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Mitigating Progressive Collapse of RC Buildings with Shear Walls and Flat Slab System

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: Recently, due to the increasing number of attacks on embassies, commercial buildings and industrial facilities, considerable attention has been focused on the consequences of blast loads on RC building structures. One of the major consequences of bomb attacks, from the perspective of structural performance, is the possibility of a progressive collapse resulting in significant life and monetary losses, where key element failure is normally the primary cause of such progressive failure in RC building structures. In fact, ground columns of RC buildings are the key load-bearing elements in these structures, as these load paths are the most vulnerable to terrorist attacks. In many attack events worldwide, the damage to the ground columns of RC building structures proliferated into a failure disproportionate to the local damage caused by the initiating event, which accordingly led to partial or total collapse of the attacked structure. This study focuses on the effect of losing one of the ground columns in a 5-storey RC shear wall building with flat slab system satisfying the current NBCC code, where flat slabs are the building's sole defense against propagating collapse. Based on the obtained results, this study introduces technical modifications in the design and reinforcement detailing of RC slab components designed in accordance with the NBCC 2005. These modifications are mainly intended to increase the load-carrying capacity of the slab components that are affected by column loss scenarios by providing alternate load paths to these structural components in case primary load paths are lost.

1 Introduction

Collapse in RC building structures occurs when major structural load carrying members are suddenly damaged or destroyed and the remaining structural elements cannot support the weight of the building, causing these elements to fail. This failure usually occurs in a domino effect and leads to a progressive collapse failure in the structure. The attention of the engineering community has been strongly drawn to this issue by many dramatic events, which resulted in catastrophic failures following explosive devices loaded in vehicles that were detonated remotely or by suicide drivers. The bombing of the Alfred P. Murrah Federal Building in Oklahoma City is a typical example of progressive collapse failure, where the initial bomb blast caused only 10% of the structure's damage, but the resulting progressive collapse lead to 90% of the structure failing. Given this, professionals have been alerted to the particular behaviour of RC buildings when subjected to extreme loading-induced progressive collapse, and at the same time, to the importance of considering the influence of changing the boundary/support conditions of RC building structures to account for any unforeseen abnormal event in the initial design stage.

RC building structures that are vulnerable to extreme loadings include government and public buildings, embassies, financial institutions, and landmark structures of tourist interest. Supplying a sufficient standoff distance from these buildings is a primary design strategy that can usually help in mitigating the effects of extreme loadings, such as close-in explosions, and highly decreases the possibility of progressive collapse occurring. Demonstrating this, Yagob et al. (2009) concluded that standoff distances larger than 15 m from RC building structures would cause moderate to minor damage to the structural members but, generally, would lead to the protection of the building from progressive collapse for detonations of up to 500 kg TNT. However, this approach, which is known as an event control approach, is impractical and may not be possible due to a building's location or other constructional circumstances, where these structures are often located in congested urban environments and a safe stand-off distance is not easy to maintain. Nevertheless, even when the distance can be provided, it still does not eliminate the risk entirely, as the structure should be resistant to progressive collapse resulting from other causes. It is of prime important, in aim of extreme/blast-resistant design, to ensure that RC buildings are not at risk of progressive collapse, thereby permitting the safe and timely exit of all occupants in the event of any extreme loading.

2 Alternate Path Method for Assessing Progressive Collapse

The U.S. General Services Administration (GSA) guidelines are the only available criteria intended to be used primarily for new federal office buildings and major modernization projects. Thus, the recommendations of the GSA guidelines were adopted in this study, which focuses mainly on the progressive collapse of civilian building structures. In the GSA guidelines, the main direct design procedure is the alternate path (AP) method. The AP method aspires to limit the amount of total damage which results from local failure by effectively transferring the gravity loads along alternate load paths. Thus, the structure must be capable of bridging over a missing structural element and, hence, progressive collapse does not initiate in the structural system.

The AP methodology considers the removal of a key element from the structural system due to presumed abnormal loading, where the structure is required to redistribute its gravity loads to the remaining undamaged structural elements. The structure is analyzed to ensure that deflection or stress limits are not exceeded and that progressive collapse does not take place. The main advantage of this approach is that it promotes structural systems with ductility, continuity, and energy absorbing properties that are desirable in averting progressive collapse. In general, this method is attractive not only because the overall structural performance of the damaged structure is considered, but also, a specific abnormal load event needs not be identified.

The AP method was used in this study to investigate the vulnerability of RC shear wall buildings with flat slab system to progressive collapse and, then, to investigate the ability of these RC buildings to mitigate the progressive collapse once their slab components are technically modified in the design and reinforcement detailing. A separate analysis for the instantaneous loss of a single column at the ground-level of the building structure in different plan locations is conducted. It is assumed that only one column is instantaneously removed at a time. The following four scenarios, representing different threat locations caused by bomb explosions, on the ground level, around a typical RC shear wall building structure, are analyzed in this study:

(i) Column at the corner of the building is removed.
(ii) Penultimate column on the perimeter, at the short side of the building, is removed.
(iii) Column on the perimeter, at the middle of the long side of the building, is removed.
(iv) Penultimate interior column of the building is removed.

In these scenarios, it is implicitly assumed through AP method that the initial failure corresponds to a blast or impact loading scenario or any natural disaster resulting in a full column removal. The residual structure must be able to accommodate this loss by developing an alternate load path to redistribute gravity loads. It is essential to mention that in the AP method, the concern is mainly with the vertical displacement or the chord rotation at the removed column location.

3 Analysis Method and Applied Load Combination

As with the GSA (2003) criteria, there are different levels of analysis that can be used to perform the alternate load path analysis for the assessment of the progressive collapse potential of building structures. These include: (i) linear static analysis, (ii) nonlinear static analysis, (iii) linear dynamic analysis and (iv) nonlinear dynamic analysis. As progressive collapse is purely a dynamic and nonlinear event, nonlinear dynamic analysis is seen as the most accurate method in this study. In the GSA guidelines, this analysis method is highly recommended for the assessment of the progressive collapse potential of buildings. 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. The GSA guidelines specify the following loading combination to be applied in the dynamic simulation of progressive collapse for the entire structure for each of the column removal scenarios previously mentioned:

[1] G_{ND} = 1.0 DL+0.25 LL
where G_{ND} = Gravity loads, DL = Dead load, and LL = Live load.

