ANALYTICAL AND EXPERIMENTAL EVALUATION OF PROGRESSIVE COLLAPSE RESISTANCE OF REINFORCED CONCRETE STRUCTURES

A Dissertation Presented

by

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to

The Department of Civil and Environmental Engineering

in partial fulfillment of the requirements

for the degree of

Doctor of Philosophy

in

Civil Engineering

Northeastern University

Boston, Massachusetts

August 2012
Abstract

Experimental and analytical studies are carried out on three full-scale actual reinforced concrete (RC) buildings to characterize system level resisting mechanisms against progressive collapse following an initial local damage. The results obtained from the analytical models are verified with the experimental data and important modeling issues, assumptions, and requirements for structural elements as well as infill walls are identified and discussed. In order to evaluate the effects of initial damage location on progressive collapse of structures, a seven-story RC building structure is designed. Using an analytical model of the structure based on the knowledge gained in this study, the structure is analyzed under 15 different initial damage scenarios.

None of the experimentally evaluated structures experienced full or partial collapse. Vierendeel frame action is found to be the dominant progressive collapse resisting mechanism following element removal in all evaluated structures. The seven-story building designed in this study is found to be susceptible to collapse under the initial damage scenario of top floor corner column removal and top floor middle column removal on its short edge. The capability of the structure to develop Vierendeel Frame Action is crucial in resisting progressive collapse. Structures are less susceptible to collapse if there are at least two floors (and connecting columns) above the removed column such that Vierendeel frame action can effectively develop. Vierendeel frame action can be characterized by double curvature deformations of beams, slabs, and columns. Such a deformed shape provides shear forces in beams and slabs required to redistribute gravity loads following column removal. The direction of bending moments in the elements in the vicinity of the removed column changes after column removal. A potential brittle failure mechanism is identified and described which can develop due to insufficient reinforcement and the change in the moment direction. It is shown that axial compressive force develops in beams and slabs due to their growth at small displacements. This axial force enhances the flexural capacities of floor elements (beams and slabs) and in turn improves the performance of the Vierendeel frame action considerably. In the building structures discussed above, the level of displacements and deformations were not large enough to develop Catenary action.

In order to study Catenary action response in RC structures, progressive collapse resistance of a scaled two-dimensional frame structure is studied experimentally and analytically. The frame experienced small deformations and resisted collapse after being subjected a column removal on its first floor. Following this test, the frame was also subjected to monotonically increasing displacement at the top of the removed column to further study the resisting mechanism(s). The Catenary action was observed experimentally and evaluated analytically.
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Acknowledgements

I would like to gratefully and sincerely thank my advisor, Professor Mehrdad Sasani for his continuous guidance and support throughout the course of this research. This work would not have been possible without his great expertise, knowledge and practically unlimited accessibility.

I would also like to thank Professor Dionisio Bernal, Professor Luca Caracoglia and Professor Jerome F. Hajjar for serving as readers and members of my dissertation committee. Having such wise faculty members and great staff like David Whelpley made my experience at Northeastern memorable.

I also thank my friends Ali Kazemi, Leila Keyvani and Justin Murray for providing valuable feedback and comments when reviewing my dissertation. Thanks are also due to my friends Omer Faruk & Gulhan Tigli, Arif Orcun Soylemez, Fatih & Zeynep Alemdar, Mustafa Ayazoglu, Selcuk & Begum Altay and Ragab & Samira Hamdoun. I will never forget their support and friendship.

Finally I would like to express my deep thankfulness to my family for their constant encouragement, support and patience. This dissertation is dedicated to them.
Chapter 1

Introduction

1.1 Overview

In the conventional design of buildings, the designer usually takes into account the self-weight of the structure (dead load), operational loads (live load), and depending on the location of the building, seismic, and climate related loads (wind and snow loads). While the vast proportion of the existing buildings experience only the types of loads mentioned above during their lifetimes, some of them could be subjected to abnormal loadings which they were not explicitly designed for. Past experience indicates that buildings may be vulnerable to blast-induced air pressures and related local damage. The source of the blast could be accidental or intentional as in the terrorist attacks to government or private buildings. If the initial damage caused directly by the blast propagates in the structure due to the incapability of the structure to redistribute the loads that were carried by the initially damaged elements to the neighboring elements, then the damage can be classified into two parts; damage due to blast and damage due to progressive collapse. The progressive collapse is defined in ASCE/SEI 7 (2010) as 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. The key characteristic here is that the total damage at the end is not proportional to the original cause.

1.2 Progressive Collapse Examples

One of the earliest well known examples of progressive collapse was the partial collapse of Ronan Point apartment in England in 1968 (Pearson and Delatte, 2003). The building had 22 stories and was built using precast concrete panel construction. An accidental gas explosion in the kitchen of the 18th floor that was located in the corner of the building blew out the load bearing exterior walls, removing the structural supports to
the four floors above. Since there was no alternative load paths for the upper floors when the 18th floor corner external walls were blown out, the four upper floors collapsed onto the 18th floor. The sudden impact loading on the 18th floor initiated a second phase of collapse, failure of the 17th floor and progressing in the lower floors until it reached the ground. While the initial damage due to the gas explosion was only on the 18th floor, at the end, the entire corner of the building collapsed. Four people were killed in the incident, and seventeen were injured. Figure 1.1 shows the final state of the Ronan Point apartment building after collapse.

The collapse of the Alfred P. Murrah building in Oklahoma City in 1995 can be given as another example of progressive collapse cases in history (Corley et al., 1998). The building was the target of a terrorist attack in which a truck bomb was detonated in front of the building. The blast caused extensive damage to the Alfred P. Murrah Building and various degrees of damage to other buildings in the surrounding area. 168 people were killed in the incident, and more than 800 were injured.

The Alfred P. Murrah Building was a nine-story reinforced concrete (RC) ordinary moment frame structure with shear walls. Different from the upper floors, there was a transfer girder at the third floor level in the north side of the building. The exterior columns that were supporting the transfer girder were 40 ft apart and had a dimension of 20” by 36”. The transfer girder was supporting the columns on the upper floors that were spaced at 20 ft. The curtain wall located in the north side was set back about 3 ft in the first two levels, providing an open space below the third level. The transfer girder at the third floor level and the open space can be clearly seen in the Figure 1.2 that shows the building before explosion.

Due to the blast, three exterior columns that supported the transfer girder in the third floor were destroyed. With the loss of these columns, the transfer girder at the third floor collapsed causing the progressive collapse of the upper stories. Corley et al. pointed out that most of the devastation was due to progressive collapse rather than direct effects of the explosion. It was estimated that the exterior frame would not have had the capacity
to resist its self-weight if any one of the first floor exterior columns that supports the transfer girder in the third floor level were lost.

After the bombing attack, almost half of the usable space in the building collapsed. Figures 1.3 and 1.4 show the failure boundaries and a view of the building after the explosion, respectively.

The World Trade Center in New York City was a target of terrorist attack in 1993, 8 years before the collapse of two towers in 2001. The attack occurred on February 26, 1993, when a car bomb was detonated below the North Tower of the World Trade Center. The bomb exploded in the underground garage, generating an estimated pressure of 150,000 psi. The bomb opened a 98 ft wide hole through four floors. Although explosion caused heavy damage in the garage (see Figure 1.5) the building did not collapse. Six people were killed and 1,042 others were injured (most during the evacuation that followed the blast) in the incident (Reeve, 1999).

The terrorist attacks on September 11, 2001 caused collapse of three buildings in the World Trade Center Complex, namely 1 WTC, 2 WTC (Twin Towers) and 7 WTC. Two commercial passenger jet airliners crashed into the 1 WTC and 2 WTC. The crash of the airliners caused extreme damage on the two towers but the buildings did not collapse immediately due to the strike of aircrafts (See Figure 1.6). According to the report prepared by the National Institute of Standards and Technology (NIST, 2008a) the fireproofing on the Twin Towers' steel infrastructures was blown off by the initial impact of the planes and that, if this had not occurred, the towers would likely have remained standing. The fires weakened the trusses supporting the floors, making the floors sag. The sagging floors pulled on the exterior steel columns to the point where exterior columns bowed inward. With the damage to the core columns, the buckling exterior columns could no longer support the buildings, causing them to collapse (NIST, 2008b). When the 1 WTC collapsed, debris that fell on the nearby 7 WTC building (See Figure 1.7) damaged it and initiated fires. NIST concluded that uncontrolled fires in 7 WTC caused floor beams and girders to heat and subsequently "caused a critical support column to
fail, initiating a fire-induced progressive collapse that brought the building down" (NIST, 2008c).

### 1.3 Scope and Objectives

In the context of progressive collapse, it may be unavoidable having some elements through the structure, especially close to the initiation spot, exceed their capacities and collapse. The crucial issue is if the damage would be contained in a limited area and if the structure would stabilize without partial or full collapse. To address this issue, the problem should be examined in system level.

In terms of experimental studies in the literature, while there are many tests conducted on the element level, very few tests on the system level behavior of structures are available. Specifically for progressive collapse of structures, other than the post-incident observations and evaluations of the damaged structures that were subjected to accidental events or terrorist attacks, almost no experimental data is available for the system level response of the structures that were subjected to localized damage. The main reasons for that are the high cost of full scale tests and the limitations of the laboratories.

Two guidelines that directly focus on reducing the potential of progressive collapse in buildings in the case of a local damage to the building, “Progressive Collapse Analysis and Design Guidelines” by General Services Administration (2003) and “The Unified Facilities Criteria Design of Buildings to Resist Progressive Collapse” by the Department of Defense (2010), mandate removal of one column or vertical load bearing element in the first story level to evaluate the performance of the building against progressive collapse (DOD (2010a) guidelines also mandate removal of one column or vertical load bearing element in other floors such as top and mid-height floors). First story building columns are especially vulnerable to car or truck bombs and they carry the largest axial force compared to the other floors.

Utilizing the initial damage scenarios stated in these guidelines, the experimental studies are carried out on real RC building structures as a part of this research. The buildings utilized in this study were to be demolished by implosion. Prior to the total
destruction of the buildings, selected first floor columns were removed by explosion of the columns. The behavior of the buildings is monitored through carefully implemented instrumentation and data collection. Analytical studies were also carried out for the buildings that were studied experimentally.

One objective of this research is to characterize the system-level resisting mechanisms of RC buildings that prevent the structure having partial or total collapse following an initial local damage. Characterization of the resisting mechanisms has been done using the results of the experimental and analytical studies performed on RC buildings as well as on a two-dimensional scaled frame structure. Another objective of this study is to identify and investigate important modeling issues and assumptions based on the evaluation of the structures mentioned above.

1.4 Organization of Dissertation

In Chapter 1, the progressive collapse phenomenon is introduced. Some examples of progressive collapse cases in history are presented. Scope and the objectives of the study as well as the organization of the dissertation are presented.

Chapter 2 presents the approaches given in two main guidelines prepared by the General Services Administration (GSA) and Department of Defense (DOD) that are currently used to evaluate the behavior of the structures against progressive collapse.

In Chapter 3, experimental and analytical studies performed on three full scale RC building structures are presented. The buildings studied experimentally were scheduled to be demolished. The instrumentation used and the data obtained from the buildings that were subjected to removal of one (or two) first floor column are presented. The analytical results are compared and presented along with the experimental results. The mechanisms developed following column removal that prevent the progressive collapse of the structures are discussed. Potential failure mechanisms are also discussed.

In Chapter 4, a 1/8\textsuperscript{th} scale model of a two-dimensional three-story four-bay frame structure is evaluated against progressive collapse. The 1/8\textsuperscript{th} scale frame was built and tested in the laboratory in two steps. In the first step, the frame was subjected to removal
of the center column in the first story while the all gravity loads were applied to the frame. Gravity loads were calculated based on the tributary areas of the building. The frame was able to transfer the loads of the removed column to the neighboring elements and did not collapse in the first step. In the second step, all external loads were removed and the frame was subjected to a monotonic vertical downward displacement at the top of the removed column. The data from the experimental study is presented along with the analytical results to evaluate the behavior of the frame.

Chapter 5 presents the issues and assumptions related to modeling of structures when progressive collapse analysis is considered. The modeling techniques used in the analytical modeling of the structures in Chapter 3 and Chapter 4 are described. The key factors that affect the estimated behavior of the structure in the analytical process are discussed.

In Chapter 6, a seven-story RC building structure designed according to current design codes is presented. Analytical model of the structure is developed based on the knowledge gained in this study and analyzed under nine initial damage scenarios. In each scenario, initial damage is removal of one column but the location of the removed column is different either in plan or over the height of the structure. Behavior of the structure for each case is studied and resisting mechanisms are characterized. Also the effects of the initial damage location on progressive collapse of structures are discussed.

Finally, Chapter 7 presents a summary of the dissertation and the conclusions derived from this study.
Figure 1.1 Collapse of Ronan Point Apartment (The Daily Telegraph, 1968)

Figure 1.2 Alfred P. Murrah Building before explosion (FEMA 277)
Figure 1.3 Failure boundaries in Alfred P. Murrah Building (Corley et al., 1998)

Figure 1.4 Alfred P. Murrah Building after explosion (Encyclopædia Britannica, 2012)
Figure 1.5 A view of the garage in WTC after explosion in 1993 attack

Figure 1.6 September 11 attacks to 1 WTC and 2 WTC Buildings (Photo Courtesy of FEMA)
Figure 1.7 A view of 7 WTC after the collapse Twin Towers (Photo Courtesy of FEMA)
Chapter 2

Current Practice in Evaluating Response of Structures Following Loss of Load Bearing Elements

2.1 Introduction

General Services Administration (GSA) and Department of Defense (DOD) developed two guidelines to address the issue of progressive collapse in the case of an abnormal loading in the structure in the design process of new buildings and in the evaluation of existing ones: “Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects” by GSA (2003) and “Unified Facilities Criteria (UFC): Design of Buildings to Resist Progressive Collapse” by DOD (2010a). In this chapter, these two guidelines are presented.

2.2 GSA Guidelines

2.2.1 Overview

The guidelines prepared by GSA aim to satisfy the requirements of Interagency Security Committee (ISC) Security Criteria regarding progressive collapse. The purposes of the guidelines are stated as follows:

- Assist in the reduction of the potential for progressive collapse in new Federal Office Buildings
- Assist in the assessment of the potential for progressive collapse in existing Federal Office Buildings
- Assist in the development of potential upgrades to facilities if required
To minimize the potential for progressive collapse in the design of new and upgraded buildings, and for assessing the potential for progressive collapse in existing buildings, the GSA guidelines provide a threat-independent approach.

In this approach, the structure is required to be analyzed and evaluated for the removal of a column or a vertical load bearing element such as a shear wall. Element removal is used as a load initiator. However, the element removal approach does not intend to mimic any specific threat scenario. Since it is not feasible to examine all potential sources of collapse initiation, the approach requires removal of columns (or shear walls) in a few representative locations.

The objective of the approach is not necessarily to prevent collapse initiation from a specific cause. It is to prevent or mitigate the potential for progressive collapse after having an initial damage as a result of an abnormal loading (e.g. blast). The philosophy of the guidelines focuses on arresting the progression of the collapse and reducing the extent of the damage by providing continuity, ductility and, robustness to the structure. At the element level, the concern of the guidelines is the post-event capacity and the behavior of the elements in the vicinity of the initial damaged area that are the key elements which control the progression of collapse rather than the robustness of the elements that are immediately affected by the original cause of damage.

GSA guidelines require the structure be analyzed for the case of an instantaneous loss of a primary vertical support and then be evaluated by utilizing the given analysis and acceptance criteria that are explained below. The recommended initial damage scenarios to be used for the analysis are also described below.

