Vulnerability of Buildings with Flat Plates and Flat Slabs to Progressive Collapse

By

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Dedicated to my family, especially to my parents

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I thank the All Mighty God (Allah) for the blessings including enabling me to complete this work and ask him to turn it into a tool that helps making this world a better place.

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Abstract

The investigation of progressive collapse of buildings has been of special interests during the last decade. The aim of this study is to assess the influence of selected design parameters on the vulnerability of structures with flat plates or flat slabs to progressive collapse triggered by the severe damage or failure of a column as a result of accidental or maliciously intended terrorist activity. The variables of frames with two-way slab systems investigated include: the number of stories in buildings (low-, medium- and high-rise buildings), span ratio (the ratio of span lengths in two orthogonal directions), span length for square panels, as well as the effect of having flat plates compared to flat slabs detailed either as ordinary gravity load carrying systems or by following the seismic detailing requirements of the Canadian practice as per CSA A23.3-04.

The linear-elastic static analysis method, outlined in the General Services Administration (GSA) guidelines of the US, was employed to conduct a parametric investigation to assess the significance of the design variables considered on progressive collapse potentials of buildings. This involved the computation of demand/capacity ratios (DCR) Higher DCRs were obtained for buildings with increased number of stories and span ratios, while the change in span lengths did not show a clear trend. Also, flat slabs were less vulnerable, and experienced smaller DCRs, than flat plates. Buildings detailed according to the seismic provisions of CSA A23.3-04 were less vulnerable than those detailed without these provisions. However, both slabs with non-seismic and seismic detailing required improvements in terms of the percentage of top and bottom reinforcement and bar lengths, depending on the slab type and the design parameters considered.

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Chapter 1

Introduction

1.1 Background

The topic of Progressive Collapse has been of interest to researchers in recent years, especially in the last decade after the failure of gravity load structural systems in buildings due to accidental or terrorist attacks. In particular, structural engineers became concerned with this phenomenon after the collapse of the 22-storey Ronan Point Apartment Building in London, England in 1968 due to a gas explosion (Shankar 2004). This explosion occurred at the 18th floor of the building and caused the collapse of the corner slabs at higher floors, which in turn resulted in the collapse of all corner columns of the building.

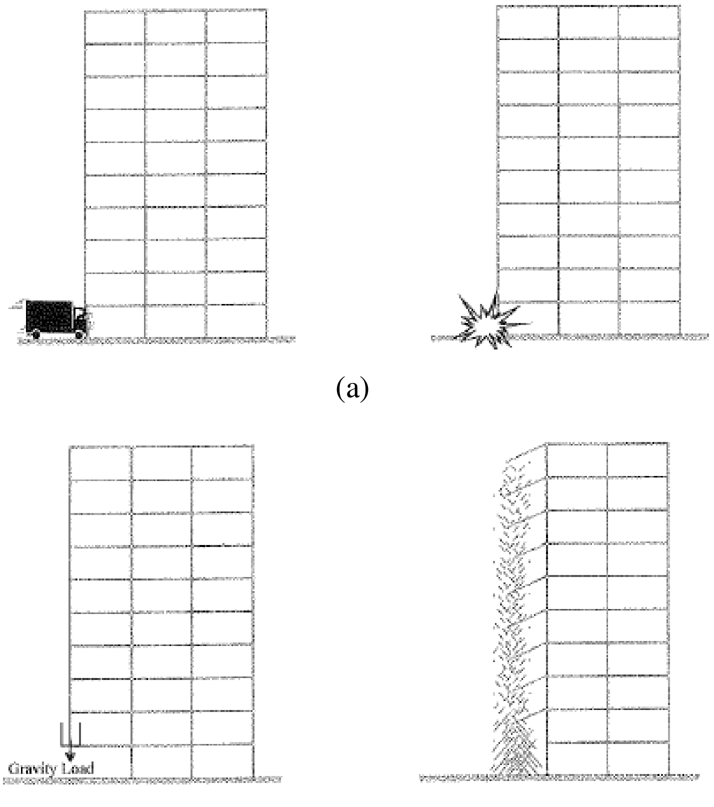
Another recent catastrophe grabbed the attention of civil engineers in 1995 after the collapse of the Alfred P. Murrah Federal Building in Oklahoma, USA. A bomb explosion occurred at the ground level, leading to the collapse of the front half of the building as a result of the failure of three of the structural supporting systems (columns).

Other examples of flat plate failures include the collapse of a condominium in Coca Beach, as well as the Baily Crossroads, which occurred in 1973. These failures were triggered by the punching failure of an interior column during the construction stage (Hawkins & Mitchell 1979).

The most devastating case of progressive collapse that had lasting impact on global international affairs is the airplane attacks of September 11th, 2001 on the World Trade Center Building in New York, USA. These airplanes crashed into the buildings and led to a complete progressive collapse of the towers.

According to the General Services Administration of the US (GSA 2003), progressive collapse can be 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”. Another definition by the ASCE 7-2005 is “the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or disproportionately large part of it”.

Progressive collapse results from abnormal loading events that could be accidental, such as fire or gas explosions or vehicle impacts, or terrorist attacks such as those that result from bomb or missile attacks. Fig 1.1 illustrates the causes and the sequence of progressive collapse.



(a)

(b)

Fig. 1.1 Progressive collapse (a) causes and (b) failure of column and triggering of collapse (Naumoski et al. 2009).

Many recent codes address the issue of progressive collapse as it pertains to improving structural integrity, but do not provide detailed provisions against progressive collapse (Naumoski et al. 2011).

1.2 Objectives and Scope of Study

The objective of the current research project is to investigate the progressive collapse potential of reinforced concrete systems with flat plate or flat slab systems. More specifically, the objective includes the investigation of the effects of; i) number of stories in buildings (low-, medium- and high-rise buildings), ii) span ratios, iii) span length of square panels, and iv) type of slab (flat plates versus flat slabs) on vulnerability of reinforced concrete structures to progressive collapse, when detailed either as ordinary non-seismic buildings or by following the seismic detailing requirements for slabs.

The scope consists of design and analysis of the following six two-way slab systems forming part of 1, 5, 10 and 15 storey buildings:

1. Flat plate system having 7x7 m spans (FP-7x7).
2. Flat plate system having 7x3.5 m spans (FP-7x3.5).
3. Flat slab system having 7x7 m spans (FS-7x7).
4. Flat slab system having 7x3.5 m spans (FS-7x3.5).
5. Flat slab system having 5x5 m spans (FS-5x5).
6. Flat slab system having 9x9 m spans (FS-9x9).

Figures 1.2 and 1.3 illustrate typical flat plate and flat slab construction. Flat plates are directly supported by columns, whereas flat slabs are supported by columns through drop panels. The slabs and buildings considered in the current investigation were designed in accordance with the National Building Code of Canada (NBCC 2010) and the Canadian standard for design of concrete structures (CSA A23.3-04). The vulnerability to progressive collapse assessment was carried out following the General Services Administration guidelines (GSA 2003) which involves the evaluation of the demand/capacity ratios in positive and negative moment regions, as well as the presence of reinforcement in critical locations of structural elements using linear-elastic static analysis.

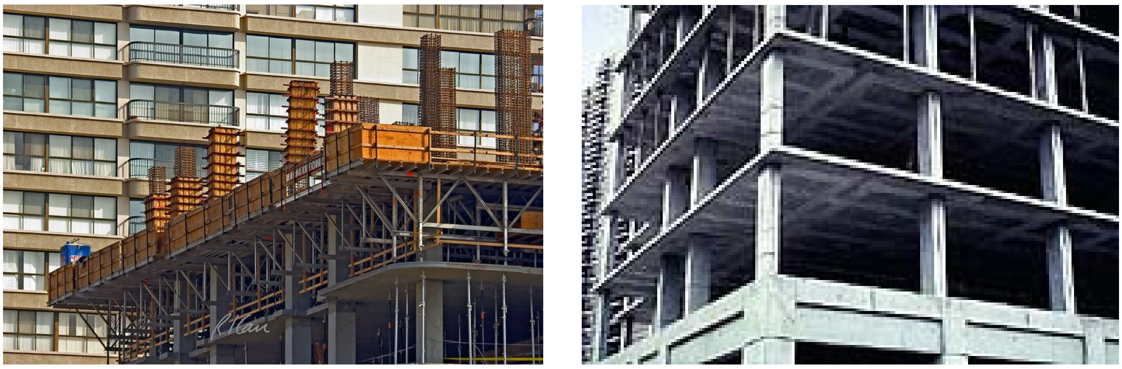


Fig. 1.2 Typical flat plate construction where slabs are directly supported by columns


Fig. 1.3 Typical flat slab construction where slabs are supported through drop panels

The current research investigation focuses on progressive collapse analysis triggered by flexural action. In some structures, shear could be critical and punching shear of slab should be checked.

1.3 Outline of Thesis

The content of the thesis is divided into six main chapters and two appendices. These chapters include discussions on the methodology, analysis and design of buildings, as well as the presentation of results and conclusions. Chapter 2 summarizes the literature review on progressive collapse of different types of structural systems, including flat plates and flat slabs. Chapter 3 describes the layout of slabs, geometry of buildings and detailed explanation of designs for the buildings considered in the current research project. A brief summary about the assumptions made and the reinforcement detailing provided are also presented. Chapter 4 describes the analysis approach employed, in addition to the provisions of the guidelines followed. The software used and the building models devised, as well the results of analyses are presented. Chapter 5 provides discussion of results and detailed recommendations for improved design against progressive collapse. The sixth and final chapter summarizes the conclusions obtained as a result of the current investigation, as well the recommendations made for future research.

Chapter 2

Literature Review

2.1 Prevention of Progressive Collapse of Two-Way Slab Systems

Flat slabs have become increasingly more popular among designers within the last decade. This may be attributed to ease in construction and economic benefits offered by these slab systems, especially because of the simplicity of formwork. However, this type of slab system poses a punching failure risk, which often results in sudden and brittle failures at slab-column connections. They become especially critical under abnormal loading.

Many research programs were undertaken and workshops were organized to address the challenges associated with progressive collapse. This interest increased after each event that led to progressive collapse. Examples include the terrorist attacks in 1970s (Vonier 1997) as well as other explosions that led to partial or full progressive collapse, such as the gas explosion that occurred in the Ronan Point Apartment Building (1968). As a result, it became essential to include provisions related to this phenomenon in recent codes and standards.

The National Building Code of Canada (NBCC) begun to address progressive collapse by including some related regulations in 1975 (Dusenberry and Juneja 2002). Subsequent additions of the code included further improvements. The NBCC 1995 required structures to sustain loads through adequate structural integrity. This requirement was stated as “the ability of the structure to absorb local failure without widespread collapse”. In addition, it recommended the consideration of probable accidents, and the development of new demands, but did not provide further details on how to achieve these objectives, how to conduct the analysis, or which load combinations to use in design (Mirzaei 2010).

Breen (1975) reviewed the regulations related to progressive collapse with concentration on the design details of precast concrete structures. The study illustrates the required resistance to progressive collapse, and methods to evaluate existing structures. Popoff (1977) investigated weaknesses in connections, especially in precast concrete slabs, and suggested some retrofit methods that could be applied to reduce the vulnerability of these critical regions to progressive collapse.

One of the papers published by Hawkins & Mitchell in 1979 addressed factors that could start progressive collapse of buildings with flat plates. They also stated that these types of structures are most vulnerable to progressive collapse, as punching shear often tends to occur prior to the yielding of bottom reinforcement. The researchers outlined the following points to reduce vulnerability to progressive collapse:

- The adoption of higher values for design live loads.
- Designing the bottom steel for tensile membrane action.
- Provisions related to continuity and anchoring of bottom reinforcement
- Provisions related to the stirrup reinforcement for an integral beam.

It was found that a well-detailed flat slab could survive beyond the failure of one of its columns by hanging from the other supports.

Regan (1981) suggested that in the case of a failure of an internal column, shear in adjacent columns can increased by about 25%. This failure can easily propagate horizontally and could lead to a complete failure of the slab. It can also fall on to the lower floor, which then experiences almost double the design load, magnified by dynamic effects. After the initial shear failure of a column-slab connection, the only remaining resistance would be the reinforcement passing through the damaged area. Negative (top) reinforcement could participate in carrying part of the load. However, it would easily tear out of the slab. This could extend further, until it compromises anchorage, resulting in the detachment of the bars. On the other hand, bottom bars are more reliable compared to top bars, as they are less likely to tear out. Resistance provided by bottom steel increases rapidly with increasing deflections in the vertical direction after the initial failure. The final mechanism of failure is not fully understood, but believed to be caused by the yielding of steel due to flexural bending and the crushing of concrete. Regan (1981) concluded that the bottom bars could enhance the resistance of punching shear. A small increase in bottom steel could reduce the risk of progressive collapse. Fig. 2.1 explains the behavior of top and bottom reinforcement after the initial punching failure of a column-slab connection:

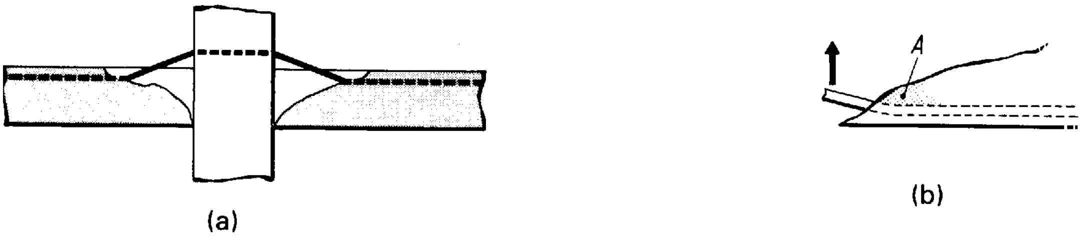


Fig. 2.1 Post punching behavior of top and bottom reinforcement (Regan 1981).
a. Tearing out of top reinforcement. b. Yielding of bottom reinforcement and crushing of surrounding concrete (area A).

Most of the research conducted during the period of 1976 to 1994, was based on probabilistic approaches to determine load factors that would lead to an abnormal load and eventually failure (Kim 2006). These approaches were illustrated in detail by Ellingwood and Leyendecker (1983).

Other researchers (Chu 1981, Balazic 1982, Mitchell and Cook 1984) concentrated on the enhancement of reinforcement detailing so that tensile behavior can be developed. This was done by ensuring that the two-way punching load would not exceed post-punching resistance. This implies that slabs would withstand the sudden failure of a column by developing a secondary mechanism that can carry and transfer the load. This would be achieved by:

- Detailing continuous bottom reinforcement over the area of column in order to sustain post punching shear resistance at areas that connect slabs and columns.
- Good detailing of slab reinforcement that would enable the slab to behave as a tensile membrane and allow for the adequate transfer of loads.

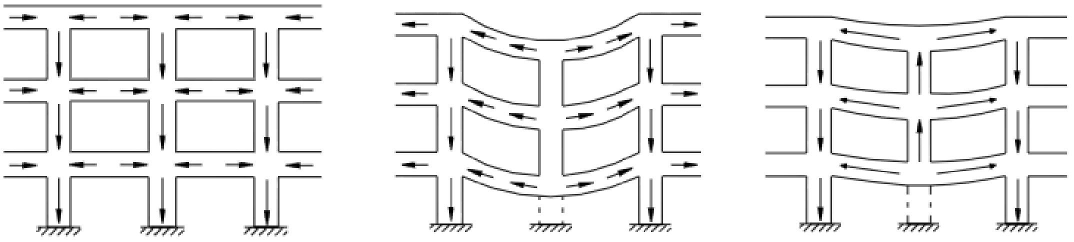
Recent research and papers are more oriented towards supplying designers with guidelines to prevent the collapse of the building following the failure of one of the supports.

A workshop was held after the September 11, 2001 event to discuss and review the state of knowledge on hazard mitigation, including features of progressive collapse (Multi-Hazard Mitigation Council Workshop 2002). The GSA, as well as numerous researchers and engineers participated in the workshop and discussed different mitigation methods that are available. Burns et al. (2002) suggested design provisions that accounted for progressive collapse. Cagley (2002) suggested the use of performance-based design as a guideline. Corley (2002) discussed the importance of detailing structures with moment resisting frames according to seismic provisions in preventing or mitigating progressive collapse.

Recommendations made by Baldridge and Humay (2003) include design with appropriate continuity, redundancy and ductility to create secondary load paths, following the loss of a primary load carrying mechanism such as a gravity-load carrying support. The researchers also indicated that structures designed according to modern seismic provisions would in most cases resist progressive collapse. Starossek and Wolff (2005) suggested methods to prevent progressive collapse as outlined below:

- “Direct Method,” where higher loads can be adopted by the designer to provide higher local resistance (specific load resistance).
- “Indirect Method,” where prescriptive design rules are usually given when detailed analysis cannot be performed.
- “Event Control Method,” where external protective measures are used such as creating a secondary path for the loading in the event of failure a of primary load carrying system.

The importance of ductility in reinforced concrete flat slab-column connections, particularly after the application of peak-load, was emphasized by Polak (2005). Excessive flexural cracking of concrete upon yielding could initiate punching shear, jeopardizing structural integrity. Therefore, inelastic deformations in these regions should be controlled through adequate ductility. Also, the article reveals the importance of transverse reinforcement in increasing rotational capacity, and the role of shear bolts used for retrofitting purposes in increasing shear resistance and ductility. Studies made by Elgabry and Ghali (1990) and Mokhtar et al. (1985) discussed the role of shear reinforcement in slabs, if effectively utilized to increase punching shear and rotational capacities by ensuring sufficient anchorage. The conclusions of the experimental and analytical research confirm the importance of shear bolts and their ability to control failure, even if they may not eliminate the failure completely. Fig. 2.2 shows the distribution of forces in a frame structure before and after the failure of a column and the newly developed demands afterwards.



a) Before column failure

b) After column failure
(catenary action)

(c) After column failure
(tension in columns)

Fig. 2.2 Flow of forces in a structure (Kim 2006).

Egberts (2009) studied the importance of secondary load paths, following the loss of a support, and emphasized the importance of the ability of secondary load carrying members to resist the new loads transferred to them. Kokot et al. (2010) performed a study on static and dynamic analysis of a reinforced concrete flat slab frame building for progressive collapse. The study revealed the following points:

1. Linear static analysis shows “limited or no damage” when removing the columns statically. However, using the same analysis but removing the column dynamically results in a more vulnerable structure.

2. Linear dynamic analysis shows slightly better performance. However, the local dynamic factor, reflecting the ratio of dynamic and static demand-resistance ratios is “unworkable”.
3. Nonlinear dynamic analysis indicates that progressive collapse “would not have happened”. This type of analysis is time consuming, and requires accurate modeling and special attention to plastic hinge locations.
4. Inconsistencies are observed in behavior when different analysis techniques are employed in terms of the location and severity of critical sections.

Ramos et al. (2010) conducted an experimental investigation of the slab-column connection. The experiment evaluated the behavior of flat slab systems under punching load, when concurrently subjected to in-plane forces. It was found that flexural and shear cracking were delayed compared to slabs without the in-plane forces. Also, smaller strains and deflections were observed in the latter case. Choi et al. (2007) reported that the most critical region of slabs, leading to progressive collapse is the perimeter region around the column, failure of which jeopardizes structure’s ability to carry its gravity load, potentially resulting in the separation of the column-slab connection.

Park (2011) investigated the causes of the progressive collapse of Sampoong Department Store in Seoul-South Korea, which consisted of flat slabs systems. The failure resulted in the death of more than 500 people after the complete collapse of structure in 1995. Unlike most cases of progressive collapse, in this case, the cause of the collapse was concluded to be the result of changes in design and heavier design loads that have not been accounted for. Poor management and supervision during construction and improper planning were considered to be contributing factors. The direct triggering factor was believed to be the shear failure that occurred in the top floor and roof slabs along the perimeter of the supporting pillars, leading in turn to the damage of the adjacent slabs and eventually the collapse of the structure.

Recently, a study was conducted by Namouski et al. 2011 on progressive collapse behavior of reinforced concrete buildings. Six buildings were designed according to the seismic requirements of the National Building Code of Canada (NBCC 2005) for Ottawa and Vancouver, representing seismic regions of moderate and highly intensity to assess the vulnerability of seismically resisting structures having frame systems. Three buildings; with 5, 10 and 15 stories; were analyzed for each city. The buildings in Ottawa consisted of different span ratios in two orthogonal directions and were designed as moderately ductile frames, while the buildings in Vancouver had a span ratio of 1, and were designed as ductile frame structures. The GSA approach was followed to assess the progressive collapse potential. The most important findings of the study can be summarized as below:

1. Buildings are more vulnerable when there are large differences in longitudinal to transverse span ratios. This is because stiffer shorter beams go under larger demands.
2. Buildings are less vulnerable when span lengths in longitudinal direction approach to those in transverse direction.
3. Structures designed according to the seismic provisions of NBCC could show satisfactory results in terms of their progressive collapse potential, with the exception of high-rise buildings.

The review of previous literature indicates that an urgent need exists for comprehensive guidelines that would include analysis and evaluation techniques to assess the vulnerability of buildings to progressive collapse in Canada. Two main U.S. guidelines currently exist. They were published specifically for government and military structures, as listed below:

- The U.S. General Services Administration (GSA) published as “Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects (GSA 2003)”.
- The U.S. Department of Defense (DoD) published as “Design of Buildings to Resist Progressive Collapse (DoD 2005)”.

These guidelines provide a thorough evaluation of progressive collapse potential of buildings and include types and procedures of relevant analysis, as well as the sequence to be adopted. Most importantly, the section on the evaluation criteria provides estimates of building vulnerability to progressive collapse, providing a clear picture to the designer. These guidelines are discussed with more details in Chapter 4.

In summary, the previous literature has indicated that a number of progressive collapse prevention methods are available for two-way slab systems, focusing on improved detailing of slab reinforcement by ensuring proper anchorage. Furthermore, creating an alternate load path is probably the most practical and efficient strategy to prevent progressive collapse. However, the new system should be capable of withstanding the additional load transferred by the loss of a primary supporting structural system.

Chapter 3

Description and Design of Buildings Investigated

This chapter explains the configurations of the slabs and buildings analyzed throughout the current research program. All buildings consisted of two-way reinforced concrete slabs and were designed and analyzed according to the National Building Code of Canada NBCC 2010 and the CSA A23.3-04 standard.

3.1 Types of Slabs and Buildings Considered

Six main reinforced concrete two-way slab systems were considered for analysis. The purpose was to investigate the influence of the span ratio (the ratio of longitudinal span to transverse span), span length for square panels, and the type of slab system (two-way flat plates without drop panels and two-way flat slabs with drop panels). Neither interior nor exterior (spandrel) beams were used. Also, shear-walls were not included in any of the buildings. However, the vulnerability of slab systems supported only by columns is likely to be more critical than those with shear-walls. Hence, the current research focuses on slabs supported only by the columns.

Six main slab systems considered are as follows:

1. Flat Plate 7x7 m spans (FP-7x7).
2. Flat Plate 7x3.5 m spans (FP-7x3.5).
3. Flat Slab 7x7 m spans (FS-7x7).
4. Flat Slab 7x3.5 m spans (FS-7x3.5).
5. Flat Slab 5x5 m spans (FS-5x5).
6. Flat Slab 9x9 m spans (FS-9x9).

Each of the six main slabs was designed and analyzed for 4 buildings with different number of stories, representing low-, medium- and high-rise buildings with a typical storey height of 3.65 m, as follows:

- 1 Storey Building
- 5 Storey Building
- 10 Storey Building
- 15 Storey Building

Figures 3.1 through 3.6 illustrate typical floor plans and slab systems used. All the stories in a given building had the same slab system. Figure 3.7 illustrates the elevation views of buildings. Each building was analyzed for different scenarios of column removal, which represents sever damage or failure of the removed column. The total number of buildings analyzed was 24, while the total number of analyses consisting of all different cases of column removal was 86.

3.2 Slab Design

The frame elements, consisting of slabs and columns, were analyzed and designed for specified live and dead loads. The same materials were used for all building designs. Slab thickness, column dimensions and the required reinforcement were computed as presented in the following subsections, following the current design practice in Canada. Standard grade reinforcement with yield strength of f_y = 400 MPa was used in design. The compressive strength of concrete was selected to be f_c' = 30 MPa. Unfactored design loads were taken as DL = 1.5 kN/m^2 for the Dead Load and LL = 2.4 kN/m^2 for the Live Load. A typical storey-height of 3.65 m was used for each floor level, irrespective of the slab system. This value was used in the analytical building model as centre-to-centre distance between the slabs along building height.

3.2.1 Determination of Member Dimensions

The first step for analysis and design involved computation of member dimensions so that an analytical model, with appropriate stiffnesses could be constructed for computer analysis. This implies that the slab and column dimensions had to be established first.

The slab dimensions were computed with due considerations given to the minimum slab thickness requirements outlined in CSA A23.3 (2004) as described in the following subsections. In each case a slab thickness, slightly higher than the minimum was selected.