It should be mentioned that, in the GSA guidelines, the use of 3-dimensional models in progressive collapse analysis is highly recommended. This is in order to account for 3-dimensional effects and to avoid overly conservative results. Using 3-dimensional models for progressive collapse analysis, while considering their geometric and material nonlinearity, is of prime importance. In light of this, in this study, 3-dimensional models are used in all cases of column removal mentioned above.

4 Building Description and Design Parameters

A typical RC office building structure designed for Vancouver, BC, was used for the purpose of this study. The building was designed according to the 2005 edition of the National Building Code of Canada (NBCC 2005). This RC shear wall building is classified as low-rise (5-storey). A span ratio of 1 was considered in the design. The building 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). The building has a typical storey height of 3.65 m (Fig. 1(b)). The gravity load resistance system used in this building is a flat slab with drop panels. Drop panels of 90 mm were used with 180 mm thickness flat slabs, but without column capitals. It is essential to mention that for this slab system, the required slab thickness and reinforcing steel are the same at all floor levels and at the roof. It should be mention that all structural components were detailed in accordance with the CSA standard A23.3-04 *design of concrete structures*.

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. A 25 mm concrete clear cover for the slab was used. Each floor slab supports its own dead load plus 2.25 kPa average weight for partitions and finishes, in addition to 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 this slab system, as shown in Fig. 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 RC columns. 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 study.

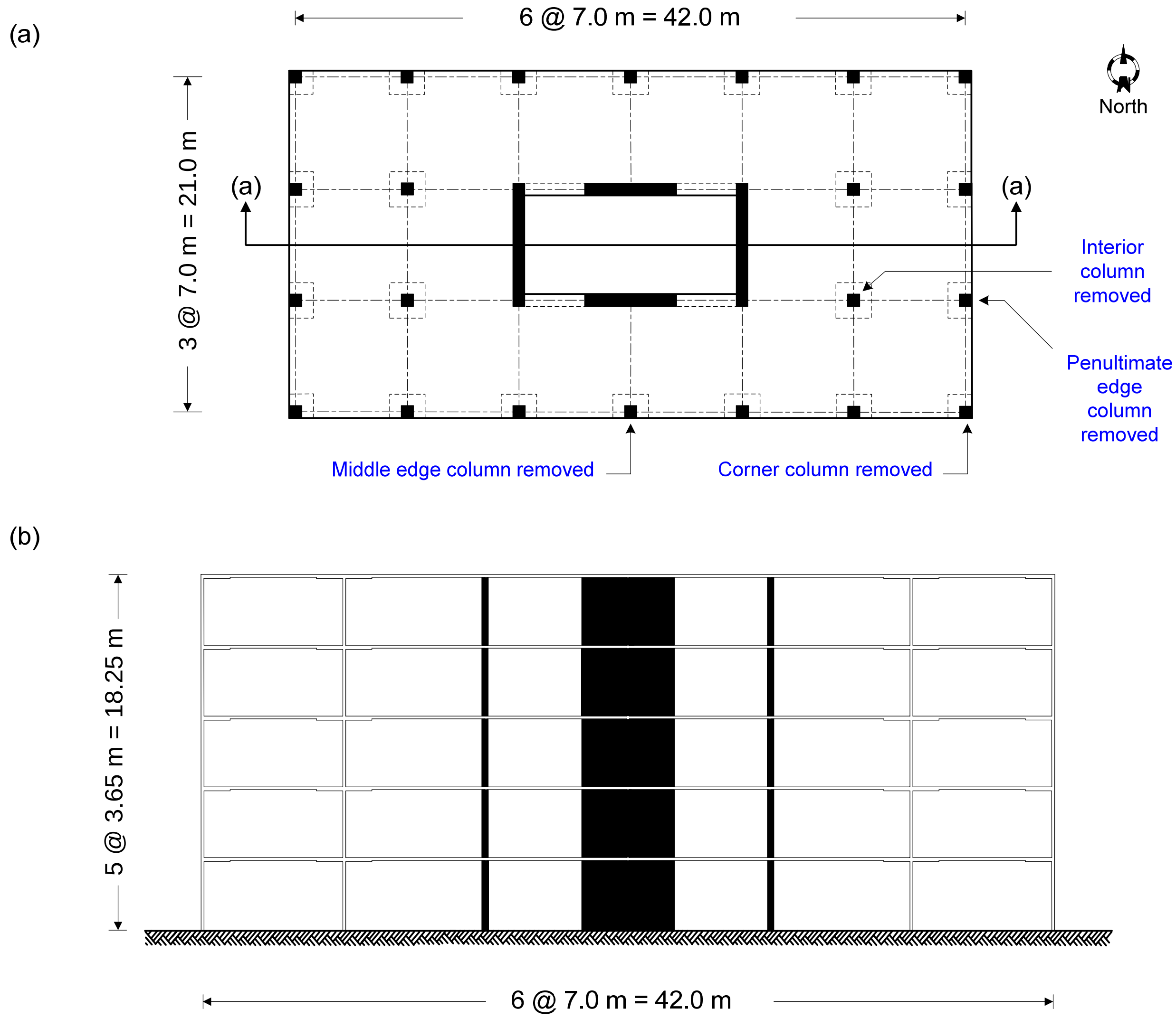

Figure 1: (a) Plan of slab floors and (b) cross section (a-a) of the RC shear wall building.

5 Modeling RC Shear Wall building for Progressive Collapse Analysis

Structural testing and analysis is a basic requirement in every research area. For the sensitivity of progressive collapse analyses in terms of capability and reliability, choosing structural analysis software that can grow with the evolving needs of the present research work is of prime importance. For the purpose of progressive collapse analysis and based on the features of available structural tools, the building structure in this study is modeled and analysed using the nonlinear program Extreme Loading for Structures (ELS®). This program is based on the Applied Element Method (AEM) (Tagel-Din, H. 1998) and, nowadays, one of the most frequently used software in progressive collapse research.

In progressive collapse analysis, the ELS® software is used 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 dynamic forces and displacement caused my membrane action and P-Delta effect, elastic and plastic bending under compressive loads, 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, which is the most unique feature of this software. All consequences of these failures are automatically considered and calculated during the analysis.

The mode of failure or collapse in ELS® software is a direct and visual output of the analysis, where the extent of the expected collapse, if any, due to any applied case of loading will automatically be seen in a visual representation. This includes the initiation of cracking, the deformation shape before the onset of propagation failure, the rigid body motion of debris, the collision between falling debris and other structural components and, finally, the dimensions of the expected collapse area based on the conducted progressive collapse analysis.

