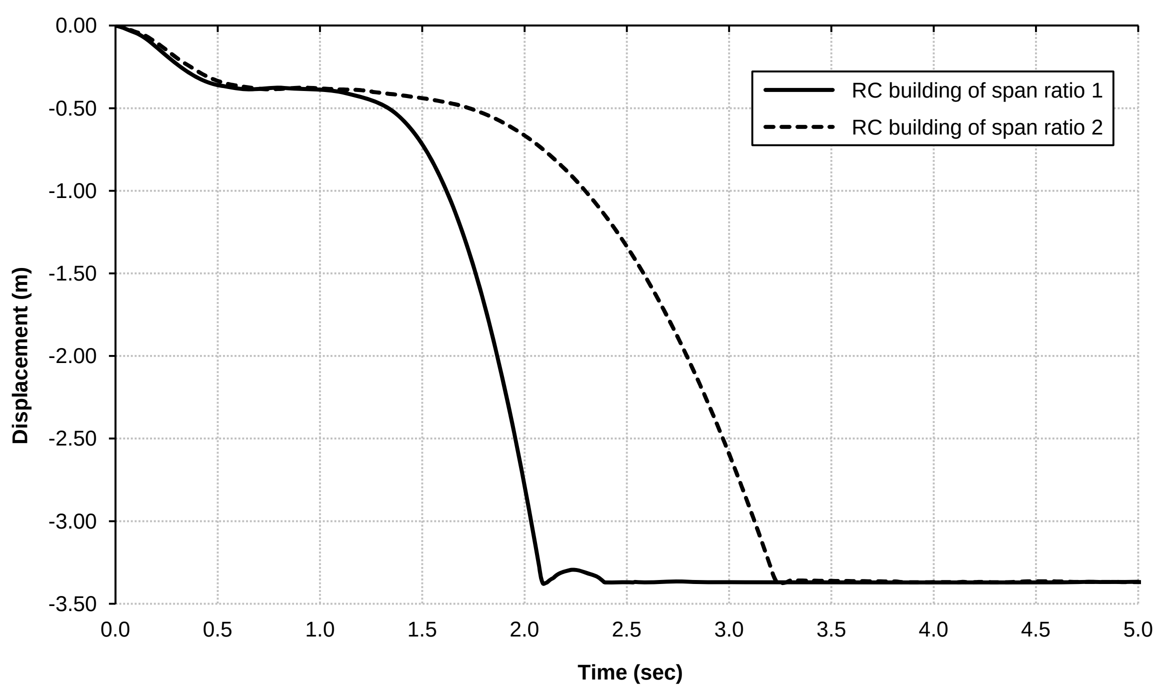

Figure 9: Response of RC shear wall buildings to the removal of IC.
(Designed according to NBCC 2005)

In this scenario of column loss, the moments in the slab sections of the affected slab panels were significantly larger than the capacity of the slab (approximately 4 times larger due to the doubling of the span upon loss of the interior column). Thus, slab panels deflected, though more slowly compared to the other scenarios of column removal, and then failed. In fact, the positive moment capacity of the slab segments connecting the four slab panels adjacent to the removed column was nearly limited to the required structural integrity reinforcement over the damaged column. About 50% of the required bottom reinforcement in both the column and the middle strips connecting the removed column are not continuous, where the slab in the vicinity of the removed column is designed to resist only negative moments. As the positive moment capacity in the removed column region was limited to the structural integrity reinforcement and the resulting dynamic gravity load redistribution was much higher than the capacity of the affected slab section, this led to a catastrophic propagation of progressive collapse throughout the RC buildings.

It is clear that, in this scenario, the slab components lack the ability to resist the moment reversal that occurs over the removed column. On the other hand, the failure mechanism under this column loss scenario indicates that the improper length of top reinforcement within the designed column strips connecting the neighboring supports to the removed column also play a significant role in this failure. The length of the negative moment region at the neighboring supports shifts towards the removed column, coupled with a larger magnitude. Thus, the regenerated negative moment demand by the redistributed gravity loads causes the slab failure at the curtailments of this reinforcement. In fact, this is similar to the failure mechanism of the scenario of edge column removal, where the regenerated negative moments within the outer column strips were over wider ranges compared to those used in the initial design.

9 Summary and Conclusions

This study analyzed the inherent ability of mid-rise RC shear wall buildings, satisfying the current NBCC code requirements with two different span ratios, to resist progressive collapse. The obtained results demonstrated that RC shear wall buildings with two-way flat plate systems are prone to propagating progressive collapse due to a failure of any key element support at the ground level, when their slabs are designed and detailed for gravity loading only, which is the case with the structural requirement of current design codes. None of the analyzed RC buildings could safely transfer the redistributed gravity loads caused by any of the removed column scenarios to the surrounding undamaged members, which destabilized the analyzed structures and triggered a catastrophic collapse within these buildings. In fact, in all considered cases of column removal in this study, the results showed that progressive collapse was initiated and propagated throughout the analyzed buildings as the slab reinforcement detailing in both structures was insufficient to carry the redistribution of gravity loads within the slab sections, not to mention passing these tremendous loads to the surrounding load paths. The slab reinforcement detailing significantly impacts the structural performance of the analyzed buildings, where a lack of appropriate reinforcement for the generated moments within the slab sections, caused by the redistribution of gravity loads, results in a progressive collapse that propagates throughout the analyzed buildings in some column removal scenarios.

Through analyzing mid-rise buildings with different span ratios of the two-way flat slab, an interesting observation was made that, with the increase of span ratio, the buildings' failure due to the considered scenarios of removal columns was somehow delayed, coupled with some passive resistance. This is most likely because, as the moment arm of the affected slab panel decreases and, thus, the moment demands are lower in magnitude than of those obtained in the other arm of the affected slab panels, the redistributed gravity loads seek an alternative load path through the short span of the affected slab panels, when the span ratio is more than 1. This shorter alternative load path will be sturdily cantilevered and provide some resistance to the affected slab segments by the redistributed gravity loads before the failure is initiated due to the insufficient reinforcement length within the slab segments along the short column strip of the affected slab panels. Given this, a delay in failure of RC building of span ratio more than 1 would be expected compared to the other building of span ratio 1, once a key gravity load path is removed.

Overall, the obtained results in this study reveal an urgent need to accelerate the development of standards with specific emphasis on progressive collapse resistance, as there are no specific Canadian guidelines to protect RC building structures against this important threat. Practical guidelines need to be developed for use in design practice in order to reduce the risk of progressive collapse of RC building structures under extreme loadings, as the vast majority of building structures in Canada and, more generally, in North America are not currently being designed for this threat.

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