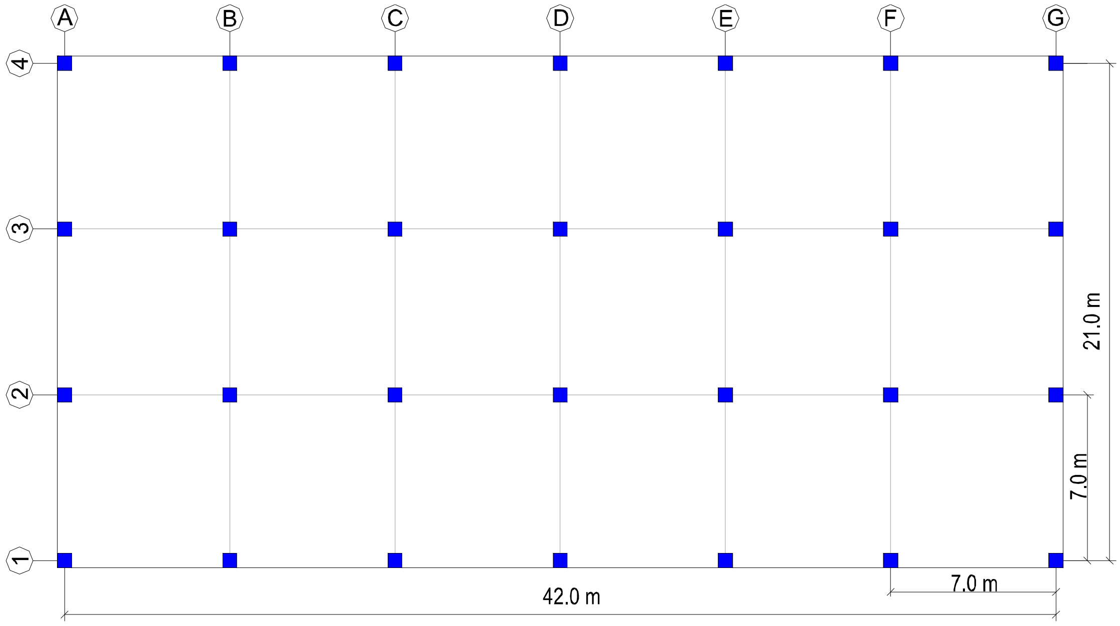


Fig. 3.1 Typical floor plan for FP-7x7

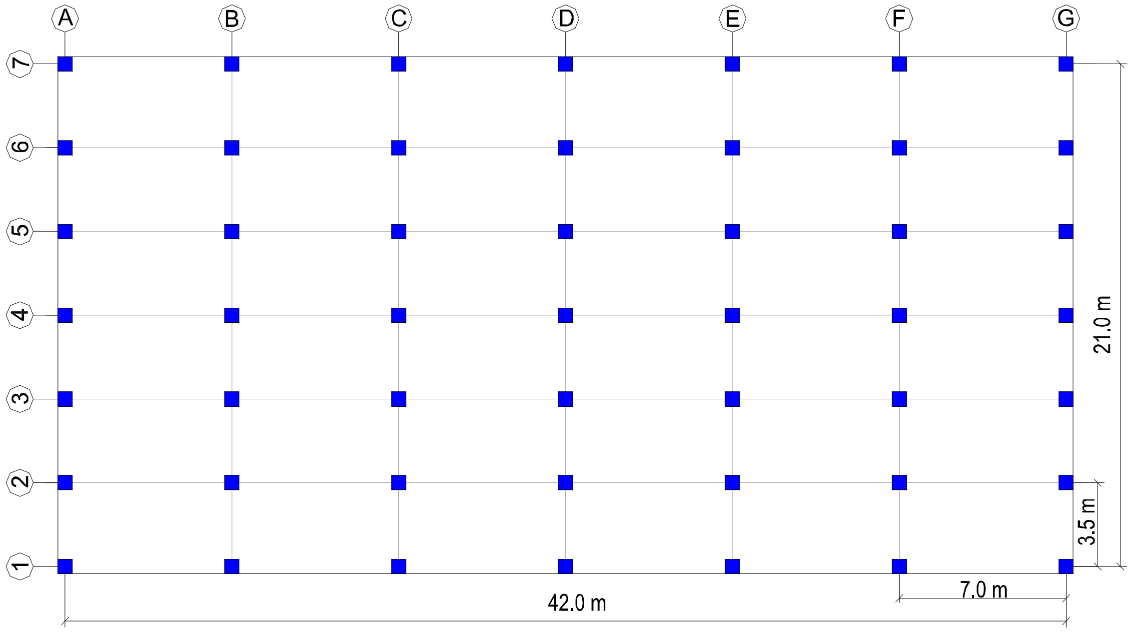


Fig. 3.2 Typical floor plan of FP-7x3.5

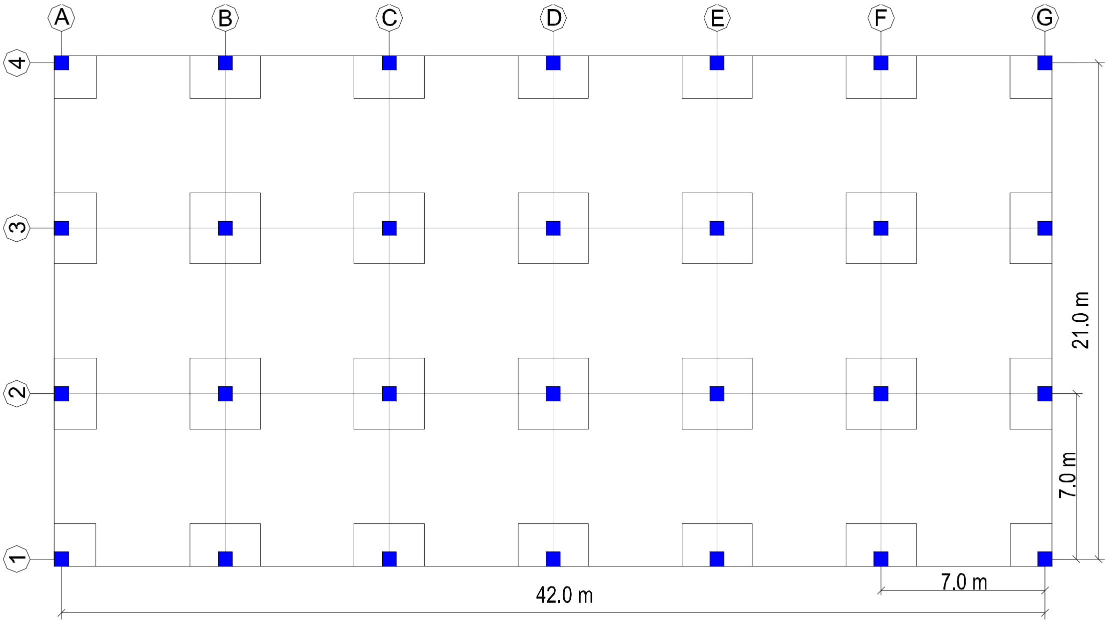


Fig. 3.3 Typical floor plan of FS-7x7

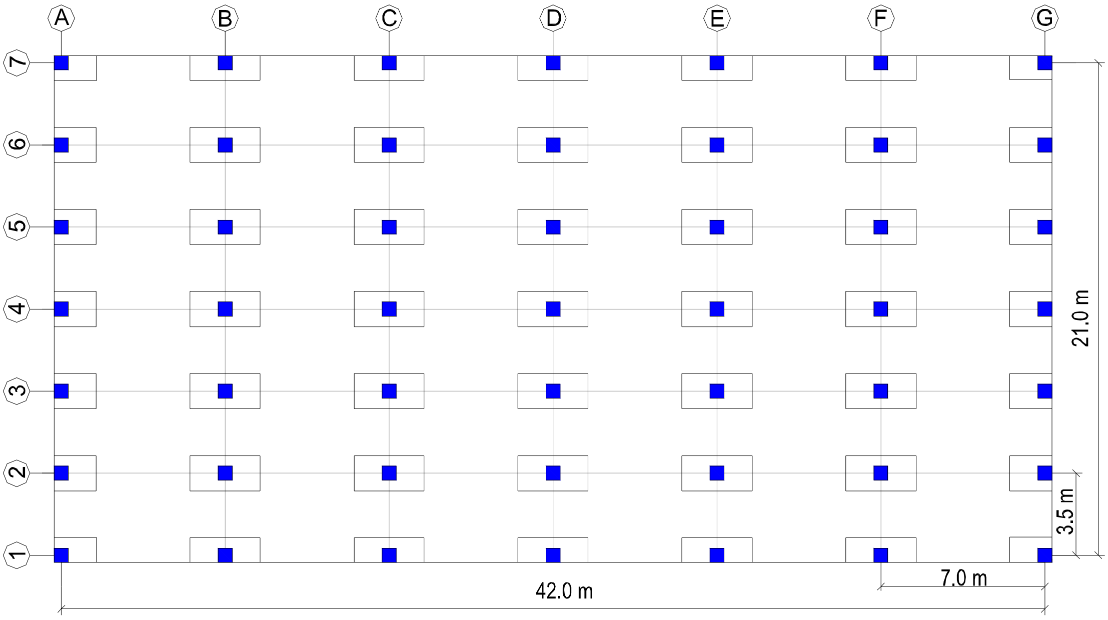


Fig. 3.4 Typical floor plan for FS-7x3.5

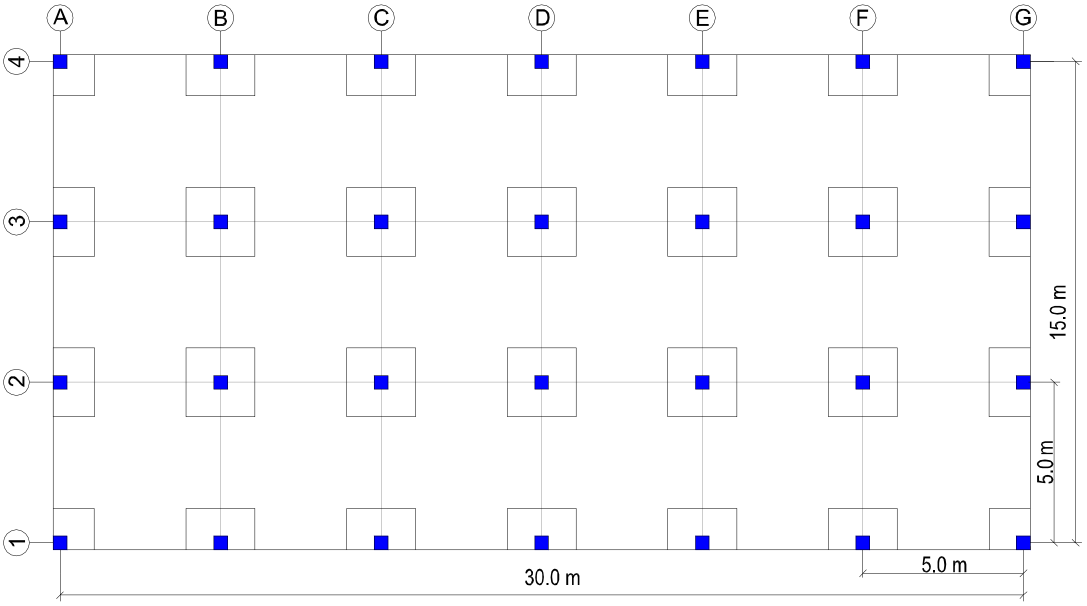


Fig. 3.5 Typical floor plan for FS-5x5

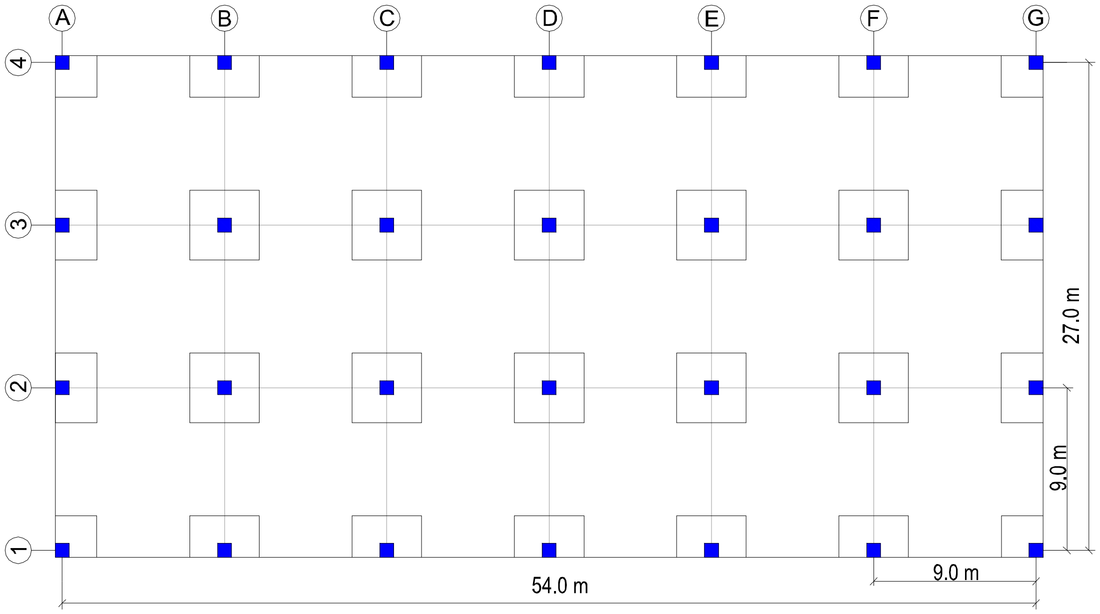


Fig. 3.6 Typical floor plan for FS-9x9

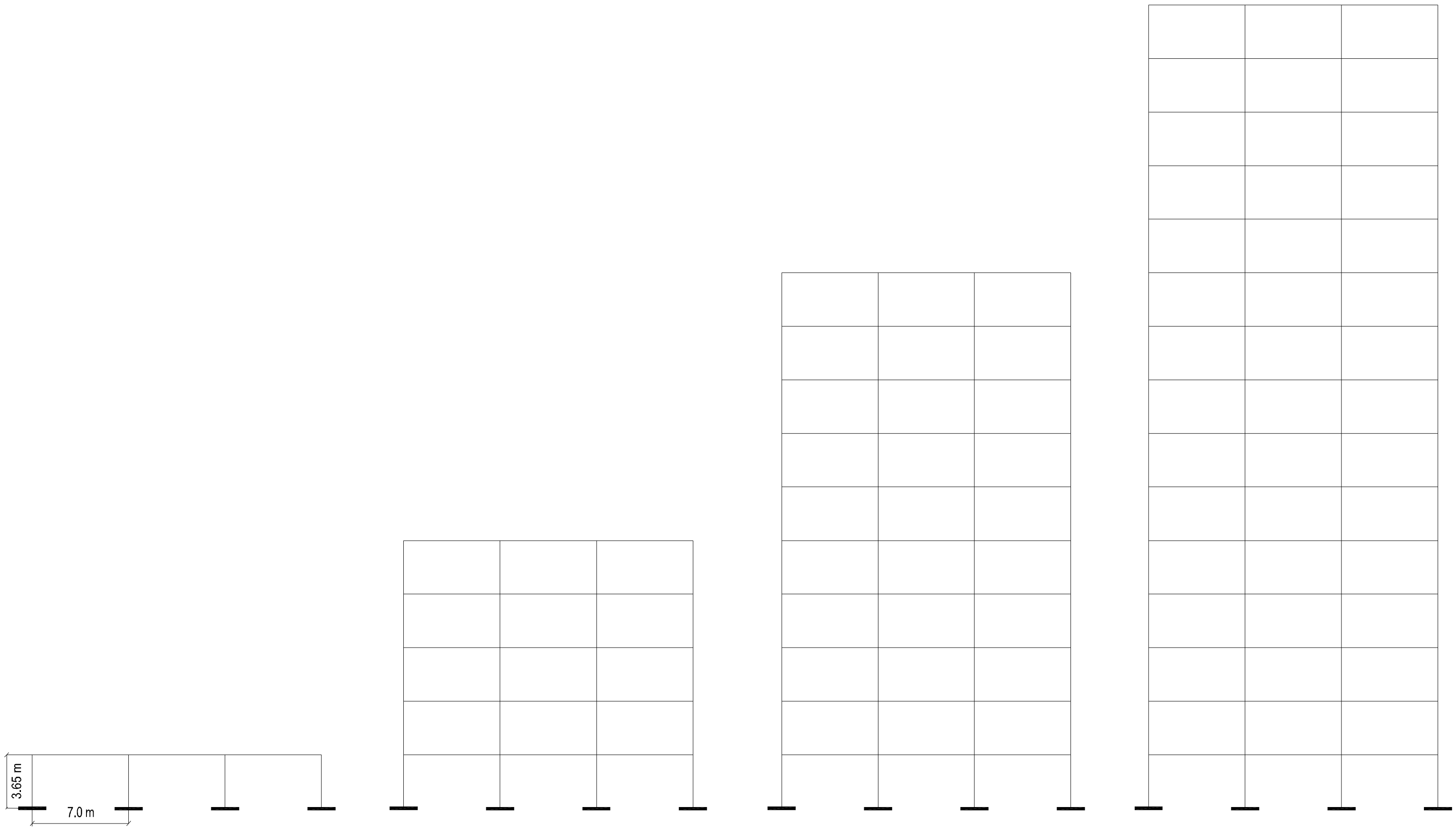


Fig.3.7 Side view (transverse direction) of 1, 5, 10 and 15 storey buildings with typical floor height of 3.65 m and varying span length (3.5, 5, 7 and 9 m) horizontally.

The thickness used for flat plates (FP-7x7 and FP-7x3.5) was identical, and the thickness used for flat slabs and drop panels (FS-7x7 and FS-7x3.5) were the same, with different dimension drop panels used for FS-7x3.5 in the short direction**.**

Slab Thickness for FP-7x7 and FP-7x3.5

The minimum thickness of the slab can be obtained from Clause 13.2.3, Equation 13-1 in CSA A23.3-04, which is reproduced below.

$$h_s \leq \left(\frac{l_n \, (0.6 \, + \, f_y/1000)}{30} \right)$$

Where h_s is the minimum slab thickness. The longest clear span length (measured face to face of supports), l_n was taken as 6400 mm, and the steel grade used was f_y = 400 MPa. This resulted in h_s = 213 mm. The slab thickness selected was 250 mm in all buildings with flat plates, in all stories, when the span ratio was 1 and 2 (FP-7x7 and FP-7x3.5) so that the results could be compared under different scenarios. This satisfies clause 13.2.1 of CSA A23.3-04 which requires that the minimum thickness provided should always be larger than 120 mm.

Slab Thickness for FS-7x7

Flat slabs are flat plates with drop panels. Drop panels are used for economical and practical reasons for reducing punching shear stresses at slab-column joints, and for increasing the negative moment capacity by providing a larger effective depth for a smaller slab thickness. The effective depth below the slab thickness, however, should not be considered larger than 1/4th the distance from the edge of the column or column capital to the edge of the drop panel according to clause 13.10.7 of CSA A23.3-04. The minimum thickness of the slab is expressed in Clause 13.2.4, Equation 13-2 of CSA A23.3-04 is reproduced below.

$$h_s \leq \left(\frac{l_n \, (0.6 \, + \, f_y/1000)}{30} \right) - \left(\frac{2 \, X_d}{ln} \, \Delta_h \right)$$

In the above expression, the distance from the edge of the support to the edge of the drop panel is denoted by X_d. This length may be taken between 1/4th and 1/6th of the largest clear span. The value of Δ_h, which represents the projection of the drop below the slab, should be larger than half the thickness of the slab and smaller than the total thickness of the slab, as illustrated in Fig. 3.8. Using these values, the minimum slab thickness was computed to be 180 mm. This thickness should not be less than 120 mm, as per Clause 13.2.1 of CSA A23.3-04.

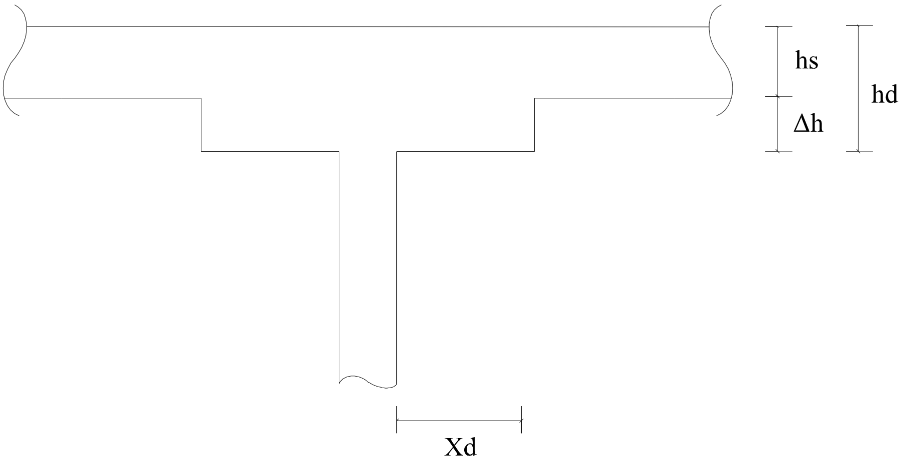


Fig. 3.8 Cross-section of flat slab

The following values were used in design:

h_s = 180 mm

h_d = 270 mm

X_d = 1200 mm.

Slab Thickness for FS-7x3.5

For the building with a span ratio of 2.0, one of the spans has double the length of the other span in the orthogonal direction. Rectangular drop panels were used for the building as they are more economical and appropriate for this plan layout. Therefore, drop panel was designed twice; in the short and long directions with the same thickness. The only difference was in the value of X_d. The following was used for this slab.

Long direction:

The same dimensions used for FS-7x7 applied in the long direction.

h_s = 180 mm.

h_d = 270 mm.

X_d = 1200 mm.

Short direction:

Trying minimum $X_d = l_n / 6 = 2900 / 6 = 483$ mm, and therefore choosing 500 mm.

$$h_s = \left(\frac{2900\ (0.6 + 400/1000)}{30} \right) - 2 \left(\frac{500}{2900} \right) (90) = 65.6 \text{ mm}$$

The values used in design:

h_s = 180 mm.

h_d = 270 mm.

X_d = 500 mm.

The selected slab thickness satisfies the requirements of Clause 13.2.1 of CSA A23.3-04, which requires that the minimum thickness provided in any case to be larger than 120 mm.

Slab Thickness for FS-5x5

The selected design dimensions were:

h_s = 150 mm.

h_d = 230 mm.

X_d = 800 mm.

$$h_s = \left(\frac{4500\ (0.6 + 400/1000)}{30} \right) - 2 \left(\frac{800}{4500} \right) (80) = 121.6 \text{ mm}$$

These dimensions satisfy the requirements of the minimum and the maximum values recommended by the code.

Slab Thickness for FS-9x9

The selected dimensions were:

h_s = 230 mm.

h_d = 350 mm.

X_d = 1400 mm.

$$h_s = \left(\frac{8400\ (0.6\ +\ 400/1000)}{30} \right) - 2 \left(\frac{1500}{8400} \right) (80) \quad = \quad 226.5\ \text{mm}$$

These dimensions satisfy the requirements of the minimum and the maximum values recommended by the code.

Column Dimensions

Using the loading considered on the buildings, it was decided to use 600 mm by 600 mm square columns in all buildings and building models. This section would require different percentages of reinforcement for different columns.

3.2.2 Analytical Modeling and Structural Analysis for Design

Once member sizes were established, the next step in the design process involved the modeling of buildings for analysis. ETABS computer software was employed for structural analysis (CSI 2008). The analytical model for ETABS consisted of slabs supported by columns. The number of spans used in two orthogonal directions varied among the buildings depending on the span ratios. Slabs with 1:1 span ratio had 6 spans in the longitudinal direction and 3 spans in the transverse direction. Slabs with 2:1 span ratio had 6 spans in the longitudinal direction and 6 short spans in the transverse direction. The slab elements were modeled as thin plates, which imply that transverse shearing deformations were neglected. The thick plate option available in the computer software incorporated out-of-plane transverse shearing deformations, which when considered reduced force demands on slabs with normal thicknesses.

One of the most important parameters that had to be decided was the mesh size for slabs. Different scenarios were considered, including mesh aspect ratios of 32, 16, 8, 4, 2, 1and 0.5. The aspect ratios of 32 and 16 were far from the actual value. On the other hand, aspect ratios of less than 1 were extremely complicated and took hours to execute. When convergence could not be attained quickly, other issues arouse during the analysis, and the program terminated prematurely. It was concluded that an aspect ratio of 1.0 was the most suitable option for analysis (mesh size of 0.25 x 0.25 x 0.25 m) in the case of FP-7x7 and FP-7x3.5. The same configuration of mesh size (0.25 x 0.25 m) was used in all slabs. Figure 3.9 summarizes the results for different mesh aspect ratios.

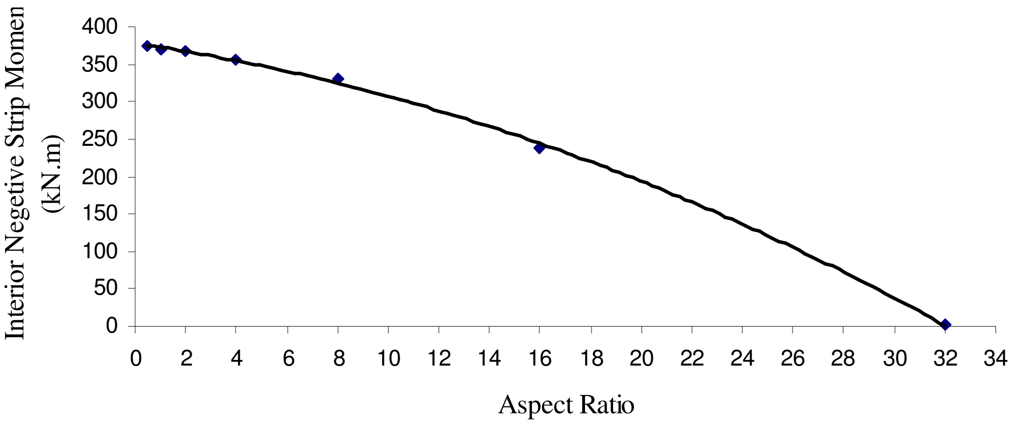


Fig. 3.9 The effect of aspect ratio of mesh on results

The automatic rigid zone option of the software over the columns was excluded from the analysis as it underestimated force demands in these regions.

Linear-elastic analysis of buildings was conducted for design using ETABS. Two gravity load combinations were considered as per CSA A23.3-04:

- 1.4 (Dead Load)
- 1.25 (Dead Load) + (1.5 Live Load)

The second case always governed the slab design.

3.2.3 Reinforcement Design and Detailing of Slabs

ETABS analysis provided column distortions at slab-column connections. The distortions for a single storey were then exported to SAFE software for slab analysis and design as per CSA A23.3-04 provisions. SAFE has the capability to design slabs, whereas ETABS does not. On the other hand, ETABS analysis provides accurate support restraints at slab-column joints. This approach also allowed the incorporation of column distortions in all the stories in multi-storey buildings, which could not otherwise be done by SAFE. The design was performed and reinforcement detailing was determined while also imposing the minimum reinforcement requirement. Column and middle strip widths were specified as input parameters. The column strip was taken as the smaller of 0.25 l_1 or 0.25 l_2 on either side of the column line, as defined in Clause 2.2 of CSA A23.3-04, where l_1 and l_2 are longitudinal and transverse span lengths. The remaining portions of the slab strip formed the middle strip. The reinforcement determined by the software was manually checked in order to verify the results.

Column and middle strips were designed for positive and negative reinforcement for flexure at mid-span and strip ends near the columns. The structural integrity reinforcement, which is required to form part of the positive (bottom) reinforcement near the columns, was calculated based on Clause 13.10.6 of CSA A23.3-04. Accordingly, the structural integrity steel should be the larger of:

- $\sum A_{sb} = \left(\frac{2\ V_{se}}{f_y} \right)$
- Two continuous bars or tendons

Where, f_y is the specified yield strength of steel and V_{se} is the shear transmitted to column or column capital due to specified loads, but not less than the shear corresponding to twice the self-weight of the slab as described by CSA A23.3-04. In all cases, the above equation, which is a function of shear force, always produced higher area of steel than the two No. 10 continuous bars used as minimum reinforcement. The minimum area of reinforcement in each direction was computed as required by Clause 7.8.1 of CSA A23.3-04 with a value of $0.002A_g$, where A_g is the gross area of slab cross section. The reinforcement curtailment rules regarding minimum reinforcement lengths for flat slabs (without beams), specified in Clause 13.10.8 of CSA A23.3-04, and also illustrated in Fig. 3.10, were implemented. The area of steal used in the subsequent progressive collapse analysis was taken to be the actual steal area required rather than the steel area provided.

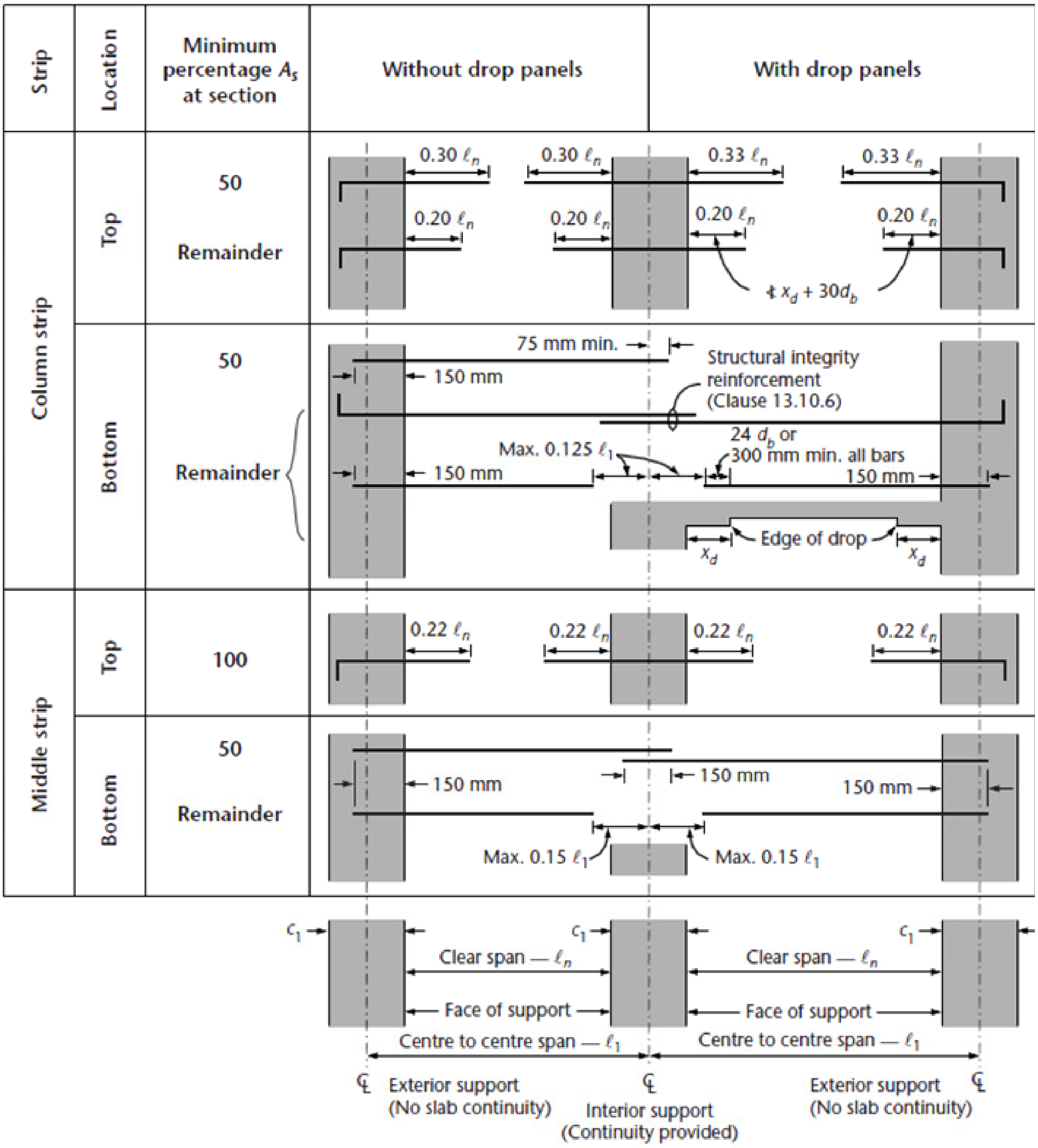
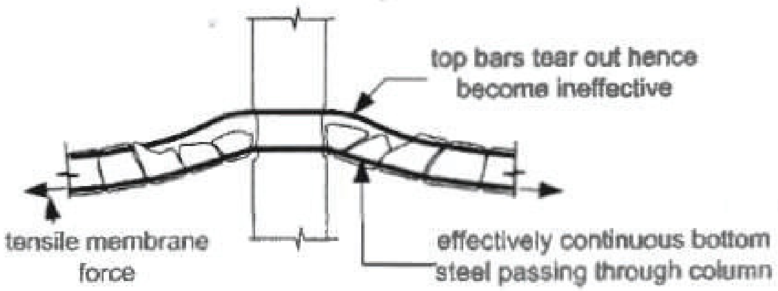
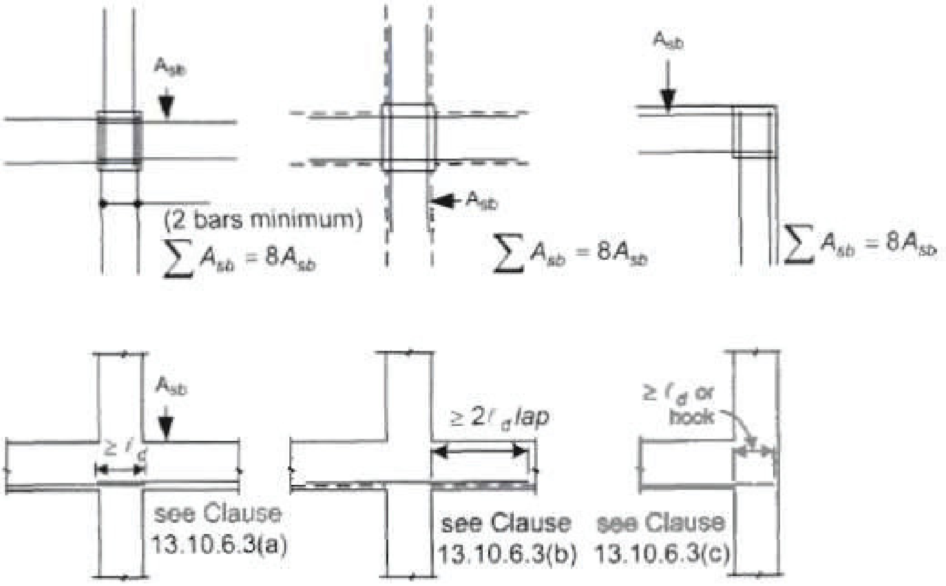


Fig. 3.10 Guidelines for minimum slab reinforcement lengths
(CSA A23.3-04 Clause 13.10.8.1)

The structural integrity reinforcement described in CSA A23.3-04, with details shown in Fig. 3.11, was provided as positive reinforcement at column locations.