### 2.2.2 Analysis Techniques

The guidelines allow the use of linear analysis techniques (e.g. linear static or linear dynamic) for the evaluation of progressive collapse potential of the new as well as existing structures that have a typical structural configuration and have 10 or less stories above grade. The use of linear analysis techniques is intended to determine the potential for progressive collapse (i.e., a high or low potential for progressive collapse), not to
predict the response of the structure when it is subjected to the instantaneous removal of a primary vertical element. The assessment of the potential for progressive collapse using the results of a linear analysis is achieved by using the acceptance criteria in the form of appropriate Demand-Capacity Ratios.

For the structures that have more than 10 stories above the grade and/or have an atypical structural configuration, nonlinear analysis techniques (e.g. nonlinear static or nonlinear dynamic) in which the material and geometric nonlinearities are accounted for are recommended. For the assessment of the potential for progressive collapse using the results of a nonlinear analysis, less restrictive acceptance criteria are permitted to be used, recognizing the improved results that can be obtained from a more sophisticated procedure. The guidelines provide damage criteria that are outlined in Department of Defense Construction Standard (DOD, 2010a). The damage criteria to be used with nonlinear analysis provide the maximum allowable ductility and rotation limits for structural elements for different types of construction (e.g. RC, steel, masonry etc.) to limit the possibility of collapse.

Although it is recommended that 3-dimensional analytic models be used to account for potential 3-dimensional effects, the guidelines also allow use of 2-dimensional models if the analysis can adequately account for the general responses and 3-dimensional effects.

**2.2.3 Load Combinations to be used in Analysis**

The following vertical load combinations are recommended to be applied to the structure when assessing the potential for progressive collapse according to GSA guidelines.

When a static analysis procedure (either linear or nonlinear) is used the structure is required to be analyzed under following vertical load combination:

\[ \text{Load} = 2(DL + 0.25LL) \]
When a dynamic analysis procedure (either linear or nonlinear) is used the structure is required to be analyzed under following vertical load combination:

\[
\text{Load} = (\text{DL} + 0.25\text{LL})
\]

where, DL is dead load and LL is live load (higher of the design live load or the code live load). The coefficient of 2 in the load combination to be used in the static analysis procedure is used to account for the dynamic effects in the static analysis.

### 2.2.4 Recommended Initial Damage Scenarios

The guidelines classify the structures into two categories based on their configurations: typical and atypical structures. Typical structures are defined as having a relatively simple layout with no atypical structural configurations. Combination structures, vertical discontinuities, variations in bay size/extreme bay sizes, plan irregularities and closely spaced columns are considered as examples of possible atypical structural configurations. The guidelines recognize that most of the structures do not fall into the typical structures category since buildings often contain distinguishing structural features or details.

The guidelines provide the locations of the columns to be removed for the initial damage scenario for typical structures. For atypical structures, the same scenarios suggested for the typical structures can be used. In addition to those, however, the engineer who performs the evaluation is expected to determine initial damage scenarios that are critical for the specific structure under assessment using his/her engineering judgment.

The following scenarios are proposed to be used in the assessment for progressive collapse of **framed or flat plate structures** with typical structural configurations. Note that all the columns to be removed are located in the first story (above the grade). The suggested locations of the columns are schematically shown in Figure 2.1.

1. Instantaneous loss of a first floor column located at or near the middle of the short side of the building (Figure 2.1(a)).
2. Instantaneous loss of a first floor column located at or near the middle of the long side of the building (Figure 2.1(b)).

3. Instantaneous loss of a first floor column located at the corner of the building (Figure 2.1(c)).

If the building has an underground parking or uncontrolled public ground floor areas, then the following scenario is also required to be considered.

4. Instantaneous loss of one column that extends from the floor of the uncontrolled underground parking area or uncontrolled public ground floor area to the next floor (1 story). The column considered should be interior to the perimeter column lines (Figure 2.1(d)).

For the shear/load bearing wall structures, the following scenarios are proposed to be used in the assessment for progressive collapse. The suggested locations of the walls are schematically shown in Figure 2.2.

1. Instantaneous loss of one structural bay or 30 linear feet of an exterior wall section (whichever is less) for one floor above the grade, located at or near the middle of the short side of the building (Figure 2.2(a)).

2. Instantaneous loss of one structural bay or 30 linear feet of an exterior wall section (whichever is less) for one floor above the grade, located at or near the middle of the long side of the building (Figure 2.2(b)).

3. Instantaneous loss of the entire bearing wall along the perimeter at the corner structural bay or for the loss of 30 linear feet of the wall (15 feet in each major direction) (whichever is less) for one floor above the grade (Figure 2.2(c)).

If the building has an underground parking or uncontrolled public ground floor areas, then the following scenario is also required to be considered.

4. Instantaneous loss of one structural bay or 30 linear feet of an interior wall section (whichever is less) at the floor level of the underground parking area or uncontrolled
public ground floor area. The wall section considered should be interior to the perimeter bearing wall line (Figure 2.2(d)).

In GSA Progressive Collapse Analysis and Design Guidelines (2003), the vertical element removal is defined as the removal of the vertical element only. It is stated in the guidelines that the removal should not impede into the connection/joint or horizontal elements that are attached to the vertical element at the floor levels.

If the damage that the removed column had was extended into the connection/joint or horizontal elements at the top or bottom of the removed column, then the behavior of the structure would be completely different. Figure 2.3 shows the extents of the removal of a column as suggested in GSA Guidelines (2003).

The approach (i.e., the removal of a column or other vertical load bearing member) suggested in the GSA Guidelines (2003) is not proposed to imitate any specific extreme load or physical attack on the structure. The objective of the approach is to avert or mitigate the potential for progressive collapse, not necessarily to avert collapse initiation from a specific scenario.

2.2.5 Analysis Criteria

Analysis criteria in GSA guidelines define the boundaries of the allowable collapse area in the structure after instantaneous removal of a primary vertical support. The allowable extent of collapse is defined separately for exterior and interior element loss scenarios. The analysis criteria limits the allowable collapse area quantitatively in addition to using structural bay size as a measure to define the boundaries of the allowable collapse area since structural configurations may have abnormally large structural bay sizes.

The maximum allowable extents of collapse resulting from the instantaneous removal of an exterior primary vertical support member one floor above grade is limited to the structural bays directly associated with the instantaneously removed vertical member in the floor level directly above the instantaneously removed vertical
member or 1,800 ft$^2$ at the floor level directly above the instantaneously removed vertical member, whichever is the smaller area.

The allowable extents of collapse resulting from the instantaneous removal of an interior primary vertical support member in an uncontrolled ground floor area and/or an underground parking area for one floor level is limited to the structural bays directly associated with the instantaneously removed vertical member or 3,600 ft$^2$ at the floor level directly above the instantaneously removed vertical member, whichever is the smaller area.

2.2.6 Acceptance Criteria

GSA guidelines use an approach similar to the m-factor approach currently used in FEMA 273 (1997) and FEMA 356 (2000) to estimate the magnitude and distribution of the potential inelastic demands utilizing the results of linear elastic analysis. The approach relies on calculating Demand-Capacity Ratios (DCR) for structural elements and comparing DCR values with threshold values to identify the collapsed elements and connections for the assessment of the potential for progressive collapse. The DCR values are calculated with the following formula:

$$DCR = \frac{Q_{ud}}{Q_{ce}}$$

Where $Q_{ud}$ is acting force (demand) determined in component or connection/joint (moment, axial force, shear, and possible combined forces) obtained from the linear elastic analysis and $Q_{ce}$ is expected ultimate, un-factored capacity of the component and/or connection/joint (moment, axial force, shear and possible combined forces).

In the calculation of capacities of the components or connections (i.e. $Q_{ce}$ values), GSA guidelines allow to increase the design material strengths by strength-increase factors to determine the expected material strengths. Strength-increase factors given in the guidelines for both concrete and reinforcing steel are 1.25.
As explained above, the guidelines classify the structures into two categories based on their configurations: typical and atypical structures. The limit values for the DCR values are given separately for these two categories:

- **For typical structural configurations**, structural elements and connections that have DCR values that exceed **2.0** are considered to be severely damaged or collapsed.

- **For atypical structural configurations**, however, structural elements and connections that have DCR values that exceed **1.5** are considered to be severely damaged or collapsed.

The guidelines recognize the structures that have localized atypical configuration through the structure. In such cases, the criteria for atypical structural configurations (i.e., DCR < 1.5) may be only limited to the ‘atypical’ region.

**2.2.7 Suggested Linear Procedure for Assessing Potential for Progressive Collapse**

The GSA guidelines provide an iterative procedure to evaluate progressive collapse potential of the new as well as existing structures. The procedure utilizes linear static analysis since it is a less sophisticated analysis tool compared to nonlinear analysis techniques and hence it does not require the engineer to have qualifications and experiences in advanced structural engineering and nonlinear analysis techniques.

First, one column (or load bearing wall) according to one of the initial damage scenarios explained above is removed from the model and the structure is analyzed under the static load of 2(DL + 0.25LL) as mentioned before.

Secondly, DCR values for all members and connections are calculated and compared with the acceptance criteria to determine the failed members. The member end connections whose DCR values have exceeded the acceptance criteria based on shear force are to be considered a failed member. Moreover, the elements whose flexural DCR values exceed the acceptance criteria at both ends and in the span (creating a three hinged failure mechanism) are to be considered as a failed member. For the next analysis step,
failed members are removed from the model, and all gravity loads associated with these members are redistributed to other members in adjacent bays.

If the flexural DCR value exceeds the acceptance criteria at a member end or connection (and if the member does not have a three hinged failure mechanism) then a hinge is placed appropriately at the member end or connection to release the moment. At each inserted hinge, equal but opposite moments with a value of expected flexural capacity of the section are applied to each side of the hinge. The direction of the moments should be consistent with the analysis results.

After all failed members are removed from the model and hinges are placed for the yielding sections with associated moments, the new model is re-analyzed. The results are again evaluated to determine the failed members and yielded sections. This process continues until no DCR values exceed the acceptance criteria. In the final analysis, if the DCR values that exceed the acceptance criteria are inside the allowable collapse region, then the structure is considered to have a low potential for progressive collapse. Otherwise, the building is considered to have a high potential for progressive collapse.

2.3 Unified Facilities Criteria (UFC)

Department of Defense published the first Unified Facilities Criteria (UFC) for Design of Buildings to Resist Progressive Collapse in 2005. The document is updated in 2010 including significant changes. The document provides the design requirements necessary to reduce the potential of progressive collapse for new and existing facilities that experience localized structural damage as a result of accidental events.

Similar to GSA guidelines, the requirements in the UFC for Design of Buildings to Resist Progressive Collapse are not intended to directly limit or eliminate the initial damage since the initiating event is unpredictable. The objective of the UFC is to reduce the risk of mass casualties resulting from progressive collapse of the structure in the event of an attack. Moreover, it is stated in the document that the progressive collapse design requirements shall still apply even though a structure is designed to resist an identified explosive threat.
The progressive collapse design approaches in the UFC for Design of Buildings to Resist Progressive Collapse are primarily based on the occupancy of the building, although the structure’s function is also considered. The UFC defines four Occupancy Categories (OC) based on the level of occupancy and building function or criticality and then sets different levels of design requirements for each category. Three design approaches are proposed in the document to design new and existing structures to resist progressive collapse; Tie Forces, Alternate Path Method and Enhanced Local Resistance. Tie Forces is considered as an indirect design approach in ASCE/SEI 7-10 while Alternate Path Method and Enhanced Local Resistance are mentioned under direct design approaches.

Tie Forces Method prescribes a tensile force capacity of the floor or roof system, to allow the transfer of load from the damaged portion of the structure to the undamaged portion. In the Alternate Path Method, the building must bridge across a removed element and be capable to redistribute the load of the removed element to neighboring elements. In the Enhanced Local Resistance approach, the shear and flexural capacity of the perimeter columns and walls are increased to provide additional protection by reducing the probability and extent of initial damage.

The design requirements for different Occupancy Categories are shown in table 2.1. Note that the level of progressive collapse design of the structure becomes more intense as its level of occupancy or its criticality increases. The definitions of the Occupancy Categories can be found in UFC 3-301-01, Structural Engineering (DOD, 2010b).

2.3.1 Tie Forces Approach

In the Tie Forces approach, the building is designed to be able to develop alternate load paths to redistribute the loads of the damaged portion to the undamaged regions through the axial tensile forces (tie forces) of the structural elements. The existing structural elements that are designed without concerning about the ties could provide required tie forces. The approach categorizes the ties to be provided in the structure into four categories based on their location and alignment in the structure:
1. Longitudinal horizontal ties
2. Transverse horizontal ties
3. Peripheral horizontal ties
4. Vertical ties

The continuity and ductility play the key roles in the redistribution of the loads over a damaged region. It must be ensured that adequate splicing and anchorage are provided that allows development of the transverse, longitudinal, and peripheral tie forces. The different tie forces listed above are illustrated in Figure 2.4 for a frame structure.

In the approach, the structural elements are expected to provide required tie forces while undergoing rotations of 0.20 rad. (11.3 deg). If the beams, girders or spandrels and their connections were not enough to provide required tie forces then the floor system and roof are expected to carry the tie forces. The elements are designed such that their design strength (considering the strength reduction factors as specified in the appropriate material specific codes) is at least the required tie strength.

If the structural elements or connections cannot provide the required longitudinal, transverse, or peripheral tie strength, the element and connection is required to be redesigned or retrofitted to make it able to develop sufficient design tie strength. For the vertical ties, however, if any structural element or connection fails to provide vertical required tie strength, redesigning can be omitted if it can be proven that the structure is capable of bridging over this deficient element using the Alternate Path Method.

**2.3.1.1 Calculation of Required Tie Force**

The required tie forces are calculated based on the floor load and bay size. The following floor load is to be used in the calculation of the required tie strengths:

\[ w_f = 1.2D + 0.5L \]

Where \( w_f \) is floor load, D and L are dead and live load, respectively.
In the case of a significant variation of the floor load over the plan, one of the following approaches is used based on the level of variation of the loads over different parts:

a. Using an effective $w_f$ calculated by computing the total force acting on the floor and dividing by the total plan area.

b. Using the maximum floor load as the effective $w_f$.

c. Dividing the floor plan into sub areas according to their floor loads and providing longitudinal and transverse ties and peripheral ties separately for each sub-area. In other words, different required tie strengths are calculated for the areas under different floor loads. In this case, additional peripheral tie is also required between the sub-areas.

Cladding and façade loads are included in the calculation of peripheral and vertical tie forces while they are excluded for the transverse and longitudinal tie calculations.

2.3.1.2 Longitudinal and Transverse Ties

Ties are required to be provided in both orthogonal directions (longitudinal and transverse directions) of the floor system. If the floor system has beams, girders, etc. then they can be used to provide some or all of the required tie strength; if they and their connections can be shown capable of carrying the tie force while undergoing a 0.20-rad (11.3-deg) rotation. If the floor is made of flat slab without beams or girders, etc. then all the ties will be provided by the slab. In both cases, the spacing of the ties is required to be less than one fifth of the bay size in the corresponding direction.

The following formula is used to calculate the required tie strength for the longitudinal or transverse ties for framed structures as well as for load bearing wall structures:

$$F_i = 3 \, w_f \, L_i$$
Where \( F_i \) is the required tie strength, \( w_F \) is the floor load and \( L_i \) is the greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the direction under consideration. For the calculation of the required tie strength for the transverse direction of a one way span in a load bearing wall structure \( L_1 \) is calculated as \( 5h_w \) where the \( h_w \) is the clear story height.