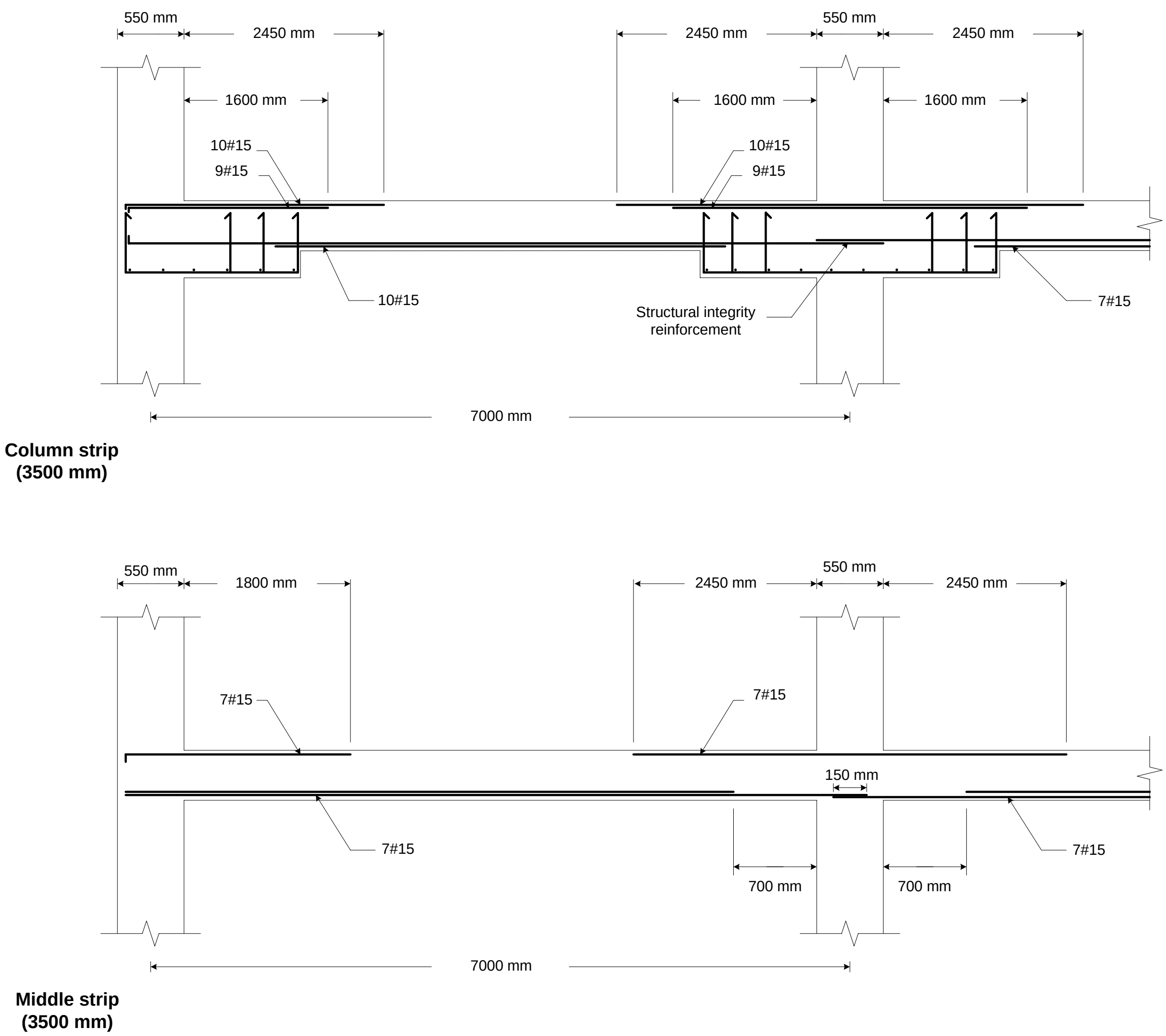

Figure 2: Interior strip of a flat slab floor system designed and detailed for the effects of gravity loads using Canadian Standard A23.3.04 (CSA 2004).

6 Progressive Collapse Analysis and Results

In this study, progressive collapse analysis was conducted on the RC building with shear walls and flat slab described in section 4. The objective of the analysis was to evaluate the potential for progressive collapse of such building structures designed according to NBCC 2005. Using the structural analysis program ELS®, a 3-dimensional model was developed and nonlinear dynamic analyses were conducted, as mentioned in section 2. Columns at the first storey of the building were removed suddenly. The columns were removed in 0.001 seconds. The following four scenarios were investigated: (i) corner column removed, (ii) Penultimate exterior column removed at the short side of the building, (iii) exterior middle column removed at the long side of the building, and (iv) Penultimate interior column removed. Note that only one column at a time was removed, as shown in Fig. 1(a). The load combination as required by GSA guidelines was applied to the entire structure. The loading scenario in this study consisted of two stages, Stage 1 and Stage 2. During Stage 1, the self-weight of the structure was applied over 75 increments. Stage 2 was a dynamic stage during which the element removal takes place. The duration of Stage 2 was 5 seconds, allowing enough time for element removal and complete collapse (if any). 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, i.e. the total number of calculation steps was 50,000. This time step was used in all the analysis cases conducted in this study. The nonlinear dynamic analysis for the RC shear wall building and for all cases of column removal considered in this study shows that the building could not resist progressive collapse in any of the column removal scenarios, as proper placement and detailing of reinforcing steel within the flat plate for such a threat was not considered in the design. It is clear that reinforcement detailing is the key aspects of progressive collapse resistance of RC structures.

6.1 Corner column removed

When a corner column was removed, the corner portion of the building, along its entire height, collapsed. Fig. 3 illustrates the slab failure of the RC building. The first cracking started at the curtailments of the top reinforcement of the neighboring supports, before the capacity of the slab section at these curtailments failed under the redistributed gravity loads, as can be seen in Fig. 4. It is clear that the discontinuity of the top reinforcement, in the middle of the spans of the corner bay, led to the collapse of the entire corner portions of the building structure. This is because the bending moments in the slab (when the corner column is removed) are very different from those used in the design. In this case, the curtailments in the top reinforcement are considered to be the main factors responsible for progressive collapse. It was observed that the structural integrity reinforcement that is required in the slab system for bonding with the corner columns would not help in preventing slab collapse once these supports are damaged or removed.

