Assessment of the Vulnerability of Buildings to Progressive Collapse due to Blast Loads

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Abstract: Progressive collapse analyses were conducted on reinforced concrete frame buildings designed according to the seismic provisions of the 2005 edition of the National Building Code of Canada. The purpose of the analyses was to estimate the vulnerability of the buildings to progressive collapse. Three buildings designed for Ottawa and three buildings designed for Vancouver, with heights of 5 storeys, 10 storeys, and 15 storeys were used in this study. The longitudinal and the transverse frames of the Ottawa buildings have different spans and were designed as moderately ductile frames, while those of the Vancouver buildings have equal spans and were designed as ductile frames. The performance of the buildings against progressive collapse was evaluated following the widely used guidelines for progressive collapse analysis and design, prepared by the U.S. General Services Administration. The Ottawa buildings were found to be more vulnerable to progressive collapse than the Vancouver buildings. The results from this study show that the progressive collapse vulnerability of seismically designed regular buildings depends greatly on the differences between the spans of the longitudinal and the transverse frames, i.e., larger differences between the spans lead to higher vulnerability.

1. Introduction

According to the guidelines for progressive collapse analysis and design of the U.S. General Services Administration (GSA 2003), progressive collapse of a structure is defined as "a situation where a local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause". There are many cases of progressive collapse of buildings that occurred in the past due to different reasons. The first case that triggered special attention in the engineering community was the progressive collapse of a part of the 22-storey Ronan Point apartment building in London, England in 1968 (Shankar 2004). A gas explosion at the 18th floor of the building triggered the collapse of the corner slabs at the upper floors (above the 18th floor) that was followed by the collapse of all corner slabs of the building. A typical example of progressive collapse is the collapse of the Alfred P. Murrah Federal Building in Oklahoma City, in 1995, due to a bomb explosion at the ground level. Three columns at the first storey were badly damaged and that caused total collapse of almost half of the building. The most dramatic case was the progressive collapse of the twin towers of the World Trade Center that occurred on Sept. 11, 2001, when airplanes crashed into each tower. This type of collapse, which was due to an extraordinary impact, is almost impossible to prevent and it is beyond the considerations of most common cases of progressive collapse caused by bomb explosions or vehicle impacts.

Since the progressive collapse of the Ronan Point apartment building in 1968, many codes and standards have attempted to consider the issue of this type of collapse. However, most building codes address the progressive collapse through implicit and very general requirements for the integrity of the structure without more explicit provisions on progressive collapse analysis and design (e.g. NBCC 2005; ASCE 2005).

The most comprehensive documents related to progressive collapse are the guidelines of the U.S. General Services Administration (GSA) entitled *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects* (GSA 2003) and those of the U.S. Department of Defense (DoD) entitled *Design of Buildings to Resist Progressive Collapse* (DoD 2005). The GSA guidelines are intended for governmental buildings and the DoD guidelines are for military buildings. Both documents are for specific threats. The requirements of the GSA guidelines are focused on the prevention of progressive collapse due to instantaneous loss of first storey columns/walls as a result of bomb blast or vehicle impact, while those of the DoD guidelines consider loss of columns/walls at any storey of the structure due to terrorist acts with ballistic missiles. Both guidelines provide detailed information for column/wall removal scenarios, loading, analysis methods, and criteria for the assessment of the progressive collapse potential.

In general, it is assumed that buildings designed to resist seismic actions have good robustness against progressive collapse. However, no detailed investigations have been conducted so far to assess this robustness. The objective of this study is to evaluate the performance of seismic resistant frame buildings against progressive collapse. Buildings designed according to the seismic provision of NBCC 2005 were used in the study. Progressive collapse analyses were conducted in accordance with the GSA guidelines. Detailed discussions of the design of the buildings, the progressive collapse analyses, and the results from the analyses are given in the following sections.

2. Description of Buildings and Design Parameters

The buildings considered in this study were designed for Ottawa and Vancouver, which are in regions with moderate and high seismicity respectively. In each location, three buildings were designed with heights of 5 storeys, 10 storeys, and 15 storeys, which are considered representative of low-rise, medium-rise, and high-rise buildings respectively.