### 2.3.1.3 Peripheral Ties

The peripheral ties are required to be provided all around the floor or roof, around the openings and between the sub-areas where different floor loads are present (i.e. heavily and lightly loaded areas). A proper anchorage at the corners of the peripheral ties must be provided. The peripheral ties are to be placed within 3 feet from the edge of the floor. If beams, girders, spandrels, etc can carry the peripheral tie force while undergoing a 0.20-rad (11.3-deg) rotation, then they can be utilized to provide some or all of the required tie strength.

The following formula is used to calculate the required peripheral tie strength for framed structures as well as for load bearing wall structures

\[
F_p = 6 \, w_F \, L_i \, L_p
\]

where \( w_F \) is floor load, \( L_i \) is the greater of the distances between the centers of the columns, frames, or walls at the perimeter of the building in the direction under consideration (for exterior peripheral ties) or the length of the bay in which the opening is located, in the direction under consideration (for peripheral ties at openings), and \( L_p \) is 3 ft.

### 2.3.1.4 Vertical Ties

Columns or load-bearing walls must carry the required vertical tie strength. Each column and load-bearing wall is required to be tied continuously from the foundation to the roof level. The required vertical tie strength in tension is equal to the largest vertical load received by the column or wall from any one story based on the tributary area and the floor load.
2.3.2 Alternate Path Method

The Alternate Path Method (APM) in the UFC for Design of Buildings to Resist Progressive Collapse follows the LRFD approach as in Tie Force Method. The design strength of members as well as their connections are required to have a design strength that is the product of the strength reduction factor and the nominal strength of the member (or its connections), including any over-strength factors, equal or more than required strength.

\[ \Phi R_n = \sum \gamma_i Q_i \]

where \( \Phi R_n \) is Design Strength
\( \Phi \) = Strength reduction factor (as specified in the appropriate material specific code)
\( R_n \) = Nominal Strength
\( \sum \gamma_i Q_i \) = Required Strength
\( \gamma_i \) = Load factor
\( Q_i \) = Load Effect

Three analysis procedures are proposed to be used:
1. Linear Static Analysis
2. Nonlinear Static
3. Nonlinear Dynamic

2.3.2.1 Element Classification

UFC classifies the structural elements into two categories: primary and secondary elements. The primary elements are the ones that contribute to the resistance of the structure to prevent collapse following removal of a vertical load-bearing element. All other elements are classified as secondary elements.

2.3.2.2 Structural Action Classification

The internal actions of the structural elements are classified into two groups: Force-controlled and Deformation-controlled actions. The classification is made by
comparing force deformation relationship of the structural actions of the elements with
the given three typical types of force deformation curves (See figure 2.5).

A primary component action is defined as deformation-controlled if it has a Type 1 curve and e = 2g, or, it has a Type 2 curve and e = 2g (See figure 2.5) A primary component action is defined as force-controlled if it has a Type 1 or Type 2 curve and e < 2g, or, if it has a Type 3 curve (See Figure 2.5).

A secondary component action is defined as deformation-controlled if it has a Type 1 curve with any e/g ratio or if it has a Type 2 curve and e = 2g. A secondary component action is defined as force controlled if it has a Type 2 curve and e < 2g, or, if it has a Type 3 curve (See Figure 2.5).

2.3.2.3 Force and Deformation Capacities of Elements

Linear Static Procedure

Capacities for deformation-controlled actions are defined as the product of \( m \)-factors (linear acceptance criteria in the form of demand to capacity ratio) and expected strengths, \( Q_{CE} \), multiplied by the appropriate strength reduction factor \( \Phi \). Capacities for force-controlled actions are defined as lower-bound strengths, \( Q_{CL} \), multiplied by the appropriate strength reduction factor \( \Phi \).

Nonlinear Procedures

Capacities for deformation-controlled actions are defined as permissible inelastic deformation limits. Capacities for force-controlled actions are defined as lower-bound strengths, \( Q_{CL} \), multiplied by the appropriate strength reduction factor \( \Phi \).

As can be seen above, UFC prescribes using the expected strength, \( Q_{CE} \) when evaluating the behavior of deformation-controlled actions. When evaluating the behavior of force-controlled actions, a lower bound estimate of the component strength, \( Q_{CL} \) is used.
2.3.2.4 Removal of Load-Bearing Elements for Alternate Path Method

2.3.2.4.1 Extent of Removed Load-Bearing Elements

Either for deficient vertical tie force case or for other APM cases, for column removal, the clear height between lateral restraints of the column in consideration is removed for the Alternate Path Method. For load bearing wall removal, a section of the wall with a length of twice the clear story height is removed. If the length of the wall is smaller than that, then the wall is removed completely.

Note that in both instances, the clear height of the element (column or wall) between lateral restraints is removed and the continuity and the strength of the joints below and above the removed element are not affected.

2.3.2.4.2 Location of Removed Load-Bearing Elements

If the Alternate Path Method is employed due to a Deficient Vertical Tie Force case, then the column (or load bearing wall) which fails to provide the required vertical tie strength is required to be removed from the structure. As mentioned above, if the deficient element is a load bearing wall then a section of the wall with a length of twice the clear story height is to be removed. If the length of the wall is more than that length, then the location of the removed section of wall within the length of deficient wall will be chosen to obtain the worst case scenario.

For other Alternate Path cases following locations are suggested to remove the load bearing elements to perform Alternate Path Method analysis.

**External Columns**

The locations below are suggested to remove external columns:

1. Near the middle of the short side
2. Near the middle of the long side
3. At the corner of the building
The engineer is expected to determine other critical column locations by using engineering judgment. (e.g. where the plan geometry of the structure changes significantly). The removal of the columns that the locations in the plan are mentioned above will be repeated at the different elevation levels listed below:

1. First story above grade
2. Story directly below roof
3. Story at mid-height
4. Story above the location of a column splice or change in column size

**Internal Columns**

If the structure has an underground parking or uncontrolled public access area, the internal columns in underground parking or uncontrolled public access area level (one story height) at the following locations are required to be removed to perform APM analyses:

1. Near the middle of the short side
2. Near the middle of the long side
3. At the corner of the uncontrolled space.

The engineer is expected to determine other critical column locations within the uncontrolled public access area by using engineering judgment.

**External Load-Bearing Walls**

The locations below are suggested to remove external load-bearing walls:

1. Near the middle of the short side
2. Near the middle of the long side
3. At the corner of the building

The length of the wall section to be removed is twice the clear story height. If two load bearing walls are intersecting at an exterior corner, then a length of the wall equal to the clear story height is required to be removed in each direction.
The engineer is expected to determine other critical load-bearing wall locations by using engineering judgment. (e.g. where the plan geometry of the structure changes significantly)

The removal of the load-bearing walls that the locations in the plan mentioned above will be repeated at the different elevation levels listed below:

1. First story above grade
2. Story directly below roof
3. Story at mid-height
4. Story above the location of a change in wall size

**Internal Load-Bearing Walls**

If the structure has an underground parking or uncontrolled public access area, the internal load-bearing walls in underground parking or uncontrolled public access area level (one story height) at the following locations are required to be removed to perform APM analyses:

1. Near the middle of the short side
2. Near the middle of the long side
3. At the corner of the uncontrolled space.

The engineer is expected to determine other critical column locations within the uncontrolled public access area by using engineering judgment.

The length of the wall section to be removed is twice the clear story height. If two load bearing walls are intersecting at an interior corner, then a length of wall equal to the clear story height is required to be removed in each direction.

**2.3.2.5 Structure Acceptance Criteria**

As mentioned before three analysis procedures are suggested in UFC for Design of Buildings to Resist Progressive Collapse; Linear Static Analysis Procedure, Nonlinear Static Analysis Procedure and Nonlinear Dynamic Analysis Procedure. For each
procedure, UFC defines acceptance criteria for primary and secondary elements, components, or connections. If the primary elements and components meet the acceptance criteria for the corresponding procedure, then the building satisfies the progressive collapse requirements, otherwise it must be re-designed or retrofitted.

2.3.2.6 Linear Static Analysis Procedure

The Linear Static Procedure can be employed only if the structure meets certain conditions about irregularity and Demand-Capacity Ratios (DCR). If there are no irregularities in the structure, then UFC allows using the Linear Static Procedure. If there are irregularities in the structure, then the use of Linear Static Procedure depends on the level of the Demand-Capacity Ratios of the structural elements under a given loading. A linear model of the structure with the specified column or load bearing wall removed is analyzed under gravity dead and live loads increased by the load increase factor $\Omega_{LD}$. Calculation of load factor $\Omega_{LD}$ is explained in the following sections. Then the DCRs are calculated with the following formula:

$$DCR = \frac{Q_{UDLin}}{Q_{CE}}$$

where $Q_{UDLin}$ is the resulting actions (internal forces and moments) in the element and $Q_{CE}$ is expected strength of the component or element.

If all of the component DCRs calculated are less than or equal to 2.0, then the linear static procedure can be used. If one or more of the DCRs exceed 2.0, then a linear static procedure cannot be used.

2.3.2.6.1 Analytical Modeling

In Linear Static Procedure, only three-dimensional models are allowed to be employed to model, analyze, and evaluate a building. Two-dimensional models are not permitted.

In the analytical model of the structure, only the primary elements and components are expected to be included. If the model includes also the secondary elements and components, their stiffness and resistance should be set to zero.
2.3.2.6.2 Loading

The structure is required to be analyzed separately for the deformation-controlled actions and the force-controlled actions. UFC defines different load combinations to calculate the deformation-controlled actions and the force-controlled actions.

2.3.2.6.2.a Load Case for Deformation-Controlled Actions ($Q_{UD}$)

In the calculation of deformation-controlled actions, the structure is required to be analyzed under gravity and lateral loads simultaneously. Four different analyses are required to be performed. In each analysis, the lateral loads are to be applied in one of each principal direction of the building while the gravity loads are the same. Gravity loads on the plan are not applied uniformly. Two different gravity load combinations are defined:

1. **Increased Gravity Loads for Floor Areas Above Removed Column or Wall**

   For the bays immediately adjacent to the removed element and at all floors above the removed element, the following load combination is required to be applied:

   \[ G_{LD} = \Omega_{LD} [(0.9 \text{ or } 1.2) \ D + (0.5 \text{ or } 0.2 \ S)] \]

   where  
   \( G_{LD} \) = Increased gravity loads for deformation-controlled actions for Linear Static Analysis  
   \( D \) = Dead load including façade loads  
   \( L \) = Live load including live load reduction per ASCE/SEI 7-10  
   \( S \) = Snow load  
   \( \Omega_{LD} \) = Load increase factor for calculating deformation-controlled actions for Linear Static analysis.

2. **Gravity Loads for Floor Areas Away From Removed Column or Wall**

   For all other areas over the plan, the following load combination is required to be applied:
\[ G = (0.9 \text{ or } 1.2) \ D + (0.5 \ L \text{ or } 0.2 \ S) \]

where \( G \) = Gravity loads

3. Lateral Loads

The following lateral load is required to be applied to each side of the building in combination with the gravity loads.

\[ L_{LAT} = 0.002\Sigma P \]

where
\( L_{LAT} \) = Lateral load
\( 0.002\Sigma P \) = Notional lateral load applied at each floor; this load is applied to every floor on each face of the building, one face at a time
\( \Sigma P \) = Sum of the gravity loads (Dead and Live) acting on only that floor; load increase factors are not employed.

2.3.2.6.2.b Load Case for Force-Controlled Actions \( Q_{UF} \)

Similar to the calculation of deformation-controlled actions, in the calculation of force-controlled actions, the structure is required to be analyzed under gravity and lateral loads simultaneously. Again a separate analysis is required to be performed for each principal direction of the building for the lateral loads to apply (four analyses in total) and two different gravity load combinations are defined for the floor areas above removed element and for the floor areas away from removed element.

1. Increased Gravity Loads for Floor Areas above Removed Column or Wall

For the bays immediately adjacent to the removed element and at all floors above the removed element, the following load combination is required to be applied:

\[ G_{LF} = \Omega_{LF} [(0.9 \text{ or } 1.2) \ D + (0.5 \ L \text{ or } 0.2 \ S)] \]

where
\( G_{LF} \) = Increased gravity loads for force-controlled actions for Linear Static analysis
\( D \) = Dead load including façade loads  
\( L \) = Live load including live load reduction per ASCE/SEI 7-10  
\( S \) = Snow load  
\( \Omega_{LF} \) = Load increase factor for calculating force-controlled actions for Linear Static analysis

2. Gravity Loads for Floor Areas Away From Removed Column or Wall

For all other areas over the plan, the following load combination is required to be applied:

\[
G = (0.9 \text{ or } 1.2) \ D + (0.5 \ L \text{ or } 0.2 \ S)
\]

where \( G \) = Gravity loads

3. Lateral Loads

The following lateral load is required to be applied to each side of the building in combination with the gravity loads.

\[
L_{LAT} = 0.002 \Sigma P
\]

where

\( L_{LAT} \) = Lateral load

0.002\( \Sigma P \) = Notional lateral load applied at each floor; this load is applied to every floor on each face of the building, one face at a time

\( \Sigma P \) = Sum of the gravity loads (Dead and Live) acting on only that floor; load increase factors are not employed.

2.3.2.6.3 Load Increase Factor

As stated above, UFC requires applying increased gravity loads to the floor areas above the removed column or wall. Amplification of the gravity loads is achieved by means of load increase factors. UFC provides load increase factors for the analyses to
calculate deformation-controlled and force-controlled actions separately. Table 2.2 shows the load increase factors to be used for reinforced concrete structures.

In Table 2.2, \( m_{LIF} \) is the smallest \( m \) of any primary beam, girder, spandrel or wall element that is directly connected to the columns or walls directly above the column or wall removal location. For each primary beam, girder, spandrel or wall element, \( m \) is the \( m \)-factor defined in Chapter 4 of UFC for RC. Columns are omitted from the determination of \( m_{LIF} \).

2.3.2.6.4 Component and Element Acceptance Criteria

The component and element acceptance criteria are given below for deformation-controlled or force-controlled actions:

**Deformation-Controlled Actions**

All primary and secondary components and elements shall satisfy the requirement below for deformation-controlled actions

\[
\Phi m Q_{CE} = Q_{UD}
\]

where

\( Q_{UD} \) = Deformation-controlled action, from Linear Static model
\( m \) = Component or element demand modifier (\( m \)-factor)
\( \Phi \) = Strength reduction factor.
\( Q_{CE} \) = Expected strength of the component or element for deformation-controlled actions.

**Force-Controlled Actions**

All primary and secondary components and elements shall satisfy the requirement below for force-controlled actions

\[
\Phi Q_{CL} = Q_{UF}
\]

where

\( Q_{UF} \) = Force-controlled action, from Linear Static model
\( Q_{CL} \) = Lower-bound strength of a component or element for force-controlled actions
\( \Phi \) = Strength reduction factor.

2.3.2.7 Nonlinear Static Analysis Procedure

Without any limitation, Nonlinear Static Procedure can be used for buildings with or without irregularities.

2.3.2.7.1 Analytical Modeling

In the nonlinear static procedure, only three-dimensional models are allowed to be employed to model, analyze, and evaluate a building. Two-dimensional models are not permitted. Model must have all primary elements and components. Model may also have secondary elements to make their check against the allowable deformation-controlled criteria easy but their stiffness and resistance must be set to zero.