(a) effective continuous bottom reinforcement providing post punching failure resistance



(b) ways of proving effectively continuous bottom reinforcement

Fig. 3.11 Detailing of reinforcement provided at the bottom of the slab at the location of column to serve as structural integrity reinforcement
(CSA A23.3-04 Notes N13.10.8.6.1).

When the effects of seismic detailing are investigated, additional slab reinforcement was provided in slabs, detailed according to the seismic provisions of CSA A23.3-04. These slabs have better detailing and higher moment capacities as required by Clause 21.8.4 for two-way slabs without beams. These requirements improve continuity of reinforcement in slabs, both top and bottom, while also providing some minimum positive and negative moment capacity everywhere in the slab. The main improvement for negative reinforcement is stated in Clause 21.8.4.4, which requires at lease one-quarter of the negative (top) reinforcement to be continuous throughout the entire span within the column strip. The main improvement for positive (bottom) reinforcement is specified in Clause 21.8.4.5, which indicates that the minimum amount of positive reinforcement in the column strip at the location of columns to be equal to one-third of the maximum negative reinforcement at that location. Another improvement in bottom reinforcement is to have at least one half of the positive reinforcement at mid-span to be continuous along the entire span, including the strip ends where column are located. These provisions provide significant enhancement to reinforcement detailing, as compared to non-seismic slabs, which require only the structural integrity reinforcement to be continuous. However, the structural integrity reinforcement is often significantly less than 50% of the positive reinforcement. The seismic requirements equally apply to column and middle strips.

3.3 Column Design

Columns for all buildings were sized to have 600 mm square sections. This information was used in structural analysis. However, it was not necessary to design columns for reinforcement. The objective of the project is to investigate slab behaviour following the removal of a column. Therefore, the column strength was not required in progressive collapse analysis, and hence they were not designed for strength.

Chapter 4

Progressive Collapse Analysis

4.1 General

The first measure that should be considered for mitigating and/or preventing risk associated with progressive collapse is the implementation of soft measures to minimize and/or eliminate the level of threat. This can be done through protection, controlled access and/or increased standoff (ex: Use of bollards). However, this is not always possible. Therefore, hard measures, either in the form of hardening critical supports (columns of frame buildings) or strengthening of the framing elements (beams and/or slabs) should be considered as part of risk mitigation measures.

There are techniques to assess the progressive collapse potential of a building. These techniques are outlined in two U.S. guidelines, as described below:

- The U.S. General Services Administration (GSA) published as "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects (GSA 2003)".
- The U.S. Department of Defense (DoD) published as "Design of Buildings to Resist Progressive Collapse (DoD 2005)".

The purpose of the first document is to account for the progressive collapse in government buildings while the second document is intended for military structures. Another difference between the two is that GSA considers the removal of one column from the ground level due to bomb blast or vehicles attack, while DoD considers the removal of any vertical supporting element at any storey, one at a time, due to aimed missile attacks. Other differences include the level of protection and whether there is exemption, tie requirements, loads for static and dynamic analysis, upward loads on floor slabs and the method of analysis, whether it is linear static, nonlinear static, linear dynamic or nonlinear dynamic (Bilow and Kamara 2005).

The DoD and the GSA guidelines supply the designer with all the information needed to investigate the vulnerability of the structure to progressive collapse. This includes different cases of column removal (corner, exterior or interior column), the loading pattern, as well as the analysis procedure and the assessment and evaluation criteria. After the assessment, if the results are found to be satisfactory, it would be concluded that the structure would be resistant to progressive collapse. On the other hand, if the evaluation results indicate unsatisfactory performance, corrective measures should be taken and the evaluation should be made based on the new design.

The GSA approach is used in the current study, as it is more relevant to civilian structures, which constitute the scope of this investigation. A brief review of the GSA guidelines is provided in the following section.

4.2 GSA Guidelines

GSA guidelines investigate the demand/capacity ratios (DCRs) resulting from linear-elastic static analysis of the structure after the failure of one of the structural supporting systems. In most cases this translates into a column loss. Nonlinear dynamic analysis may also be used for more detailed investigation.

4.2.1 Evaluation Criteria

The evaluation criterion involves the computation of demand/capacity ratio (DCR). This is calculated according to the following equation:

$$\mathrm{DCR} = \left(\frac{Q_{UD}}{Q_{CE}} \right)$$

Where:

Q_{UD} is the factored force or demand (moment, shear, axial force…) acting on an element or area.

Q_{CE} is the expected capacity or resistance of the section or element, designed according to the code of practice, by considering different load combinations.

The GSA guidelines suggest the following condition to be satisfied to declare the structure to be safe against progressive collapse:

- DCR ≤ 2.0 for structures with typical or regular configurations.
- DCR ≤ 1.5 for structures with atypical or irregular configurations.

The guideline defines atypical structure as a structure that may contain one or more of the following configurations:

- Combination structures
- Vertical discontinuities/transfer girders
- Variations in bay size/extreme bay sizes
- Plan irregularities
- Closely spaced columns

The guidelines also suggest that, if localization of the irregularity can be made by engineering judgment, that identified area may be treated considering the ratios of less than or equal to 1.5, while the rest of the typical area can be evaluated based on the ratio of 2.0 (GSA 2003).

4.2.2 Strength Increase Factor

The GSA Guidelines allow the use of strength increase factors based on Table 4.1 to account for high strain rate effects on materials. As a result, the capacity of section may be recalculated using the strength increase factors, which in turn may produce higher capacity for a given section.

4.3 Progressive Collapse Analysis of the Slab Systems Considered

After the completion of slab design, as presented in the previous chapter, the following steps were followed to conduct the progressive collapse analysis:

Table 4.1 Strength increase factor for different materials (GSA 2003)

Construction Material	Strength Increase Factor
Reinforced Concrete	
Concrete Compressive Strength	1.25
Reinforcing Steel (tensile and yield strength)	1.25
Concrete Unit Masonry	
Compressive Strength	1.0
Flexural Tensile Strength	1.0
Shear Strength	1.0
Wood and Light Metal Framing	
All Components	1.0

1. Software ETABS was used to conduct linear elastic analysis of 1, 5, 10 and 15 storey buildings as per GSA requirements for column removals between the base and the first storey (ground floor level):
   - Corner column removal.
   - Edge column removal (from longitudinal and transverse direction when span ratios exceeded 1.0).
   - Interior column removal.

   The load combination of 2 (Dead Load) + 0.5 (Live Load) was used on all floors according to the GSA 2003 requirement. This step provided slab moment demands.

2. Following the completion of ETABS analysis of the entire structure, the results for the first-storey slab was imported into SAFE and analyzed using the load combination suggested by the GSA approach for progressive collapse analysis. Only the first storey slab was considered since it experienced the largest demands compared to slabs in higher stories, as verified and illustrated in Section 4.4. This step provided slab moment capacities. The thickness used in capacity calculations for the negative reinforcement was either equal to the slab thickness (for flat plates) or equal to the drop panel thickness (for flat slabs). In both cases, the effective depth (lever arm) was calculated as shown below, with a clear cover of 20 and bar diameter of 200 mm:

$$d_e = h - \text{clear cover} - \frac{\text{diameter of one bar}}{2}$$

3. Calculation of demand/capacity ratios at different locations by dividing the moment demands obtained in Step 1 by the moment capacity values of designed sections obtained in Step 2.

4.4 Verification of the Use of First Storey Slab for Multi-Storey Buildings

DCRs were always calculated for the first storey slab in the case of multi-storey buildings since it represents the most critical floor as demand/capacity ratios decrease in upper floors. However, this was verified through analysis. Interior column B2 was removed from the ground floor of a 10 storey building with slab Type FP-7x7, as shown in Fig. 4.1 for this purpose. Moment DCRs were investigated at locations 1, 2, 3, and 4 representing negative moment regions; and 5 representing positive moment regions at each floor slab of the 10 storey building. The results, shown in Table 4.2, clearly show a decrease of positive and negative DCRs with increasing distance from the first floor (a column of which has been removed). This is in conformity with GSA guidelines, which require only the removal of the first storey columns. The capacities used in DCRs given in Table 4.2 are based on Chapter 13 of CSA A23.3-04.

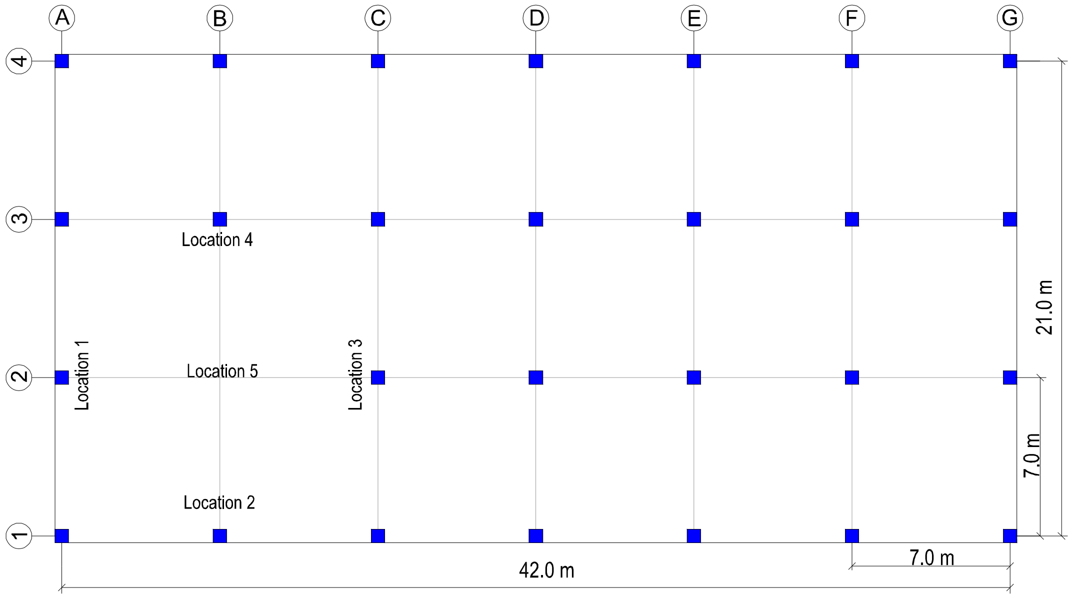


Fig. 4.1 Removal of interior column B2.

Table 4.2 DCRs at Locations 1, 2, 3, 4 and 5 of 10 storey building.

Number of storey	Demand/capacity ratios				
	Location 1	Location 2	Location 3	Location 4	Location 5
10th	0.91	0.91	1.22	1.21	2.96
9th	0.99	0.99	1.25	1.23	2.98
8th	0.99	0.98	1.26	1.25	3.01
7th	1.00	1.00	1.28	1.28	3.06
6th	1.02	1.01	1.32	1.32	3.15
5th	1.03	1.03	1.36	1.36	3.49
4th	1.06	1.05	1.42	1.42	3.92
3rd	1.08	1.08	1.47	1.50	4.43
2nd	1.12	1.12	1.56	1.59	5.06
1st	1.13	1.13	1.64	1.67	5.68

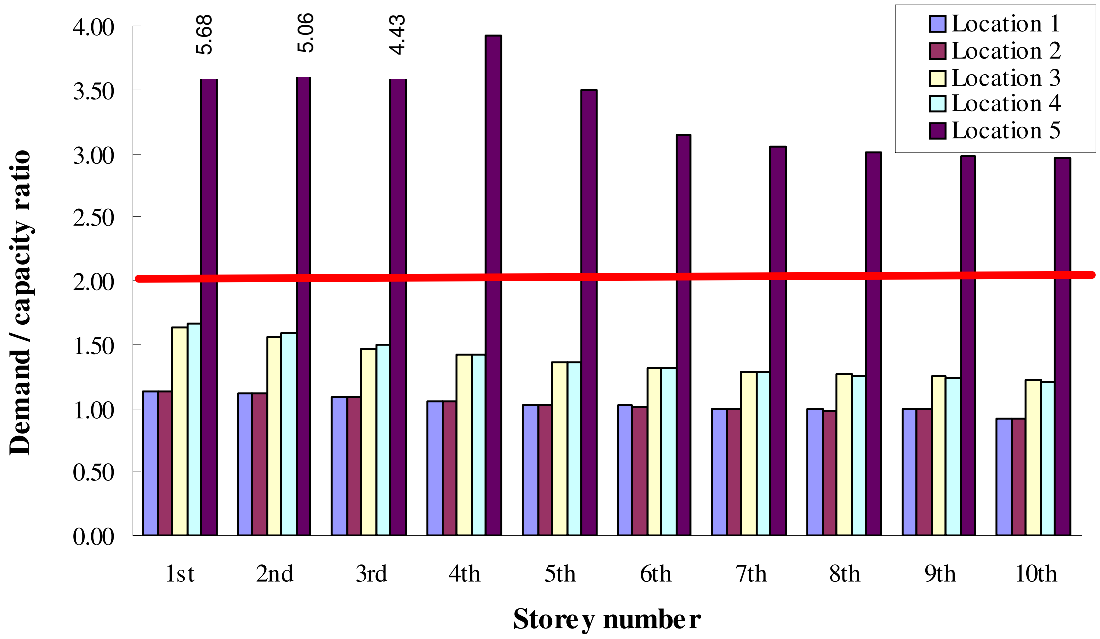


Fig. 4.2 DCRs for all stories after removing the first-storey interior column of the 10 storey building FP-7x7

4.5 Results of Progressive Collapse Analyses

Results of progressive collapse analysis for slabs with and without seismic detailing are presented in this section in the following sequence:

1. FP-7x7 (Span Ratio of 1:1)
    - Case 1 (FP-7x7-C): Corner column removed.
    - Case 2 (FP-7x7-E): Edge column removed.
    - Case 3 (FP-7x7-I): Interior column removed.
2. FP-7x3.5 (Span Ratio of 2:1)
    - Case 1 (FP-7x3.5-C): Corner column removed.
    - Case 2 (FP-7x3.5-EL): Edge column removed in longitudinal direction.
    - Case 3 (FP-7x3.5-ET): Edge column removed in transverse direction.
    - Case 4 (FP-7x3.5-I): Interior Column removed.
3. FS-7x7 (Span Ratio of 1:1)
    - Case 1 (FS-7x7-C): Corner column removed.
    - Case 2 (FS-7x7-E): Edge column removed.
    - Case 3 (FS-7x7-I): Interior column removed.
4. FS-7x3.5 (Span Ratio of 2:1)
    - Case 1 (FS-7x3.5-C): Corner column removed.
    - Case 2 (FS-7x3.5-EL): Edge column removed in longitudinal direction.
    - Case 3 (FS-7x3.5-ET): Edge column removed in transverse direction.
    - Case 4 (FS-7x3.5-I): Interior Column removed.
5. FS-5x5 (Span Ratio of 1:1)
    - Case 1 (FS-5x5-C): Corner column removed.
    - Case 2 (FS-5x5-E): Edge column removed.
    - Case 3 (FS-5x5-I): Interior column removed.
6. FS-9x9 (Span Ratio of 1:1)
    - Case 1 (FS-9x9-C): Corner column removed.
    - Case 2 (FS-9x9-E): Edge column removed.
    - Case 3 (FS9x9-I): Interior column removed.

Values of DCRs in the negative moment region were investigated at following locations:

- At the center of the support (column).
- At a distance 0.20 l_n from the face of the support.
- At 0.30 l_n from the face of the support for flat plates.
- At 0.33 l_n from the face of the support for flat slabs.

These locations were chosen because of the reinforcement detailing required by Clause 13.10.8 of CSA A23.3-04, which changes slab capacities at these locations. Clause 13 requires that 100% of the negative reinforcement to be continuous up to a distance equal to 0.20 l_n from the face of the support. Therefore, the amount of reinforcement used to calculate the capacity at the center of the column is 100% of the negative reinforcement used in detailing that section, while 50% was used at the point where the distance is equal to 0.20 l_n. The remaining 50% of negative reinforcement that extends from 0.20 l_n to a distance of 0.30 l_n for flat plates and 0.33 l_n for flat slabs is discontinued beyond these locations with zero negative moment capacity. The DCR ratios computed for all cases considered are presented in Appendix B. When there is no moment capacity provided because of lack of reinforcement the Tables indicate whether a positive or negative moment (+M or –M) demand was observed in the analysis. Also, the location of zero negative moment is indicated to determine if negative reinforcement needs to be extended beyond 0.30 l_n for flat plates and 0.33 l_n for flat slabs. Refer to Appendix B for more details.

Positive moment demand was investigated at locations of column removal**.** The **r**einforcement used to compute positive flexural capacity at these locations was the structural integrity reinforcement since this is the only bottom reinforcement continuous over the support region when seismic detailing is not implemented. The slab capacity is improved in these regions when reinforcement continuity is improved due to seismic detailing. Smaller DCRs were computed for positive moment regions at column removal locations. Also, there was no need to determine the location of zero negative moment in these regions since there would always be top and bottom continuous reinforcement in seismically detailed slabs. The value of negative moment DCR at the center of the column, as well as at 0.20 l_n from the face of the support are unaffected by seismic detailing. For the majority of the slabs with seismic detailing, 33% of the top reinforcement at the location of the column was larger than 50% of the bottom reinforcement at midspan. Therefore for most cases, the amount of negative reinforcement at the location of the support governed the amount of the bottom reinforcement to be added at column locations. The only exception was observed when the section was designed based on minimum reinforcement.

Figures 4.3 through 4.9 show the column removal scenarios for slabs with different span ratios. The results of progressive collapse analysis, corresponding to these scenarios for flat plates and flat slabs, with or without seismic detailing are shown in Figs. 4.10 through 4.43 are presented. Negative DCRs at the location of center of columns as well as positive DCRs are presented in the figures in this chapter. For more details, refer to tables in Appendix B.