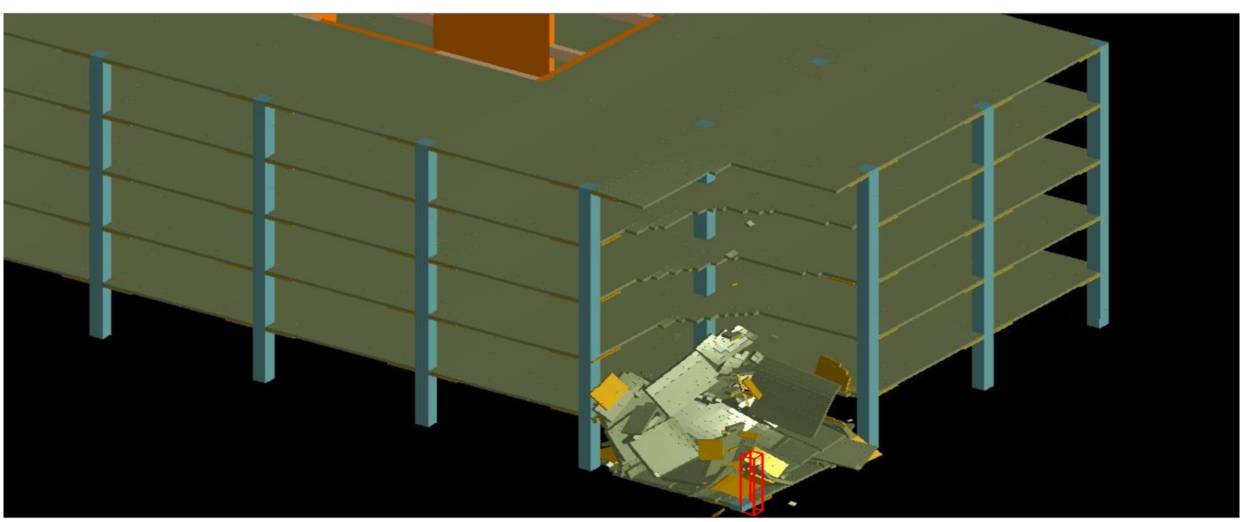

Figure 3: Corner column removed in a 5-storey shear wall building with flat slab system.
(at 5.0 sec of the analysis)

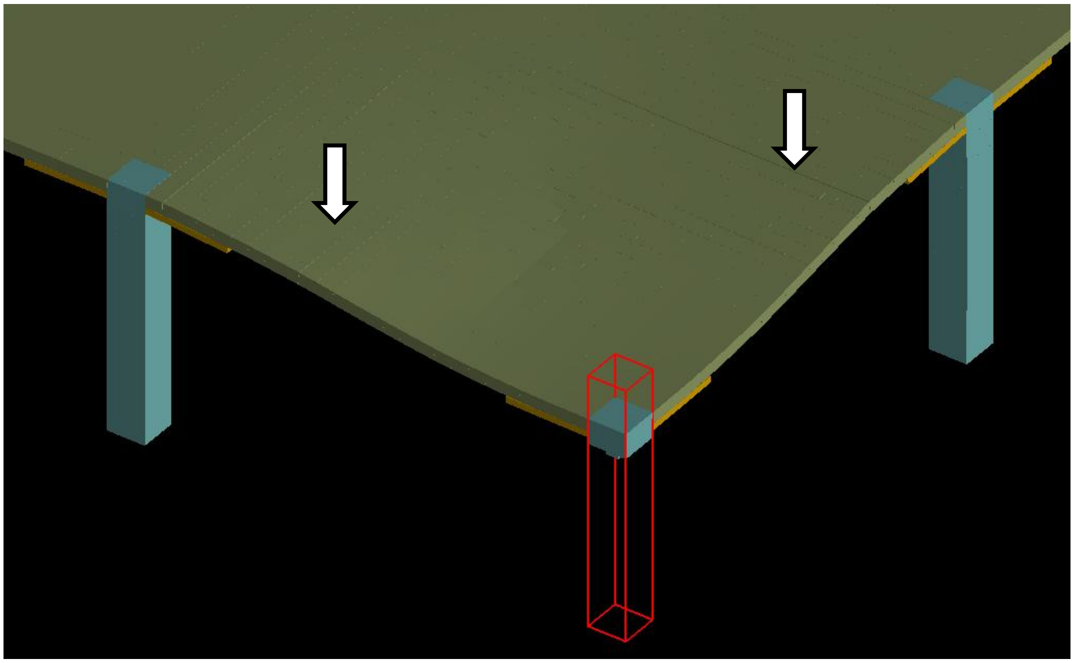

Figure 4: Flat slab failure at the top reinforcement curtailment once corner column is removed. (at 0.75 sec of the analysis)

Considering these observations, analyses were conducted by using continuous top reinforcement in the column strips connecting the edge columns to the corner columns, as illustrated in Fig. 5. The results show that the building could resist the collapse initiated by the removal column, as seen in Fig. 6, which shows the displacement vs. time at the point of the slab where the corner column is removed. Figure 7 shows the strain-stress curves for the top reinforcement (tension). This is at the face of the neighboring edge column. It is seen that the top reinforcement yield, but it does not reach the ultimate strength.

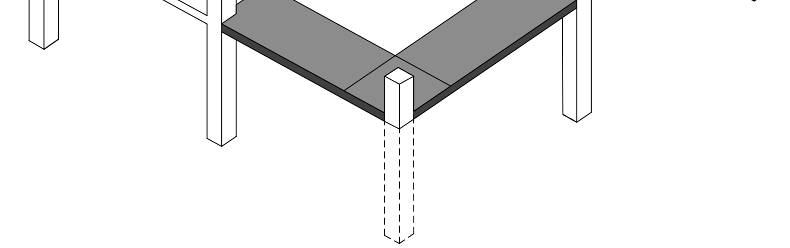

Figure 5: Improvement of the design of floor slabs in the corner column strip regions only, when corner column is removed (continuity of top reinforcement).