The force-deformation behavior of all components is required to be explicitly modeled, including strength degradation and residual strength. The model of the RC structure is required to be developed by using the stiffness requirements of ASCE 41 (2007) Chapter 6. Adequate discretization must be provided to identify the inelastic regions. For the deformation controlled components, the nominal strengths shall be reduced by appropriate strength reduction factors \( \Phi \). The connections are required to be modeled explicitly when the connection is weaker or has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10%.

2.3.2.7.2 Loading

In contrast to Linear Static Procedure, the structure is required to be analyzed once for both the deformation-controlled actions and the force-controlled actions in Nonlinear Static Procedure.

As in the Linear Static Procedure, the structure is required to be analyzed under gravity and lateral loads simultaneously. Four different analyses are required to be performed in which the lateral loads are to be applied in one of each principal direction of
the building while the gravity loads are the same. Again, the gravity load combinations for floor areas above the removed element and for floor areas away from the removed element are defined separately as follows.

1. **Increased Gravity Loads for Floor Areas Above Removed Column or Wall**

   For the bays immediately adjacent to the removed element and at all floors above the removed element, the following load combination is required to be applied:

\[
G_N = \Omega_N [(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)]
\]

where
\(G_N\) = Increased gravity loads for Nonlinear Static Analysis
\(D\) = Dead load including façade loads
\(L\) = Live load including live load reduction per ASCE/SEI 7-10
\(S\) = Snow load
\(\Omega_N\) = Dynamic increase factor for calculating deformation-controlled and force-controlled actions for Nonlinear Static Analysis procedure (see Section 2.3.2.7.4).

2. **Gravity Loads for Floor Areas Away From Removed Column or Wall**

   For all other areas over the plan, following load combination is required to be applied:

\[
G = (0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)
\]

where \(G\) = Gravity loads

3. **Lateral Loads Applied to Structure**

   The following lateral load is required to be applied to each side of the building in combination with the increased gravity loads \(G_N\) and \(G\).

\[
L_{LAT} = 0.002\Sigma P
\]

where
**Chapter 2: Current Practice**

$L_{LAT}$ = Lateral load

$0.002 \Sigma P$ = Notional lateral load applied at each floor;

$\Sigma P$ = Sum of the gravity loads (Dead and Live) acting on only that floor; dynamic increase factors are not employed.

### 2.3.2.7.3 Loading Procedure

In the nonlinear static procedure, the loads are applied incrementally. Minimum 10 load steps are required to be used to apply loads.

### 2.3.2.7.4 Dynamic Increase Factor for NSP

In the Nonlinear Static Procedure, the gravity loads to be applied in the vicinity of the removed element are increased by the Dynamic Increase Factor (DIF), $\Omega_N$. While the DIF for load bearing wall structures is 2, DIF for framed RC structures is calculated with formula below:

$$\Omega_N = 1.04 + 0.45/(\theta_{pra}/\theta_y + 0.48)$$

where $\theta_{pra}$ is the plastic rotation angle and $\theta_y$ is the yield rotation. The values for $\theta_{pra}$ and $\theta_y$ for the structural elements are provided in the acceptance criteria tables in UFC. The determination of the DIF to be used in the procedure requires calculation of DIF for every primary element and connection in the model within or touching the area that is loaded with the increased gravity load and then picking the largest value as DIF for the analysis of the entire structure.

### 2.3.2.7.5 Component and Element Acceptance Criteria

**Deformation-Controlled Actions**

The deformation demands of primary and secondary elements and components shall not exceed their expected deformation capacities that are determined considering all coexisting forces and deformations.

**Force-Controlled Actions**
All force-controlled actions in all primary and secondary elements and components are required to satisfy following criterion:

$$\Phi Q_{CL} = Q_{UF}$$

where

- $Q_{UF}$ = Force-controlled action from Nonlinear Static model
- $Q_{CL}$ = Lower-bound strength of a component or element.
- $\Phi$ = Strength reduction factor.

### 2.3.2.8 Nonlinear Dynamic Analysis Procedure

Similar to Nonlinear Static Procedure, Nonlinear Dynamic Procedure can be used for buildings with or without irregularities, without any limitation.

#### 2.3.2.8.1 Analytical Modeling

In the nonlinear static procedure, only three-dimensional models are allowed to be employed to model, analyze, and evaluate a building. Two-dimensional models are not permitted. Model must have all primary elements and components. Model may also have secondary elements to make their check against the allowable deformation-controlled criteria easy but their stiffness and resistance must be set to zero.

The force-deformation behavior of all components is required to be explicitly modeled, including strength degradation and residual strength. The model of the RC structure is required to be developed by using the stiffness requirements of ASCE 41 (2007) Chapter 6. Adequate discretization must be provided to identify the inelastic regions. For the deformation controlled components, the nominal strengths shall be reduced by appropriate strength reduction factors $\Phi$. The connections are required to be modeled explicitly when the connection is weaker or has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10%.
2.3.2.8.2 Loading

Similar to Nonlinear Static Procedure, the structure is required to be analyzed once for both the deformation-controlled actions and the force-controlled actions in Nonlinear Dynamic Procedure. The structure is required to be analyzed under gravity and lateral loads simultaneously. A separate analysis is required to be performed in which the lateral loads are to be applied in one of each four principal directions of the building while the gravity loads are same (four analyses in total). Different from Linear Static and Nonlinear Static Procedures, the gravity load combinations for floor areas above removed element and for floor areas away from removed element are not defined separately. One load combination is used for the whole plan.

Gravity Loads for Entire Structure.

The following load combination is required to be applied to the entire structure.

\[ G_{ND} = (0.9 \text{ or } 1.2) \, D + (0.5 \, L \text{ or } 0.2 \, S) \]

where

\[ G_{ND} = \text{Gravity loads for Nonlinear Dynamic Analysis} \]
\[ D = \text{Dead load including façade loads} \]
\[ L = \text{Live load including live load reduction per ASCE/SEI 7-10} \]
\[ S = \text{Snow load} \]

Lateral Loads Applied to Structure Side

The following lateral load is required to be applied to each side of the building in combination with the gravity load \( G_{ND} \).

\[ L_{LAT} = 0.002 \Sigma P \]

where

\[ L_{LAT} = \text{Lateral load} \]
\[ 0.002 \Sigma P = \text{Notional lateral load applied at each floor;} \]
\[ \Sigma P = \text{Sum of the gravity loads (Dead and Live) acting on only that floor} \]
2.3.2.8.3 Loading Procedure.

First, gravity loads and lateral loads are applied to the structure gradually. Then the selected column or wall section is removed from the system instantaneously and dynamic analysis is continued until the maximum displacement is reached or one cycle of vertical motion occurs. If the computer analysis software is not capable of removing an element instantaneously, then the duration for removal must be selected being at most one tenth of the period that is associated with the structural response mode for the vertical motion of the bays above the removed column. Therefore a separate modal analysis of the structure without removed elements is required prior to column removal analysis.

2.3.2.8.4 Component and Element Acceptance Criteria

Deformation-Controlled Actions.

The deformation demands of primary and secondary elements and components shall not exceed their expected deformation capacities that are determined considering all coexisting forces and deformations.

Force-Controlled Actions.

All force-controlled actions in all primary and secondary elements and components are required to satisfy following criterion:

\[ \Phi Q_{CL} = Q_{UF} \]

where

- \( Q_{UF} \) = Force-controlled action from nonlinear dynamic model
- \( Q_{CL} \) = Lower-bound strength of a component or element.
- \( \Phi \) = Strength reduction factor.
2.3.3 Enhanced Local Resistance

In the Enhanced Local Resistance approach, the shear and flexural capacity of the perimeter columns and walls are increased to provide additional protection by reducing the probability and extent of the initial damage.

The Enhanced Local Resistance approach is required along with other approaches (e.g. Tie Forces, Alternate Path) depending on the occupancy category of the structure (See Table 2.1). As can be seen in Table 2.1, the number and location of the columns or walls that the Enhanced Local Resistance approach will be applied depends on occupancy category of the structure. Moreover, the extent of the requirements of the approach varies for different occupancy categories as explained below.

**For Occupancy Category II and III**

**Baseline Flexural Resistance**

In the Enhanced Local Resistance approach, the flexural resistance is defined as the magnitude of a uniform load acting over the height of the wall or load-bearing column which causes flexural failure, i.e. the formation of a three hinge mechanism or similar failure mode.

**Shear Resistance**

The shear resistance of the column, load-bearing wall, and their connections must be equal to or greater than the shear capacity associated with the baseline flexural resistance.

**For Occupancy Category IV**

**Enhanced Flexural Resistance**

The enhanced flexural resistance is determined based on two flexural resistances: baseline flexural resistance and existing flexural resistance. The baseline flexural resistance is calculated using the design of the structure when only gravity loads are
considered. The existing flexural resistance is calculated using the column and load-bearing wall design determined after the Alternate Path procedure was applied to the structural design that incorporated all applied loads (wind, earthquake, gravity, etc.).

For columns, the enhanced flexural resistance is the larger of the existing flexural resistance or 2.0 times the baseline flexural resistance. If the enhanced flexural resistance is greater than the existing flexural resistance, the column is required to be re-designed to match the enhanced flexural resistance load.

For load-bearing walls, the enhanced flexural resistance is the larger of the existing flexural resistance or 1.5 times the baseline flexural resistance. If the enhanced flexural resistance is greater than the existing flexural resistance, the load-bearing wall is required to be re-designed to match the enhanced flexural resistance load.

**Shear Resistance**

The shear resistance of the column, load-bearing wall, and their connections must be equal to or greater than the shear capacity associated with the baseline flexural resistance.
Table 2.1 Occupancy Categories and Design Requirements

<table>
<thead>
<tr>
<th>Occupancy Category</th>
<th>Design Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>No specific requirements</td>
</tr>
<tr>
<td>II</td>
<td>Option 1: Tie Forces for the entire structure and Enhanced Local Resistance for the corner and penultimate columns or walls at the first story. OR Option 2: Alternate Path for specified column and wall removal locations.</td>
</tr>
<tr>
<td>III</td>
<td>Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first story columns or walls.</td>
</tr>
<tr>
<td>IV</td>
<td>Tie Forces; Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first and second story columns or walls.</td>
</tr>
</tbody>
</table>

Table 2.2 Load increase factors for RC structures

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>$\Omega_{LD}$, Deformation-controlled</th>
<th>$\Omega_{LF}$, Force-controlled</th>
</tr>
</thead>
<tbody>
<tr>
<td>Framed</td>
<td>$1.2 , m_{LIF} + 0.80$</td>
<td>$2.0$</td>
</tr>
<tr>
<td>Load-bearing Wall</td>
<td>$2.0 , m_{LIF}$</td>
<td>$2.0$</td>
</tr>
</tbody>
</table>
Figure 2.1 Element removal scenarios for framed structures based on GSA (2003)

Figure 2.2 Element removal scenarios for shear/load bearing structures based on GSA (2003)
Figure 2.3 Boundaries of removed elements based on GSA (2003)
Chapter 2: Current Practice

Figure 2.4 Tie Forces in a Frame Structure (DOD, 2010a)

Figure 2.5 Definition of Force-Controlled and Deformation-Controlled Actions, from ASCE 41 (DOD, 2010a)
Chapter 3

Experimental and Analytical Evaluation of Response of Actual Buildings to Loss of Columns

3.1 Introduction

In general, while vast amount of data obtained from the tests done for various types of structural elements for different type of loadings are available in the literature; there are relatively few tests done using full scale structures. The main reasons for that are the high cost of full scale tests and the limitations of the laboratories. Although studying the behavior of a structure analytically is always an option; the assumptions and the techniques made through the modeling and the potential issues due to the limitations of the programs used need to be checked and verified with the available experimental data. This verification could be done in element level or in system level. Particularly for the progressive collapse of RC structures, there is a very limited data available to be used to study the behavior of the structures in system level or to verify the analytical models.

The behavior of a series of real structures subjected to loss of one or more columns are experimentally and analytically evaluated and presented in this chapter. Buildings that are scheduled to be demolished by implosions are utilized to obtain experimental data. In the process of demolition of a structure by implosion, prior to implosion of whole structure, only a few columns are exploded as a part of the demolition process which is carried by the demolition contractors. The damage scenario for the structures explained in 3.3 and 3.4 was the individual explosion of one column. In other words, there was not any other column exploded in the vicinity of the removed column afterwards. During the main implosion, however, large numbers of columns in the structure are exploded. There is a certain explosion sequence and there are time lags between the different groups of columns throughout the building during the main implosion. The columns removed in the evaluation of the structure explained in 3.2 were
the first two exploded columns of the main implosion sequence. The columns are carefully selected based on the negotiation with the demolition contractor and enough time lag is provided between first and second group so that after the first group of columns (two columns) are exploded, the structure had enough time to stabilize before the second group of elements are exploded. The buildings are carefully instrumented with number of sensors as well as monitored with surveillance cameras through the experimental program.

During this research three buildings that were demolished by implosion are visited and instrumented. In addition to experimental studies, analytical models of the buildings are also developed based on the structural drawings of the structures and analyzed for the same initial damage scenarios and the results are compared with the experimental results. The material characteristics used in the analytical studies are calculated from the tests of the specimens that are obtained from the buildings visited.

3.1.1 Hotel San Diego in San Diego, CA

Hotel San Diego was a 6 story RC structure. The south annex of the structure is evaluated during the experimental program. The building had exterior infill walls in all floors except the first (ground) and 3rd floors. The initial damage scenario for this building was the removal of two adjacent exterior columns. One of the removed columns was a corner column. The structure resisted progressive collapse with a maximum vertical displacement of 0.25” measured at the top of removed corner column. The dominant resisting mechanism for this structure is found to be the bi-directional Vierendeel action of the beams and columns above the removed columns with the participation of the infill walls existing in the structure. The results of the study are also presented in Sasani and Sagiroglu (2008) and Sasani (2008).

3.1.2 University of Arkansas Medical Center Dormitory in Little Rock, AR

The University of Arkansas Medical Center Dormitory was a 10 story RC structure. There were neither infill walls nor partitions in the building prior to removal of the column. The resisting mechanism was purely based on the performance of structural
elements without contribution from any nonstructural element. The initial damage scenario implemented in this building was the removal of an exterior ground floor column. The structure resisted progressive collapse with a maximum vertical displacement of 0.25” measured at the top of the removed column. The results of the study are also presented in Sasani et al. (2007).

3.1.3 Baptist Memorial Hospital in Memphis, TN

The Baptist Memorial Hospital was a 20 story RC structure consisting of four wings connected to a core. All wings were separated from the core with an expansion joint. The initial damage scenario studied for this building was the removal of an interior ground floor column of one of the wings.

The structure resisted progressive collapse with a maximum vertical displacement of 0.38” measured at the top of removed column. The dominant resisting mechanism for this structure is found to be the Vierendeel action of the beams and columns above the removed column with the participation of infill walls existed in the structure. The results of the study are also presented in Sasani and Sagiroglu (2010).

3.2 Hotel San Diego

3.2.1 Building Characteristics

Hotel San Diego was located in downtown San Diego, CA. It was composed of one main building built in 1914 and two annexes built in 1924 (see Figure 3.1). During the experimental program, the South-East annex of the building as highlighted in Figure 3.1 is evaluated. Figure 3.2 shows a picture of the building from street level with the removed columns marked. The evaluated building was a 6 story RC structure. The first story height was 19’-8” and the top story height was 16’-10”. The heights of other floors were 10’-6”. Figure 3.3 shows the typical floor plan. The building had 6 spans in the longitudinal direction and two spans in the transverse direction. The floor system was a one-way joist floor running in the longitudinal direction. Sizes of the elements between axes A and B in the longitudinal direction and between axes 1 and 3 in the transverse direction are shown in Figure 3.4. The cross section of joists and longitudinal beams
between axes A and B can be seen in Figure 3.5. Elevation views of axes A and B are shown in Figure 3.6.