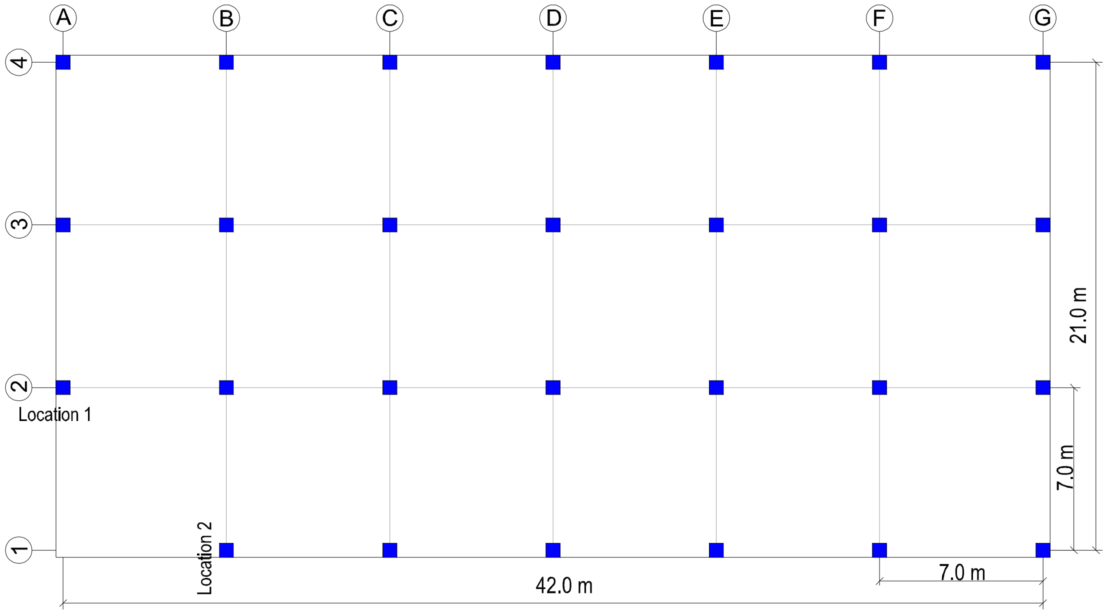


Fig. 4.3 Removal of corner column A1

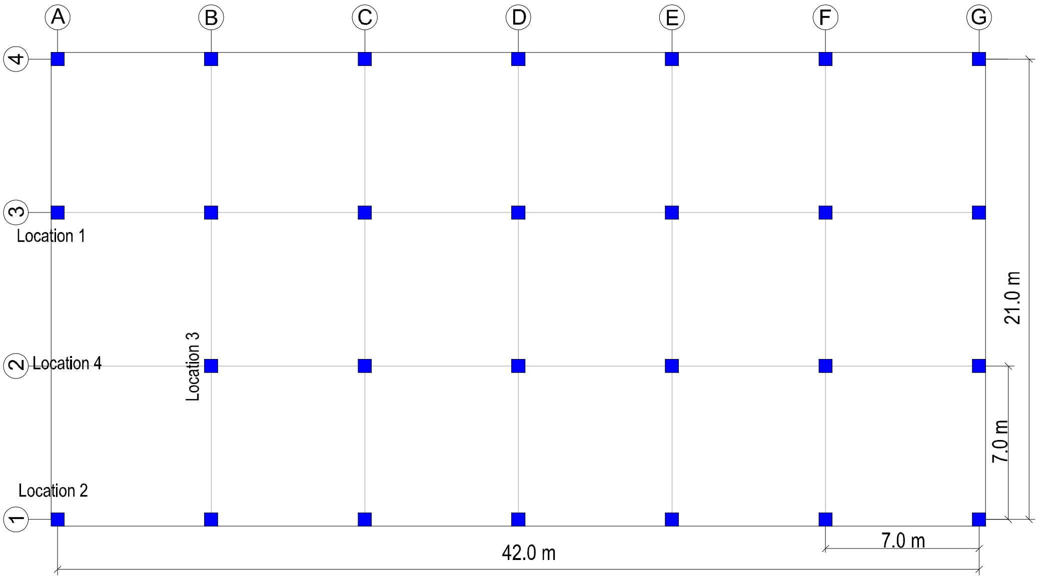


Fig. 4.4 Removal of Edge column A2

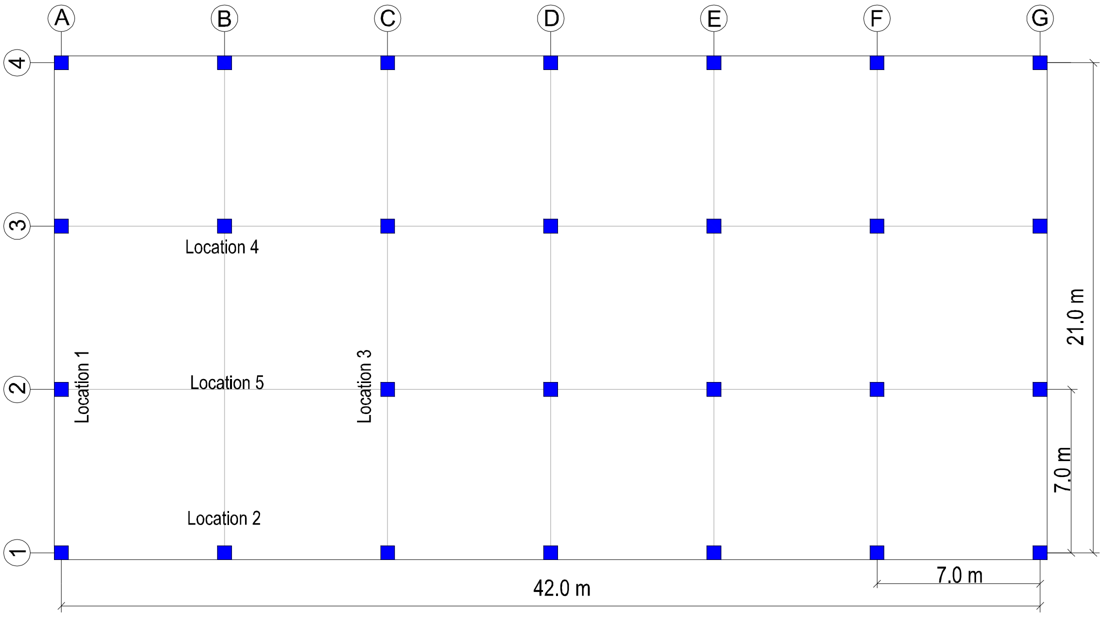


Fig. 4.5 Removal of interior column B2

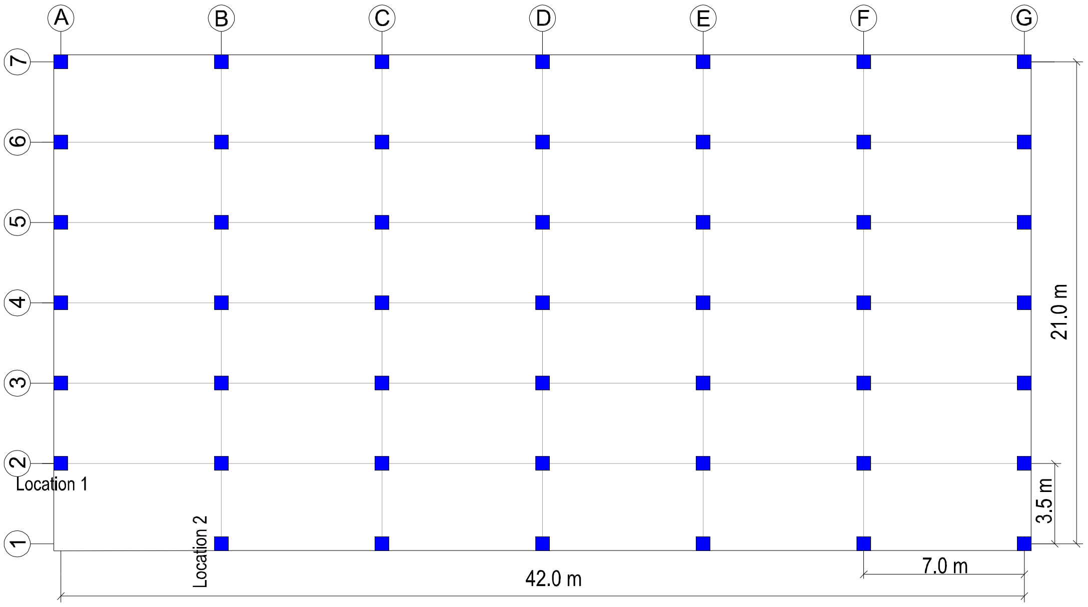


Fig. 4.6 Removal of corner column A1

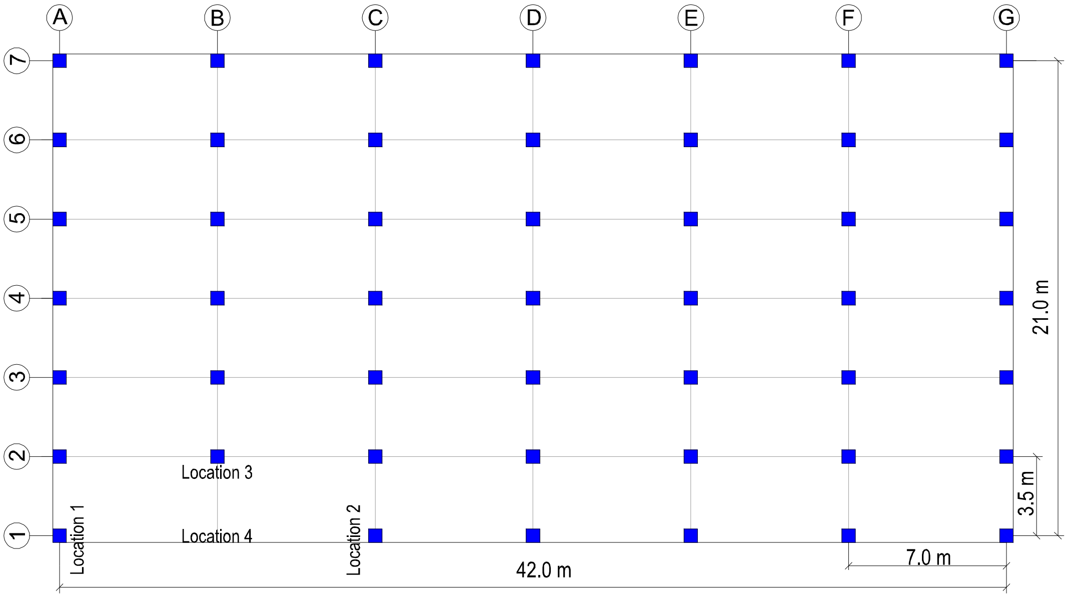


Fig. 4.7 Removal of edge column B1 in longitudinal direction

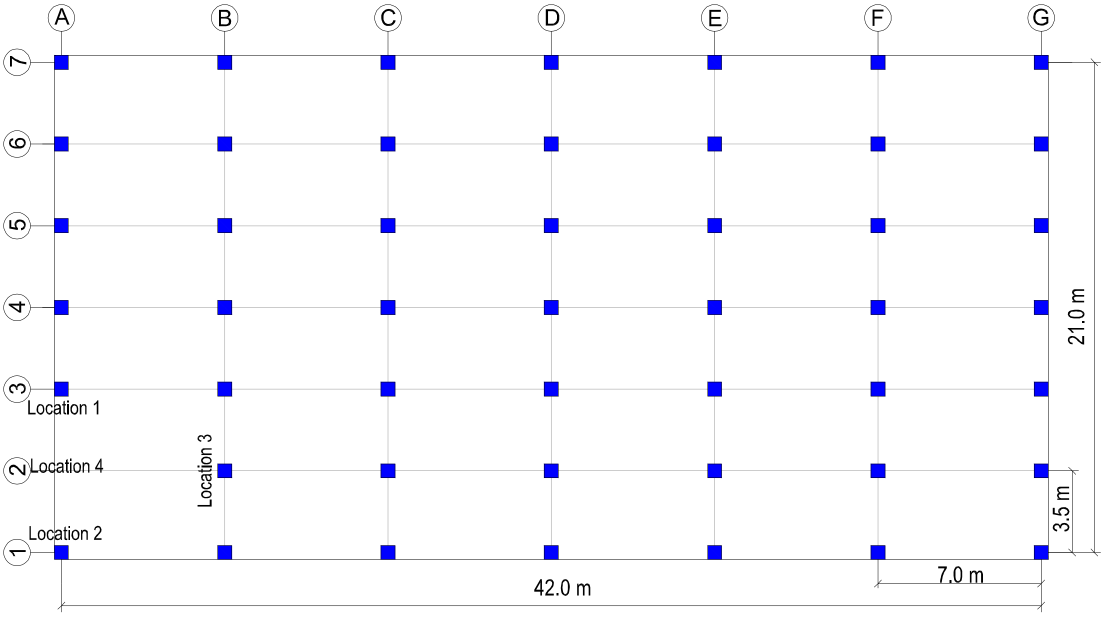


Fig. 4.8 Removal of edge column A2 in transverse direction

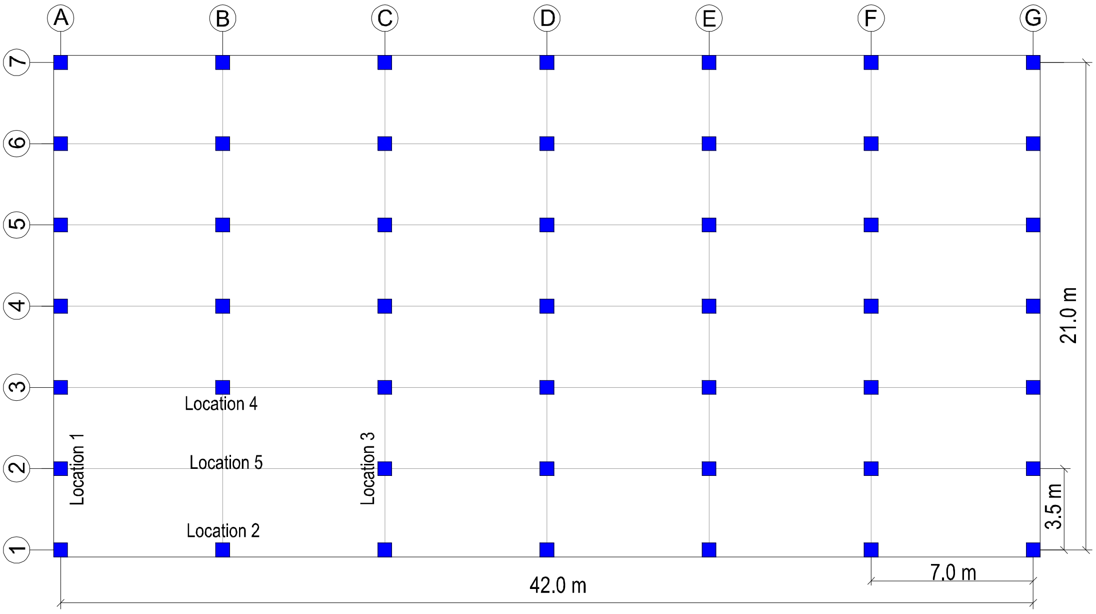


Fig. 4.9 Removal of interior column B2

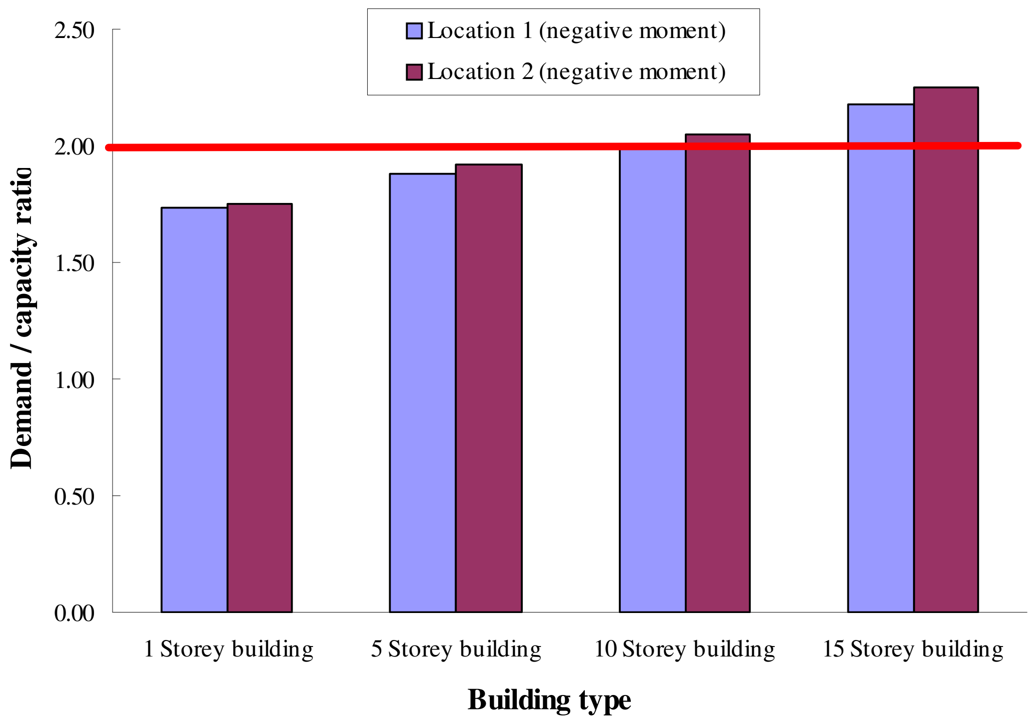


Fig. 4.10 DCRs for FP-7x7-C with corner column removed
(For locations 1 and 2 see Fig. 4.3)

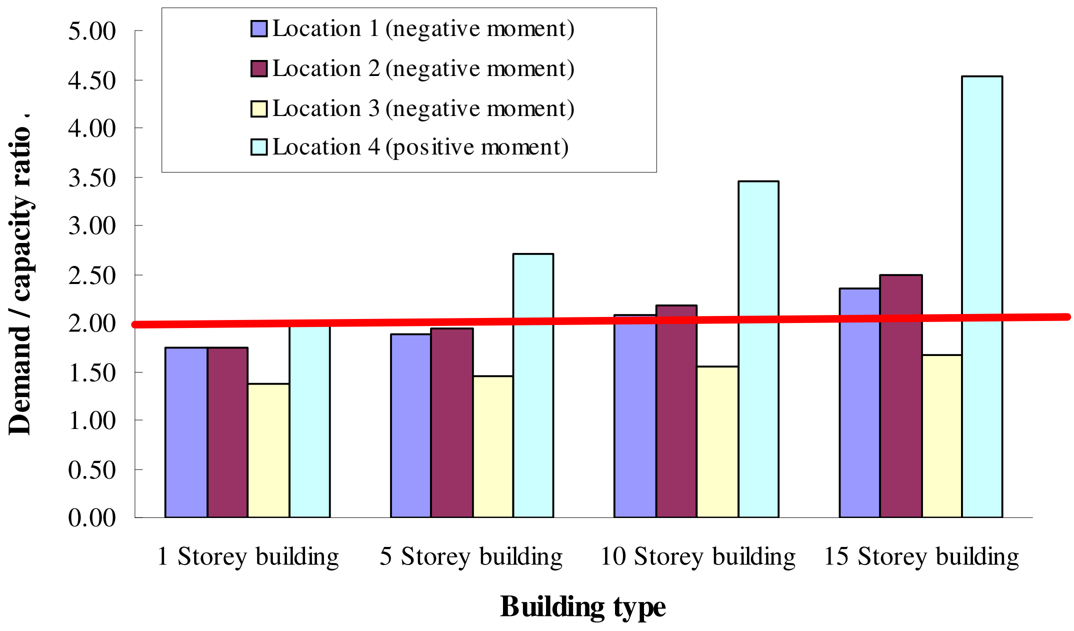


Fig. 4.11 DCRs for FP-7x7-E with edge column removed; slab with seismic detailing
(For locations 1 through 4 see Fig. 4.4)

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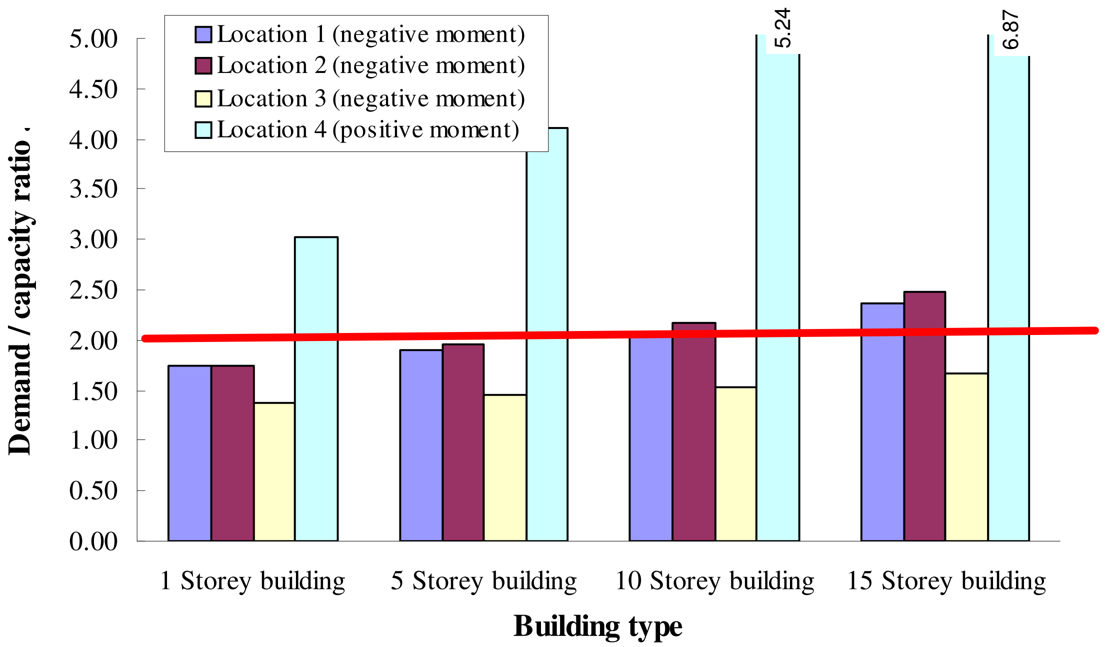

Fig. 4.12 DCRs for FP-7x7-E with edge column removed; slab without seismic detailing (For locations 1 through 4 see Fig. 4.4)

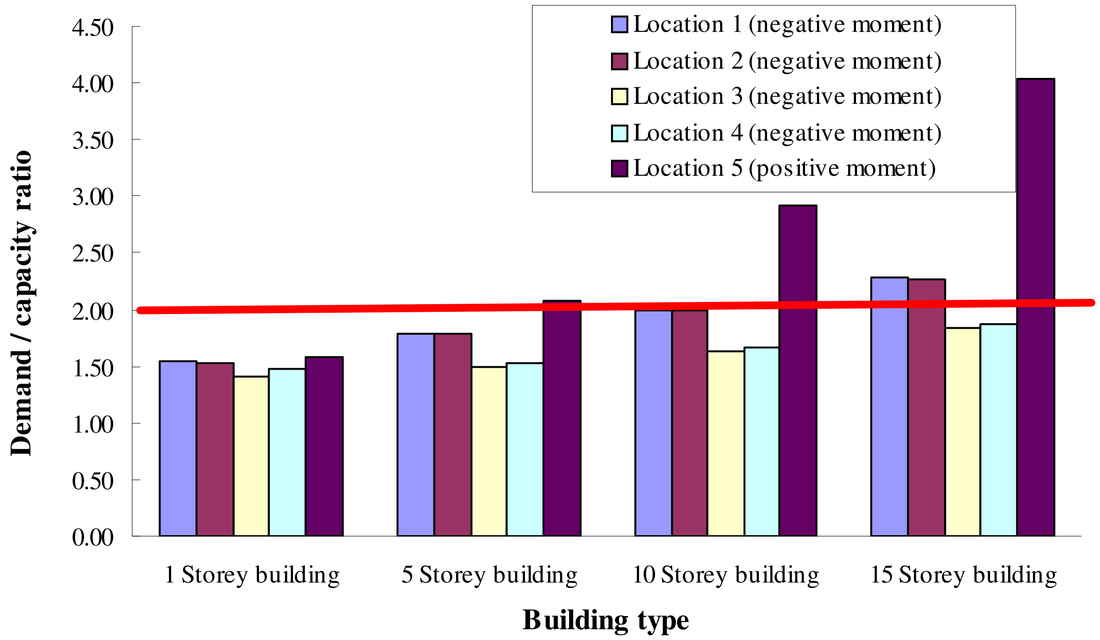

Fig. 4.13 DCRs for FP-7x7-I with interior column removed; slab with seismic detailing (For locations 1 through 5 see Fig. 4.5)

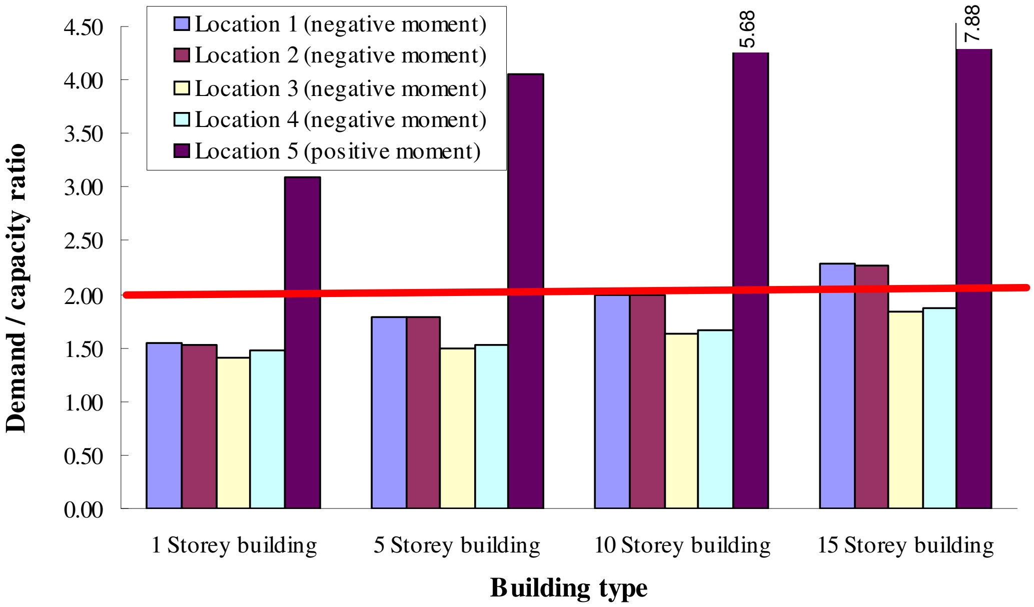

Fig. 4.14 DCRs for FP-7x7-I with interior column removed; slab without seismic detailing (For locations 1 through 5 see Fig. 4.5)

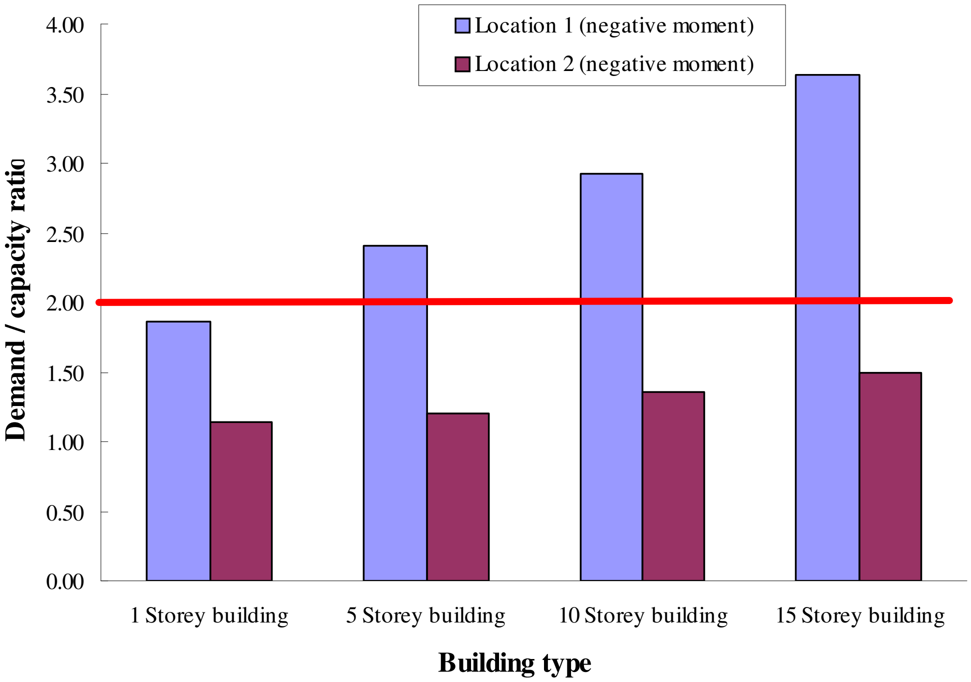

Fig. 4.15 DCRs for FP-7x3.5-C with corner column removed
(For locations 1 and 2 see Fig. 4.6)

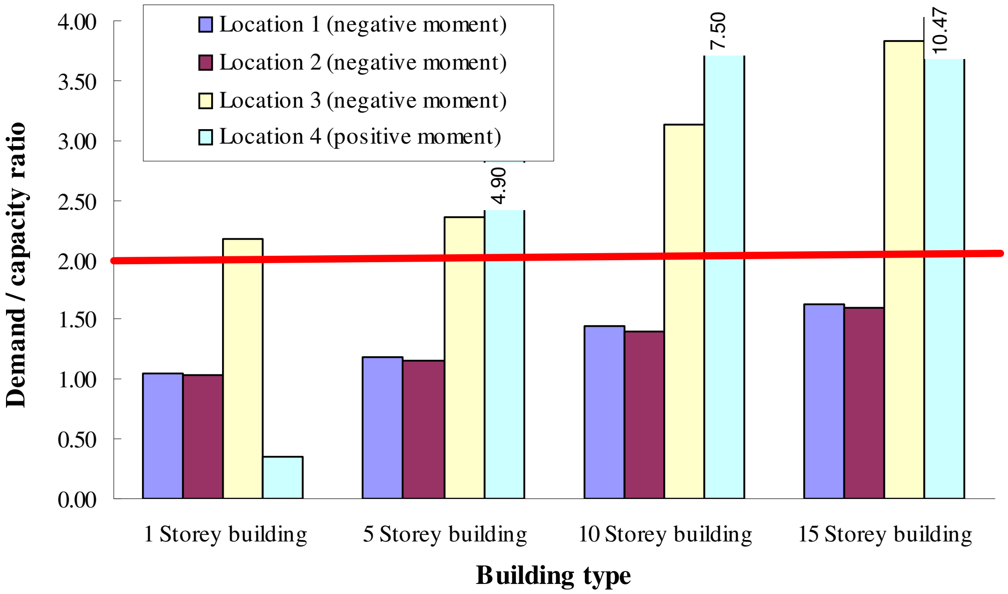

Fig. 4.16 DCRs for FP-7x3.5-EL with edge column in longitudinal direction removed; slab with seismic detailing (For locations 1 through 4 see Fig. 4.7)

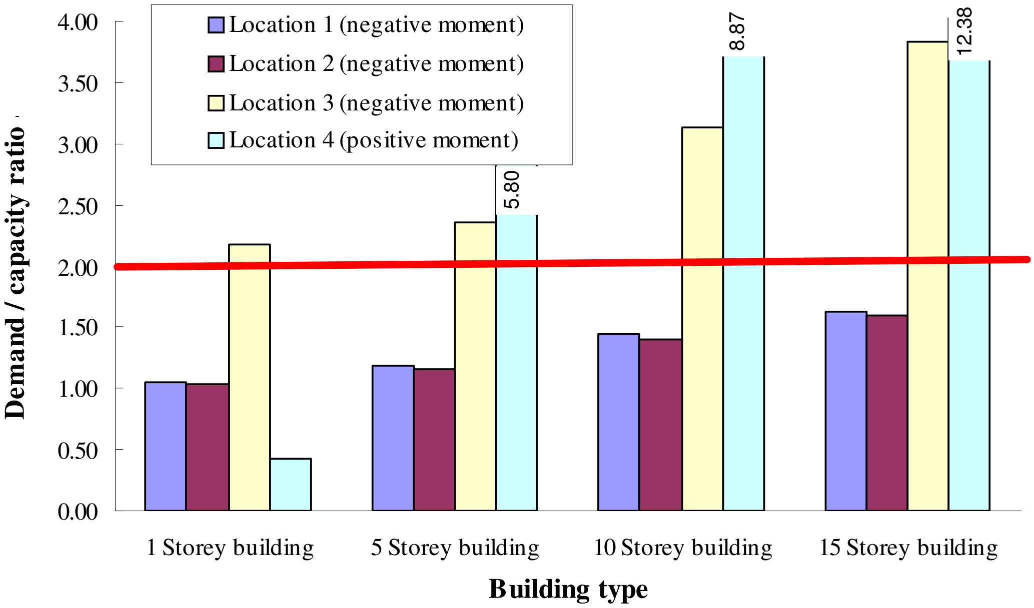

Fig. 4.17 DCRs for FP-7x3.5-EL with edge column in longitudinal direction removed; slab without seismic detailing (For locations 1 through 4 see Fig. 4.7)

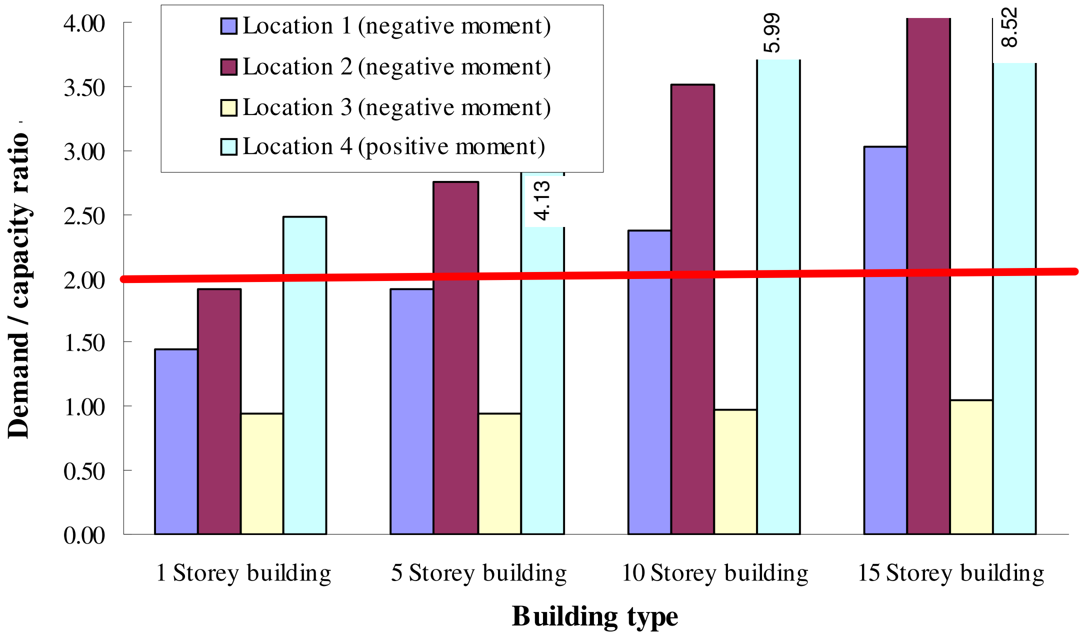

Fig. 4.18 DCRs for FP-7x3.5-ET with edge column in transverse direction removed; slab with seismic detailing (For locations 1 through 4 see Fig. 4.8)

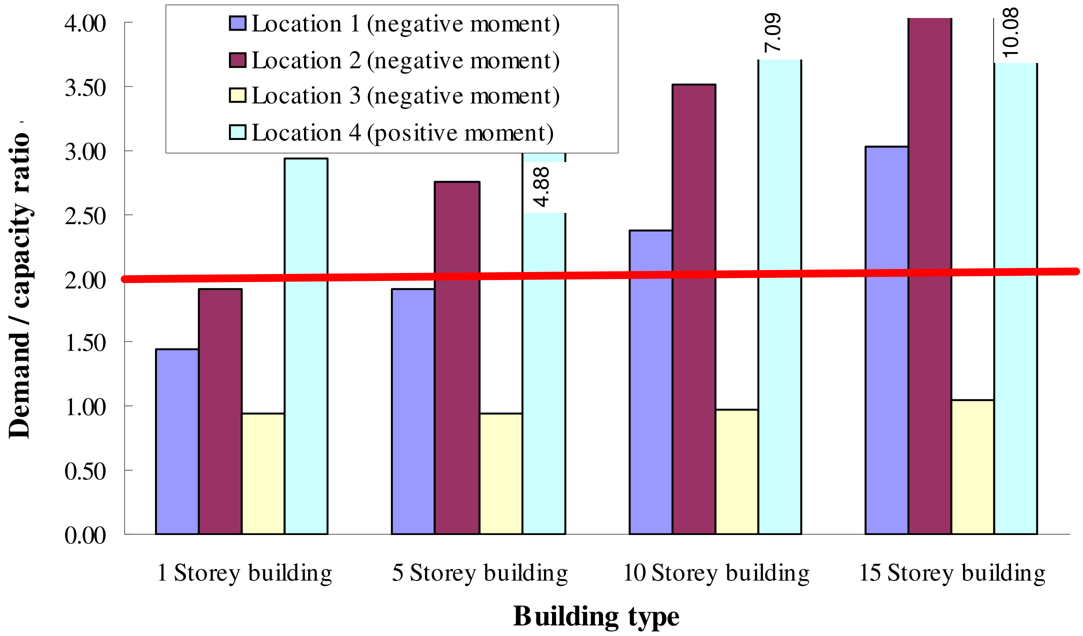

Fig. 4.19 DCRs for FP-7x3.5-ET with edge column in transverse direction removed; slab without seismic detailing (For locations 1 through 4 see Fig. 4.8)

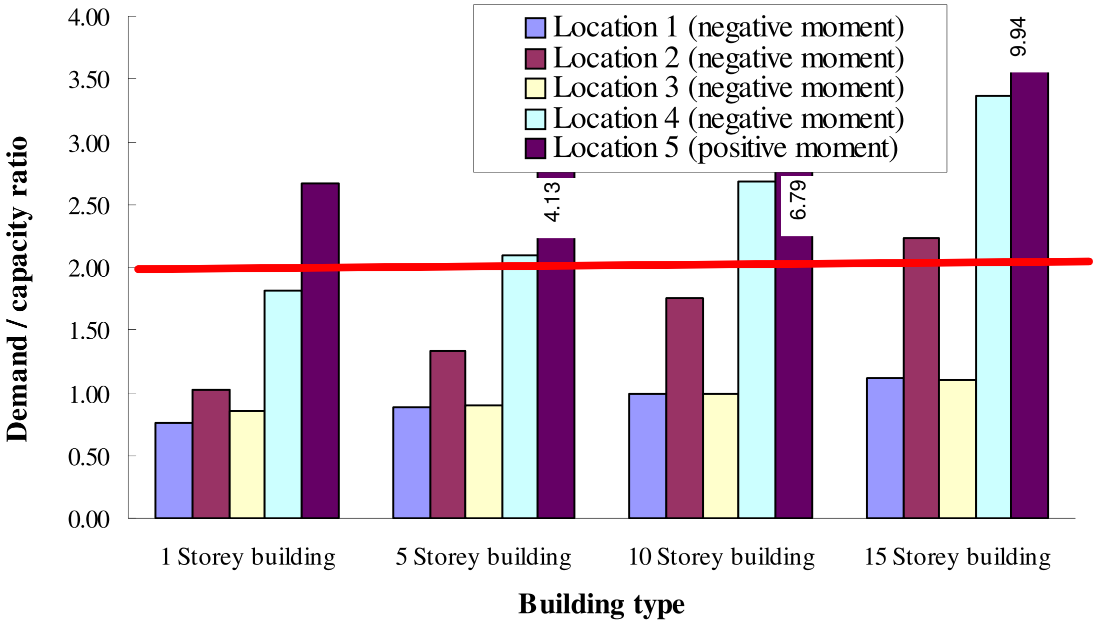

Fig. 4.20 DCRs for FP-7x3.5-I with interior column removed; slab with seismic detailing (For locations 1 through 5 see Fig. 4.9)