The plans of the Ottawa and the Vancouver buildings are shown in Figure 1. The dimensions of the plans are the same, i.e., 30 m x 18 m. The spans in the transverse direction of the Ottawa and the Vancouver buildings are also the same, i.e., three spans of 6.0 m each. However, the number and the dimensions of the spans in the longitudinal directions of the buildings are different. The Ottawa buildings have four spans of 7.5 m each, and the Vancouver buildings have five spans of 6.0 m each. The storey heights for all buildings are 4.0 m (Figure 2).

The lateral load resisting systems of the buildings consist of reinforced concrete moment resisting frames in both the longitudinal and the transverse directions of the buildings. There are four frames in the longitudinal direction of both the Ottawa and the Vancouver buildings (these are designated Li – interior frames, and Le – exterior frames in Figure 1). In the transverse direction, the Ottawa buildings have 5 frames, and the Vancouver buildings have 6 frames. The additional frame in the Vancouver buildings is introduced in order to keep the interstorey drift within the limit value of 2.5% prescribed by the code.

The buildings were designed by Cimilli (2008) in accordance with the seismic provisions of the 2005 edition of the National Building Code of Canada (NBCC) (NRCC 2005). The Ottawa buildings were designed as *moderately ductile* frame buildings, and those in Vancouver were designed as *ductile* frame buildings. Ductile buildings were not considered for Ottawa since the seismic forces for such buildings in locations with moderate seismicity (such as Ottawa) are quite small and the seismic design is governed by the code requirements for minimum structural member dimensions and reinforcement amount. Similarly, moderately ductile buildings for Vancouver (with plan configurations as shown in Figure 1(b)) were not considered since the seismic forces for Vancouver would require very large structural members in order to keep the interstorey drift below the code limit of 2.5% (i.e., frame structural system is not practical in such a case).

The buildings were designed for office occupancy. The foundations were assumed to be on soil class C, as specified in NBCC 2005. Dead loads of 5 kPa for the floors, and 3.5 kPa for the roofs were used in the design. Live loads of 2.4 kPa for the floors, and 2.2 kPa for the roofs were used. The seismic design forces were computed using a ductility-related force modification factor (R_d) and an overstrength-related force modification factor (R_o) of R_d=2.5 and R_o=1.4 for the Ottawa (i.e., the moderately ductile) buildings, and R_d=4.0 and R_o=1.7 for the Vancouver (i.e., the ductile) buildings. Compressive strength of concrete f_c'=40 MPa, and yield strength of reinforcement f_y=400 MPa were used in the design. More detailed explanations of the design of the buildings, sizes of structural members, and reinforcement can be found in Cimilli (2008).

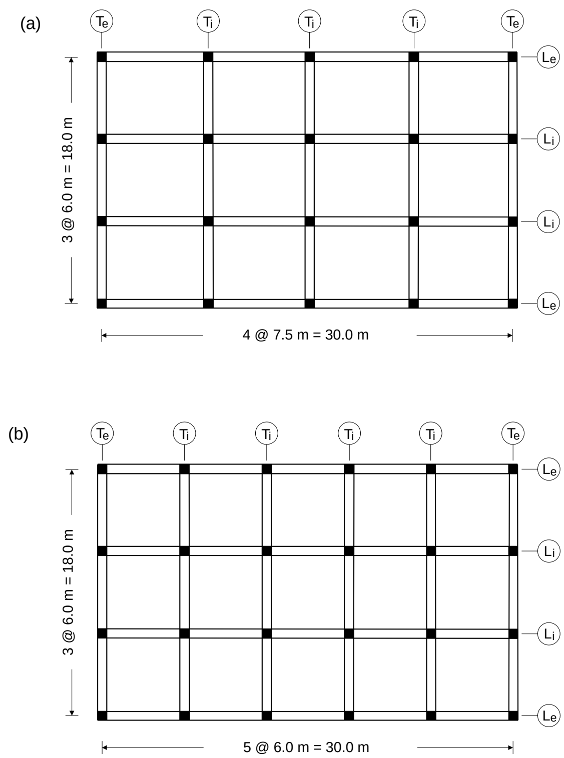

Figure 1: Floor plans of the buildings: (a) buildings in Ottawa, (b) buildings in Vancouver.