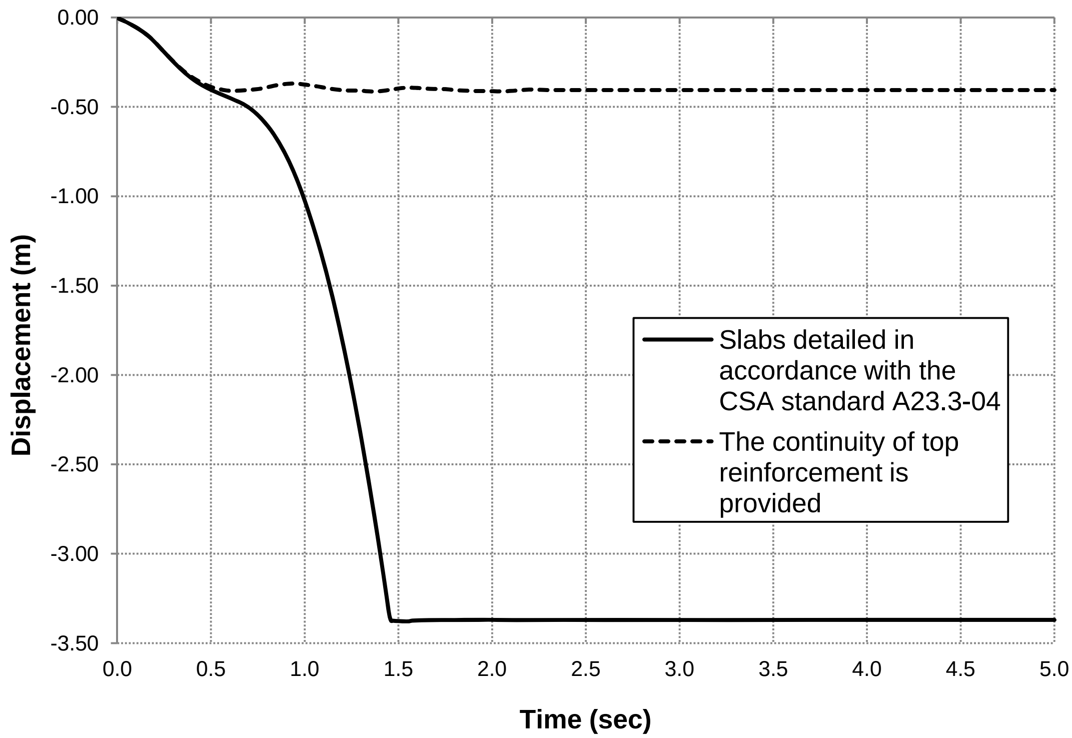

Figure 6: Response of RC shear wall building with flat slab system to corner column removal.

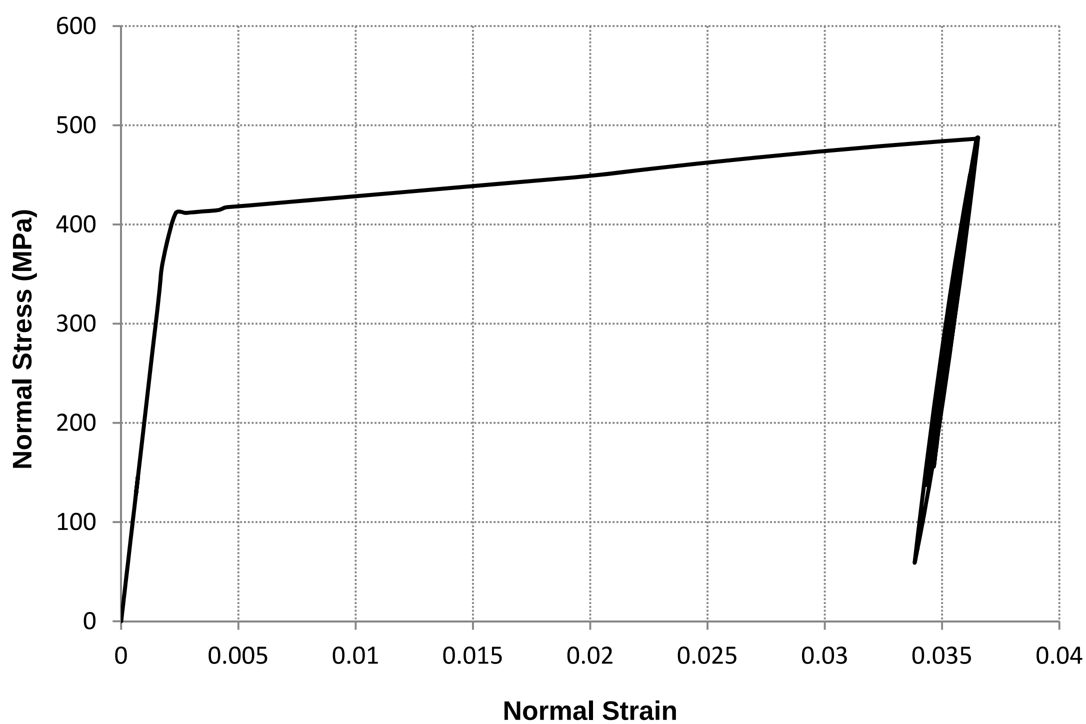

Figure 7: Stress-strain curves in the continuous top reinforcement of RC shear wall building when corner column is removed.

6.2 Edge column removed

When edge columns were removed, either from the short side or the long side of the building, the RC building could not resist the loads and collapsed partially. The exterior two bays along the height of the building, which were supported by the removed columns, completely failed (see Fig. 8). This is mainly due to the redistribution of gravity loads, which the double span could not resist. This caused a failure of the slab at each of the neighboring supports along the height of the building. None of the neighboring supports has any contribution to preventing the collapse of the exterior slab panels.

While providing the continuity of the top reinforcement in the middle of the spans in the interior strip that connects the removed columns to the interior columns might increase the resistance, it is certain that the failure caused by the insufficient reinforcement passing through the removed column area, i.e. column strips, at the bottom of the slab level, where about 50% of the reinforcement is not continuous. Moreover, the overlap length of the U-shaped bottom reinforcement of the drop panel was not sufficient in transferring the generated forces to the bottom reinforcement in the column strip of the slab. On the other hand, the non proper length of top reinforcement within the column strips connecting the removed column to the neighboring columns, is an addition reason for the failure, as the negative moments are over wider ranges compared with those used in the design.