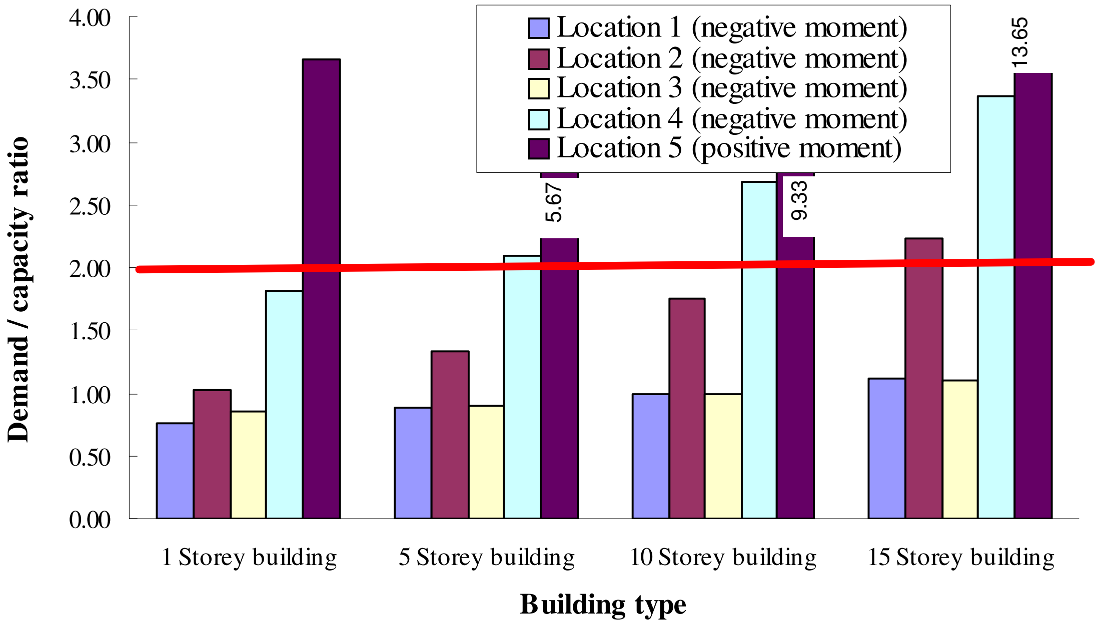

Fig. 4.21 DCRs for FP-7x3.5-I with interior column removed; slab without seismic detailing (For locations 1 through 5 see Fig. 4.9)

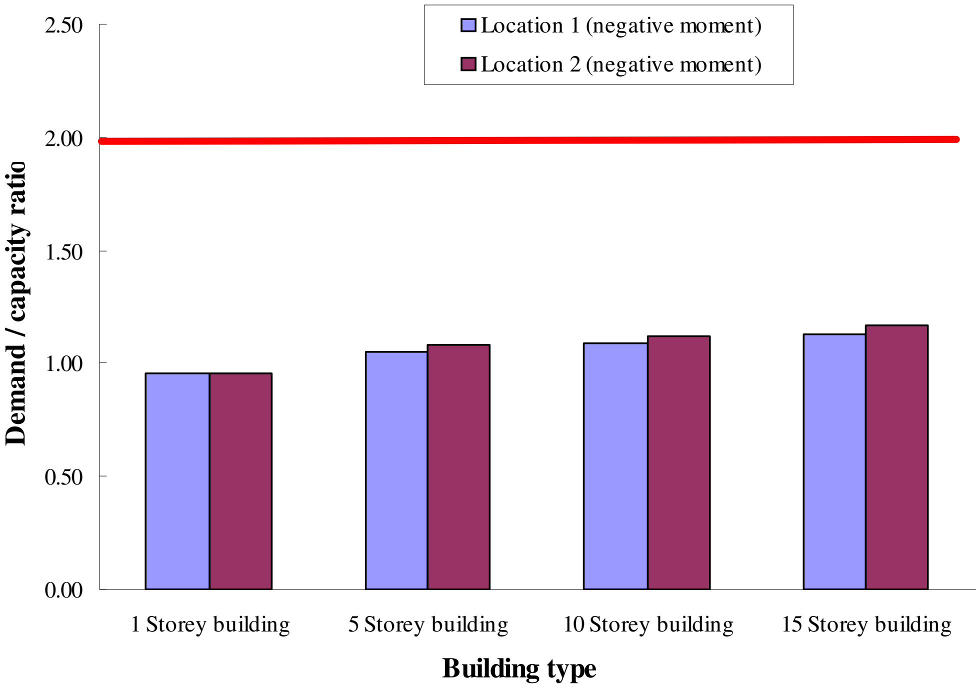

Fig. 4.22 DCRs for FS-7x7-C with corner column removed
(For locations 1 through 2 see Fig. 4.3)

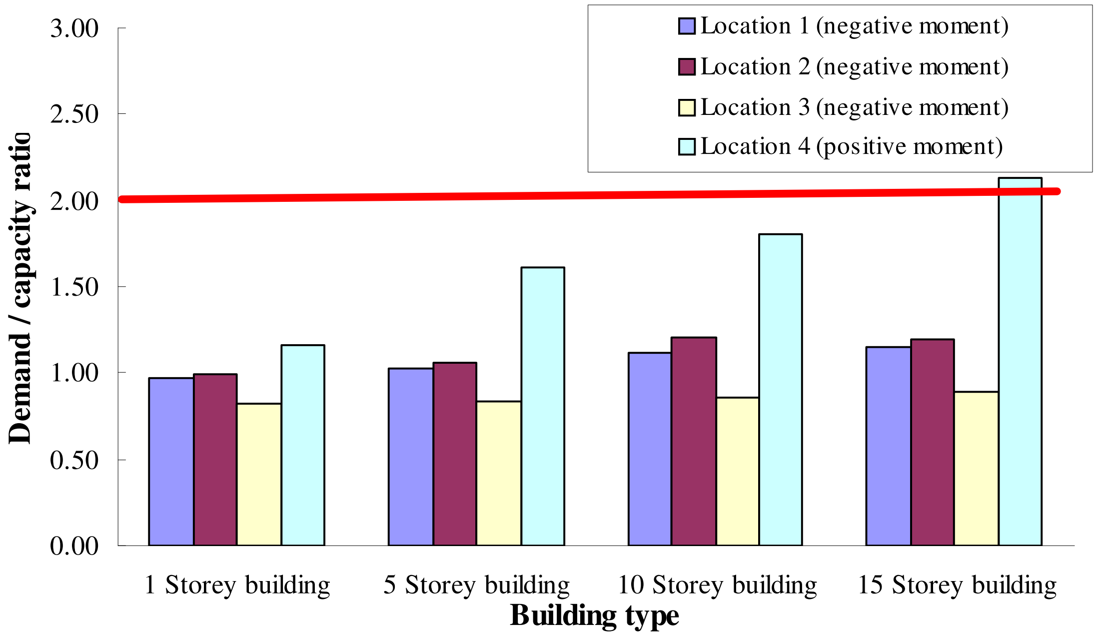

Fig. 4.23 DCRs for FS-7x7-E with edge column removed; slab with seismic detailing
(For locations 1 through 4 see Fig. 4.4)

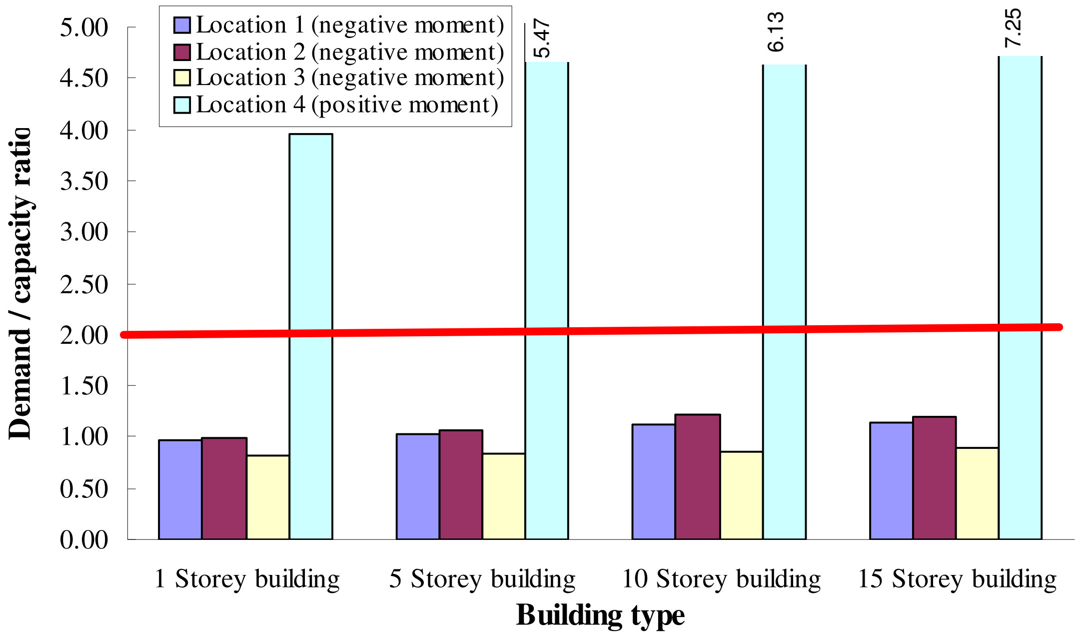

Fig. 4.24 DCRs for FS-7x7-E with edge column removed; slab without seismic detailing (For locations 1 through 4 see Fig. 4.4)

Fig. 4.25 DCRs for FS-7x7-I with interior column removed; slab with seismic detailing (For locations 1 through 5 see Fig. 4.5)

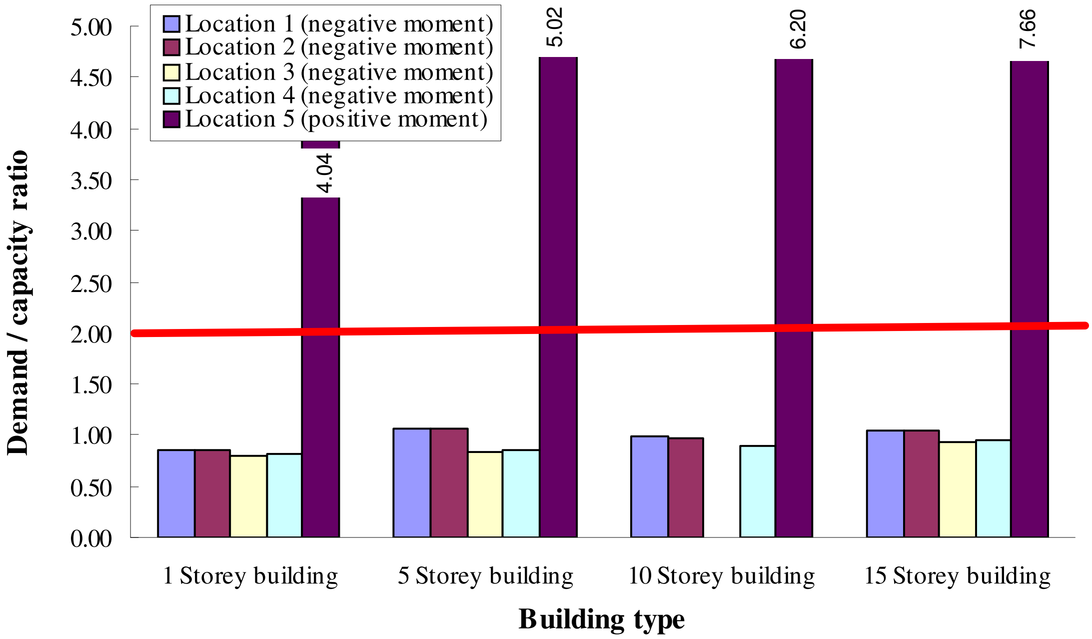

Fig. 4.26 DCRs for FS-7x7-I with interior column removed; slab without seismic detailing (For locations 1 through 5 see Fig. 4.5)

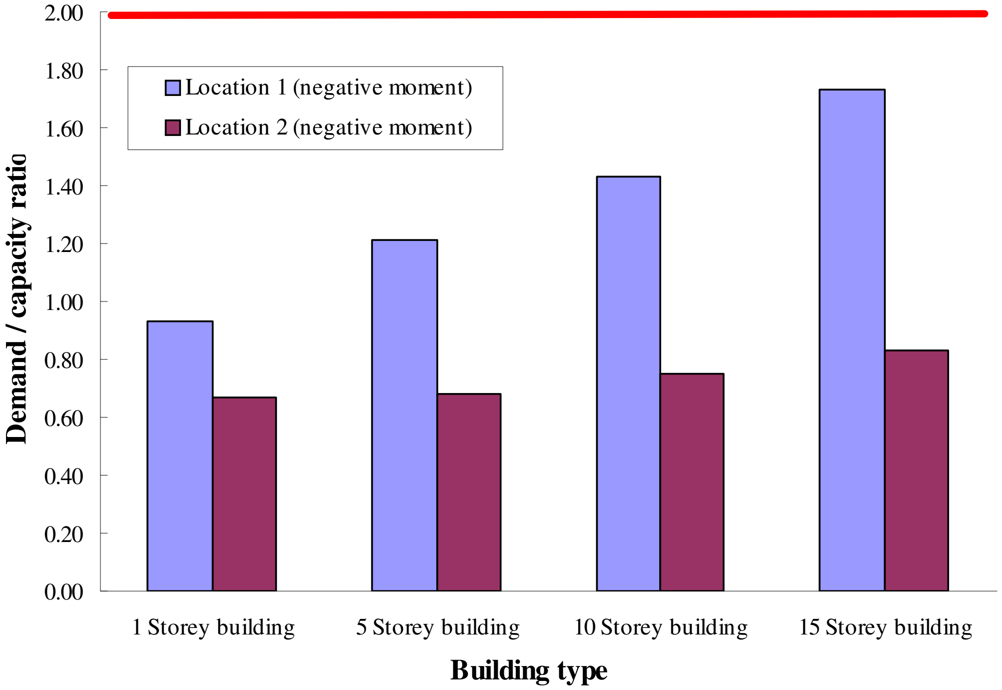

Fig. 4.27 DCRs for FS-7x3.5-C with corner column removed (For locations 1 and 2 see Fig. 4.6)

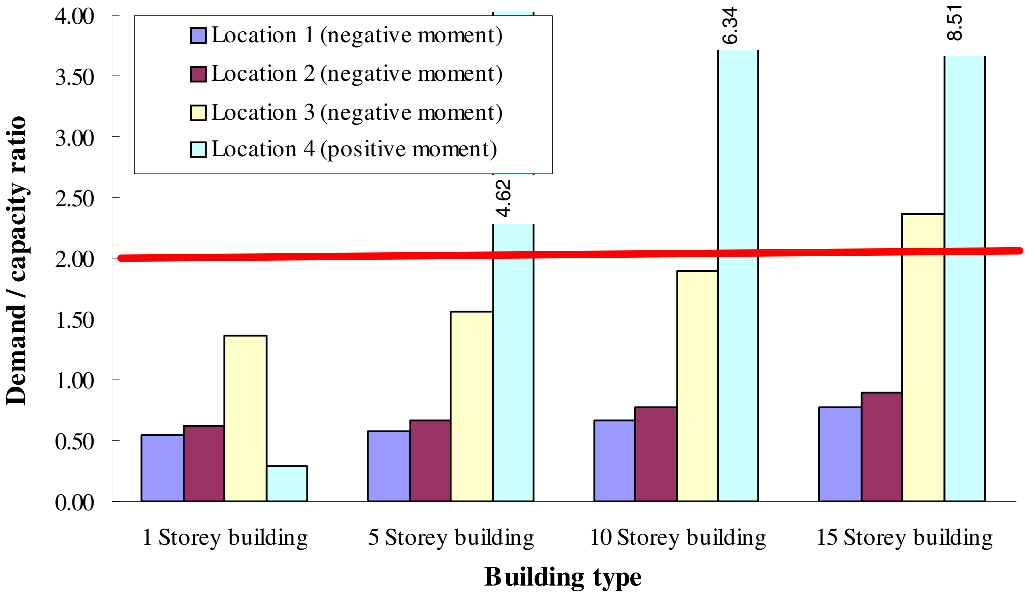

Fig. 4.28 DCRs for FS-7x3.5-EL with edge column in the longitudinal direction removed; slab with seismic detailing (For locations 1 through 4 see Fig. 4.7)

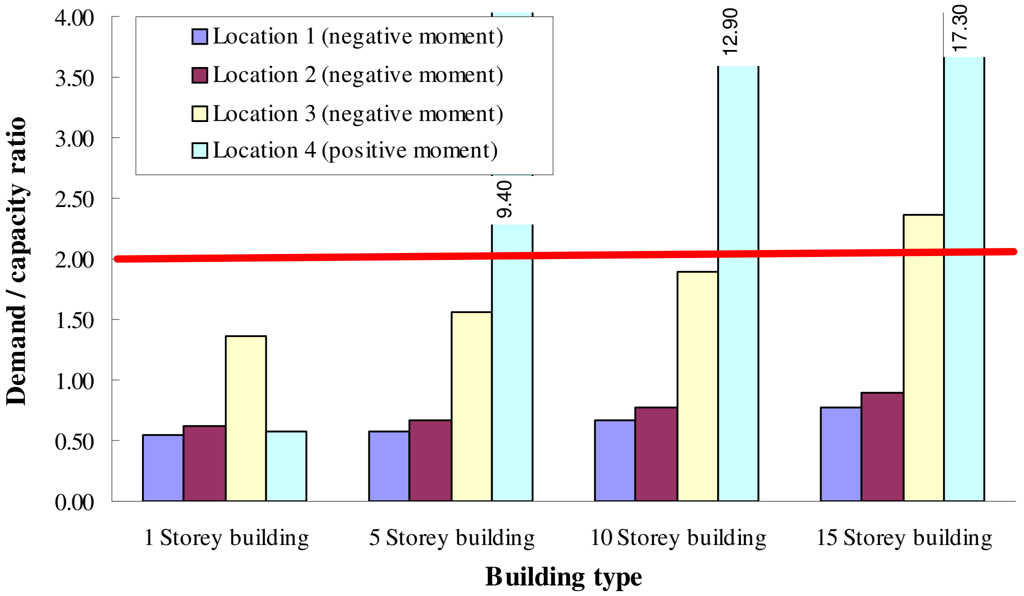

Fig. 4.29 DCRs for FS-7x3.5-EL with edge column in longitudinal direction removed; slab without seismic detailing (For locations 1 through 4 see Fig. 4.7)

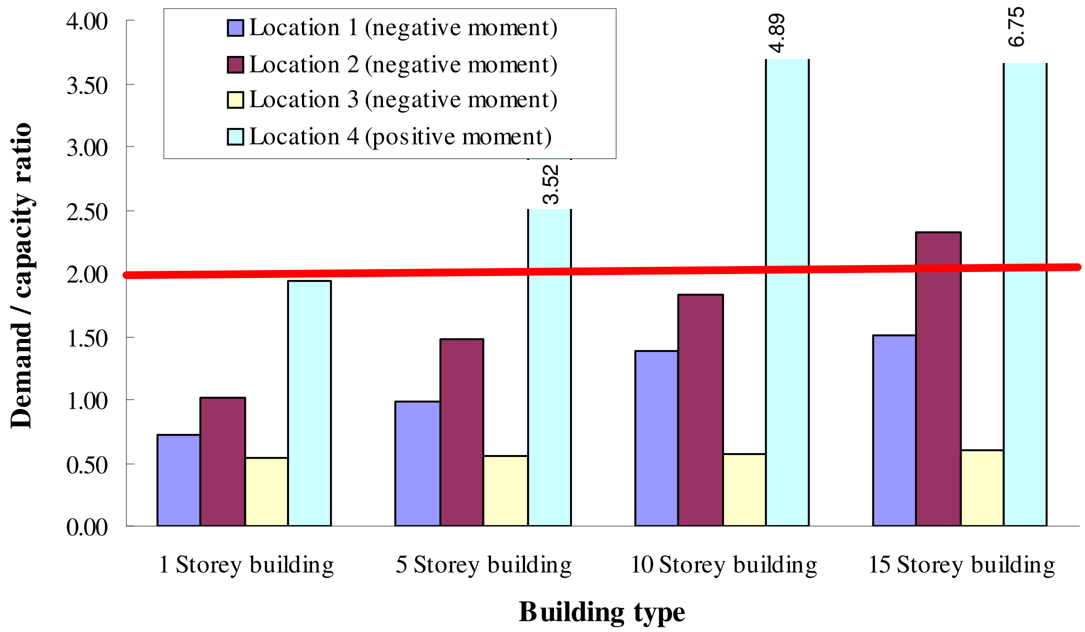

Fig. 4.30 DCRs for FS-7x3.5-ET with edge column in transverse direction removed; slab with seismic detailing (For locations 1 through 4 see Fig. 4.8)

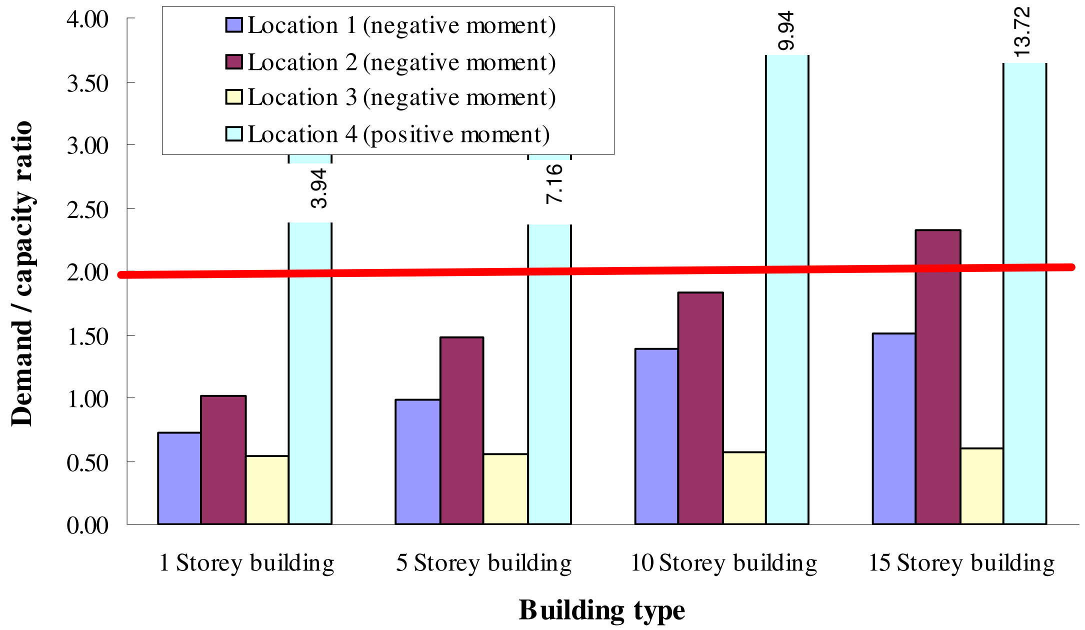

Fig. 4.31 DCRs for FS-7x3.5-ET with edge column removed in transverse direction; slab without seismic detailing (For locations 1 through 4 see Fig. 4.8)

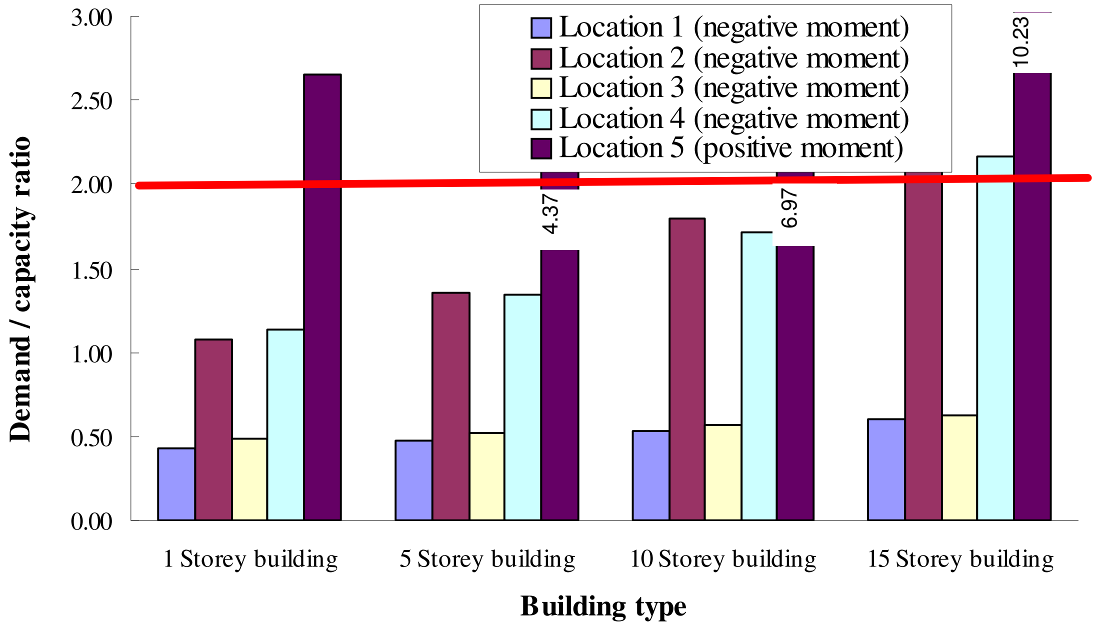

Fig. 4.32 DCRs for FS-7x3.5-I with interior column removed; slab with seismic detailing (For locations 1 through 5 see Fig. 4.9)

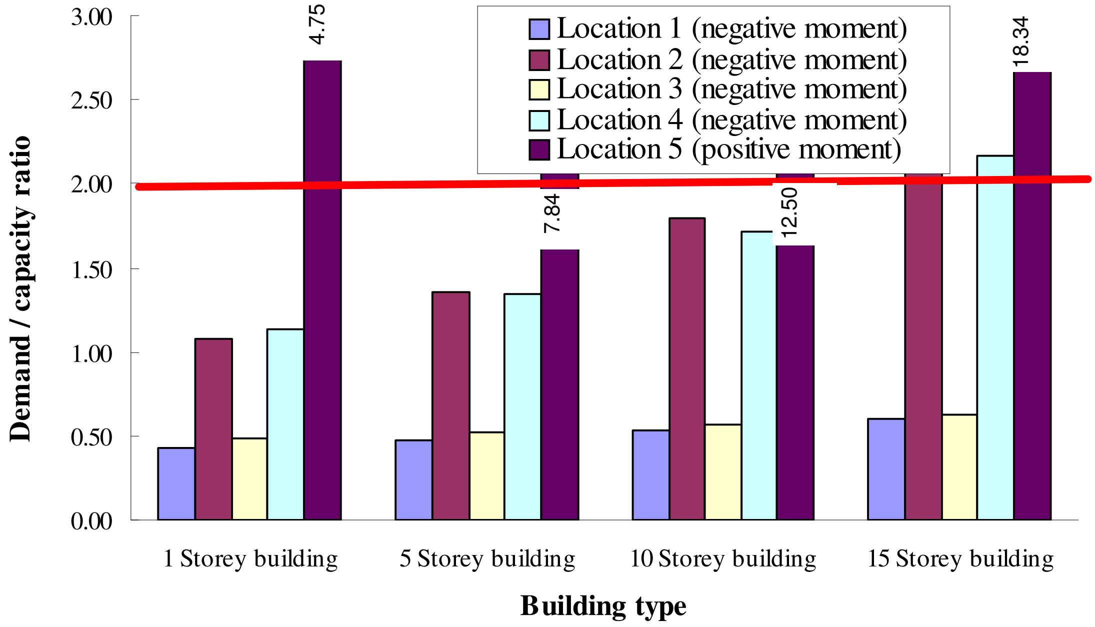

Fig. 4.33 DCRs for FS-7x3.5-I with interior column removed; slab without seismic detailing (For locations 1 through 5 see Fig. 4.9)