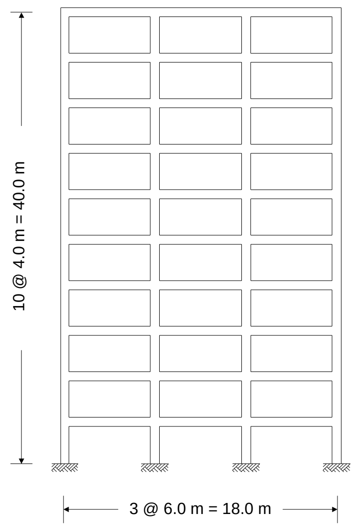

Figure 2: Elevation view of the transverse frame of the 10-storey building.

3. Analysis Considerations

As mentioned earlier, the analyses in this study were conducted in accordance with the GSA guidelines. The guidelines require considerations of scenario cases for analysis, method for analysis, and acceptance criteria for the progressive collapse evaluation. These considerations are briefly discussed in this section.

3.1 Scenario Cases for Progressive Collapse Analysis

For moment resisting frame systems as used in this study, GSA requires that separate analyses be conducted for instantaneous loss of a column at the first storey due to bomb blast or vehicle impact. The following four cases are required to be considered for the removal of columns:

Case 1: Column on the perimeter, approximately at the middle of the long side of the building, should be removed.
Case 2: Column on the perimeter, approximately at the middle of the short side of the building, should be removed.
Case 3: Column at the corner of the building should be removed, and
Case 4: Interior column should be removed.

Figure 3 illustrates the four cases for column removal used in this study.

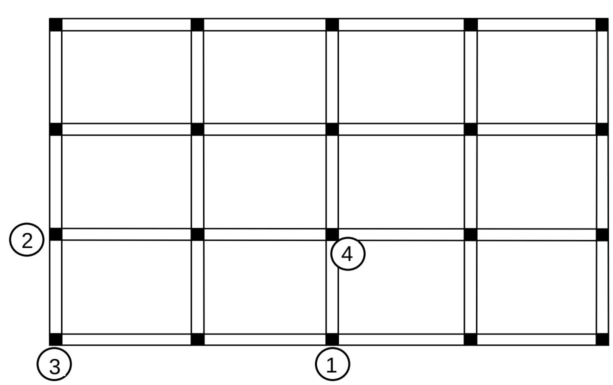

Figure 3: Scenario cases of column removals considered in the progressive collapse analysis.

3.2 Analysis Methods

According to the GSA guidelines, three types of analyses can be used in the assessment of the potential for progressive collapse of buildings, i.e., linear-elastic static analysis, nonlinear static analysis, and nonlinear dynamic analysis. The GSA guidelines prefer the use of linear elastic static analysis. Research has shown that the elastic static analysis is quite appropriate for the assessment of the progressive collapse potential, and the findings from the analysis of regular buildings are consistent with those from the nonlinear static and nonlinear dynamic analyses (Applied Research Associates, Inc., 2009; personal communication).

The guidelines prefer the use of 3-D models in the analysis in order to account for 3-D effects and to avoid conservatism in the results. However, 2-D models are also allowed provided that the general response and the 3-D effects are adequately accounted for.

3.3 Acceptance Criteria

Acceptance criteria are specified in the GSA guidelines for the case when elastic static analysis is used. The criteria are expressed in terms of Demand/Capacity Ratios (DCR), which are defined as

[1] $DCR = D/C$

where,

D = demand (i.e. moment, axial force, or shear force acting on the member) resulting from the elastic static analysis, and

C = capacity of the member (i.e. moment, axial force, or shear force that the member can resist).

The allowable DCR values for the structural members are $DCR \leq 2.0$ for regular buildings, and $DCR \leq 1.5$ for irregular buildings. According to the GSA guidelines, demand/capacity ratios larger than the foregoing values indicate that the building has a high potential for progressive collapse. Since the buildings considered in this study are regular in plan and elevation, the criterion for regular configurations (i.e., $DCR \leq 2.0$) was used in the evaluation of the vulnerability of the buildings to progressive collapse. It is important to note that the benchmark value DCR=2.0 should be considered as an approximate value. This is because there are uncertainties in the modeling of the structures, the analysis, the loading, and the computation of the capacities of the structural members.