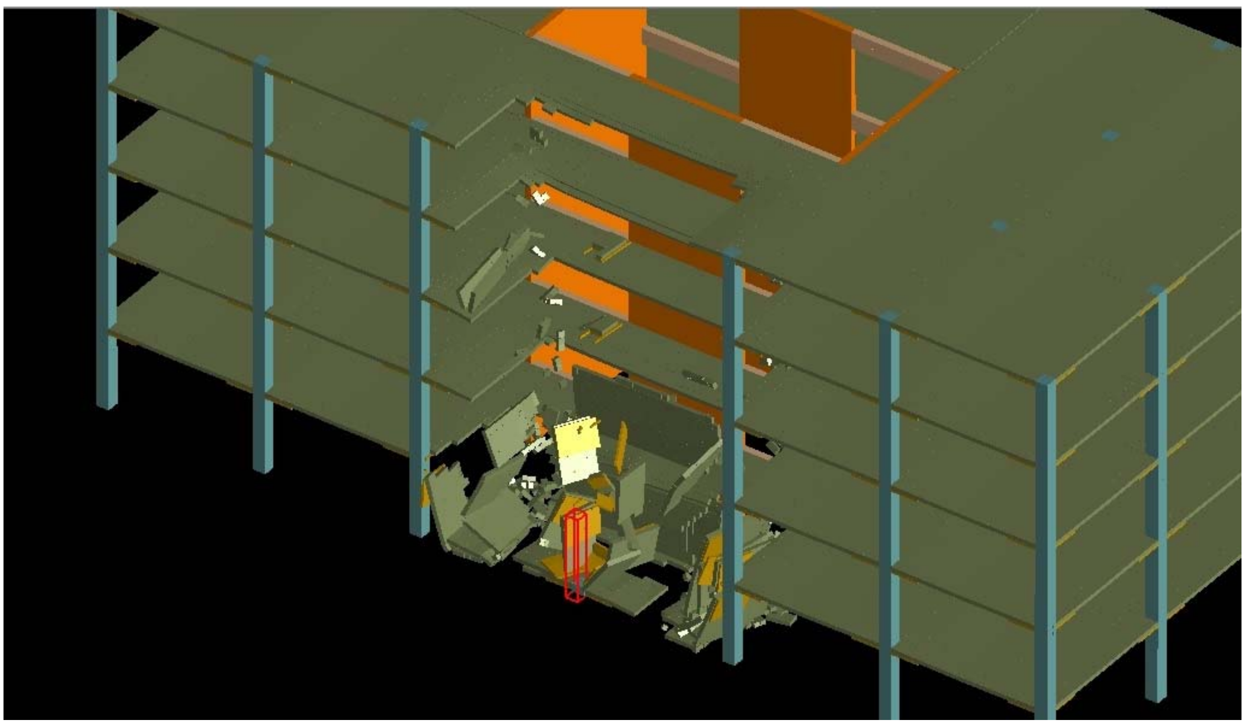

Figure 8: Edge column at the long side of the building removed.
(at 5.0 sec of the analysis)

Based on the foregoing discussion, continuity of all bottom reinforcement of the outer column strips was assumed in the subsequent analyses, as illustrated in Fig. 9. In addition, within the same column strips, continuity of the top reinforcement was assumed to cover the shifted moment diagram among each column strip in each floor level. Then, each building was analyzed under the same loading and other conditions. The results show that the building could resist the collapse initiated by the column removal, as seen in Fig 10.

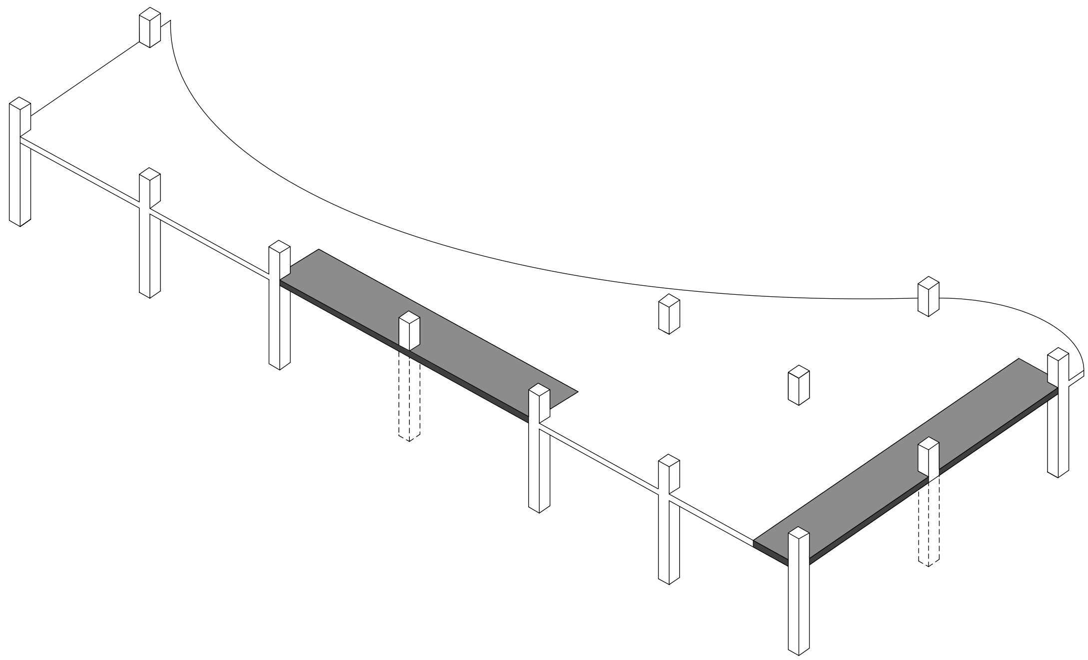

Figure 9: Improvement of the design of floor slabs in the outer column strip regions only, when edge column is removed (provided continuous bottom and top reinforcement).