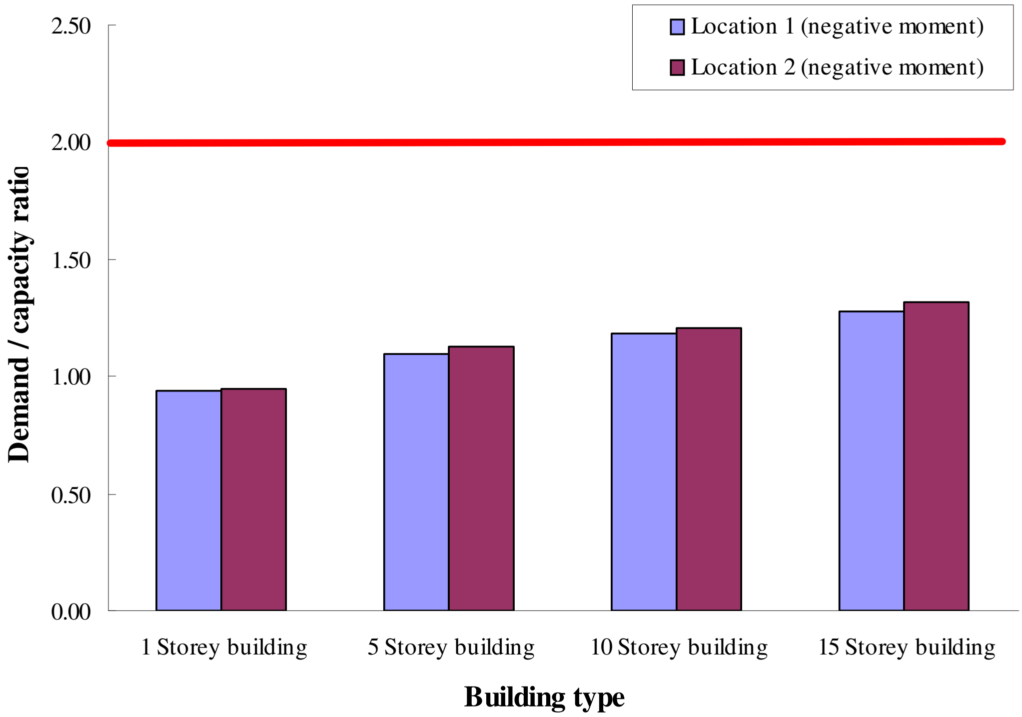

Fig. 4.34 DCRs for FS-5x5-C with corner column removed
(For locations 1 and 2 see Fig. 4.3)

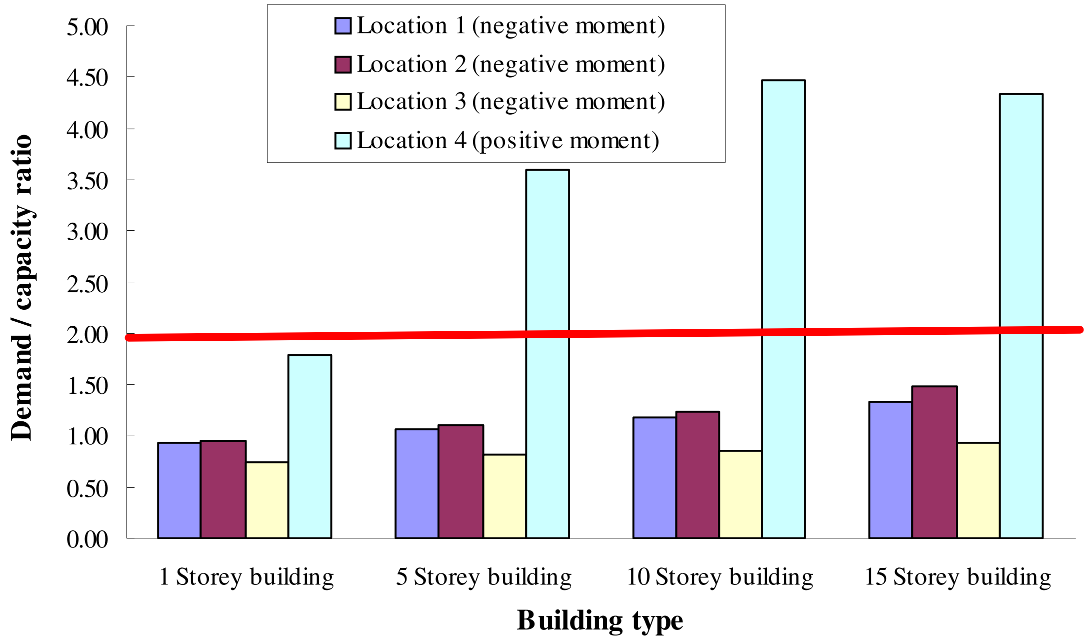

Fig. 4.35 DCRs for FS-5x5-E with edge column removed; slab with seismic detailing
(For locations 1 through 4 see Fig. 4.4)

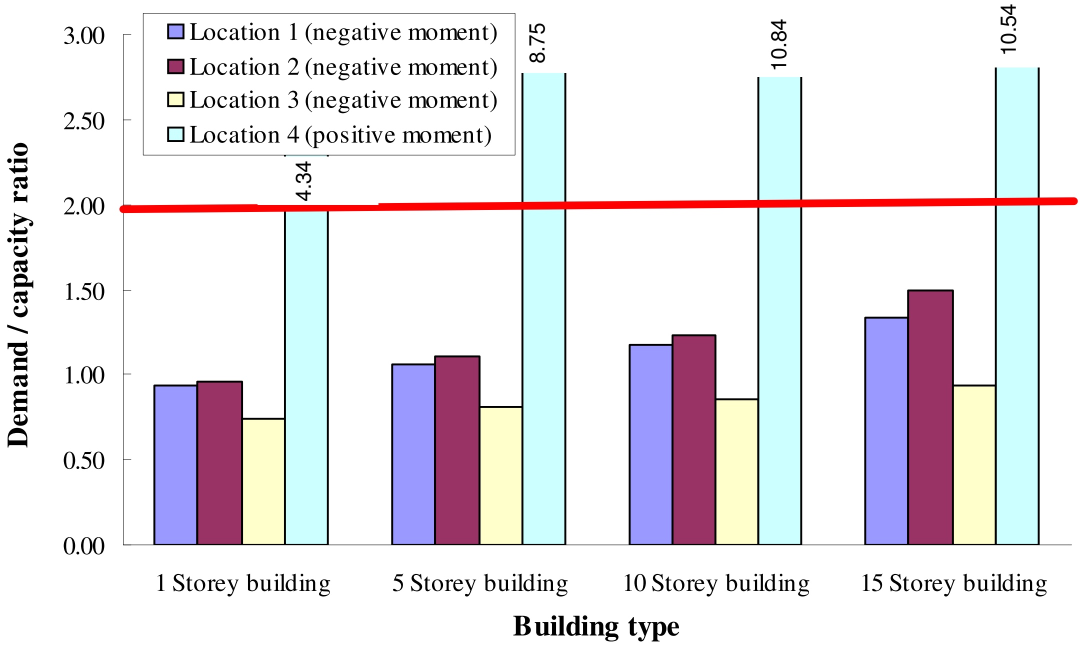

Fig. 4.36 DCRs for FS-5x5-E with edge column removed; slab without seismic detailing (For locations 1 through 4 see Fig. 4.4)

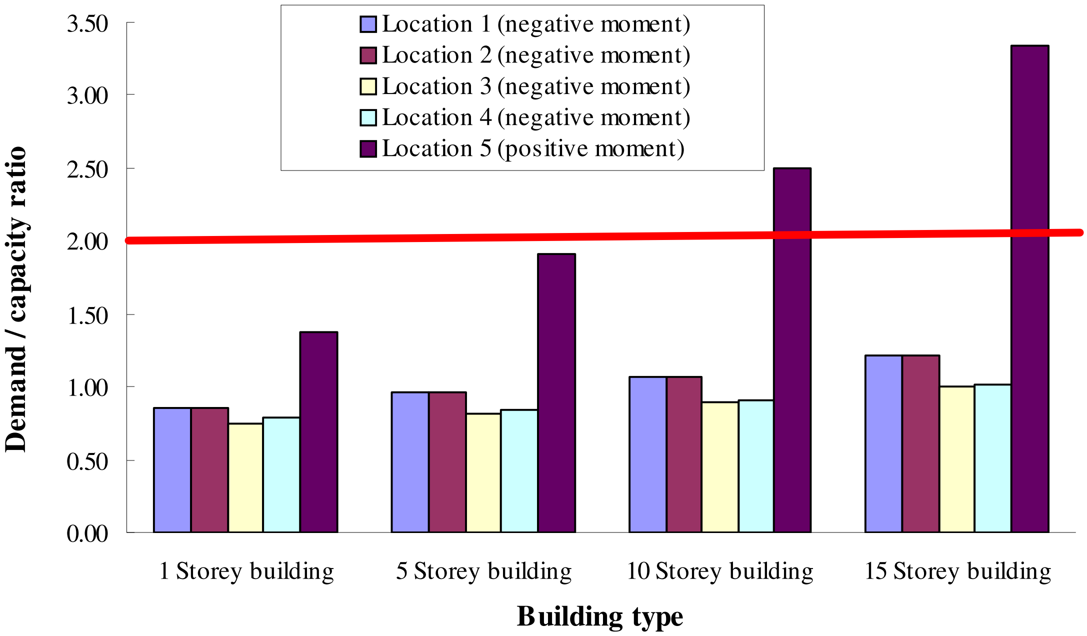

Fig. 4.37 DCRs for FS-5x5-I with interior column removed; slab with seismic detailing (For locations 1 through 5 see Fig. 4.5)

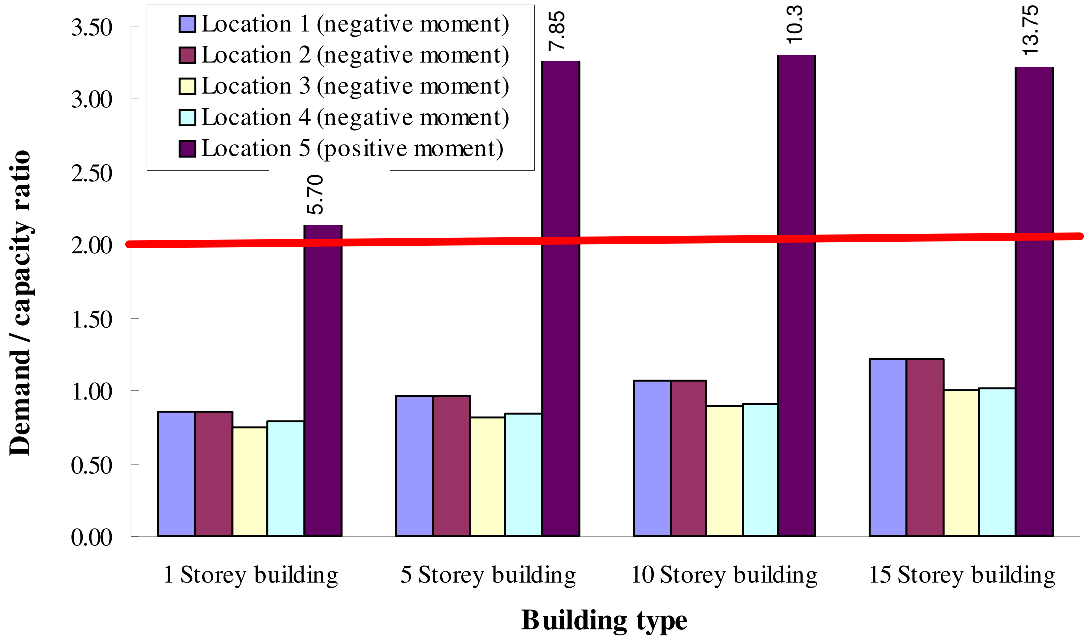

Fig. 4.38 DCRs for FS-5x5-I with interior column removed; slab without seismic detailing (For locations 1 through 5 see Fig. 4.5)

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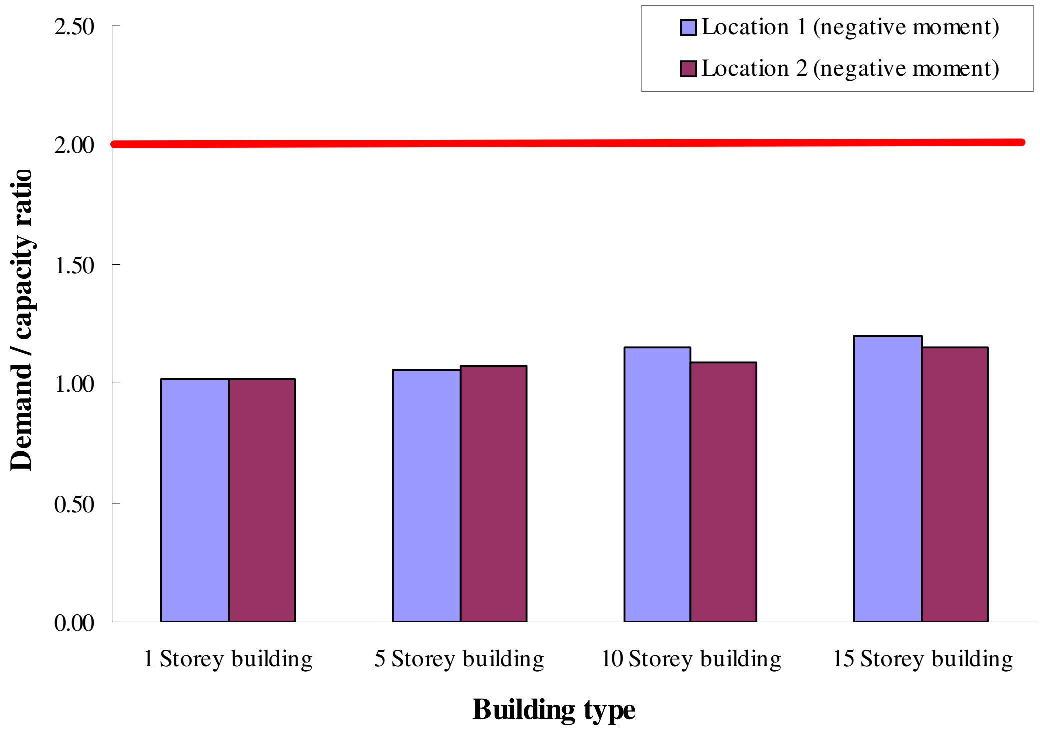

Fig. 4.39 DCRs for FS-9x9-C with corner column removed
(For locations 1 and 2 see Fig. 4.3)

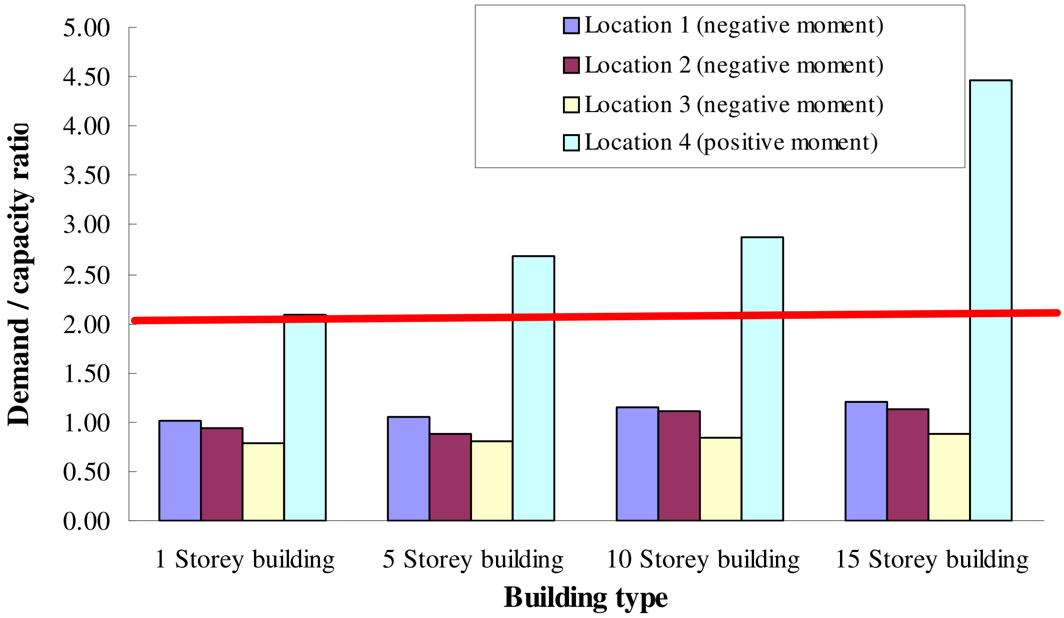

Fig. 4.40 DCRs for FS-9x9-E with edge column removed; slab with seismic detailing
(For locations 1 through 4 see Fig. 4.4)

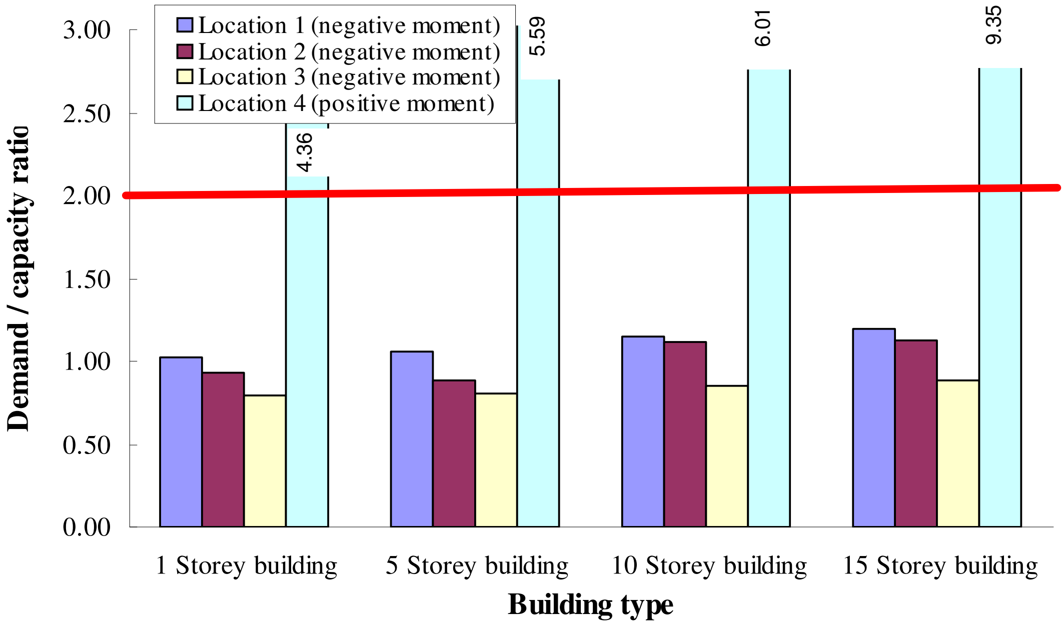

Fig. 4.41 DCRs for FS-9x9-E with edge column removed; slab without seismic detailing
(For locations 1 through 4 see Fig. 4.4)

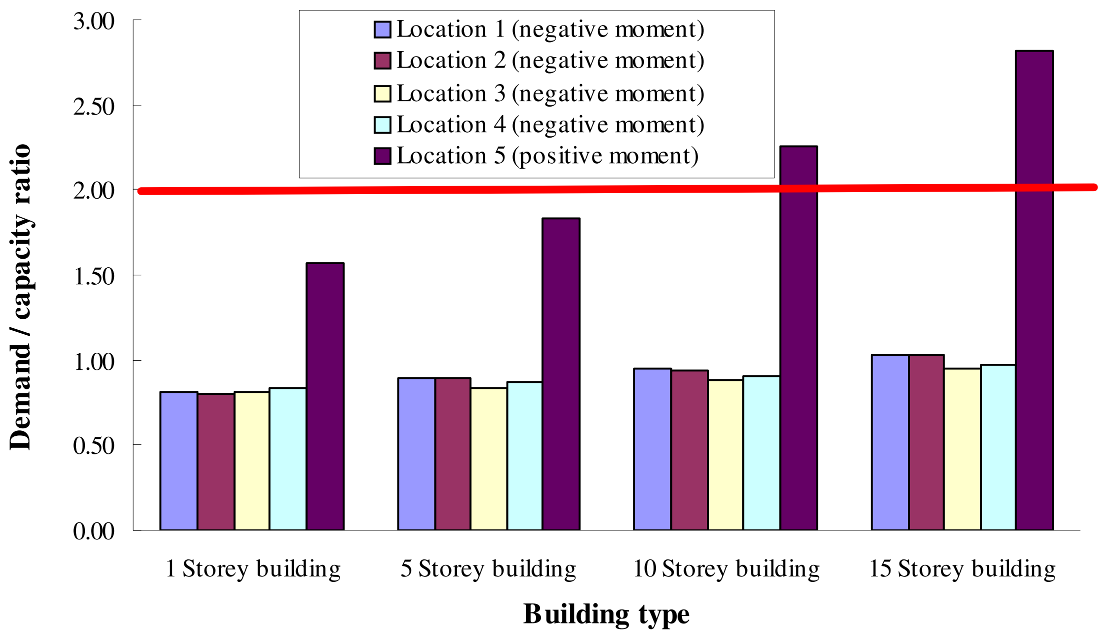

Fig. 4.42 DCRs for FS-9x9-I with interior column removed; slab with seismic detailing (For locations 1 through 5 see Fig. 4.5)

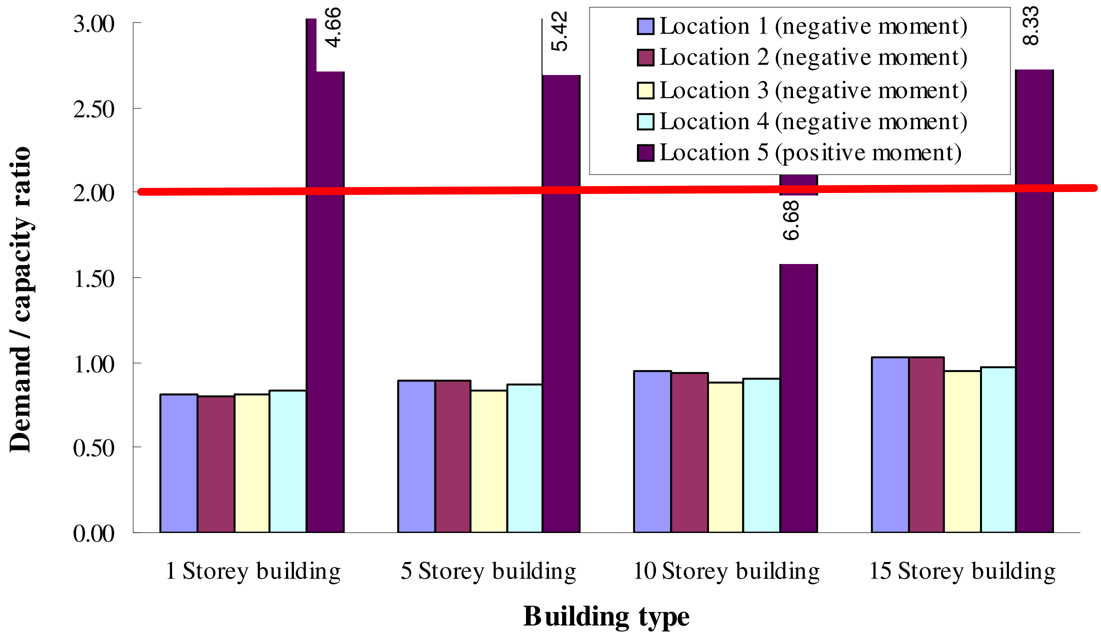

Fig. 4.43 DCRs for FS-9x9-I with interior column removed; slab without seismic detailing (For locations 1 through 5 see Fig. 4.5)

Chapter 5

Discussion of Results

5.1 General

The results of progressive collapse analyses of buildings are discussed in this Chapter in terms of demand capacity ratios (DCR) and reinforcement design and detailing requirements. A discussion of each building is provided in terms of the adequacy of design in the event of a column loss. Both negative and positive moment regions are considered with recommendations for possible improvements in behavior. Reinforcement detailing, both in terms of steel area requirements and bar cut-off locations are discussed.

5.2 Discussion of Results for Slabs without Seismic Detailing

This section provides a detailed discussion of results that are presented in Chapter 4. The section is divided into six sub-sections, one for each slab type as listed below:

- 5.2.1 Flat Plate with 7.0 m by 7.0 m Square Panels (FP-7x7)
- 5.2.2 Flat Plate with 7.0 m by 3.5 m Rectangular Panels (FP-7x3.5)
- 5.2.3 Flat Slab with 7.0 m by 7.0 m Square Panels (FS-7x7)
- 5.2.4 Flat Slab with 7.0 m by 3.5 m Rectangular Panels (FS-7x3.5)
- 5.2.5 Flat Slab with 5.0 m by 5.0 m Square Panels (FS-5x5)
- 5.2.6 Flat Slab with 9.0 m by 9.0 m Square Panels (FS-9x9)

Each sub-section will discuss the results obtained by analyzing that particular slab in 1, 5, 10 and 15 storey high buildings. Design shortcomings for each slab will be identified and recommendations will be made for appropriate measures to be taken during the design stage to produce a structure that will resist progressive collapse. The recommendations are intended to obtain a maximum DCR of about 1.95. They represent the least corrective measure that can be taken to provide a structure strong enough to sustain the new loads generated immediately after a column loss, with appropriate load paths forming to transfer the loads without failure.

The critical DCR values obtained for the four buildings analyzed with four different heights are discussed below. The most critical values obtained are almost always observed in the 15-storey building. Therefore, the DCRs indicated apply to 15-storey building slabs, unless otherwise indicated. Similarly, the recommendations made for improved performance are also intended for 15-storey buildings. The requirements for low-rise and medium-rise buildings may be less stringent, especially for the positive reinforcement, as explained in Section 5.4.4.

5.2.1 Flat Plate with 7.0 m by 7.0 m Square Panels (FP-7x7)

The DCRs obtained for this slab are similar and comparable in longitudinal and transverse directions. Therefore, both directions need to be assessed for vulnerability to progressive collapse, as both will undergo critical changes upon column removal. The only acceptable DCRs of less than 2.0 are those for the negative moment regions at interior columns, though the reinforcement cutoff locations in these regions may be critical.

- The removal of the first edge column next to the corner column results in a DCR of 2.49 for the negative moment region at the nearby edge column (location 2). The negative moment demands in the nearby exterior column regions (locations 1 and 2) extend beyond $0.3l_n$, where no negative reinforcement is provided. Similar observations are made for corner and interior column removals. In addition, for the case of interior column removal, though the DCR is below 2.0 in negative moment regions of nearby interior columns (ex: location 3), the negative moment region extends beyond $0.3l_n$ where no negative reinforcement is provided.
- The removal of the first interior column produces DCR of 7.9 for the positive moment at the location of column removal.

The following improvements may be introduced to provide corrective measures against progressive collapse:

For negative moments:

- 50% of negative reinforcement to be extended to a distance equal to 0.4 l_n instead of 0.3 l_n from the face of the support around the perimeter of the slab for all exterior columns, as well as from the face of the support around all interior columns.
- The percentage of negative reinforcement to be increased by 30% for all exterior columns in both longitudinal and transverse directions.

For positive moments:

- 100% of positive reinforcement used for flexural resistance should be extended into the other side of the adjacent columns, ensuring an adequate overlap equal to at least the development length. In other words, all the positive reinforcement should perform as structural integrity reinforcement.
- Positive reinforcement used at mid span and extended into the column to be increased by 75% at the location of the supports.

5.2.2 Flat Plate with 7.0 m by 3.5 m Rectangular Panels (FP-7x3.5)

The majority of the high DCRs are observed to occur along the transverse (short) direction. The longitudinal direction on the other hand, showed satisfactory ratios for most cases with a value less than 2.0. This may be explained by the fact that, in rectangular panels, the newly developed load paths transfer a larger portion of stresses in the short direction (stiffer direction), making it more critical than the longer direction (more flexible direction). As a result, the short direction becomes more critical with the increase in span ratio. The most critical values obtained are almost always observed in the 15-storey building. Therefore, the DCRs indicated below apply to 15-storey building slabs, unless otherwise indicated.

In transverse direction:

- The removal of the first edge column next to the corner column in the long direction results in a DCR of 3.83 for negative moment at the nearby interior column location in the short direction. The ratio becomes 3.16 at 0.20 l_n from the face of the support where only 50% of negative reinforcement is provided. The negative moment demand extends beyond 0.30 l_n where no negative reinforcement is provided.
- The removal of a corner column results in a DCR of 3.63 for negative moment at the nearby edge column location. The negative moment demand extends beyond 0.30 l_n where no negative reinforcement is provided around the perimeter of the slab.
- The removal of the first edge column next to a corner column in the short direction produces a DCR of 4.54 for negative moment at the nearby corner column location.
- The removal of the first interior column produces a DCR of 13.65 for positive moment at the location of column removal.

In longitudinal direction:

- The removal of edge column next to a corner column results in lower values of DCR than the critical ratio of 2.0 in negative moment regions. However, the negative moment demand extends beyond 0.30 l_n from the face of the edge column (location 2) where no negative reinforcement is provided.
- The removal of the first interior column (at location 5) produces a DCR of 5.33 for positive moment at column removal location.

The following improvements may be introduced to provide corrective measures against progressive collapse:

For negative moments in transverse (short) direction:

- The amount of negative reinforcement should be increased by 100% for interior column regions, by 90% for exterior column regions, and by 130% for corner column regions. All reinforcement should be extended to a distance equal to 0.20 l_n from the face of the support.

- 50% of the new negative reinforcement calculated above should be continued up to 0.50 l_n from the face of support around all exterior and interior columns in the transverse (short) direction.

For negative moment in longitudinal (long) direction:

- 50% of negative reinforcement should be extended to a distance equal to 0.30 l_n from the face of all exterior columns.

For positive moment in longitudinal and transverse directions:

- Positive reinforcement used at mid-span should be increased by 80% and should be continuous in both directions for interior and exterior columns. This reinforcement should be extended into the other side of the supporting column, ensuring adequate overlap equal to at least the development length of reinforcement. In other words, the entire bottom steel should be detailed as the structural integrity reinforcement.