Infill walls made of 8” thick hollow clay tiles existed on the peripheral beams with some openings. The infill wall pattern can be seen in Figure 3.7 for axes A, 1 and 3. There were no infill walls or partitions inside the building. As a part of the demolition process, the infill walls on the peripheral beams were completely removed from the first and third floors.

Prior to column removal, all nonstructural elements including partitions, plumbing, and furniture were removed from the structure. The loading was only due to the self weight of the structural elements (i.e. columns, beams, joists and slab) and infill walls.

Mechanical characteristics of the concrete and steel were calculated by performing tests on the samples obtained from the building. The concrete compressive strength was estimated as 4500 psi. Yield and ultimate stresses for the steel were estimated as 62 ksi and 87 ksi respectively.

The structural drawings that show the reinforcement details of the structural elements were available at the time of implosion. The reinforcement detail of beam A1-A2 and beam A3-B3 in the second floor are shown in Figure 3.8.

3.2.2 Initial Damage Scenario

The initial damage scenario for Hotel San Diego was the removal of columns A2 and A3 in the first (ground) floor. Removed columns are indicated with red circles in Figure 3.3 and highlighted in Figure 3.2. Column removal is achieved by explosion of the columns. Explosives are inserted into holes along the height of the columns by demolition contractors. These two columns were the first exploding columns of the implosion sequence. Enough time is allocated between the explosion of these two columns and the next group of columns to make sure that the structure would be stabilized before the next group of columns is exploded. However there was no time lag between the explosion of columns A2 and A3.
3.2.3 Experimental Evaluation

3.2.3.1 Sensors

In the experimental evaluation of Hotel San Diego, 20 linear potentiometers, 7 concrete strain gages, and one steel strain gage are used to capture the behavior of the structure. In addition to sensors, 4 video cameras recorded the response of the structure during the removal of the columns, two of which were located inside the 2nd floor and another two were located outside.

Potentiometers are used to capture the global displacements as well as the rotations of the beam end regions. 7 potentiometers used to capture global displacements are shown in Figure 3.9. 14 potentiometers that are used to capture the end rotations of beams A1-A2 and A3-B3 are shown in Figure 3.10. The rotation between the two cross-sections of a beam is calculated by first finding the difference in the top and bottom elongations and then dividing it by the height of the section. Because of that the potentiometers are located at the top and bottom of the beams. Top potentiometers on beams A1-A2 and A3-B3 are connected to the sides of the beams to not damage the infill walls. Note that after column removal, as explained later, beams A1-A2 and A3-B3 deformed in double curvature. As a result, compressive struts were expected to develop in the infill walls on top of these beams. Damaging the corners of the infill walls would change the development of these compressive struts.

In total, 7 concrete strain gages and one steel strain gage are used to measure the change in the strain of the beams and columns. Figure 3.11 shows the strain gages attached to the first and second floor columns. One strain gage (CS1) was attached to Column A2 in the first floor which was one of the exploding columns to determine the exact removal time of the columns in the evaluation of other sensor recordings. Note that all sensor recordings were synchronized. The time that columns A2 and A3 in the first floor were exploded obtained from the recording of strain gage CS1 is assumed as 0 (zero) in the time axis of other sensor recordings plots. The strain gages attached to first floor columns were located close to the top of the columns due to the field limitations. The strain gages attached to 2nd floor columns, however, were located at the mid-height
of the columns to minimize the effect of the bending moment developed in the columns on the strain readings. Fig 3.12 shows the strain gages attached to the second, third and fourth floor beams. After column removal, beams A1-A2, A2-B2 and A3-B3 are expected to deform in double curvature. Based on that, all concrete strain gages were located where the compressive strain is expected except BS1. Figure 3.13 shows the location of the steel strain gage attached to bottom rebar of beam A3-B3 at the A3 side. Again, based on the expected deformation pattern, this rebar is expected to experience tensile strains.

The concrete strain gages used in the Hotel San Diego were 3.5” long having a maximum strain limit of ± 0.02. The steel strain gage had a maximum strain limit of ± 0.2. The sampling rate used in the recording of the data collected from the sensors was 1000 Hz. The potentiometers had a resolution of about 0.0004” and a maximum operational speed of about 40 in/s. The maximum recorded speed in the experiment was about one third of the maximum operational speed. The potentiometers used in the building had three different displacement capacities; 1”, 2” and 4”. Selection of the type of potentiometer for each location is done based on the severity of expected deformation at that location.

3.2.3.2 Global Displacements

Figure 3.14 shows the vertical displacement of joint A3 in the second floor. The vertical displacement of joint A3 is calculated utilizing the recordings of three potentiometers that are connected to this joint (i.e. P1, P2 and P3) as shown in Figure 3.9. The readings obtained from these three potentiometers make it possible to calculate displacement of joint A3 in three dimensions (i.e. two horizontal and vertical). As can be seen from Figure 3.14, joint A3 had a first peak vertical displacement of 0.242” at 0.069 sec after the columns A2 and A3 in the first floor were removed. Following some vibration, it stabilizes at a vertical displacement of 0.242 after about 1.2 seconds.

Figure 3.15 shows the vertical displacement of joint A2 in the second floor. The vertical displacement of joint A2 on the second floor is calculated utilizing the recordings of two potentiometers that are connected to this joint (P4 and P5) as shown in Figure 3.9...
and assuming joint A2 and A3 had the same horizontal movement in the transverse direction. Similar to joint A3, joint A2 also had the same vertical permanent displacement of 0.242”. The displacement of joint A2 stabilizes at 0.6 second after column removal.

The vertical displacement of joint A1 in the second floor is presented in Figure 3.16. The vertical displacement of joint A1 is calculated utilizing the recording of potentiometer P6 as shown in Figure 3.9 and by ignoring horizontal movements of this joint. The validation of this assumption is checked with the analytical models later. Joint A1 had a first peak vertical displacement of 0.024”. After some fluctuation, it increased to a permanent displacement of 0.026”. The downward vertical displacement of joint A1 is due to an increase in the axial load of column A1. As explained in the later sections, most of the load that was carried by the columns A2 and A3 prior to removal of these columns was transferred to columns A1, B2 and B3. Increase in the axial load caused additional compressive strains in column A1 and as a result joint A1 on the second floor moved downwards.

Figure 3.17 shows the vertical displacement of joint A2 in the third floor. The vertical displacement of joint A2 on the third floor is calculated utilizing the recording of a diagonal potentiometer that was running from joint A1 on the second floor to joint A2 on the third floor (P7) as shown in Figure 3.9, using the known vertical movement of joint A1 on the second floor. Joint A2 in the third floor had a peak displacement of 0.215”. The vertical movement of the joint stabilizes after 0.4 seconds with a permanent displacement of 0.215” which is equal to the peak displacement. In figure 3.18 the vertical displacements of joint A2 in the second and third floors are plotted together. As can be seen from this figure the vertical displacement of Joint A2 in the third floor is smaller than the vertical displacement of Joint A2 in the second floor. After removal of columns A2 and A3 in the first floor, columns A2 and A3 in the third floor lost their supports and unbalanced forces developed at joints A2 and A3 in the second floor. These unbalanced forces caused the downward movement of these joints. As joints A2 and A3 moved downward while the joints A2 and A3 on the third floor had not experienced any unbalanced force, the columns A2 and A3 on the second floor elongated and the compressive strains on these columns reduced. As a result axial force dropped and caused
unbalanced forces at joints A2 and A3 in the third floors. This sequence continued up to the roof. Therefore, joint A2 in the third floor had experienced less vertical movement than the joint A2 in the second floor because of the elongation of column A2 in the second floor that is the result of loss in its compressive axial force.

### 3.2.3.3 Beam Deformations

As explained before, potentiometers are used to find the rotations at the end regions of selected beams. Based on the preliminary numerical analysis of the building before the removal of columns, beams A1-A2, A2-B2 and A3-B3 are expected to be the ones that would deform more severely than the other beams and transfer most of the loads that used to be carried by columns A2 and A3 to the neighboring columns. In total, 14 potentiometers were allocated to find the rotations at the end regions of beams A1-A2 and A3-B3 in the second floor. A set of two potentiometers was used to find the rotation between two cross sections of beam. One of them is connected to the bottom of the beam while the other one is connected to the side near the top of the beam. The potentiometers to measure the deformation at the top were attached to the sides near to the top edge instead of the top surface of the beam because infill walls were present on the top of the second floor beams.

The rotation of second floor beam A1-A2 on the A1 end is captured over two segments using 2 sets of potentiometers while one set of potentiometers was used on the other end of the beam (A2 end) to capture the rotation over one segment as shown in Figure 3.10(a). The rotations of second floor beam A3-B3 on the A3 end is captured over one segment using one set of potentiometers while three sets of potentiometers were used on the other end of the beam (B3 end) to capture the rotation over three segments.

Deformed shapes of beams A1-A2 in the second floor drawn by using the rotations of measured segments mentioned above at the peak displacement of joint A2 is shown in Figure 3.19(a). The expected deformed shape between the measured segments of beam is drawn with dashed lines.
Based on the measured rotations of end regions of beam A1-A2, it can be deduced that the A1 end of beam A1-A2 experienced a negative bending (i.e. tension at the top and compression at the bottom) while A2 end of the same beam experienced a positive bending (i.e. tension at the bottom and compression at the top). Figure 3.20 shows the recording of the strain gage BS1 that was attached to bottom of second floor beam A1-A2 at the A2 end (see Figure 3.12). As can be seen from this figure, the bottom of A2 end of beam A1-A2 experienced tensile strains following the removal of columns A2 and A3 in the first floor which is compatible with the deformed shape.

Figure 3.21 shows the recording of the strain gage BS2 that was attached to top of beam A1-A2 at the A2 end in the third floor. As can be seen from this figure the top of the A2 end of third floor beam A1-A2 experienced compressive strains following the removal of columns A2 and A3 in the first floor. The recording of strain gage BS2 indicates the same direction of rotation for the A2 end of A1-A2 beam in the third floor as in the second floor. Having similar vertical displacements of joints A2 in the second and third floors (see Figure 3.18) and based on the strain recording of BS2, it can be said that beam A1-A2 in the third floor deformed in double curvature and had a similar deformed shape as in the second floor.

The deformed shape of beam A3-B3 is drawn by using the rotations of measured segments at the peak displacement of joint A3 (see Figure 3.19(b)). Again, the expected deformed shape between the measured segments of beam A3-B3 is drawn with a dashed line. The measured rotations of end regions of second floor beam A3-B3 indicates that the A3 end of beam A3-B3 experienced a positive bending (i.e. tension at the bottom and compression at the top) while the B3 end experienced a negative bending (i.e. tension at the top and compression at the bottom).

The recording of the strain gage BS3 that was attached to top of third floor beam A3-B3 at the A3 end (see Figure 3.12) is shown in Figure 3.22. Increase in the compressive strain at this location indicates positive bending for the A3 end of the third floor A3-B3 beam as in the second floor.
Figure 3.23 shows the recordings of the steel strain gage RS1 that was attached to bottom rebar at the A3 end of second floor beam A3-B3 (see Figure 3.12). Bottom rebars at this location experienced tensile strain that is compatible with the measured rotations.

### 3.2.3.4 Load Redistribution Mechanism(s)

After removal of columns A2 and A3 in the first floor, the loads carried by these columns are transferred to neighboring columns without having partial or total collapse in the structure. The structure stabilized after the top of the removed columns displaced downward 0.242”. The elements in the vicinity of the removed columns were capable of transferring the loads carried by the removed columns to the neighboring columns without failure.

The concrete strain gages are used to monitor the change in the column strains and observe the redistribution of loads. Note that the recordings of strain gages show the change in the strain from the time of installation, not the total strain on the surface that the strain gage is attached.

Two strain gages were attached to each A1 and B3 columns in the first floor. CS2 and CS3 strain gages were attached to the north and south sides of column B3 respectively. The strain gages were located close to top along the height of the column due to field limitations. Figure 3.24 and 3.25 show the recordings of strain gages CS2 and CS3, respectively. The north side of column B3 experienced a compressive strain of 0.000044 while the change in the strain on the south side was 0.000075 (compressive). Both strain gages were located at the same height and were centered along the width of the column. Note that after removal of columns A2 and A3 in the first floor, column B3 was subjected to an additional axial load which was transferred from the removed columns and additional bending because of the new deformed shape of the structure after column removal. Based on the geometry of the structure and the measured deformation of the beam A3-B3 as shown in Figure 3.19(b), one can expect the top of column B3 in the first floor to bend mainly in a plane parallel to the longitudinal direction of the building. The direction of additional bending would be in the direction to satisfy the additional bending in the B3 end of A3-B3 beam which is expected to impose tension on the north
side and compression on the south side. The tension strain on the north side of the top of column B3 in the first floor due to additional bending would decrease the compressive strain in the column due to additional axial load. On the south side, however, the compressive strain due to additional bending would add up to the compressive strain due to additional axial load. Therefore, having more compressive strain on the south side than the north side of column B3 at its top height is the effect of additional bending at the top of column. Even though the additional bending imposes tensile strains to the north side of column B3 at the strain gage location, the compressive strains due to additional axial load overcomes the tensile strains and results in compressive strain in the recording of CS2.

Column A1 in the first floor had two strain gages attached, one on the north side (CS4) and the other on the east side (CS5). Both strain gages were located at the top part of column A1 along its height and centered along the width of column. Figure 3.26 and 3.27 show the recordings of strain gages CS4 and CS5 respectively. CS4 and CS5 show compressive strain with a value of 0.000038 and 0.000113 respectively. Again these values reflect the change in the strain after removal of columns not the total strains on the surfaces where the strain gages attached. Based on the geometry of the structure and the deformed shape of beam A1-A2 as presented in Figure 3.19(a), the additional major bending in column A1 would be in a plane parallel to the transverse direction of the building. To be in equilibrium with the bending at the A1 end of beam A1-A2, the top of column A1 in the first floor would bend developing tension on the west side and the compression on the east side. Therefore, having more compressive strain on the east side than the north side of column A1 at its top height is due to additional bending in the east-west plane.

As the recordings of strain gages CS2 and CS3 indicate, some portion of the load carried by the removed columns is transferred to column B3. Similarly, the recordings of strain gages CS4 and CS5 indicate another portion of the load carried by the removed columns is transferred to column A1.
Figure 3.28 shows the recording of strain gage CS6 which was attached to the south side of column A3 in the second floor at its mid-height. The recording shows tensile strain with a permanent value of 0.000146. Before removal of columns A2 and A3 in the first floor, columns A2 and A3 on the upper floors were carrying the load coming from the beams and floor system of their own floor as well as the axial force coming from the upper floors, transferring it to a lower floor. Following the loss of their supports in the first floor, the loads coming from the beams and floor found a new path to the neighboring columns and then to the foundation. When the axial load on columns A2 and A3 in the upper floors diminished, the existing compressive strains vanished and in turn these columns elongated. This elongation led to the recording of tensile strain in column A3 in the second floor, above one of the removed columns.

A finite element model as well as an applied element model of the building was developed to study the behavior of the structure analytically after removal of two columns. Results of the two models were compared with the measured experimental data. The conclusions about the load resisting mechanism(s) of Hotel San Diego after removal of two columns were made based on experimental and analytical results in the next section.