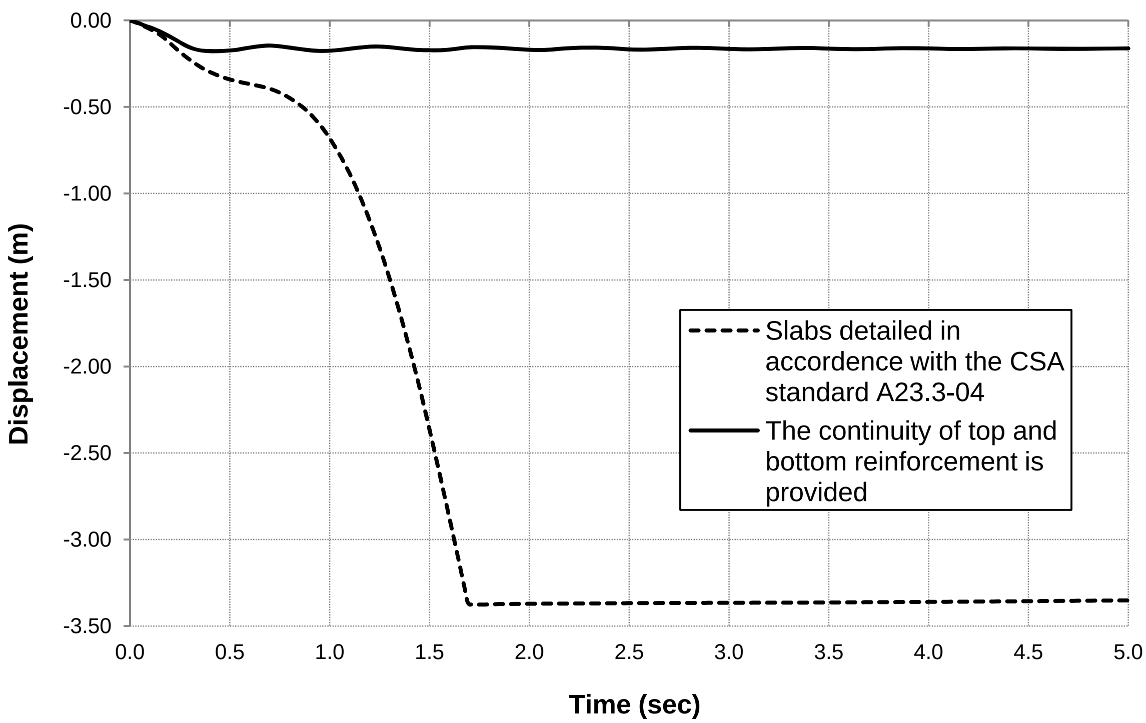

Figure 10: Response of RC shear wall buildings with flat slab system to edge column removal. (at the long side of the building)

6.3 Interior column removed

In the fourth scenario, where an interior column was removed, although the bottom slab was provided with the required structural integrity reinforcement, the buildings' slabs could not sustain the double span in each direction, and progressive collapse occurred, as seen in Fig. 11. This is because the moments in the slab sections were significantly larger than the capacity of the slab (approximately 4 times larger due to doubling of the span upon the loss of the interior column). The positive moment capacity of the slab sections connecting the four slab panels that are sharing the removed column was almost limited to the required structural integrity reinforcement over the damaged column. This is because about 50% of the required bottom reinforcement in both the column and the middle strips is not continuous, and 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 such reinforcement only, where the way the bottom reinforcement of the drop panel is detailed did not deem it effective in transferring the generated forces to bottom slab reinforcement, in addition to the fact that 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 through the building. It is clear that, in this scenario, the slab lacks the ability to resist the moment reversal occurring over the removed column.

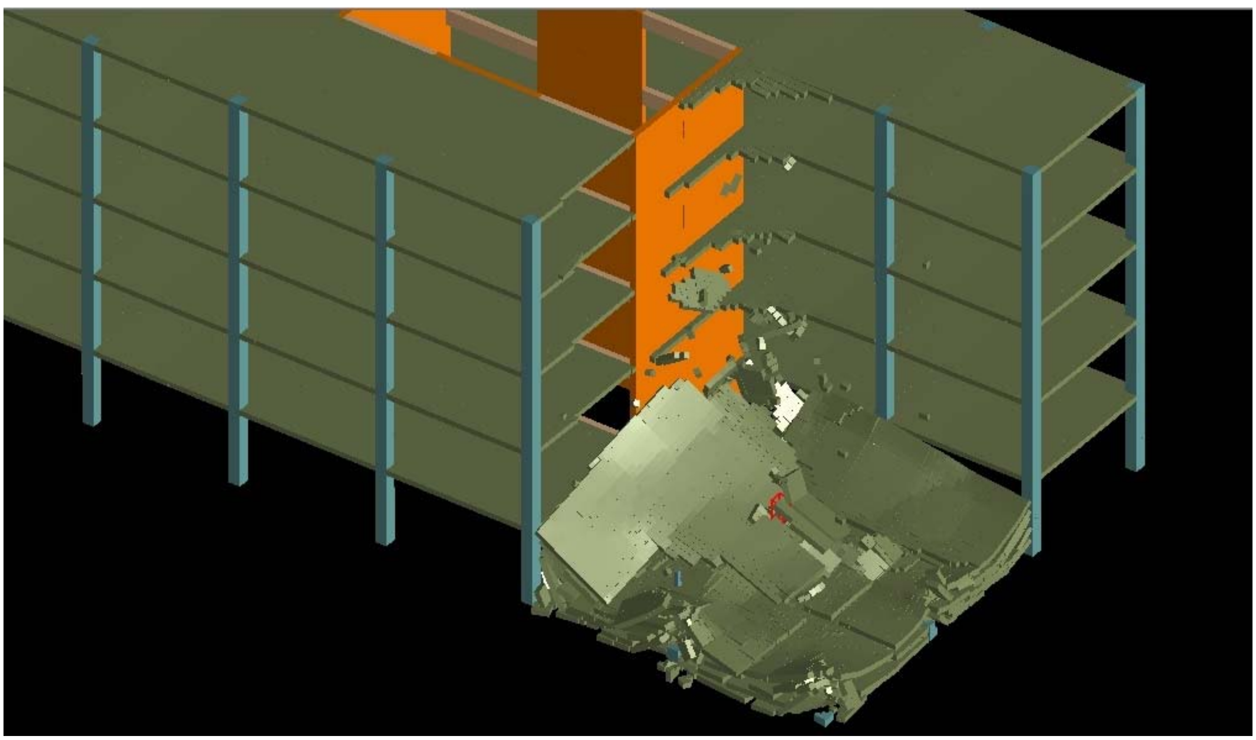

Figure 11: Penultimate interior column of the building removed.
(at 5.0 sec of the analysis)

Considering these observations, analyses were conducted by assuming continuity of all bottom reinforcement in the interior column strips at all floors, as shown in Fig. 12. In addition, within the same column strips, continuity of bottom reinforcement was assumed in the analysis, in order to cover for the shifted moment diagram among each column strip in each floor level. Then, the building was analyzed under the same loading and other conditions. The results show that all building could resist collapse when interior column is removed, as seen in Fig. 13.