5.2.3 Flat Slab with 7.0 m by 7.0 m Square Panels (FS-7x7)

The DCRs for negative moments are satisfactory for this slab, at all interior and exterior column regions, including locations at a distance of 0.20 l_n in longitudinal and transverse directions. The negative moment demands are significantly smaller for this slab relative to those for FP-7x7. This is attributed to the presence of drop panels with larger effective section depths. The positive moment regions at locations of column removal are critical for all slabs, irrespective of the building height. However, the 15-storey building remains to be the most critical among all the building heights considered.

The following are the most critical regions in terms of moment demands for all four buildings:

- Interior column removal results in a DCR of 7.66 for positive moment at column removal location of the 15-storey building.

- Edge column removal results in a DCR of 7.25 for positive moment at column removal location of the 15-storey building.
- Edge column removal produces negative moment demands that extend beyond 0.33 l_n from the column face of the nearby interior column (at location 3) where no negative reinforcement is provided. Similarly, the removal of a corner column generates negative moment demands that extend beyond 0.33 l_n from the face of the nearby edge columns (locations 1 and 2) where no negative reinforcement is provided.

The following improvements may be introduced to provide corrective measures against progressive collapse:

For negative moment:

- 50% of the negative reinforcement should to be extended to a distance equal to 0.37 l_n from the face of the support around all edge columns around perimeter of the slab.

For positive moment:

- 100% of the positive reinforcement used for flexural resistance should be extended into the other side of the adjacent columns, ensuring an adequate overlap equal to at least the development length. In other words, all the positive reinforcement should perform as structural integrity reinforcement.
- Positive reinforcement used at mid span and extended into the column regions should be increased by 25% at the location of interior column removal, and by 30% at locations of edge column removal.

5.2.4 Flat Slab with 7.0 m by 3.5 m Rectangular Panels (FS-7x3.5)

The DCRs for rectangular flat slabs are smaller than those computed for the equivalent flat plate slabs. However, the behavior of these two types of slab systems is very similar, with higher DCRs in the short span direction due to the relatively higher rigidity of the slab system in this direction. The negative moment DCRs in the longitudinal (long) direction at 0.2 l_n from the column face are lower than the critical value. Also, no negative moment demand was observed at a distance equal to 0.33 l_n from the face of the support along exterior or interior columns. Therefore no additional measures need to be taken along the long direction, in terms of negative moment regions, to prevent progressive collapse.

The most critical values of DCRs are obtained for the 15-storey building. Therefore, the DCRs indicated below apply to 15-storey building slabs.

In transverse direction:

- The removal of an edge column in the long direction, next to the corner column, produces a DCR of 3.36 for the negative moment region at the first interior column location. The DCR at 0.2 ln from the face of the same interior column is 3.11, where only 50% of negative reinforcement is extended. The negative moment demand however, is extended beyond 0.22 l_n where no negative reinforcement is provided.
- The removal of an edge column in the short direction, next to the corner column, produces a DCR of 2.32 for the negative moment region at the corner column location.
- The removal of a corner column generates negative moment demands beyond 0.33 l_n from the face of the nearby exterior column in the short span direction where no negative reinforcement is provided.
- The removal of the interior column produces a DCR of 18.34 for positive moment at the location of column removal in the short span direction.

In longitudinal direction:

- The removal of the interior column produces a DCR of 5.33 for positive moment at the location of column removal in the long span direction.

The following improvements may be introduced to provide corrective measures against progressive collapse:

For negative moment in short span direction:

- The amount of negative reinforcement should be increased by 20% in interior and corner column regions. All reinforcement should be extended to a distance equal to 0.20 l_n from the face of the supports.
- At all exterior and interior column locations, 65% of the new negative reinforcement (instead of 50% as suggested by the code) should be extended to a distance equal to 0.50 l_n from the face of columns in the short span direction.

For positive moment in both directions:

- 100% of the positive reinforcement used for flexural resistance should be extended into the other side of the adjacent columns, ensuring an adequate overlap equal to at least the development length. In other words, all the positive reinforcement should perform as structural integrity reinforcement.

Additional requirement in the short span direction:

- Positive reinforcement used at mid span and extended into the columns should be increased by 190% at the location of the supports for interior columns and by 80% for exterior columns.

Additional requirement in the long span direction:

- Positive reinforcement used at mid span and extended into the column to be increased by 40% at the location of the supports for interior columns and by 15% for exterior columns.

5.2.5 Flat Slab with 5.0 m by 5.0 m Square Panels (FS-5x5)

The DCRs for negative moments are all below the critical value at all interior and exterior column locations. The same is true for locations 0.20 l_n distance from the face of the column in longitudinal and transverse directions. The moment demands are smaller compared to those computed for 7.0 m span slabs (FS-7x7). It is observed that the increase in DCR with span length is more pronounced in the one-storey building. The effect of span length on DCR decreases with the number of stories.

The following are the most critical values obtained from the analysis of the four buildings considered with four different heights:

- The removal of the interior column produces a DCR of 13.75 for positive moment in the 15-storey building, at the location of column removal.
- The removal of the edge column produces a DCR of 10.84 for positive moment in the 10-storey building, at the location of column removal.
- The removal of the edge column results in negative moment demands beyond 0.33 l_n from the face of the nearby interior column where no negative reinforcement is provided.
- The removal of the corner column generates negative moment demands beyond 0.33 l_n from the face of the nearby edge column where no negative reinforcement is provided.

The following improvements may be introduced to provide corrective measures against progressive collapse:

For negative moment:

- 50% of negative reinforcement should be extended to a distance equal to 0.40 l_n from the face of all interior and exterior columns.

For positive moment:

- 100% of the positive reinforcement used for flexural resistance should be extended into the other side of the adjacent columns, ensuring an adequate overlap equal to at least the development length. In other words, all the positive reinforcement should perform as structural integrity reinforcement.

- Positive reinforcement used at mid span and extended into the column regions should be increased by 50% at interior column locations and by 70% at exterior column locations.

5.2.6 Flat Slab with 9.0 m by 9.0 m Square Panels (FS-9x9)

The DCRs are satisfactory at all interior and exterior column locations, including at a distance 0.20 l_n from the face of columns in longitudinal and transverse directions. The demands were smaller compared to those for FP-7x7, which is a slab without drop panels.

The following are the most critical values obtained from the analysis of four buildings with four different numbers of stories:

- The removal of the interior column produces a DCR of 8.33 for positive moment in a 15-storey building, at the location of column removal.
- The removal of the edge column produces a DCR of 9.35 for positive moment in a 10-storey building, at the location of column removal. In this case the negative moment demand extends beyond 0.33 l_n from the face of the nearby interior column where no negative reinforcement is provided.
- The removal of the corner column generates negative moment demands, extending beyond 0.33 l_n from the face of the nearby edge columns where no negative reinforcement is provided.

The following improvements may be introduced to provide corrective measures against progressive collapse:

<u>For negative moment</u>:

- 50% of negative reinforcement should be extended to a distance equal to 0.36 l_n from the face of all exterior and interior supports.

<u>For positive moment</u>:

- 100% of the positive reinforcement used for flexural resistance should be extended into the other side of the adjacent columns, ensuring an adequate overlap equal to at least the development length. In other words, all the positive reinforcement should perform as structural integrity reinforcement.
- Positive reinforcement used at mid span and extended into the columns should be increased by 20% at locations of interior columns and by 60% at locations of exterior columns.

5.3 Discussion of Results for Slabs with Seismic Detailing

Buildings detailed according to the seismic provisions of CSA A23.3-04 experienced less DCRs at locations of column removal (in regions of positive moment demands) and were less problematic as compared to buildings with identical geometry and loading but detailed as ordinary buildings without the seismic detailing. The continuity requirement of reinforcement in Chapter 21 of CSA A23.3-04 reduces the requirement for additional steel against progressive collapse.

Negative moment demands are the same as those for buildings detailed without seismic provisions, with the exception of the extension of reinforcement beyond 0.30 l_n for flat plates and 0.33 l_n for flat slabs because of the integrity steel requirement, which provides continuity and some percentage of reinforcement required against progressive collapse.

5.4 Effects of Design Parameters

The 6 slab systems considered in the current investigation was selected to conduct an analytical parametric study. Two different slab systems (flat plates and flat slabs) were considered to assess the performance of flat plates relative to flat slabs (with drop panels). In each case two different span ratios (or aspect ratios of slab panels) were used, consisting of either 7 m by 7 m square panels or 7 m by 3.5 rectangular panels with span ratios of 1:1 and 2:1, respectively. Furthermore, the effect of span length was investigated by employing flat slab systems with square panels, and span lengths of 5.0 m, 7.0 m and 9.0 m. In addition, the building height was considered as one of the primary parameters. Four different building heights were considered (1; 5; 10; and 15-storey buildings) in the analyses. In almost all cases, the 15-storey building produced higher moment and reinforcement demands, and is used in the comparisons made in the following sections. The effect of number of stories is specifically addressed in Section 5.4.4.

In addition to the above geometric parameters, each slab was designed and detailed twice, first as ordinary gravity load structural system and secondly by implementing the seismic detailing requirements of CSA A23.3-04, which call for continuity of reinforcement with some minimum negative and positive moment capacity along the length of slab panels.

The following sections provide discussions on the effects of design parameters considered in assessing the progressive collapse potential of buildings.

5.4.1 The Effects of Drop Panels

The progressive collapse potential for flat plates and flat slabs can be compared by examining the results of FS-7x7 and FP-7x7. The comparison indicates superior performance of flat slabs over flat plates. This is because they experience smaller moment demands, while having larger negative moment capacities because of drop panels. The results indicate that flat slabs have adequate negative reinforcement at column locations, whereas they need positive reinforcement of up to 125% to 130% of mid-span reinforcement to be continued into the potential column loss regions. This is significantly less than the increased reinforcement required for comparable flat plates, i.e., 30% increased negative steel over the columns and 175% of positive reinforcement to be continuous over the potential column loss regions.

5.4.2 The Effects of Span Ratio

Two different span ratios were investigated in the current projects, i.e., 1:1 and 2:1. Both flat plates and flat slabs consisting of 7.0 m by 7.0 m square panels and 7.0 m by 3.5 m rectangular panels were analyzed. The comparison of FP-7x7 and FP-7x3.5, as well as FS-7x7 and FS-7x3.5 indicate that slabs with span ratios closer to 1.0 are less vulnerable to progressive collapse. As the span ratio increases, the slab strips in the short direction become more rigid and they attract more loads compared to slab strips in the long direction. This has been confirmed by DCRs and corresponding reinforcement requirements. Square flat plates require additional 30% negative steel over supports, whereas rectangular panels require 90% to 130% more negative steel in the short direction, depending whether the column loss is a corner, edge or interior column. Similarly, square panels require 75% increase in positive moment reinforcement to continue into the column removal location, whereas rectangular panels require 80% increase in positive reinforcement. Similar behavior is observed when flat slabs with different panel aspect ratios are compared. FS-7x7 does not require additional negative moment reinforcement at column location due to the loss of a nearby column, whereas FS-7x3.5 would require 20% increase in top reinforcement in the short direction. FS-7x7 requires only 25% to 30% increase in positive moment reinforcement to be continuous in the short span direction at column removal location, whereas this amount increases to 80% to 190% in FS-7x3.5 depending on which column is lost. The effect of panel aspect ratio appears to be more dominant in flat slabs than in flat plates, probably because of the more dominant role of drop panels in providing additional stiffness in the short direction.

5.4.3 The Effects of Span Length

The effect of span length was investigated by analyzing flat slabs having 5.0 m, 7.0 m, and 9.0 m square panels (FS-5x5, FS-7x7 and FS-9x9). The results indicate that, contrary to intuitive expectation of increase in percentage reinforcement with increasing spans, a slightly higher percentage of increase is attained in the positive reinforcement requirements for the slab with 5.0 m span (shortest span considered). This may not necessary translate in more steel in absolute terms. However, the additional positive reinforcement requirement for continuous bottom steel in interior spans are 50% for the 5.0 m panel, 25% for the 7.0 m panel and 20% for the 9.0 m panel. The increases in bottom steel requirement for exterior spans are 70% for the 5.0 m panel, 30% for the 7.0 m panel and 60% for the 9.0 m panel. No additional negative steel is needed for any of the three different span cases (though cutoff points are affected somewhat as explained in Section 5.2). Based on these comparisons, there appears to be no clear trend between the percentage of increase in positive reinforcement and the span length as the trend changes with the change in number of stories.

5.4.4 The Effects of Number of Stories

The observations made in the preceding sections in terms of relative importance of the parameters considered equally apply to slabs in buildings with different number of stories, though the DCRs and corresponding reinforcement requirements gradually increase with the number of stories. This can be clearly seen in Fig. 5.1. For one storey building with square slab panels, it would be reasonable to use 60% of bottom reinforcement used at mid-span at the column removal location. This percentage increases with number of stories almost linearly up to 120% (for 9.0 m span) to 150% (for 5.0 m span) in flat slabs (with drop panels) and up to 175% in flat plates. Furthermore, it is clear in Fig. 5.1 that rectangular panels of flat slabs with 2:1 aspect ratio requires higher percentage of increase in positive reinforcement in the short span direction, 80% for the one-storey building up to 230% for the 15-storey building, though the effect of panel aspect ratio on the relationship between positive steel requirement and the number of stories is not pronounced.

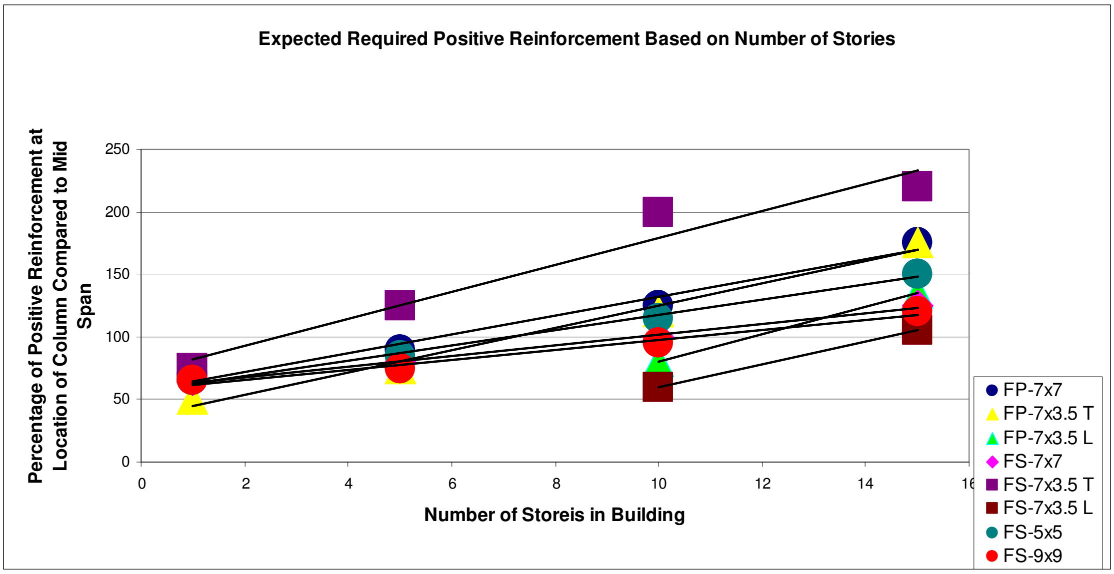

Fig. 5.1 The relationship between the percentage of positive reinforcement required to prevent progressive collapse and the number of stories

Based on the above discussion, the following expressions may be used to estimate the positive steel requirements in slabs with square panels that lose their supports due to blast.

$$\rho^{+} = [0.65 + (N - 1)\,c]\,\rho^{+}_{ms} \qquad (5\text{-}1)$$

c = 0.06 for flat slabs (with drop panels)

c = 0.08 for flat plates

Where; ρ^+ is the positive continuous reinforcement ratio required to mitigate the effects of progressive collapse, ρ^{+}_{ms} is the positive flexural steel ratio required at mid-span for gravity loads, N is the number of stories and c is a coefficient that reflects the effect of slab type. Eq (5-1) can be used to estimate the positive steel requirements for rectangular panels, with an aspect ratio of 2:1 if modified to reflect the differences in short and long directions. This can be done approximately by increasing the steel requirement computed by Eq. (5-1) by 12% to estimate the amount of positive steel required in the short direction, and reducing the amount by 12% to estimate the amount required in the long direction.

5.4.5 The Effects of Seismic Detailing

Slabs detailed with seismic provisions had better continuity for top and bottom reinforcement. This resulted in smaller positive moment demand/capacity ratios at the location of column removal. However, the flexural reinforcement considered in this investigation was based on gravity loads only. Any additional moments imposed on slabs due to seismic loads were not considered, except for the detailing requirement as it affects continuity of reinforcement. Therefore, the same recommendations mentioned above for slabs detailed without the seismic provisions can be implemented since the recommended reinforcement is based on the gravity load requirements.

Chapter 6

Conclusions and Recommendations

6.1 Conclusions

The following summarizes the most important findings of the research project reported in this thesis. The conclusions are primarily based on the results of over 40 analyses of flat plate and flat slab buildings with different heights, spans and panel aspect ratios, and reinforcement detailing.

- The most critical region of slabs with highest vulnerability to progressive collapse occurs at the location of the column removal due to blast loads, and the subsequent formation of load paths producing additional positive flexural demands. Negative moments at the neighboring columns are significantly smaller compared to positive demands, but in some cases they may also pose slight vulnerability for collapse.
- The majority of critical negative DCRs occur at columns adjacent to the column that has been removed. These are the regions that are affected most by the new path of forces. In most of the other negative moment sections, the vulnerability to progressive collapse is smaller with the exception of newly generated negative moment demands, necessitating increase in percentage by more than 50% to a distance larger than 0.20 l_n from the face of the support and extensions of top reinforcement beyond 0.30 l_n from the face of the support.
- Flat slabs impose smaller risk compared to flat plates. The main reason is that the presence of drop panels produces sections with higher negative moment capacities. Moreover, the positive moments developed within the span become smaller in flat slabs because of the reduced slab thickness and associated reduction in dead loads.

- Seismically detailed slabs based on Chapter 21 of CSA A23.3-04 are less problematic compared to slabs designed and detailed for gravity loads as per Chapter 13 of CSA A23.3-04. However, both require improvements in terms of the additional top and bottom reinforcements, as well as increased cut-off lengths.
- Higher vulnerability to progressive collapse is observed with the increase of number of stories because of the additional load imposed on lower stories in taller buildings.
- Slabs with a span ratio of more than 1.0 become more vulnerable as compared to square panels, as the slab becomes stiffer in the short direction and attract most of the newly developed loads. On the contrary, a span ratio of 1.0 distributes the developed loads evenly among the adjoining columns, generating approximately equal increases in negative bending in each direction.
- The change in span lengths does not show a pronounced effect on DCRs.
- Within the range of the variations of spans between 5.0 m and 9.0 m and span ratios of 1:1 to 2:1, the increase in positive reinforcement requirement in one-story buildings may be sufficed by the extension of 100% the mid-span reinforcement into the face of support. The percentage requirement increases with number of stories almost linearly up to 120% (for 9.0 m span) to 150% (for 5.0 m span) in flat slabs (with drop panels) and up to 175% in flat plates.
- Eq. (5-1) produces reasonable estimates of positive reinforcement requirements in flat plates and flat slabs, within the restrictions specified.
- The loss of a column may impose negative moments in areas that are further away from the adjacent columns. This may translate into longer bars than the cut-off lengths currently used for gravity load designs.

While the above conclusions provide a summary of observations obtained from the extensive analyses conducted in the current investigation, in an effort to provide guidance to designers, it is emphasized that they are not substitutes for structural analysis results obtained under exact geometry and loading conditions for the problem at hand. Therefore, for improved progressive collapse assessment, structural engineers are encouraged to conduct custom tailored analysis for the slab system that is being investigated.

6.2 General Recommendations

The recommendations presented in this section are in addition to the specific recommendations made in Chapter 5, and are intended to highlight good design practices for improved performance of buildings that are exposed to progressive collapse risk. They are applicable to positive and negative reinforcement regions to reduce the DCRs, but do not necessarily bring them below the critical value of 2.0 in all cases:

- Always detail slabs according to the continuity requirements of the seismic provisions for top and bottom reinforcement, outlined in Chapter 21 of CSA A23.3-04.
- For positive (bottom) reinforcement:
    1. Continue 100% of the reinforcement used as flexural reinforcement throughout the entire length of the increased span, covering the column removal location to carry the increased positive moment associated with column loss.
    2. Add additional positive reinforcement at the location of potential column loss, equal to 20% - 120% of the mid-span positive steel designed for flexural resistance based on the original span length. Guidance on the amount of additional reinforcement may be obtained from the discussion presented in Chapter 5.
    3. The above recommendations are intended for slabs of 15-storey buildings. The positive steel requirements may be reduced for low-rise and medium-rise buildings as discussed in Section. 5.4.4.
- For negative (top) reinforcement:
    1. The flat slabs with square panels appear to have adequate negative reinforcement over the supports, except for rectangular panels, which may require an additional 20% of negative steel when the aspect ratio is 2:1. In flat plates, additional 30% negative steel may be needed for

square panels, with the additional steel requirement climbing up to 90% to 130% for rectangular panels having 2:1 aspect ratio.

6.3 Recommendations for Future Research

Even though extensive research has been conducted in the current investigation, the topic of progressive collapse is still relatively new and more work is needed to provide better understanding of this phenomenon. Only then, new and improved design guidelines can be developed and incorporated into building codes and standards. The following topics are recommended for future research:

- Experimental research to verify the results of this investigation, using large scale static testing and/or smaller scale shock tube testing under simulated blast loading.
- The effects of seismic forces on capacities of flat plates and slabs should be considered in accordance with seismic provisions of CSA and the new demand/capacity ratios under combined loads to assess the impact of seismic design.
- Validity of the linear-elastic static analysis and the load combination suggested by the GSA 2003; 2 (Dead Load) + 0.5 (Live Load) in replacing dynamic analysis and the effect of uplift that some slabs might experience due to blast loads or impact of large vehicles on columns.
- Differences in analysis and evaluation of results considering the GSA approach and the DoD approach.
- The effect of the presence of spandrel (exterior) beams around the perimeter of the slab for slabs with 1:1 and 2:1 span ratios.

References

1. ASCE. 2006. Minimum design loads for buildings and other structures. Standard ASCE/SEI 7-05, American Society of Civil Engineers, Danvers, MA, USA.
2. Balazic, J. M. 1982. Tests to collapse of concrete slabs. M. Eng Thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Quebec, Canada.
3. Baldridge, S. M. and Humay, F. K. 2003. Preventing progressive collapse in concrete buildings, Concrete International, Vol. 25, No. 11, November, pp. 73-79.
4. Bilow, D. N. and Kamara, M. E. 2005. GSA progressive collapse design guidelines applied to concrete moment-resisting frame buildings. Tri-Service Infrastructure Systems Conference & Exposition, St. Louis, MO, USA.
5. Breen, J. E. 1975. Research workshop on progressive collapse of building structures. Proceedings of a Research Workshop held at the University of Texas at Austin, pp. 18-20.
6. Burns, J., Abruzzo, J., and Tamaro, M. 2002. Structural systems for progressive collapse prevention. Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse, Chicago, IL.
7. Cagley, J. R. 2002. The design professional's concerns regarding progressive collapse design. Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse, Chicago, IL.
8. Choi, K., Taha, M. R., Park, H. and Maji, A. 2007. Punching shear strength of interior concrete slab–column connections reinforced with steel fibers. Cement and Concrete Composites, Volume 29, Issue 5, Pages 409-420.
9. Chu, E. K. 1982.Tests to collapse of reinforced concrete plates. M. Eng Thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Quebec, Canada.
10. CSA. 2004. CSA Standards A23.3-04 - Design of concrete structures. Canadian Standards Association, Rexdale, Ontario, Canada.
11. CSI. 2008. Analysis reference manual for SAP2000®, ETABS®, and SAFE™, Computers and Structures, Inc. Berkeley, California, USA.

12. CSI. 2002. Three-dimensional static and dynamic analysis of structures, Computers and Structures, Inc., Berkeley, California, USA.
13. CSI. 2002. SAFE reinforced concrete design manual, Computers and Structures, Inc., Berkeley, California, USA.
14. DoD. 2005. Unified Facilities Criteria (UFC) – Design of buildings to resist progressive collapse, UFC 4-023-03. U.S. Department of Defense, Washington, D.C., USA.
15. Dusenberry, D. O. and Juneja, G. 2002. Review of existing guidelines and provisions related to progressive collapse. Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse, Chicago. IL.
16. Egberts, M. 2009. Preventing progressive collapse of flat plate structures with an irregular layout of structural integrity reinforcement. M.Eng. dissertation, McGill University, Montreal, Quebec, Canada.
17. Elgabry, A. A. & Ghali, A. 1990. Design of shear stud reinforcement for slabs. ACI Structural Journal, 87(3), 350–61.
18. Ellingwood, B., Leyendecker, E. V., and Yao, J. T. P. 1983. Probability of Failure from abnormal load. Journal of Structural Engineering, ASCE, 109(4), pp. 875-890.
19. GSA. 2003. Progressive collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects, U.S. General Services Administration, Washington, D.C., USA.
20. Hawkins, N. M. and Mitchell, D. 1979. Progressive collapse of flat plate structures. Journal of the American Concrete Institute, Proceedings V.6, No. 7, July, pp. 775-808.
21. Kim, H. 2006. Progressive collapse behavior of reinforced concrete structures with deficient details. Ph.D. dissertation, The University of Texas at Austin, United States -- Texas. Retrieved January 12, 2012, from Dissertations & Theses: Full Text. (Publication No. AAT 3328259).
22. Kokot, S., Anthoine, A., Negro, P. and Solomos, G. 2010. Static and dynamic analysis of a reinforced concrete flat slab frame building for progressive collapse. JRC Scientific and Technical Reports, Ispra, Italy.

23. Mirzaei, Y. 2010. Punching shear failure in progressive collapse analysis. Department of Civil and Environmental Engineering, Northeastern University, 400 Snell Engineering Center, Boston, United States.
24. Mitchell, D. and Cook, W. D. 1984. Preventing progressive collapse of slab structures, Journal of Structural Engineering, Vol. 110, No. 7, July, pp. 1513-1532.
25. Mokhtar, A. S., Ghali, A. and Dilger,W. 1985. Stud shear reinforcement for flat concrete plates. ACI Journal, 85(2), 676–83.
26. Naumoski, N., Saatcioglu, M., Foo and Lin 2011. Assessment of the vulnerability of buildings to progressive collapse due to blast loads. Geological Survey of Canada, Natural Resources Canada, Ottawa, Ontario, Canada.
27. Naumoski, N., Glal, K. and Yagob, O. 2009. Progressive collapse of reinforced concrete structures. Structural Engineering and Mechanics, Vol. 32, No. 6, pp. 771-786, Canada.
28. NBCC. 2010. National Building Code of Canada. Institute for Research in construction, National Research Council of Canada, Ottawa, Ontario, Canada.
29. Park, T. W. 2011. Inspection of collapse cause of Sampoong Department Store, Forensic Science International,
30. Polak, M. A. 2005. Ductility of reinforced concrete flat slab-column connections. (Department of Civil Engineering, University of Waterloo, Waterloo, Ont. N2L 3G1, United Kingdom) Source: Computer-Aided Civil and Infrastructure Engineering, v 20, n 3, pp. 184-193.
31. Popoff, A., Fintel, M., Firnkas, S. and Speyer, I. J. 1977. Design against progressive collapse. PCI journal, Volume 22, No. 1,pp. 116-121.
32. Ramos, A. P., Lucio, V. J. G. and Regan, P. E. 2010. Punching of flat slabs with in-plane forces. Engineering Structures, Vol. 33, Issue 3, March 2011, pp. 894-902.
33. Regan, P. E. 1981. Behavior of reinforced concrete flat slabs. Report 89, CIRIA, London, UK.
34. Starosske, U. and Wolff, M. 2005. Progressive collapse: design strategies. IABSE Symposium Structures and Extreme Events, Lisbon, Sep. 14-17.
35. Vonier, T. 1997. Urban security zones. Urban Land; 56(l):6-7.

Appendix A

Calculation Sample for FP-7x7

The design and the detailing of the flexural reinforcements is presented below for plan one which was repeated for all the other plans.

Design of Plane 1

Demands resulting from most critical combination are summarized in Fig. A.1.

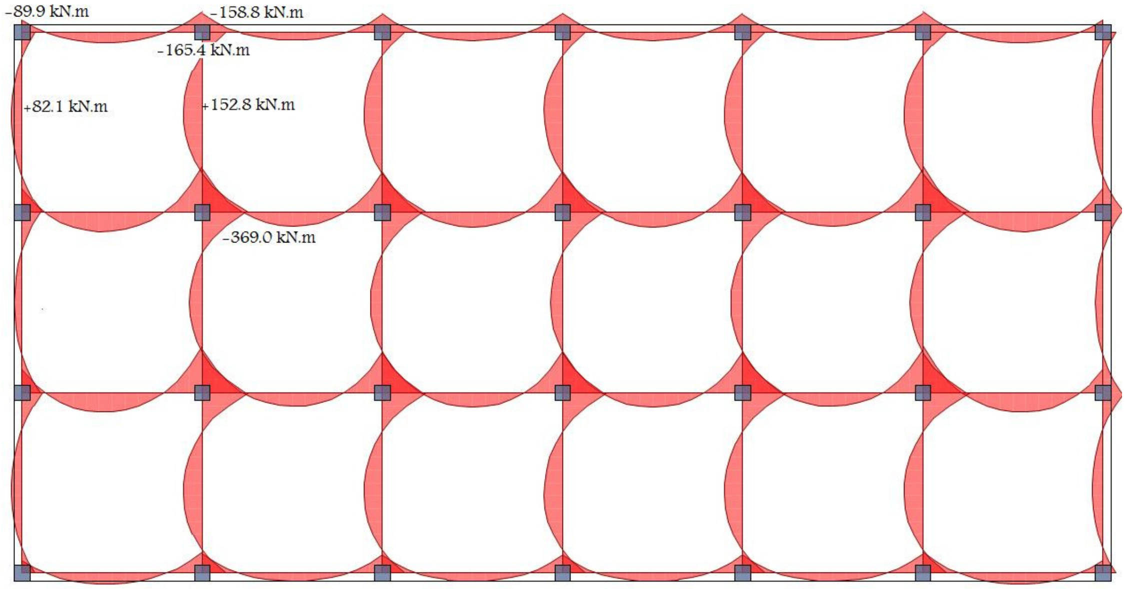

Fig. A.1 Critical moment demands.