3.2.4 Analytical Evaluation using Finite Element Model

3.2.4.1 Model Description

A finite element model of the Hotel San Diego was developed in SAP 2000 (2005) based on the structural drawings of the building. Material characteristics used for concrete and steel were measured from the tests performed on the samples obtained from the building.

The columns, beams and joists containing the adjacent slabs were modeled with Bernoulli 2-node beam elements. Beams were modeled with T sections accounting for the slab. The effective flange width on each side of the web was set equal to four times the slab thickness (ACI 318, 2005). Material nonlinearity was imposed to the model by using localized plastic hinges at the critical locations. The localized plastic hinges are
assigned to the locations where yielding can occur (e.g. end of the elements and bar cut-off locations). The force-deformation relationships of plastic hinges are calculated by performing section analysis with the dimensions and reinforcement details obtained from structural drawings and material characteristics obtained from tests. As one can see in Figure 3.8(a) and 3.8(b), beams A1-A2 and A3-B3 did not have top reinforcement at their mid-span regions. Note that the top reinforcement of beams A1-A2 and A3-B3 bend very close to columns A1 and A3 respectively. If the negative bending moment reaches the cracking moment at those sections, strength of those sections would drop to practically zero. To account for this potential brittle failure, plastic hinges are assigned to the sections where top reinforcement ends. The capacities of the plastic hinges are set equal to the cracking moment. As soon as the demand reaches the cracking moment the plastic hinges lose their strength without any strain hardening.

SAP2000 keeps the stiffness of the beam elements constant during the analysis. To account for the cracking of the elements properly an iterative approach is followed:

i) First, all elements are assumed to be uncracked and have full flexural stiffness and a nonlinear time history analysis is run simulating column removal (Analytical column removal is explained later).

ii) The bending moment demands of the beam elements are compared with the cracking moments for each beam section.

iii) The flexural stiffness of the regions that have bending moment demand exceeding the cracking moment are reduced with the coefficients of 0.35 and 0.7 for beams and columns respectively as suggested in ACI-318. The stiffness reduction due to cracking is carried out in an iterative manner. Instead of reducing the stiffness of all consecutive beam segments which has a moment demand larger than the cracking moment, only the stiffness of the segments with the largest moment demand are reduced. Therefore the fact that having smaller flexural stiffness would cause smaller bending moment demands in the next analysis has also been accounted for.
iv) Another nonlinear time-history analysis is run with the new model. The bending moments are again compared. The elements that have bending moment demands larger than the cracking moment are determined and the flexural stiffness is reduced with the logic explained in (iii). Note that one would expect the regions that the bending moment exceeds the cracking moment to be less in this analysis compared to those in step (iii) because of the reason explained in the previous step.

v) Step (iv) is repeated until all cracked regions are appropriately modeled.

Since the floor system was a one way joist, all of the load on the floor is assumed to be transferred to neighboring transverse beams only. For this reason, the floor system is modeled with beam elements running in the longitudinal direction of the building. The beam elements modeling the floor system also had T sections. Only the slab between the Axis 2 and 3 (except the joist in the middle) was modeled with rectangular beam elements.

The modeling of infill walls that existed on the peripheral beams was achieved following two different methods: one with compressive struts and another with shell elements. The performance of both methods is discussed in the next sections with comparisons to the experimental data.

The modeling of infill walls with struts is performed by following the procedure explained in FEMA 356 (2000). In this method, the compressive struts are placed in the model. The location, orientation, effective width and the other characteristics of the struts are determined based on FEMA 356 (2000). The location and orientations of the struts modeling the infill walls in Axis A and Axis 3 are shown in Figure 3.29.

In another model, the infill walls are modeled with two dimensional shell elements. The thickness of the shell elements is set equal to the thickness of bricks. The size of the shell elements is selected based on the size of the bricks. The modulus of elasticity of the infill walls is set equal to 644 ksi (FEMA356, 2000). The cracking of infill walls has a significant effect on the in-plane stiffness of the frame. SAP2000, however, was not capable of tracking the cracking of shell elements as in the beam
elements. To account for the cracking of the shell elements that model the bricks of infill walls, an iterative procedure is followed:

i) First, the model is analyzed assuming there are no cracks in the elements.

ii) The tensile stresses developed in the shells are compared with the tensile strength of infill walls. The tensile strength of the infill walls is set equal to 26 psi (FEMA356, 2000). The shell elements are separated at the nodes where the tensile stresses exceed the tensile strength.

iii) A new model is re-analyzed and the tensile stresses are compared again with tensile strength. The elements are separated at the nodes where the tensile stresses exceed the tensile strength.

Step (iii) is repeated until the cracks in the infill walls are properly modeled.

Note that the determination of the cracked regions in the beam elements and the shell elements has been done simultaneously since the cracking in some elements would affect the behavior of the structure and as a result the demand in the other elements.

Element removal during an analysis is not readily available in SAP 2000. To obtain the response of the structure following the removal of two columns analytically, the procedure below is used:

i) The structure is analyzed under dead loads. Note that the model used for this analysis includes two removed columns. The 6 internal forces (one axial force, two shear forces, two bending moments and torsion) at the top ends of the two removed columns are obtained.

ii) A new model is developed by deleting the two removed columns from the previous model. Again the structure is analyzed under dead loads. Along with the dead loads, the internal forces obtained for the removed columns in the previous step are applied to the top of the two removed columns as external loads simulating the effect of the columns that are non-existent in the model. Note that the results of this model and the model analyzed in (i) are identical.
iii) After dead loads and end forces of the removed columns are applied to the structure as explained in (ii), another set of external forces that have same magnitudes but opposite directions with the column end forces that are applied along with the dead loads are applied to the structure in 1 millisecond and a nonlinear time history analysis is run. By applying the external forces that have the same magnitudes but opposite directions with the column end forces, the effect of the columns that is simulated by applying their end forces found in (i) is simply cancelled out.

### 3.2.4.2 Global Displacements

Figure 3.30 shows the vertical displacement of Joint A3 in the second floor obtained from the analysis of two finite element models in which the infill walls were modeled using compressive struts and two-dimensional shell elements. It also shows the vertical displacement of the same joint recorded experimentally. As one can see, the displacement obtained from the model where infill walls were modeled using compressive struts was considerably larger than the one that uses two dimensional shell elements. Note that the displacement history obtained from the model which uses shell elements is closer to the displacement history obtained experimentally compared to the one obtained from the model that used compressive struts. The peak displacement obtained from the model that used shell elements is 0.252”, which is only 4% more than the peak displacement obtained experimentally (0.242”). The peak displacement obtained from the model that used compressive struts is 0.449”, which is 86% more than the peak displacement obtained experimentally. The peak displacement occurs at around 0.069 sec for the model that used shell elements to model infill walls, which is the same time obtained experimentally. For the model that struts were used to model infill walls, however, the peak displacement occurs approximately 14 milliseconds later. The permanent displacement obtained from the model that used shell elements is 0.208”, which is 14% less than the permanent displacement obtained experimentally (0.242”). The peak displacement obtained from the model that used compressive struts is, however, 0.347”, which is 43% more than the permanent displacement obtained experimentally. While the model that utilizes shell elements to model the infill walls stabilizes after 0.8 seconds, the model that utilizes compressive struts takes longer to stabilize.
Similar to Figure 3.30, Figure 3.31 shows the vertical displacement of Joint A2 in the second floor obtained from the analysis of the two finite element models as well as the experimentally obtained displacement history for the same joint. As one can see, the displacement obtained from the model in which the infill walls were modeled using compressive struts is again considerably larger than the one that used two-dimensional shell elements to model the infill walls. Again the displacement history obtained from the model which used shell elements is closer to the displacement history obtained experimentally compared to the history obtained from the model that utilized compressive struts. The peak displacement obtained from the model that used shell elements is 0.241” and occurs at 0.065 sec. After approximately 1 second, it stabilizes at 0.182” of vertical displacement, which is 25% less than the permanent displacement obtained experimentally (0.242”). The peak displacement obtained from the model that used the compressive struts is 0.404” while the permanent displacement is 0.275”. This is 14% more than the permanent displacement obtained experimentally.

Both Figures 3.30 and 3.31 indicate that using compressive struts to model the infill walls underestimates the in-plane stiffness of infill walls. Note that while the shell elements were connected to neighboring beams and columns at several joints, the struts are attached to the frame at only two end joints. As a result, the shell elements put more constraints on beams and columns than the struts, which results in a stiffer structure. Given the fact that the results of the model that used shell elements to model the infill walls has better agreement with the experimental results than the results of the model with compressive struts, from now on only the results of the model with shell elements will be presented.

Figure 3.32 shows the vertical displacement of Joint A2 in the third floor obtained from the analysis in which the infill walls were modeled using two dimensional shell elements as well as the experimental vertical displacement. The peak vertical displacement obtained analytically is 0.232” and the permanent vertical displacement is 0.173 which is 20% less than the peak vertical displacement obtained experimentally (0.215”). For this joint, the vertical displacement obtained experimentally reaches its maximum value when the structure stabilizes, not just after the removal of the columns.
Figure 3.33 and 3.34 show the vertical displacements of joint A3 and A2 in different floors, respectively. As can be seen from the figures, joints A2 and A3 in different floors move downward almost same amount. Note that the each joint moved a little bit more than the joint in floor above. This is mainly because of the fact that the columns elongated after they lost their compressive force, as observed in the experimental evaluation of the structure.

3.2.4.3 Beam Deformations

As explained before, the rotations of the end regions of beams A1-A2 and A3-B3 are measured using 7 pairs of potentiometers and presented in 3.2.3.3. The analytically obtained deformed shapes of these beams are drawn in Figure 3.35 and 3.36, respectively, along with the measured deformed shapes. Note that estimated deformed shapes of the mid regions of the beams where there were no potentiometers installed are shown with dashed lines. As can be seen from Figure 3.35 and 3.36, deformed shapes of beams A1-A2 and A3-B3 obtained experimentally and analytically show good agreement.

Figure 3.37(a) shows the bending moment diagram of beam A1-A2 in the second floor after dead loads are applied (all columns are in place). As one expects the end regions experience negative bending moment while the bending moment at the mid-span is positive. Figure 3.37(b) shows the bending moment diagram of same beams after columns A2 and A3 in the second floor are removed and the structure stabilizes (around 1.6 sec after column removal). Note that the negative bending moment demand increases at the A1 end of the beam. The bending moment at the A2 end of the beam, however, changes from negative to positive. Note that if there were no bottom rebar at these sections and if the bending moment demand reached the cracking moment of the section, brittle failure could have happened at this location. Note that the experimentally measured rotations also do not indicate any brittle failure at this location.

Similar to beam A1-A2, Figure 3.38(a) shows the bending moment diagram of beam A3-B3 after dead loads are applied (all columns are in place). Again, the end regions experience negative bending moment while the bending moment at the mid-span
is positive as expected. Figure 3.38(b) shows the bending moment diagram of same beam after columns A2 and A3 in the first floor are removed and the structure stabilizes. Note that the negative bending moment demand increases at the B3 end of the beam. The bending moment at the A3 end of the beam, however, changes from negative to positive. Note that if there were no bottom rebar at these sections and if the bending moment demand reached the cracking moment of the section, brittle failure could have happened at this location as well. Note again that the measured rotations also do not indicate any brittle failure at this location.

Moment diagrams of both beams, A1-A2 and A3-B3, are in agreement with the measured end rotations showing a change in the direction of bending moments at the A2 end of beam A1-A2 and the A3 end of beam A3-B3. The shape of the bending moment obtained analytically is explained as follows.

Figure 3.39 shows different types of loadings on beam A3-B3 schematically. Figure 3.39 (b) shows the loading due to distributed dead load on the beam and the corresponding moment diagram. This is the only loading on the beam prior to column removal. Note that the shape of the analytically obtained moment diagram of beam A3-B3 shown in Figure 3.38(a), which belongs to the dead load analysis is compatible with the shape shown Figure 3.39 (b). Figure 3.39 (c) shows the loading due to end displacement of the beam and its corresponding moment diagram. After removal of columns, joints A2 and A3 experienced vertical displacements while joints A1 and B3 did not experience any remarkable displacement. Note that joints A1 and B3 also moved downward due the compressive strains developed in columns A1 and B3 following column removal but these displacements are negligible compared to the vertical displacements of joints A2 and A3. Finally, Figure 3.39 (d) shows the loading due to the effect of infill walls on the beam after column removal and the corresponding moment diagram. Although the effect of infill walls on the beams and columns is distributed, for explanation purposes the compression strut effect of infill walls is shown in the figure. The effect of compressive struts on the beams is shown with concentrated loads. Note that the effect of infill walls on the beams is dependent on the relative end displacement of the beam. In other words, the compressive struts in the infill walls develop when the
frame holding the infill wall deforms in sway action. After removal of the columns in the first floor, beam A3-B3 was under the effect of these three types of loading. Figure 3.39 (a) shows the resulting bending moment diagram that is the superposition of the moment diagrams shown in Figure 3.39 (b), (c) and (d). Note that Figure 3.39 shows the components of the resulting moment diagram of beam A3-B3 schematically. The bending moment values are not to scale.

Figure 3.40 shows the moment diagram of beams A1-A2 and A2-A3 in different floors before and after column removal (after the structure stabilizes). Note that A1-A2 beams develop more moment than the A2-A3 beams. As shown in Figure 3.33 and 3.34, joints A2 and A3 moved downward almost the same amount. As a result, the A2-A3 beams did not deform much compared to the A1-A2 beams. In other words, the effect of end displacement as shown in Figure 3.39 (c) is not as considerable for beam A2-A3 as it is for beam A1-A2.

Note that the bending moment diagrams of the A1-A2 beams in different floors have different patterns. As shown in Figure 3.34 the displacements of joint A2 in different floors are almost same while the A1 joints in all floors have practically zero displacement compared to joints A2. So the effect of end displacement on the bending moment diagrams of A1-A2 beams on different floors is very close. The dead load coming from the floors is also same since the structural elements have same dimensions in all floors. Therefore, the difference between the patterns of bending moments in different floors is because of the fact that infill walls existed in floors 2, 4, 5 and 6 and were removed in floors 1 and 3. It can be clearly seen from the moment diagrams of beams A1-A2 in the 2nd and 4th floors as well as the moment diagrams of the same beams in the 5th and 6th floors. Both pair of floors had the same infill wall pattern in the floors above and below, resulting in the same bending moment pattern in the corresponding beams.

Figure 3.41 shows the moment diagram of beams A3-B3 in different floors before and after column removal (after the structure stabilizes). Again the bending moment diagrams of beams A3-B3 in different floors have different patterns because of the reason
explained previously. The different infill wall patterns results in different bending moment patterns. For the A3-B3 beams it can be clearly seen from the moment diagrams in the 2nd and 4th floors as well as the moment diagrams of the same beam in the 5th and 6th floors. Both pairs of floors had the same infill wall pattern in the floors above and below resulting in the same bending moment pattern.

Figure 3.42 shows the moment diagram of beams A2-B2 before and after column removal. Note that the moment diagram after column removal is obtained when the structure is stabilized. Since there were no infill walls on beams A2-B2, the moment diagram is a result of only dead loads and end displacement of joint A2. Without the effect of infill walls, and having almost same end displacement on each floor, beams A2-B2 in different floors have almost the same bending moment diagram.

Analytically obtained deformed shapes of the elements in axis A after removal of the columns are shown in Figure 3.43 with a scale factor of 200. Note that this deform shape is obtained after the structure stabilized. As can be seen, the A1-A2 beams deformed in double curvature while no significant deformations were observed for beams A2-A3. No yielding was observed in the beams in Axis A. However, two sections that did not have top reinforcement reached their cracking moment capacity and cracked. Those two sections are shown in the figure.