The GSA guidelines allow the use of increased material strengths in the calculation of the capacities of the structural members. A strength increase factor of 1.25 is specified in the guidelines for both the compressive strength of concrete and tensile strength of reinforcement (Section 4.1.2.5. in the GSA guidelines).

4. Progressive Collapse Analysis

For the purpose of the analysis, 3-D models were developed for the buildings using the structural analysis program SAP 2000. Columns were removed at the first storey, as shown by the four cases in Figure 3. These cases are referred to as the "analysis cases" in the further discussion. As required by the GSA guidelines, a load of 2DL+0.5LL (where DL represents dead load, and LL represents live load) was applied to the entire areas of all floors of the structural models.

Elastic static analysis was conducted for each of the four cases of removed columns of the buildings using the SAP program. Demands from the applied loads were computed for the beams and columns of the buildings. These included moments, axial forces, and shear forces.

Moment capacities, axial load capacities, and shear capacities were computed for the structural members of the buildings based on the dimensions and the reinforcement of the members. Increased strengths of the concrete and reinforcement were used in the computation of the capacities. The *nominal* strengths of both the concrete and the reinforcing steel were increased by a factor of 1.25, as allowed by the GSA guidelines.

Demand/capacity ratios (DCR) were computed using the demands and the capacities for moments, axial loads, and shear. It was found that the DCR values for axial loads and shear were all below 1.0, indicating that the axial and the shear deformations were not critical for the assessment of the progressive collapse potential for the buildings considered. Given this, only the moment (i.e., the flexural) DCR results for the beams are discussed below in this paper.

5. Discussion of Results

The maximum moment DCR values for the longitudinal and transverse frames of all buildings and for all analysis cases (i.e., cases of column removals) are listed in Tables 1 to 3. The tables also include the number of beams with DCR values equal or exceeding the benchmark value of 2.0 specified by the GSA guidelines (shown in brackets in the tables). In addition, the maximum vertical displacements at the beam-column joints above the columns removed, are listed in the tables. For easier visualisation of the results, the maximum DCR values for all buildings and all analysis cases are shown in Figure 4. The horizontal dashed line at DCR=2.0 shown in the figure represents the GSA benchmark level, and values above this level indicate a high potential for progressive collapse.

Table 1: Maximum demand/capacity ratios (DCR) and vertical displacements for the 5-storey buildings.

Analysis Case	Demand/capacity ratios				Vertical displacement	
	Ottawa		Vancouver		Ottawa	Vancouver
	Long.	Trans.	Long.	Trans.	(cm)	(cm)
Case 1	2.08 (4)*	2.26 (3)	1.57 ---	1.34 ---	10.1	4.2
Case 2	1.43 ---	2.85 (5)	1.40 ---	1.40 ---	7.8	4.4
Case 3	1.06 ---	1.97 ---	1.00 ---	0.93 ---	7.9	3.9
Case 4	1.94 ---	4.39 (5)	1.99 ---	1.82 ---	12.6	5.8

Table 2: Maximum demand/capacity ratios (DCR) and vertical displacements for the 10-storey buildings.

Analysis Case	Demand/capacity ratios				Vertical displacement	
	Ottawa		Vancouver		Ottawa	Vancouver
	Long.	Trans.	Long.	Trans.	(cm)	(cm)
Case 1	2.59 (5)*	2.88 (5)	1.77 ---	1.43 ---	11.9	4.1
Case 2	1.61 ---	3.49 (10)	1.44 ---	1.68 ---	9.3	4.2
Case 3	1.34 ---	2.36 (2)	1.15 ---	1.06 ---	9.6	3.6
Case 4	2.43 (2)	5.28 (10)	2.21 (2)	2.11 (2)	14.3	5.7

Table 3: Maximum demand/capacity ratios (DCR) and vertical displacements for the 15-storey buildings.