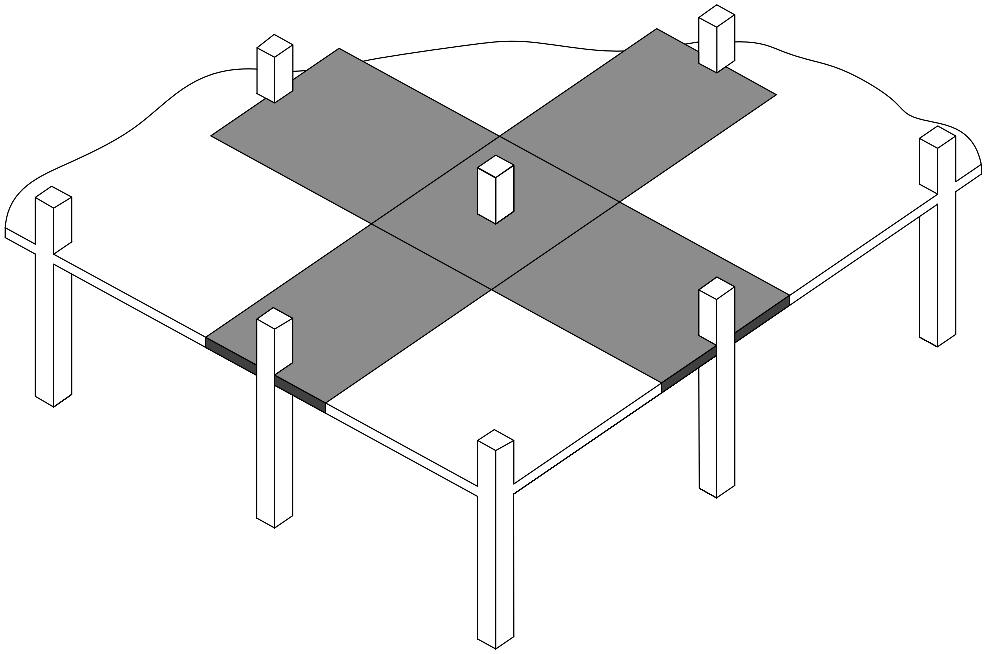

Figure 12: Improvement of the design of floor slabs in the interior column strip regions only, when interior column is removed (provided continuous bottom and top reinforcement).

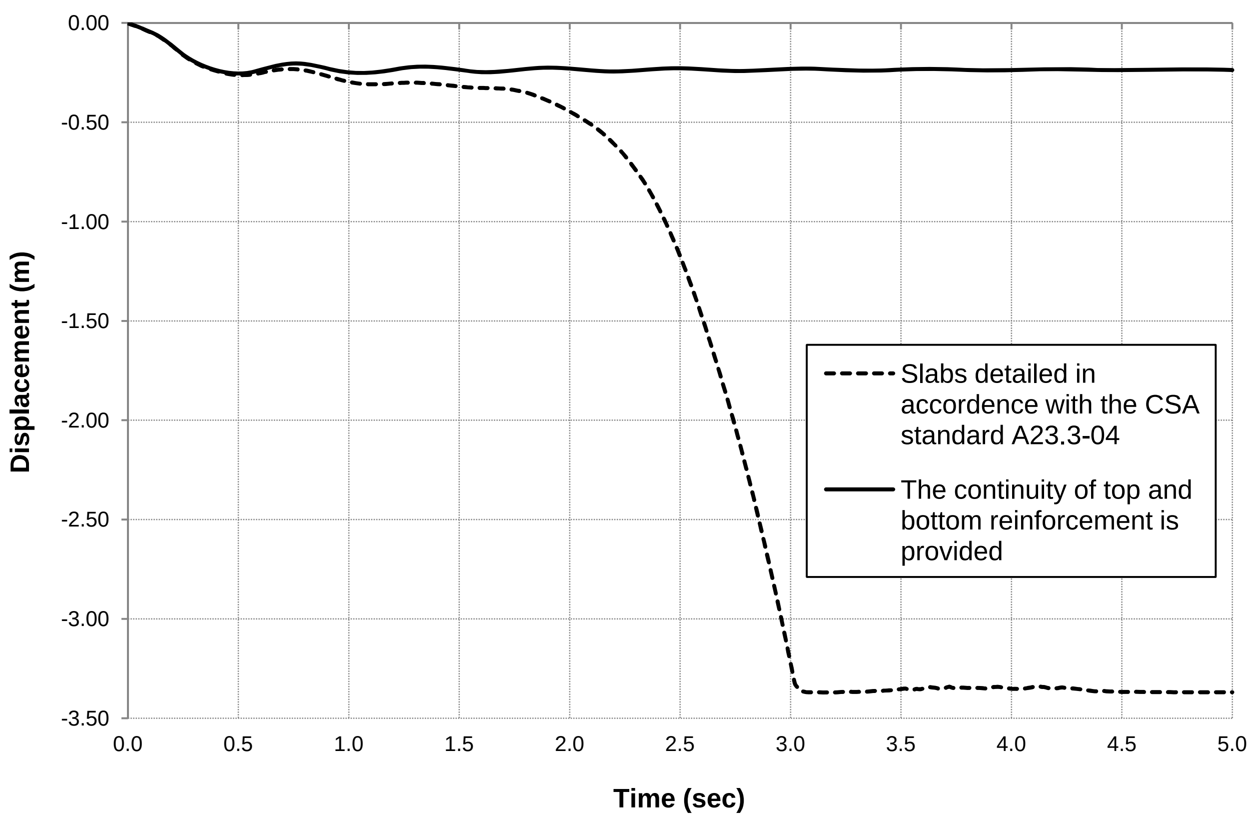

Figure 13: Response of RC shear wall buildings with flat slab system to interior column removal.

7 Summery and Conclusions

The steel reinforcement arrangement is crucial for the integrity of RC building structures exposed to conditions not considered in their initial design. As discussed, detailing of the steel reinforcement within the slab segments along their orthogonal design strips play a major role in the sole response of these structural components to their gravity load support damage or failure. According to the FEMA (Corley et al. 1996), seismic detailing of structural members may improve the structural system's ability to reduce the likelihood of subsequent progressive collapse caused by blast loadings in RC buildings. Yet, this can be only achieved if the steel reinforcement arrangement is applicable for the regenerated forces, bending moment and shear forces, within the structural member sections subjected to different load conditions during the local failure event. By analogy, flat slabs designed without moment redistribution and with bottom and top steel curtailed according to CSA A23.3-04 behave very poorly under an extreme event characterized by the removal of a column. Once the reinforcement detailing of these slabs is modified to cover generated forces caused by column loss scenarios, it is found that flat slabs are capable of bridging the removed columns thus mitigating its progressive collapse.

The main findings from the analyses described in this study can be summarized as follows:

- Shear wall buildings with flat slabs, designed according to NBCC 2005 would collapse when first storey column loses its resistance due to blast loads, accident or any other reason.
- The buildings can resist the loads (i.e. no collapse would occur) in the case of a removal of a first storey column if continuity of the reinforcement is provided in the design within the column strips (i.e. continuity of top reinforcement when corner column is removed – see Fig. 5, and continuity of both the top and the bottom reinforcement when edge or interior column is removed – Figs. 9 and 12 respectively).

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