Reinforcement Detailing of Plan 1

The slab had different detailing at the following locations:

- Exterior sections along the perimeter of the slab for Exterior columns
- Exterior Section along the perimeter of the slab for Corner Columns
- Exterior Columns in the direction Connecting Interior Columns
- Interior Sections around Interior Columns
- Structural Integrity Reinforcement for Exterior Columns
- Structural Integrity Reinforcement for Interior Columns

Design capacities of sections with and without increased strength factors are presented in the table below:

Table A.1 Section capacities.

Type of Section and Direction	Moment Values (kN.m) Per Unit Strip		
	Design demand	Design capacity after imposing minimum reinforcement	Design capacity with increased strength factor
Exterior sections along the perimeter of the slab for exterior columns	82.1	82.4	122.1
	-158.8	-170.9	-255.7
Exterior Section along the perimeter of the slab for Corner Columns	-89.9	-127.5	-189.9
Exterior Columns in the direction Connecting Interior Columns	-165.4	-209.7	-315.2
Interior Sections around interior Sections	152.5	153	226.8
	-369	-370	-554.6
Structural Integrity Reinforcement for Exterior Columns		39.2	58.2
Structural Integrity Reinforcement for Interior Columns		65.5	96.9

Notes:

Positive numbers indicate positive moment and negative indicates negative moment.

Calculation of Structural Integrity Reinforcement

Determining which produces larger shear from the following:

- Load Combinations from the code (the following combination governs (1.25 x Dead Load) + (1.5 x Live Load))

  1.25 (1.5 + 0.25 x (23.5)) + 1.5 (2.4) = 12.82 kN/m^2

- 2 x Self Wight of Slab

  2 x 0.25 x 23.5 = 11.75 kN/m^2

Therefore the first combination is chosen.

After the value of V_{se} is determined from the software, its divided by the number of direction depending on the column type and then the area required for structural integrity is calculated from the formula:

$$\sum A_{sb} = \left(\frac{2\ V_{se}}{f_y} \right)$$

The following table summarizes the results of analysis.

Table A.2 Calculation of structural integrity reinforcement.

Columns	Corner	Edge	Interior
V_{se} (kN)	142.7	321	718.2
Number of directions	2	3	4
Structural integrity reinforcement (mm^2)	356.75	535	897.75

Minimum of two bars in each direction = 2#10 = 200 mm^2.

Therefore choose 897.8 mm^2 for interior columns and 535 mm^2 for exterior columns. These areas of reinforcement produce the capacity values mentioned in Table A.1.

Design and Reinforcement Detailing of Slab (With Seismic Provisions)

Two conditions govern the amount of positive reinforcement to be continuous throughout the span:

- 0.5 x bottom reinforcement at mid span = 0.5 x 1138 = 569 mm^2
- 0.333 x top reinforcement at the location of support = 0.333 x 2454 = 817.2 mm^2

Therefore choose 817.2 mm^2.

After that, the different cases are applied to each of the plans and the demand/capacity ratios are calculated by dividing the newly developed demand by the section's designed capacity after applying the increased strength factor.

Appendix B

Results of Analysis in Tabulated Format

Results presented in the following tables represent buildings detailed without seismic provisions unless otherwise indicated which is usually whenever positive moment is investigated. The phrase “< 0.30 l_n“ or “ < 0.33 l_n” indicates that the zero moment point is located at a distance less than 0.30 l_n or 0.33 l_n.

FP-7x7

Case 1; Corner Column Removed

Location 1 (Negative Moment -M)

Table B.1 DCRs at Location 1 which represent column A2 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	2.18	1.15	-M	0.38 ln
10	1.99	0.98	-M	0.36 ln
5	1.88	0.88	-M	0.35 ln
1	1.73	0.82	-M	0.36 ln

Location 2 (Negative Moment -M)

Table B.2 DCRs at Location 2 which represent column B1 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	2.25	1.20	-M	0.38 ln
10	2.05	1.03	-M	0.36 ln
5	1.92	0.92	-M	0.35 ln
1	1.75	0.83	-M	0.36 ln

Case 2; Edge Column Removed

Location 1 (Negative Moment -M)

Table B.3 DCRs at Location 1 which represent column A3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	2.36	1.20	-M	0.36 ln
10	2.08	1.00	-M	0.34 ln
5	1.89	0.85	-M	0.34 ln
1	1.75	0.75	-M	0.33 ln

Location 2 (Negative Moment -M)

Table B.4 DCRs at Location 2 which represent column A1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	2.49	0.67	+M	< 0.30 ln
10	2.18	0.47	+M	< 0.30 ln
5	1.95	0.32	+M	< 0.30 ln
1	1.75	0.14	+M	< 0.30 ln

Location 3 (Negative Moment -M)

Table B.5 DCRs at Location 3 which represent column B2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.67	0.74	-M	0.36 ln
10	1.54	0.62	-M	0.33 ln
5	1.45	0.54	-M	0.32 ln
1	1.38	0.19	-M	0.32 ln

Location 4 (Positive Moment +M)

Table B.6 DCRs at Location 4 which represent column A2.

Number of stories in building	Demand/capacity Ratios
	At center of column
15	6.87
10	5.24
5	4.10
1	3.02

Case 3; Interior Column Removed

Location 1 (Negative Moment -M)

Table B.7 DCRs at Location 1 which represent column A2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	2.29	0.01	+M	< 0.30 ln
10	1.99	+M	+M	< 0.30 ln
5	1.78	+M	+M	< 0.30 ln
1	1.55	+M	+M	< 0.30 ln

Location 2 (Negative Moment -M)

Table B.8 DCRs at Location 2 which represent column B1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	2.27	+M	+M	< 0.30 ln
10	1.99	+M	+M	< 0.30 ln
5	1.78	+M	+M	< 0.30 ln
1	1.53	+M	+M	< 0.30 ln

Location 3 (Negative Moment -M)

Table B.9 DCRs at Location 3 which represent column C2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.83	0.76	-M	0.33 ln
10	1.64	0.62	-M	0.32 ln
5	1.49	0.52	-M	0.31 ln
1	1.41	0.49	+M	< 0.30 ln

Location 4(Negative Moment -M)

Table B.10 DCRs at Location 4 which represent column B3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.87	0.79	-M	0.33 ln
10	1.67	0.65	-M	0.32 ln
5	1.53	0.59	-M	0.31 ln
1	1.47	0.51	+M	< 0.30 ln

Location 5 (Positive Moment +M)

Table B.11 DCRs at Location 5 which represent column B2

Number of stories in building	Demand/capacity Ratios
	At center of column
15	7.88
10	5.68
5	4.06
1	3.09

DCRs of positive moments for slabs detailed according to the seismic provisions of the CSA

Table B.12 DCRs at Location 4 which represent column A2 and at Location 5 which represent column B2.

Number of stories in building	Demand/capacity ratios at center of column	
	Case2 Location 4	Case3 Location 5
15	4.53	4.04
10	3.45	2.92
5	3.70	2.08
1	1.99	1.59

FP-7x3.5

Case 1; Corner Column Removed

Location 1 (Negative Moment -M)

Table B.13 DCRs at Location 1 which represent column A2 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	3.63	2.49	-M	0.51 ln
10	2.93	2.28	-M	0.49 ln
5	2.40	1.76	-M	0.42 ln
1	1.86	1.45	-M	0.63 ln

Location 2 (Negative Moment -M)

Table B.14 DCRs at Location 2 which represent column B1 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.49	0.42	-M	< 0.30 ln
10	1.35	0.30	-M	< 0.30 ln
5	1.20	0.22	-M	< 0.30 ln
1	1.14	0.17	-M	< 0.30 ln

Case 2; Edge Column Removed in Longitudinal Direction

Location 1 (Negative Moment -M)

Table B.15 DCRs at Location 1 which represent column A1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.62	+M	+M	< 0.30 ln
10	1.45	+M	+M	< 0.30 ln
5	1.18	+M	+M	< 0.30 ln
1	1.05	+M	+M	< 0.30 ln

Location 2 (Negative Moment -M)

Table B.16 DCRs at Location 2 which represent column C1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.60	0.52	-M	0.31 ln
10	1.40	0.39	+M	< 0.30 ln
5	1.16	0.23	+M	< 0.30 ln
1	1.04	0.16	+M	< 0.30 ln

Location 3 (Negative Moment -M)

Table B.17 DCRs at Location 3 which represent column B2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	3.83	3.16	-M	0.55 ln
10	3.13	2.45	-M	0.53 ln
5	2.36	1.74	-M	0.49 ln
1	2.17	1.65	-M	0.62 ln

Location 4 (Positive Moment +M)

Table B.18 DCRs at Location 4 which represent column B1.

Number of stories in building	Demand/capacity Ratios
	At center of column
15	12.38
10	8.87
5	5.80
1	0.42

Case 3; Edge Column Removed in Transverse Direction

Location 1 (Negative Moment -M)

Table B.19 DCRs at Location 1 which represent column A3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	3.03	1.87	-M	0.38 ln
10	2.37	1.36	-M	0.37 ln
5	1.91	1.00	-M	0.35 ln
1	1.44	0.61	-M	0.32 ln

Location 2 (Negative Moment -M)

Table B.20 DCRs at Location 2 which represent column A1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	4.54	1.91	+M	< 0.30 ln
10	3.52	1.28	+M	< 0.30 ln
5	2.75	0.75	+M	< 0.30 ln
1	1.91	0.10	+M	< 0.30 ln

Location 3 (Negative Moment -M)

Table B.21 DCRs at Location 3 which represent column B2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.05	0.12	+M	< 0.30 ln
10	0.97	0.08	+M	< 0.30 ln
5	0.94	0.04	+M	< 0.30 ln
1	0.94	0.01	+M	< 0.30 ln

Location 4 (Positive Moment +M)

Table B.22 DCRs at Location 4 which represent column A2.

Number of stories in building	Demand/capacity Ratios
	At center of column
15	10.08
10	7.09
5	4.88
1	2.93

Case 4; Interior Column Removed

Location 1 (Negative Moment -M)

Table B.23 DCRs at Location 11 which represent column A2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.12	+M	+M	< 0.30 ln
10	0.99	+M	+M	< 0.30 ln
5	0.89	+M	+M	< 0.30 ln
1	0.76	+M	+M	< 0.30 ln

Location 2 (Negative Moment -M)

Table B.24 DCRs at Location 2 which represent column B1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	2.23	0.60	+M	< 0.30 ln
10	1.75	0.36	+M	< 0.30 ln
5	1.33	0.16	+M	< 0.30 ln
1	1.02	+M	+M	< 0.30 ln

Location 3 (Negative Moment -M)

Table B.25 DCRs at Location 3 which represent column C2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.10	0.18	+M	< 0.30 ln
10	0.99	0.24	+M	< 0.30 ln
5	0.90	0.05	+M	< 0.30 ln
1	0.85	+M	+M	< 0.30 ln

Location 4(Negative Moment -M)

Table B.26 DCRs at Location 4 which represent column B3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	3.36	1.95	-M	0.37 ln
10	2.68	1.46	-M	0.36 ln
5	2.10	1.05	-M	0.35 ln
1	1.81	0.86	-M	0.35 ln

Location 5 (Positive Moment +M)

Table B.27 Demand/capacity Ratios at Location 5 which represent column B2.

Number of stories in building	Demand/capacity Ratios
	At center of column
15	13.65
10	9.33
5	5.67
1	3.66

DCRs of positive moments for slabs detailed according to the seismic provisions of the CSA

Table B.28 DCRs at Location 4 which represent column B1, at Location 4 which represent column A2 and at Location 5 which represent column B2.

Number of stories in building	Demand/capacity ratios at center of column		
	Case2 Location 4	Case3 Location 4	Case4 Location 5
15	10.47	8.52	9..94
10	7.50	5.99	6.79
5	4.90	4.13	4.13
1	0.36	2.48	2.66

FS-7x7

Case 1; Corner Column Removed

Location 1 (Negative Moment -M)

Table B.29 DCRs at Location 1 which represent column A2 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.13	0.92	-M	0.37 ln
10	1.09	0.85	-M	0.36 ln
5	1.05	0.79	-M	0.35 ln
1	0.96	0.73	-M	0.36 ln

Location 2 (Negative Moment -M)

Table B.30 DCRs at Location 2 which represent column B1 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.17	1.02	-M	0.37 ln
10	1.12	0.88	-M	0.36 ln
5	1.08	0.83	-M	0.36 ln
1	0.96	0.75	-M	0.36 ln

Case 2; Edge Column Removed

Location 1 (Negative Moment -M)

Table B.31 DCRs at Location 1 which represent column A3 in the transverse direction.

Number of stories in building	Demand/capacity ratios At center of column	 At 0.2 ln from face of support	 At 0.3 ln from face of support	Location where negative moment = 0
15	1.15	0.88	-M	0.35 ln
10	1.12	0.87	-M	0.34 ln
5	1.03	0.74	-M	0.34 ln
1	0.97	0.65	+M	0.33 ln

Location 2 (Negative Moment -M)

Table B.32 DCRs at Location 2 which represent column A1 in the transverse direction.

Number of stories in building	Demand/capacity ratios At center of column	 At 0.2 ln from face of support	 At 0.3 ln from face of support	Location where negative moment = 0
15	1.20	0.44	+M	< 0.33 ln
10	1.21	0.52	+M	< 0.33 ln
5	1.06	0.30	+M	< 0.33 ln
1	0.99	0.21	+M	< 0.33 ln

Location 3 (Negative Moment -M)

Table B.33 DCRs at Location 3 which represent column B2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios At center of column	 At 0.2 ln from face of support	 At 0.3 ln from face of support	Location where negative moment = 0
15	0.89	0.61	-M	0.34 ln
10	0.86	0.59	+M	0.33 ln
5	0.83	0.52	+M	< 0.33 ln
1	0.82	0.48	+M	< 0.33 ln

Location 4 (Positive Moment +M)

Table B.34 DCRs at Location 4 which represent column A2.

Number of stories in building	Demand/capacity Ratios
	At center of column
15	7.25
10	6.13
5	5.47
1	3.96

Case 3; Interior Column Removed

Location 1 (Negative Moment -M)

Table B.35 DCRs at Location 1 which represent column A2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.05	0.05	+M	< 0.33 ln
10	0.98	+M	+M	< 0.33 ln
5	1.06	0.17	+M	< 0.33 ln
1	0.85	+M	+M	< 0.33 ln

Location 2 (Negative Moment -M)

Table B.36 DCRs at Location 2 which represent column B1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.04	0.04	+M	< 0.33 ln
10	0.97	+M	+M	< 0.33 ln
5	1.06	0.16	+M	< 0.33 ln
1	0.85	+M	+M	< 0.33 ln

Location 3 (Negative Moment -M)

Table B.37 DCRs at Location 3 which represent column C2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.93	0.59	+M	< 0.33 ln
10	0.87	0.53	+M	< 0.33 ln
5	0.84	0.52	+M	< 0.33 ln
1	0.79	0.48	+M	< 0.33 ln

Location 4(Negative Moment -M)

Table B.38 DCRs at Location 4 which represent column B3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.95	0.63	+M	< 0.33 ln
10	0.90	0.57	+M	< 0.33 ln
5	0.86	0.55	+M	< 0.33 ln
1	0.82	0.49	+M	< 0.33 ln

Location 5 (Positive Moment +M)

Table B.39 DCRs at Location 5 which represent column B2

Number of stories in building	Demand/capacity Ratios
	At center of column
15	7.66
10	6.20
5	5.02
1	4.04

DCRs of positive moments for slabs detailed according to the seismic provisions of the CSA

Table B.40 DCRs at Location 4 which represent column A2 and at Location 5 which represent column B2.

Number of stories in building	Demand/capacity ratios at center of column	
	Case2 Location 4	Case3 Location 5
15	2.13	1.76
10	1.80	1.43
5	1.61	1.16
1	1.17	0.93

FS-7x3.5

Case 1; Corner Column Removed

Location 1 (Negative Moment -M)

Table B.41 DCRs at Location 1 which represent column A2 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.73	2.18	-M	0.47 ln
10	1.43	1.71	-M	0.45 ln
5	1.21	1.35	-M	0.43 ln
1	0.93	1.20	-M	0.64 ln

Location 2 (Negative Moment -M)

Table B.42 DCRs at Location 2 which represent column B1 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.83	0.36	+M	< 0.33 ln
10	0.75	0.27	+M	< 0.33 ln
5	0.68	0.21	+M	< 0.33 ln
1	0.67	0.18	+M	< 0.33 ln

Case 2; Edge Column Removed in the Longitudinal Direction

Location 1 (Negative Moment -M)

Table B.43 DCRs at Location 1 which represent column A1 in the transverse direction

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.77	0.10	+M	< 0.33 ln
10	0.66	+M	+M	< 0.33 ln
5	0.57	+M	+M	< 0.33 ln
1	0.54	+M	+M	< 0.33 ln

Location 2 (Negative Moment -M)

Table B.44 DCRs at Location 2 which represent column C1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.89	0.45	+M	< 0.33 ln
10	0.77	0.33	+M	< 0.33 ln
5	0.67	0.22	+M	< 0.33 ln
1	0.62	0.08	+M	< 0.33 ln

Location 3 (Negative Moment -M)

Table B.45 DCRs at Location 3 which represent column B2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	2.36	3.11	-M	0.53 ln
10	1.90	2.35	-M	0.51 ln
5	1.56	1.76	-M	0.48 ln
1	1.36	1.68	-M	0.63 ln

Location 4 (Positive Moment +M)

Table B.46 DCRs at Location 4 which represent column B1.

Number of stories in building	Demand/capacity Ratios
	At center of column
15	17.30
10	12.90
5	9.40
1	0.57

Case 3; Edge Column Removed in the Transverse Direction

Location 1 (Negative Moment -M)

Table B.47 DCRs at Location 1 which represent column A3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.51	1.54	-M	0.39 ln
10	1.38	1.31	-M	0.38 ln
5	0.99	0.87	-M	0.36 ln
1	0.72	0.52	+M	0.33 ln

Location 2 (Negative Moment -M)

Table B.48 DCRs at Location 2 which represent column A1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	2.32	1.69	+M	< 0.33 ln
10	1.83	1.17	+M	< 0.33 ln
5	1.47	0.77	+M	< 0.33 ln
1	1.02	0.18	+M	< 0.33 ln

Location 3 (Negative Moment -M)

Table B.49 DCRs at Location 3 which represent column B2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.60	0.12	+M	< 0.33 ln
10	0.57	0.08	+M	< 0.33 ln
5	0.55	0.05	+M	< 0.33 ln
1	0.54	0.02	+M	< 0.33 ln

Location 4 (Positive Moment +M)

Table B.50 DCRs at Location 4 which represent column A2.

Number of stories in building	Demand/capacity Ratios
	At center of column
15	13.72
10	9.94
5	7.16
1	3.94

Case 4; Interior Column Removed

Location 1 (Negative Moment -M)

Table B.51 DCRs at Location 11 which represent column A2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.60	+M	+M	< 0.33 ln
10	0.53	+M	+M	< 0.33 ln
5	0.48	+M	+M	< 0.33 ln
1	0.43	+M	+M	< 0.33 ln

Location 2 (Negative Moment -M)

Table B.52 DCRs at Location 2 which represent column B1 in the transverse direction.

Number of stories in building	Demand/capacity ratios At center of column	 At 0.2 ln from face of support	 At 0.3 ln from face of support	Location where negative moment = 0
15	2.26	1.08	+M	< 0.33 ln
10	1.80	0.70	+M	< 0.33 ln
5	1.36	0.36	+M	< 0.33 ln
1	1.08	+M	+M	< 0.33 ln

Location 3 (Negative Moment -M)

Table B.53 DCRs at Location 3 which represent column C2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios At center of column	 At 0.2 ln from face of support	 At 0.3 ln from face of support	Location where negative moment = 0
15	0.63	0.17	+M	< 0.33 ln
10	0.57	0.11	+M	< 0.33 ln
5	0.52	0.05	+M	< 0.33 ln
1	0.49	0.02	+M	< 0.33 ln

Location 4(Negative Moment -M)

Table B.54 DCRs at Location 4 which represent column B3 in the transverse direction.

Number of stories in building	Demand/capacity ratios At center of column	 At 0.2 ln from face of support	 At 0.3 ln from face of support	Location where negative moment = 0
15	2.17	2.06	-M	0.38 ln
10	1.71	1.53	-M	0.37 ln
5	1.34	0.87	-M	0.36 ln
1	1.13	0.88	-M	0.35 ln

Location 5 (Positive Moment +M)

Table B.55 Demand/capacity Ratios at Location 5 which represent column B2.

Number of stories in building	Demand/capacity Ratios
	At center of column
15	18.34
10	12.50
5	7.84
1	4.75

DCRs of positive moments for slabs detailed according to the seismic provisions of the CSA

Table B.56 DCRs at Location 4 which represent column B1, at Location 4 which represent column A2 and at Location 5 which represent column B2.

Number of stories in building	Demand/capacity ratios at center of column		
	Case2 Location 4	Case3 Location 4	Case4 Location 5
15	8.51	6.75	10.23
10	6.34	4.89	6.97
5	4.62	3.52	4.37
1	0.28	1.94	2.65

FS-5x5

Case 1; Corner Column Removed

Location 1 (Negative Moment -M)

Table B.57 DCRs at Location 1 which represent column A2 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.28	1.23	-M	0.40 ln
10	1.18	1.08	-M	0.39 ln
5	1.10	0.96	-M	0.37 ln
1	0.94	0.78	-M	0.37 ln

Location 2 (Negative Moment -M)

Table B.58 DCRs at Location 2 which represent column B1 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.32	1.28	-M	0.40 ln
10	1.21	1.12	-M	0.39 ln
5	1.13	0.99	-M	0.37 ln
1	0.95	0.81	-M	0.37 ln

Case 2; Edge Column Removed

Location 1 (Negative Moment -M)

Table B.59 DCRs at Location 1 which represent column A3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.33	1.21	-M	0.38 ln
10	1.17	0.97	-M	0.35 ln
5	1.06	0.84	+M	0.33 ln
1	0.94	0.70	+M	0.33 ln

Location 2 (Negative Moment -M)

Table B.60 DCRs at Location 2 which represent column A1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.49	0.96	+M	< 0.33 ln
10	1.23	0.57	+M	< 0.33 ln
5	1.11	0.44	+M	< 0.33 ln
1	0.96	0.26	+M	< 0.33 ln

Location 3 (Negative Moment -M)

Table B.61 DCRs at Location 3 which represent column B2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.94	0.71	-M	0.38 ln
10	0.86	0.59	-M	0.34 ln
5	0.81	0.51	+M	0.33 ln
1	0.74	0.45	+M	0.33 ln

Location 4 (Positive Moment +M)

Table B.62 DCRs at Location 4 which represent column A2

Number of stories in building	Demand/capacity Ratios
	At center of column
15	10.54
10	10.84
5	8.75
1	4.34

Case 3; Interior Column Removed

Location 1 (Negative Moment -M)

Table B.63 DCRs at Location 1 which represent column A2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.22	0.21	+M	< 0.33 ln
10	1.07	0.08	+M	< 0.33 ln
5	0.96	+M	+M	< 0.33 ln
1	0.85	+M	+M	< 0.33 ln

Location 2 (Negative Moment -M)

Table B.64 DCRs at Location 2 which represent column B1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.21	0.21	+M	< 0.33 ln
10	1.07	0.07	+M	< 0.33 ln
5	0.96	+M	+M	< 0.33 ln
1	0.85	+M	+M	< 0.33 ln

Location 3 (Negative Moment -M)

Table B.65 DCRs at Location 3 which represent column C2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.00	0.69	+M	0.33 ln
10	0.89	0.57	+M	0.33 ln
5	0.82	0.48	+M	< 0.33 ln
1	0.75	0.41	+M	< 0.33 ln

Location 4(Negative Moment -M)

Table B.66 DCRs at Location 4 which represent column B3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.02	0.72	+M	0.33 ln
10	0.91	0.60	+M	0.33 ln
5	0.84	0.52	+M	< 0.33 ln
1	0.79	0.46	+M	< 0.33 ln

Location 5 (Positive Moment +M)

Table B.67 DCRs at Location 5 which represent column B2

Number of stories in building	Demand/capacity Ratios
	At center of column
15	13.75
10	10.30
5	7.85
1	5.70

DCRs of positive moments for slabs detailed according to the seismic provisions of the CSA

Table B.68 DCRs at Location 4 which represent column A2 and at Location 5 which represent column B2.

Number of stories in building	Demand/capacity ratios at center of column	
	Case2 Location 4	Case3 Location 5
15	4.34	3.33
10	4.46	2.5
5	3.60	1.90
1	1.79	1.38

FS-9x9

Case 1; Corner Column Removed

Location 1 (Negative Moment -M)

Table B.69 DCRs at Location 1 which represent column A2 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.20	0.94	-M	0.36 ln
10	1.15	0.87	-M	0.35 ln
5	1.06	0.81	-M	0.34 ln
1	1.02	0.83	-M	0.36 ln

Location 2 (Negative Moment -M)

Table B.70 DCRs at Location 2 which represent column B1 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.15	0.99	-M	0.36 ln
10	1.09	0.90	-M	0.35 ln
5	1.07	0.85	-M	0.34 ln
1	1.02	0.86	-M	0.36 ln

Case 2; Edge Column Removed

Location 1 (Negative Moment -M)

Table B.71 DCRs at Location 1 which represent column A3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.20	1.00	-M	0.35 ln
10	1.15	0.95	-M	0.34 ln
5	1.06	0.82	+M	0.33 ln
1	1.02	0.79	+M	0.33 ln

Location 2 (Negative Moment -M)

Table B.72 DCRs at Location 2 which represent column A1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.13	0.43	+M	< 0.33 ln
10	1.11	0.47	+M	< 0.33 ln
5	0.89	0.27	+M	< 0.33 ln
1	0.93	0.19	+M	< 0.33 ln

Location 3 (Negative Moment -M)

Table B.73 DCRs at Location 3 which represent column B2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.88	0.63	-M	0.34 ln
10	0.85	0.60	+M	0.33 ln
5	0.81	0.53	+M	< 0.33 ln
1	0.79	0.52	+M	< 0.33 ln

Location 4 (Positive Moment +M)

Table B.74 DCRs at Location 4 which represent column A2

Number of stories in building	Demand/capacity Ratios
	At center of column
15	9.35
10	6.01
5	5.59
1	4.36

Case 3; Interior Column Removed

Location 1 (Negative Moment -M)

Table B.75 DCRs at Location 1 which represent column A2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.03	0.05	+M	< 0.33 ln
10	0.95	+M	+M	< 0.33 ln
5	0.89	+M	+M	< 0.33 ln
1	0.81	+M	+M	< 0.33 ln

Location 2 (Negative Moment -M)

Table B.76 DCRs at Location 2 which represent column B1 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	1.03	0.04	+M	< 0.33 ln
10	0.94	+M	+M	< 0.33 ln
5	0.89	+M	+M	< 0.33 ln
1	0.80	+M	+M	< 0.33 ln

Location 3 (Negative Moment -M)

Table B.77 DCRs at Location 3 which represent column C2 in the longitudinal direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.95	0.65	+M	< 0.33 ln
10	0.88	0.57	+M	< 0.33 ln
5	0.84	0.52	+M	< 0.33 ln
1	0.81	0.48	+M	< 0.33 ln

Location 4(Negative Moment -M)

Table B.78 DCRs at Location 4 which represent column B3 in the transverse direction.

Number of stories in building	Demand/capacity ratios			Location where negative moment = 0
	At center of column	At 0.2 ln from face of support	At 0.3 ln from face of support	
15	0.97	0.69	+M	< 0.33 ln
10	0.91	0.61	+M	< 0.33 ln
5	0.87	0.57	+M	< 0.33 ln
1	0.84	0.54	+M	< 0.33 ln

Location 5 (Positive Moment +M)

Table B.79 DCRs at Location 5 which represent column B2

Number of stories in building	Demand/capacity Ratios
	At center of column
15	8.33
10	6.68
5	5.42
1	4.66

DCRs of positive moments for slabs detailed according to the seismic provisions of the CSA

Table B.80 DCRs at Location 4 which represent column A2 and at Location 5 which represent column B2.

Number of stories in building	Demand/capacity ratios at center of column	
	Case2 Location 4	Case3 Location 5
15	4.47	2.81
10	2.87	2.26
5	2.67	1.83
1	2.09	1.57