Figure 3.44 shows the deformed shapes (after the structure stabilizes) of the elements in Axis 3 between axes A and B. Similar to the A1-A2 beams, the A3-B3 beams deformed in double curvature. In this axis, two sections yielded while two other sections that did not have top reinforcement cracked. Both yielded and cracked sections are highlighted in the figure.

3.2.4.4 Load Redistribution Mechanism(s)

After the columns A2 and A3 in the first floor are removed, the loads originally held by these columns found a new path to reach to the foundation of the building. After removal of the columns, neighboring columns in the vicinity of the removed columns are expected to take the loads that used to be carried by the removed ones.
Table 3.1 shows the axial forces of the columns in axes A and B before and after removal of columns A2 and A3 in the first floor. As can be seen from the table, the axial force in columns A1, B2 and B3 show a significant increase after removal of columns A2 and A3 in the first floor. That is, the loads that were carried by the removed columns were transferred to these columns.

The mechanism by which the loads are transferred to columns A1, B2 and B3 is called Vierendeel action. That is, beams A1-A2, A2-B2 and A3-B3 are deformed in double curvature due to the moment connections with columns and develop shear forces to transfer the loads on them. In addition to Vierendeel action, the infill walls also helped transfer some portion of the loads to the neighboring columns.

After removal of columns A2 and A3 in the first floor, the A3 end of beam A3-B3 started to move downwards while the other end of the beam, B3, stayed relatively in place. Since moment connections existed with the columns at both ends of the beams, the A3-B3 beams deformed in double curvature. This deformation imposed additional positive bending moment to the A3 end while it imposed additional negative bending moment to the B3 end. Prior to column removal, the A3 end of the beam was under negative bending moment as a result of dead loads. As shown in Figure 3.38, the imposed positive bending moment after column removal overcame the existing negative bending moment and the resulting bending moment was positive. Only the third floor A3-B3 beam at the A3 end experienced yielding. Since the beam had bottom reinforcement that is equal to the minimum reinforcement of the section at its A3 end, this section did not experience any brittle failure. If there was not bottom rebar in this section (since there is no need as far as dead loads are concerned) and the moment demand reached the cracking moment of the section, beam A3-B3 might have experienced a brittle failure at its A3 end.

Figure 3.45 shows the axial force histories of column A3 in different floors. Columns in all floors lose their compressive force and experience some tensile forces. Axial forces of columns A3 in all floors are very close after the structure stabilizes with
the exception of the third floor. Note that the third floor is the only floor above the removed columns that does not have infill walls.

Figure 3.46 shows the axial force histories of column A3 in different floors only for the first 0.01 sec after columns are removed. As can be seen from the figure, the floor that shows the earliest reduction in the axial force is the second floor (just above the removed columns). The third floor and other floors follow the second floor with some time lag. Note that when the first floor column A3 loses all compressive force around 0.003 seconds after column removal, there was no change in column A3 in the sixth floor axial force. That is because of the fact that the axial force reduction is a chain reaction. As explained in 3.2.3.2, when the columns in the first floor are removed, the second floor columns lose their supports and unbalanced forces develop at the bottom end of second floor A2 and A3 columns. These forces cause joints A2 and A3 move downwards while the same joints in the third floor stay in place. The movements of joints A2 and A3 in the second floor cause columns A2 and A3 in the second floor to elongate and impose tensile strains. As a result, the compressive force in the second floor A2 and A3 columns drops. This reduction in the axial force in the second floor A2 and A3 columns causes unbalanced forces at joints A2 and A3 in the third floor. As a result the same joints in the third floor move downward and this causes reduction in the third floor column axial force. This repeats up to the sixth floor. This chain reaction can be clearly seen from Figure 3.46.

Figure 3.47 shows the axial force histories of columns A2 and A3 in the second floor (just above the removed columns) and those of columns A1 and B3 in the first floor. Note that there is a considerable time lag between the time that it takes for the loads to be redistributed to the neighboring columns and the time that it takes for the columns above the removed ones to lose their compressive forces. This time lag can be explained by the fact that the reduction in the axial force of columns A2 and A3 in the second floor is due to axial wave propagation, while the increase in the axial compressive force of columns A1 and B3 is due to flexural wave propagation that requires the structure to deform and redistribute the gravity loads.
3.2.5 Analytical Evaluation Using Applied Element Model

An analytical model of the structure is developed using a relatively new method, Applied Element Method (APM) (Meguro and Tagel-Din, 2000). The AEM can track the structural collapse behavior passing through early stages of loading and can account for nonlinear behavior of structures including element separation and element collisions or contacts. The structural elements are modeled with 3-dimensional cubical sub-elements. The cuboids are rigid and the connection between them is achieved by a series of spring triples (one axial and two shear element springs) which represent the structural characteristics of the connected cuboids. In other words, all structural characteristics are concentrated in the springs.

3.2.5.1 Model Description

A three dimensional applied element model of the building is developed using the computer program ELS (Extreme Loading for Structures, 2006). Beams are modeled such that there would be 200 spring triples as an array of 10 (over the width) and 20 (over the height) at each cross section. The same number of springs is used for the cross sections of the columns. The reinforcement of the beams and columns are modeled explicitly. If there is a rebar running through the interface of two cuboids, a spring representing the rebar is assigned to the interface. The characteristics of the springs are determined from the geometry and the material properties of the connected cuboids.

The floor system consisted of joists in the longitudinal direction of the building as shown in Figure 3.3. The joists are modeled similar to beams such that there would be 200 spring triples as an array of 10 (over the width) and 20 (over the height) at each cross section. The slabs between joists are also modeled with small cuboid elements. Since the joists are connected to slabs at their sides through a number of spring triples, contribution of the slabs as flanges on the behavior of joists is automatically accounted for. The reinforcement of the joists as well as the reinforcement of the slab is modeled explicitly.

As described before, the building had infill walls in the 2nd, 4th, 5th and 6th floors. The infill walls were made of hollow clay tiles, which were 12” by 12” in the plane of the
wall and 4” thick. The infill walls consisted of two layers of clay tiles with a total thickness of 8”. The infill walls were of running bond type. In the analytical model, the tiles are modeled with small cuboids having the same size as the tiles. The connection between the tile elements is modeled with another material, namely mortar. In the modeling of infill walls in ELS, it is assumed that cracking occurs only at the interface of the clay tiles and the grout and not through the tiles. It is assumed that mortar loses all of its tensile strength when it cracks.

ELS is able to remove one or more elements automatically during a dynamic analysis. Before element removal, all dead loads are applied to the structure and then the elements modeling columns A2 and A3 in the first floor are removed from the model instantaneously and a dynamic analysis is performed.

3.2.5.2 Global Displacements

The vertical displacement of joint A3 in the second floor obtained from the Applied Element Model after removal of columns A2 and A3 in the first floor is shown along with experimental vertical displacement of the same joint in Figure 3.48. As can be seen in the figure, the first peak of the analytically obtained displacement is in good agreement with the experimentally obtained displacement. Analytically obtained vertical displacement has a first peak of 0.235” while the experimental first peak was around 0.242”. However, the analytical vertical displacement shows a second peak which overestimates the experimental vertical displacement. In terms of permanent vertical displacement, the analytical model underestimates the experimentally obtained permanent vertical displacement. The analytical vertical displacement stabilizes around 0.206” while the experimental permanent vertical displacement was around 0.242”.

Figure 3.49 shows the analytical and experimental vertical displacement history of joint A2 in the second floor before and after column removal. Similar to Joint A3, the first peak vertical displacements are in good agreement. Experimental permanent vertical displacement is more than the first peak vertical displacement which is 0.242”. In the analytical model, the maximum vertical displacement occurs at the first peak after column removal. Then the vertical displacement of joint A2 goes down and stabilizes at
around 0.181". As in joint A3 in the second floor, the analytical model again underestimates the permanent vertical displacement for Joint A2 in the second floor.

The analytical and experimental vertical displacement history of Joint A2 in the third floor after column removal is shown in Figure 3.50. Similar to Joint A2 and A3 in the second floor, the first peak displacements are in good agreement. The experimental permanent displacement is equal to the first peak displacement which is 0.215". In the analytical model, the maximum displacement occurs at the first peak after column removal. Then the vertical displacement of Joint A2 decreases and stabilizes at around 0.175". Again, the analytical permanent displacement underestimates the experimental permanent displacement.

Figures 3.51 and 3.52 show the analytical vertical displacements of joints A3 and A2 in different floors. As can be seen from Figure 3.51, joint A3 in different floors moved downwards almost together which is the same for joints A2 as shown in Figure 3.52. The vertical displacement decreases as the floor goes up. The reason for this fact, which is related to the elongation of columns above the removed ones, is explained in the previous sections.

3.2.5.3 Beam Deformations

The deformed shapes of beams A1-A2 and A3-B3 in the second floor obtained from the Applied Element Model are drawn in Figure 3.53 and 3.54, respectively, along with the measured deformed shapes. The method by which the end rotations of beams were measured is explained in previous sections. Note that estimated deformed shapes of the mid regions of the beams where there were no potentiometers installed are shown with dashed lines. As can be seen from Figure 3.53 and 3.54, deformed shapes of beams A1-A2 and A3-B3 obtained experimentally and analytically from Applied Element Model show good agreement.

Figure 3.55(a) shows the bending moment diagram of beam A1-A2 in the second floor after dead loads are applied (all columns are in place). As one expects, the end regions experience negative bending moment while the bending moment at the mid-span
is positive. Figure 3.55(b) shows the bending moment diagram of same beam after columns A2 and A3 in the first floor are removed and the structure stabilized. The negative bending moment demand increases at the A1 end of the beam. The bending moment at the A2 end of the beam, however, changes from negative to positive. Note that if there were no bottom reinforcement at these sections and if the bending moment demand reached the cracking moment of the section, brittle failure could have happened at this location. Note that the measured rotations also do not indicate any brittle failure at this location.

Similar to beam A1-A2, Figure 3.56(a) shows the bending moment diagram of beam A3-B3 in the second floor after dead loads are applied (all columns are in place). Again, the end regions experience negative bending moment while the bending moment at the mid-span is positive, as expected. Figure 3.56(b) shows the bending moment diagram of same beam after columns A2 and A3 in the second floor are removed and the structure stabilized. The negative bending moment demand increases at the B3 end of the beam. The bending moment at the A3 end of the beam, however, changes from negative to positive. Note that if there were no bottom reinforcement at these sections and if the bending moment demand reached the cracking moment of the section, brittle failure could have happened at this location too. The measured rotations also do not indicate any brittle failure at this location.

Moment diagrams of both beams A1-A2 and A3-B3 are in agreement with the measured end rotations showing a change in the direction of bending moments at the A2 end of beam A1-A2 and A3 end of beam A3-B3.

The moment diagrams of beams A1-A2 and A2-A3 in different floors obtained from the Applied Element Model after column removal are presented in Figure 3.57. Note that A2-A3 beams experienced smaller moment values compared to the A1-A2 beams since joints A2 and A3 moved downward almost the same amount resulting in almost zero relative end displacement for A2-A3 beams. However, the relative end displacement is considerable for beam A1-A2 since the joint A1 stayed in place while the joint A2 was experiencing maximum displacement in the structure.
Note that the bending moment diagrams of A1-A2 beams in different floors have different patterns. The reason for having different moment diagram patterns in different floors is because of not having infill walls in the first and third floors. A detailed explanation can be found in the previous sections.

Figure 3.58 shows the moment diagram of A3-B3 beams in different floors after column removal. Again, the bending moment diagrams of A3-B3 beams in different floors have different patterns because of the reason explained previously. For the A3-B3 beams, it can be clearly seen from the moment diagrams in the 2\textsuperscript{nd} and 4\textsuperscript{th} floors as well as the moment diagrams of same beams in the 5\textsuperscript{th} and 6\textsuperscript{th} floors. Both pair of floors have the same infill wall pattern in the floors above and below resulting in the same bending moment pattern.

The moment diagrams of A2-B2 beams in different floors before and after column removal are shown in Figure 3.59. The presented moment diagram after column removal is obtained after the structure stabilized. The A2-B2 beams in different floors had almost the same bending moment diagram since the A2 ends of the beams experienced practically the same vertical displacement while the B2 ends stayed relatively in place. There were no infill walls on any of these beams.

Figure 3.60 shows the analytically obtained deformed shapes of the elements in Axis-A after removal of columns A2 and A3 in the first floor with a scale factor of 200. Note that the deform shape is obtained after the structure stabilized. As can be seen, A1-A2 beams deformed in double curvature while no significant deformations were observed for A2-A3 beams. No yielding was observed in the beams in Axis A. However, one section that did not have top reinforcement reached its cracking moment capacity and cracked. That section is shown in Figure 3.60.

The deformed shape of the elements in Axis 3 between axes A and B are shown in the Figure 3.61. Similar to beams A1-A2 and A2-B2, the A3-B3 beams are deformed in double curvature. In this axis, neither yielding in rebars nor cracking in the sections that did not have top reinforcement was observed.
3.2.5.4 Load Redistribution

The axial forces in the columns in axes A and B before and after removal of columns A2 and A3 in the first floor are shown in Table 3.2. As is seen in the Finite Element Model, columns A1, B2 and B3 experienced a significant increase in their axial force after removal of columns A2 and A3 in the first floor. That indicates that most of the loads that were carried by columns A2 and A3 were transferred to columns A1, B2 and B3.

Both of the deformed shapes as well as the moment diagrams of beams A1-A2, A2-B2 and A3-B3 obtained from the Applied Element Model after removal of columns A2 and A3 in the first floor indicate the development of Vierendeel Action. Beams A1-A2, A2-B2 and A3-B3 deformed in double curvature and developed shear forces to transfer the loads. The loads that were carried by the removed columns A2 and A3 transferred to the neighboring columns through the Vierendeel Action with the participation of the infill walls.

As in the Finite Element Model, the change in the direction and value of the bending moments of beams A1-A2, A2-B2 and A3-B3 that formed the Vierendeel action with the interaction with columns through moment connections is also observed in the Applied Element Model. Brittle failure potential at the A2 ends of A1-A2 beams and at the A3 ends of A3-B3 beams under positive bending moment is averted since the beams had enough bottom reinforcement at those sections. Note that there is no need for bottom reinforcement at those sections as far as dead loads are concerned.

Another type of brittle failure at those locations could have occurred due to potential lack of anchorage of the bottom reinforcement of the beams into the adjacent beam-column joints. There was not clear information about the anchorage of the bottom reinforcement into the joints. Therefore, the modeling of the anchorage of the bottom reinforcement into the joint was not considered in a detailed fashion. Instead, it is assumed that the bottom reinforcement is perfectly anchored into the joints. The rotation recordings of these sections during the experimental program do not indicate any brittle failure due to lack of anchorage and pull out.
The axial force histories of column A3 in different floors are shown in Figure 3.62. As was observed in the Finite Element Model, columns in all floors lose their compressive force and experience some tensile forces. Axial forces in column A3 in all floors are very similar after the structure stabilizes. The axial force in the third floor column A3 is different compared to the other floors. Note that the third floor is the only floor above the removed columns that does not have infill walls.

Figure 3.63 shows the axial force histories of the columns A2 in different floors. Similar to A3 columns, A2 columns in all floors lose their compressive force and experience some tensile forces. Again, the axial forces in columns A2 in different floors have very close values after the structure stabilizes. The axial force in the third floor, which is the only floor that did not have infill walls above the removed columns, has the least amount of tensile axial force compared to the other floors.