Analysis Case	Demand/capacity ratios				Vertical displacement	
	Ottawa		Vancouver		Ottawa	Vancouver
	Long.	Trans.	Long.	Trans.	(cm)	(cm)
Case 1	2.9 (6)*	3.05 (6)	2.01 (1)	1.56 ---	12.2	4.3
Case 2	1.75 ---	4.02 (15)	1.44 ---	1.84 ---	10.2	3.9
Case 3	1.57 ---	2.67 (5)	1.09 ---	1.27 ---	10.2	3.3
Case 4	2.63 (3)	5.81 (15)	2.5 (4)	2.38 (3)	14.3	5.7

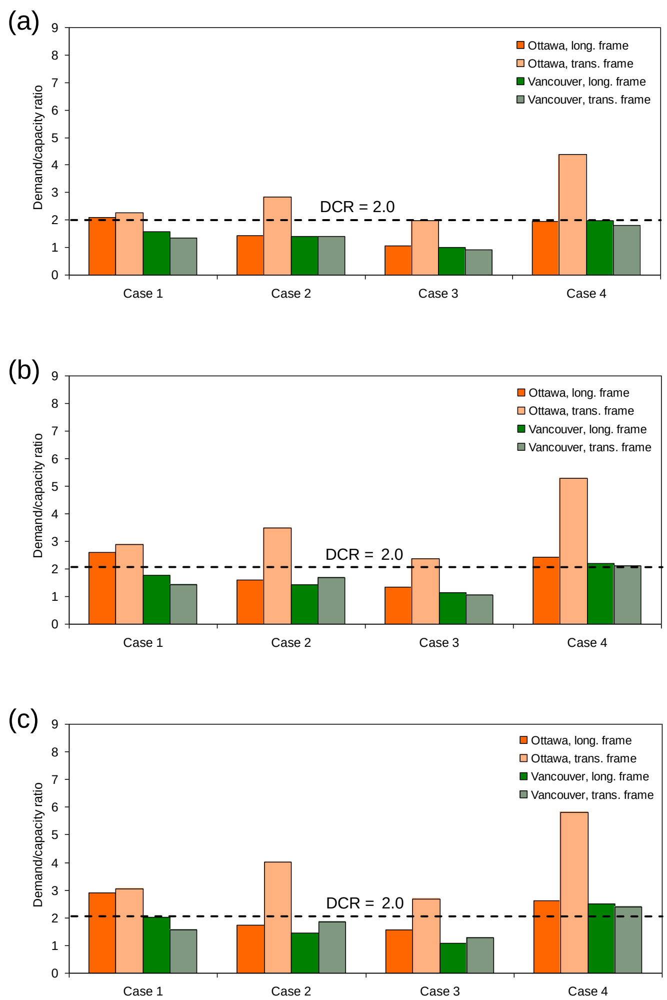

Figure 4: Demand/capacity ratios for the four cases of column removals for the Ottawa and Vancouver buildings: (a) 5-storey buildings, (b) 10-storey buildings, (c) 15-storey buildings.

The main observations from Tables 1 to 3 and Figure 4 are as follows:

- The Ottawa buildings are more vulnerable to progressive collapse than the Vancouver buildings. For all analyses cases, the maximum DCR values for the Ottawa buildings are larger than those for the Vancouver buildings by factors of 1.4 to 2.4. Also, the vertical displacements of the Ottawa buildings are larger than those of the Vancouver buildings by factors of 1.8 to 3.1; note that these displacements are at the beam-column joints where the columns are removed.
- The Ottawa buildings have high potential for progressive collapse for all analysis cases (i.e., they have DCR>2.0), with the exception of the 5-storey building for Case 3 (corner column removed), for which the maximum DCR value is slightly lower than 2.0.
- The potential for progressive collapse for the Vancouver buildings is low for all analysis cases (i.e., they have DCR<2.0), with the exception of the 15-storey building, Case 4 (interior column removed) for which DCR=2.5.
- The DCR values increase with the increase of the height of the buildings, which indicates that taller buildings are more at risk to progressive collapse than shorter buildings.
- Among the four analysis cases, Case 4 (interior column removed) is the most critical for progressive collapse. The DCR values for this case are larger than those for the other cases of column removals.
- Analysis Case 3 (corner column removed) is the least critical for progressive collapse. The maximum DCR values for this case for all buildings are smaller than 2.0, with the exception of the 10-storey and 15-storey Ottawa buildings for which the DCR values are 2.36 and 2.67 respectively.

An important observation from Figure 4 and Tables 1 to 3 is that the vulnerability of the Ottawa buildings is primarily associated with the transverse frames, i.e., the DCR values for the transverse frames are larger than those for the longitudinal frames. This is mainly due to the differences in the spans of the transverse and the longitudinal frames (i.e., 6.0 m vs. 7.5 m, as shown in Figure 3). Namely, the beams of the transverse frames, which are shorter and therefore stiffer, attract larger moment demands due to vertical loads in the vicinity of the column removed than the beams of the longitudinal frames which are longer and more flexible. On the other hand, the spans of the transverse and the longitudinal frames of the Vancouver buildings are equal, and consequently, the maximum DCR values for the frames are very close.

For illustration, Figures 5 to 8 show the computed DCR values for the 10-storey buildings for the analysis cases 1 and 4. The values shown in the figures are only for the spans adjacent to the removed columns. The DCR values for the other parts of the buildings are all below 2.0 and are not shown in the figures. The values above the beams correspond to negative moment demands and capacities, and those below the beams correspond to positive moment demands and capacities. For easier identification of the critical sections of the frames, the DCR values larger than 2.0 are shown in "red" in the figures.

(a)

(b)

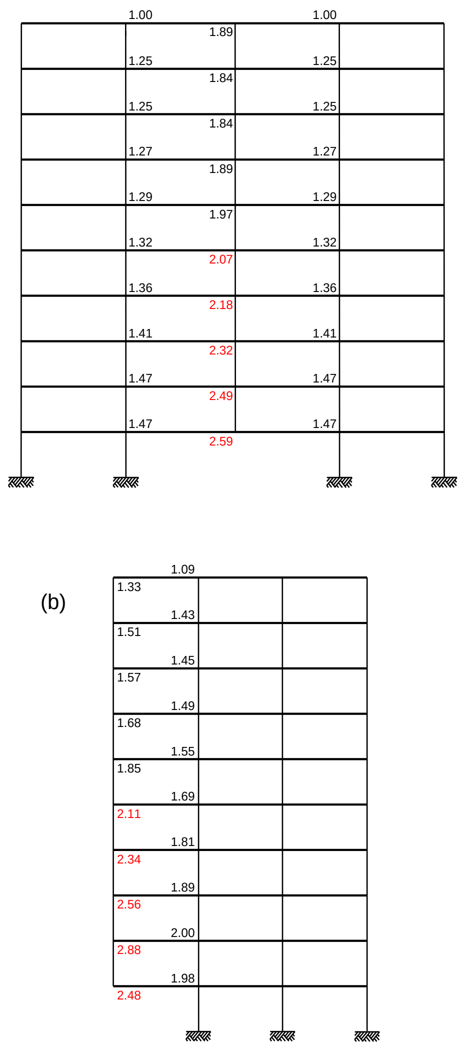

Figure 5: Demand/capacity ratios for the 10-storey Ottawa building for Case 1: (a) exterior longitudinal frame, (b) interior transverse frame.

(a)

(b)

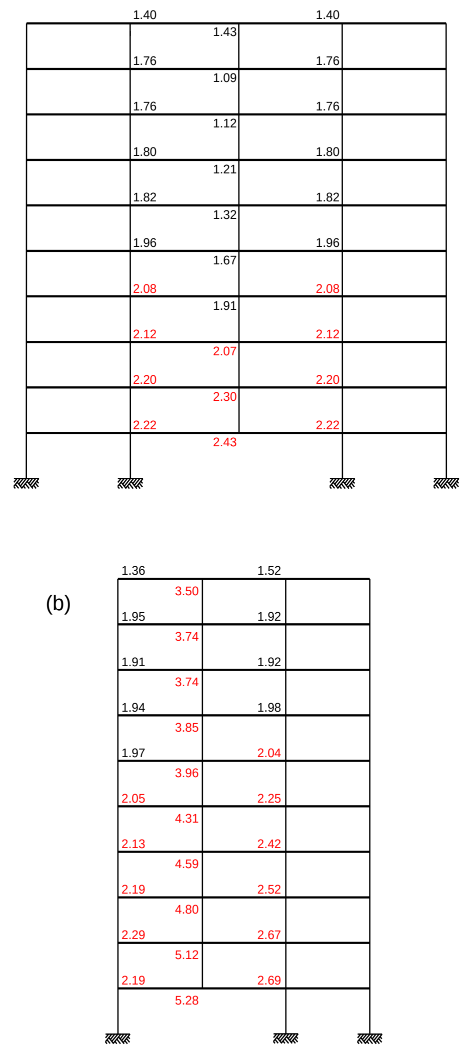

Figure 6: Demand/capacity ratios for the 10-storey Ottawa building for Case 4: (a) exterior longitudinal frame, (b) interior transverse frame.

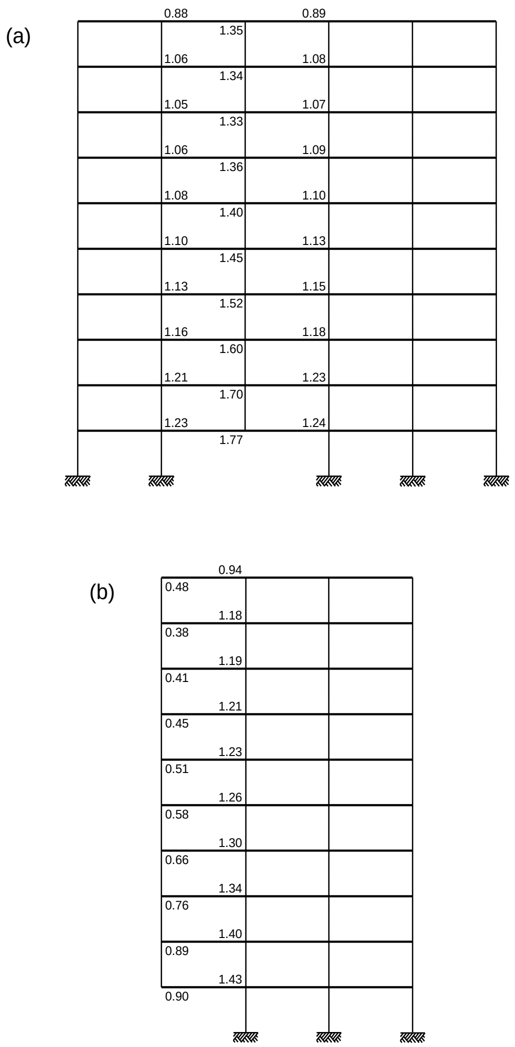

Figure 7: Demand/capacity ratios for the 10-storey Vancouver building for Case 1:
(a) exterior longitudinal frame,
(b) interior transverse frame.

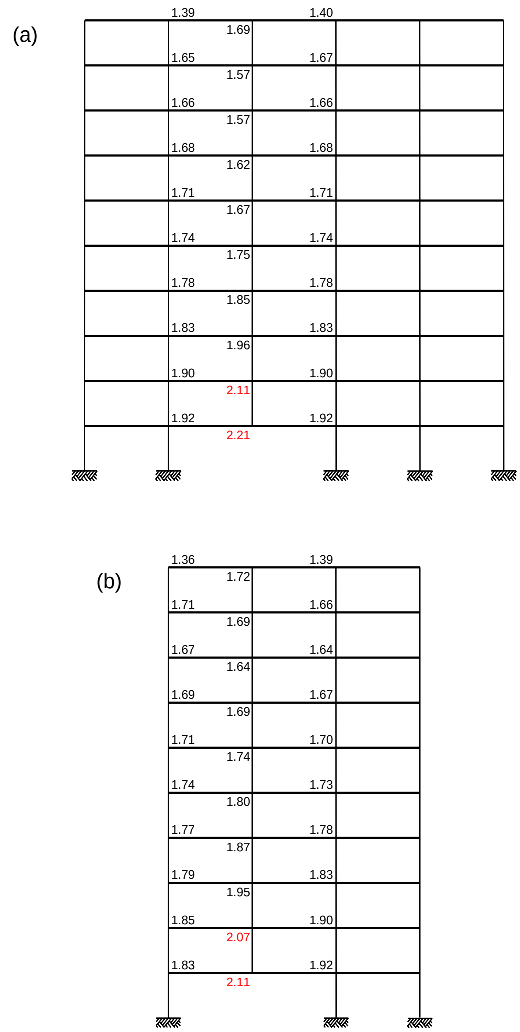

Figure 8: Demand/capacity ratios for the 10-storey Vancouver building for Case 4:
(a) exterior longitudinal frame,
(b) interior transverse frame.

For the Ottawa building, it is seen from Figures 5 and 6 that DCR values significantly larger than 2.0 are obtained for a number of beams along the height of the frames, indicating high potential for progressive collapse for both analysis cases, as discussed above. This is especially obvious for Case 4 (Figure. 6), for which DCR values from 3.5 to 5.28 are obtained for all the beams of the transverse frame in the spans above the column removed. For the Vancouver building, Figures 7 and 8 show much smaller DCR values than those for the Ottawa building. For Case 1, the DCR values for both frames are significantly smaller than 2.0 (Figure 7), while for Case 4, values slightly larger than 2.0 can be seen only for the beams at two floors above the column removed (Figure 8).

6. Summary and Conclusions

This paper describes results from progressive collapse analysis of reinforced concrete frame buildings designed according to the seismic provisions of the 2005 edition of the National Building Code of Canada (NRCC 2005). Three *moderately ductile* buildings designed for Ottawa and three *ductile* buildings designed for Vancouver were used in the analyses. The buildings considered are 5-storey, 10-storey, and 15-storey high. Progressive collapse analyses were conducted following the guidelines of the U.S. General Services Administration (GSA). Columns were removed at the first storey of each building simulating the loss of columns due to bomb blasts or vehicle impacts. The following four cases of column removals were considered: (i) exterior column removed at the long side of the building, (ii) exterior column removed at the short side of the building, (iii) corner column removed, and (iv) interior column removed. Elastic static analysis was conducted for each of these cases using 3-D models, and applying loads as required by the GSA guidelines. Demand/capacity ratios obtained from the analysis were used for the assessment of the vulnerability to progressive collapse. Based on the GSA criteria, the following are the main findings from this study:

- In most of the cases of the column removals, the Ottawa buildings considered in this study were found to be vulnerable to progressive collapse. This is because of the relatively large differences in the spans of the longitudinal and the transverse frames (i.e., 7.5 m vs. 6.0 m). When the spans of the longitudinal and the transverse frames are significantly different, then the demands in the vicinity of the column removed tend to concentrate on the shorter span beams (i.e., the stiffer beams), leading to large demand/capacity ratios.
- The Vancouver buildings showed satisfactory performance against progressive collapse. With the exception of only one case for which the analyses indicated a potential for progressive collapse (when the interior column of the 15-storey building was removed), in all other cases the buildings were found to have a sufficient resistance against progressive collapse. The results indicate that this is because the spans of the longitudinal and the transverse frames are the same, i.e., the demands from the vertical loads in the vicinity of the column removed are almost equally distributed to the beams of the longitudinal and the transverse frames.
- The results from this study suggest that regular reinforced concrete frame buildings designed for seismic actions can resist progressive collapse due to removal of first storey columns if the spans of the longitudinal and the transverse frames are relatively close, and the buildings are less than about 15 storeys high.
- This study indicates that even for regular buildings, some specific features of the buildings (e.g., span ratios of the frames) can affect significantly the resistance against progressive collapse. The prediction of the progressive collapse resistance for irregular buildings, without conducting required analysis, is much more uncertain given the variety of irregularities in plan and elevation that can be seen in practice. Therefore, the issue on the resistance against progressive collapse should be considered as a building-specific issue, and the resistance should be determined by conducting analyses as described in this paper (i.e., following the GSA guidelines).

7. References

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