The axial force histories of column A3 in different floors for the first 0.01 sec after columns were removed are shown in Figure 3.64. First, the second floor shows a reduction in the axial force. The third floor and other floors follow the second floor with some time lag. The same phenomenon observed in the Finite Element Model that is the columns above the removed columns lose their compressive axial forces with the order of floor is also observed in the Applied Element Model. Detailed explanation of this fact can be found in section 3.2.4.4.

The axial force histories of the A2 and A3 columns in the second floor (just above the removed columns) and those of the A1 and B3 columns in the first floor are presented in Figure 3.65. As can be seen from the Figure 3.65, it takes more time for the loads to be transferred to the neighboring columns following the column removal than it does for the columns above the removed ones to lose their compressive forces. As explained in 3.2.4.4, this time lag can be explained by the fact that the reduction in the axial force of columns A2 and A3 in the second floor is due to axial wave propagation, while the increase in the axial compressive force of columns A1 and B3 is due to flexural wave propagation that requires the structure to deform and redistribute the gravity loads.
3.2.6 Response of Structure in the Absence of Infill Walls

To investigate the effect of infill walls on the building response, another model is developed by removing the infill walls from the original model in the computer program ELS. The weight of infill walls are also excluded from the model. Nonlinear dynamic analysis is performed under the removal of columns A2 and A3 in the first floor. The maximum vertical displacement of the structure increased by about 2.4 times (compared to the response of the model with infill walls), yet the structure resisted progressive collapse. Figure 3.66 shows the displacement history of joint A3 in the second floor. The maximum displacement of 0.6” occurs at about 0.1 s.

Because of the lack of top reinforcement in the transverse beam A1-A2 in the vicinity of column A1 (see Figure 3.8(a)), all beams except for the roof level beams cracked and lost almost all of their flexural strength at such sections. Figure 3.67 (a) shows the deformed shape of spandrel frame A and the formation of cracks. Figure 3.67 (b) shows the bending moment diagram of frame A. In spite of the formation of the cracks in the beams, the beam bending moments in floors 2-6 developed such that the beams deformed in double curvature, providing resistance against progressive collapse.

3.2.7 Effects of Additional Gravity Loads

As mentioned before, nonstructural elements including partitions, plumbing, furniture, and mechanical systems were removed from the building prior to implosion. Only beams, columns, floor system and infill walls on the spandrel beams were present in the structure. There was no live load applied to the system. Therefore, the loading was only due to the self weight of the structural elements and infill walls.

To see the effects of these additional gravity loads that the building would have when it was functional on the building response, a unit floor load of 30 psf is added to the Applied Element Model to account for the weight of partitions, ceiling, and mechanical systems. Also an additional live load of 12.5 psf equal to 25% of the design live load is considered as suggested in GSA and DOD guidelines. These would add 42.5 psf to the gravity loads.
Considering the additional loads above, a new analysis is performed and maximum downward displacement of joint A3 in the second floor is found to be about 0.33” which is 32% more than the displacement obtained from the analysis without considering the additional loads. The permanent downward displacement of the same joint is increased by 38% to 0.285”. The loads are again transferred to the neighboring columns through Vierendeel action of the beams in the transverse and longitudinal directions with the participation of the floor system and infill walls and the structure still resisted progressive collapse.

3.2.8 Summary

The progressive collapse resistance of the Hotel San Diego is evaluated experimentally and analytically, following the simultaneous removal (explosion) of two adjacent columns, one of which was a corner column. The floor of the RC structure consisted of joists in the longitudinal direction. In addition to the transverse beams supporting the joists, the structure had spandrel beams. Exterior frames had clay infill walls. The infill walls in the first and third floors were removed prior to the explosion. Both experimental data and analytical results show that the 6-story RC structure resisted progressive collapse without partial or full collapse.

Experimental results show that both joints A2 and A3 above the removed columns had the same permanent vertical displacement following removal of two columns in the first floor. The maximum permanent vertical displacements at these joints were about 0.242”. That is, beam A2-A3 in the second floor moved downwards as a rigid body after column removal. The recorded permanent vertical displacement of joint A2 in the third floor was about 0.215”, 11% less than the recorded permanent displacement of the same joint in the second floor. The difference between the permanent vertical displacements of the same joint in two consecutive floors following the column removal is mainly due to the axial deformation of the column that connected these two joints.

Bi-directional Vierendeel (frame) action of the transverse and longitudinal frames with the participation of infill walls is identified as the major mechanism for the redistribution of the loads in this structure. That is, beams A2-B2 and A3-B3 in
longitudinal direction and beam A1-A2 in the transverse direction deformed in double curvature due to the moment connections with the columns and in turn developed shear forces to transfer most of the loads to columns A1, B2 and B3 after removal of columns. The floor system also transferred some portion of the load to adjacent beams. Furthermore, the infill walls provided the beams with constraints and supports to help carry the additional loads.

The end rotations of the two critical beams A1-A2 and A3-B3 in the second floor are captured experimentally using the recordings of 14 potentiometers attached to top and bottom of these beams at the end regions. Experimentally obtained rotations indicate the double curvature deformation of these beams after removal of two columns in the first floor and the development of Vierendeel action in the frames whose beams are measured. The change of the rotation in the beam ends in the vicinity of removed columns was captured experimentally.

Due to the change in the direction of bending moments in the beams, the beam bottom reinforcements experienced tension at the faces of joints above the removed columns. Such tensile strain was recorded experimentally with a strain gage attached to the bottom reinforcement of beam A3-B3 at the A3 end. When Vierendeel (frame) action develops, in order for the beam bottom reinforcement to be fully effective and develop stresses up to the ultimate stress at the face of a removed column, the reinforcement should be well anchored into the joint. In this structure, it was not clear if the beam bottom reinforcement was properly anchored. The recorded maximum tensile strain of the bottom reinforcement of the second floor longitudinal beam at the face of the joint above the removed column was small. Such a small strain suggests that the bar would not have pulled out, even if it had not had proper anchorage.

Strain gages were attached to the neighboring columns of the removed column as well as to the column above the removed one to capture the load redistribution. The difference between the speed of the propagation of the axial waves and that of flexural waves is discussed based on recordings of these strain gages. That is, the columns above the removed column lose their axial load only in few milliseconds after column removal.
while it takes much longer for the neighboring beams to deform and transfer the loads to the neighboring columns through Vierendeel action.

The analytical models of the structure are developed using both Finite Element Method and Applied Element Methods. Global and local deformations of the structure as well as redistribution of gravity loads following the column removal based on the results of the analytical models are compared with the experimental data and good agreement between the analytical and experimental results is observed.

The infill walls in the structure are modeled by following two different approaches. In the first approach the strut model based on FEMA 356 is used to model the infill walls. The location, orientation, effective width and the other characteristics of struts are determined based on FEMA 356. In the second approach two dimensional shell elements are used to model the infill walls. In both methods, the mechanical properties of the infill walls are obtained from FEMA 356 based on the visual examination of the infill walls. The maximum vertical displacement obtained from the analytical model that the infill walls are modeled with struts was 0.449”, 86% more than the peak displacement obtained experimentally while the maximum vertical displacement obtained from the analytical model in which the infill walls are modeled with two dimensional shell elements was 0.252”, only 4% more than the peak displacement obtained experimentally. Based on the comparison of results of the two methods with the experimental data, it is concluded that the modeling of infill walls with struts underestimates the stiffness of infill walls, especially for small deflections. If the vertical displacement of the structure was large, such that significant cracking had formed in the infill walls, modeling the infill walls with struts might have more closely predicted the structural deformation.

Another analysis is performed by excluding the infill walls from the model of the structure to see the effect of the infill walls on the behavior of the structure following column removal. The analysis results show that the maximum displacement in the structure without infill walls increased to 0.6”, 240% more than the maximum displacement of the model with infill walls. In spite of the formation of the cracks in
some beams where there were no top reinforcement to take tensile forces, the structure still resisted progressive collapse.

The response of the structure under additional dead loads (i.e. partitions and mechanical systems) as well as 25% of design live load is analytically studied. The analytical results show that maximum and permanent vertical displacement of joint A3 in the second floor, the maximum displacement in the structure, increased by about 32% and 38%, respectively, without leading to progressive collapse.

### 3.3 University of Arkansas Medical Center Dormitory

#### 3.3.1 Building Characteristics

The University of Arkansas Medical Center Dormitory was located in Little Rock, AR. It was built in 1958. Figure 3.68 shows a picture of the building from street level. The evaluated building was a 10 story RC structure with a basement. The story height was 9’-8” typical. Figure 3.69 shows the typical floor plan of the structure with the size of typical beam and column dimensions. Note that neither the element dimensions nor the span lengths in both direction show variation through the structure except the staircase regions. The floor system was one way 8” thick solid slab running in the longitudinal direction. The slab was supported by beams running on the transverse direction between columns. The elevation view of axis B is shown in Figure 3.70 indicating the removed column.

As a part of the demolition process, all nonstructural elements including partitions, plumbing, and furniture were removed prior to implosion. Prior to column removal, only beams, columns and slabs were present. There were no infill walls or partitions in the building. The loading was only due to the self weight of the structural elements (i.e. columns, beams, and slab). There was no live load in the building.

Mechanical characteristics of the concrete and steel are calculated by performing tests on the samples obtained from the building. The concrete compressive strength was estimated as 3600 psi. The modulus of rupture of the concrete is estimated as 700 psi based on the flexure test. The modulus of elasticity of the concrete is expected to be 2450
ksi based on the compressive strength of the concrete (ACI 213R-03). Yield and ultimate stresses for the steel are estimated as 48 ksi and 73 ksi respectively.

The reinforcement details of the slab between axes 4 and 5 (same with axes 5 and 6) in the longitudinal direction and between axes B and C in the transverse direction are shown in Figure 3.71. The reinforcement detail of beam B5-C5 is shown in Figure 3.72.

3.3.2 Initial Damage Scenario

The initial damage scenario for the University of Arkansas Medical Center Dormitory was the removal of column B5 in the first floor. The removed column is indicated in Figure 3.69 and Figure 3.70. Column removal is achieved by explosion of the column. Explosives were inserted to holes along the height of the column by demolition contractors. The explosion of column B5 was not a part of the implosion. Therefore there were not any other columns exploded in the vicinity of removed column B5.

3.3.3 Experimental Evaluation

3.3.3.1 Sensors

In the experimental evaluation of the University of Arkansas Medical Center Dormitory, potentiometers are used to capture the global displacements as well as the deformation of a slab end region. 4 potentiometers are used to capture vertical displacements of Joints B5 in the second and fifth floors as shown in Figure 3.73. The concrete strain gages are used to capture the change in the strain of the concrete at the critical locations. 2 strain gages are installed on the columns to capture the redistribution of loads. 3 strain gages are installed on the beams and slab to capture the deformation of these elements. The locations of the strain gages used are shown in Figure 3.73 and 3.74.

The concrete strain gages used in the University of Arkansas Medical Center Dormitory were 3.5” long having a maximum strain limit of ± 0.02. The potentiometers had a resolution of about 0.0004”. The potentiometers used to capture global displacements had a 4” capacity while the one used to capture the deformation of the slab
had a 2” capacity. The sampling rate used in the recording of the data collected from the sensors was 500 Hz.

### 3.3.3.2 Global Displacements

The vertical displacement of joint B5 in the second floor is calculated utilizing the recordings of two potentiometers (P1 and P2, see Figure 3.73) that are connected to this joint from joints B4 and B6 in the third floor. Figure 3.75 shows the vertical displacement of Joint B5 in the second floor, calculated by averaging vertical displacement components of potentiometers P1 and P2 recordings. As can be seen from Figure 3.75, joint B5 had a peak displacement of 0.25” at 0.12 sec after column B5 in the first floor was removed. Following a small vibration, it stabilizes at a displacement of 0.25” after about 0.3 seconds.

Note that the vertical displacement of Joint B5 obtained from the recordings of potentiometers P1 and P2 is not the absolute vertical displacement of that joint. The readings of potentiometers P1 and P2 reflects the relative movement of joint B5 with respect to joints B4 and B6 in the third floor. To find the absolute displacement one needs to consider the vertical displacements of joints B4 and B6 after the column removal. The change in the strain in the columns B4 and B6 in the first floor was used to estimate the vertical movement of joints B4 and B6 in the 3rd floor. The strain gages installed at the mid-heights of columns B4 and B6 in the first floor showed an increase of 22 micro-strain in compressive strain. Assuming the first floor strain recording as the average of change in the strains in the basement, first floor and second floor columns, the vertical displacement of joints B4 and B6 on the third floor is estimated as 0.01”. Therefore the absolute vertical displacement of joint B5 in the second floor would be 0.26”.

The vertical displacement of joint B5 in the fifth floor is presented in Figure 3.76. Similar to the vertical displacement of joint B5 in the second floor, the vertical displacement of joint B5 in the fifth floor is estimated utilizing the recordings of two potentiometers, P3 and P4 that are connected to this joint from joints B4 and B6 in the sixth floor. The locations of the potentiometers (P3 and P4) are shown in Figure 3.73. As can be seen from Figure 3.76, joint B5 in the fifth floor had a first peak displacement of
0.225” at 0.13 sec after column B5 in the first floor was removed. Without considerable vibration, it stabilizes at a displacement of 0.235 after about 0.23 seconds.

The vertical displacement of joint B5 in the fifth floor obtained from the recordings of potentiometers P3 and P4 is less than the vertical displacement of joint B5 in the second floor obtained from the recordings of potentiometers P1 and P2. That can be explained with two reasons. The first reason is the elongation of columns B5 in the upper floors that is the result of loss in their compressive axial forces. After removal of column B5 in the first floor, column B5 in the second floor lost its support and unbalanced forces developed at joint B5 in the second floor. These unbalanced forces caused the downward movement of this joint. As joint B5 moved downward while joint B5 in the third floor had not experienced any unbalanced force, the column B5 in the second floor elongated and the compressive strains on this column reduced. As a result, axial force dropped and caused unbalanced forces at joint B5 in the third floor. This chain action continued up to the roof. When the structure stabilized, all columns above the removed one experienced loss of axial compressive force and additional tensile strains developed in these columns resulting in elongation of these columns.

The second reason is that the axial compressive forces in the columns in the vicinity of the removed column (e.g. columns B4, B6, C5) increased since the loads that were carried by the removed column were transferred to these columns. In turn, the compressive strains in these columns increased as a result of the additional axial compressive force. Joints B4 and B6 in all floors moved downwards. Since the absolute vertical displacement of a joint includes the displacements of the same joints in the floors below, joints B4 and B6 in the fifth floor moved downwards more than those in the second floor. The combination of these two reasons explained above makes the potentiometer recordings show less vertical displacement for joint B5 in the fifth floor than joint B5 in the second floor.

3.3.3.3 Local Deformations

Figure 3.77 shows the recording of strain gage SG1 that was attached to bottom of beam B5-C5 at the B5 end in the second floor (See Figure 3.74). As can be seen from this
figure the bottom of the B5 end of the second floor beam B5-C5 experienced tensile strains following the removal of column B5 in the first floor. Prior to column removal the bottom of the B5 end of the beam B5-C5 is expected to have compressive strains due to expected negative bending moment at the support under gravity loads.

The recording of strain gage SG2 that was at the top of the C5 end of beam B5-C5 in the second floor (See Figure 3.74) is shown in Figure 3.78. As it is shown, the recording of this strain gage indicates that the top of C5 end of beam B5-C5 in the second floor experienced tensile strains after column removal.

The recordings of strain gages SG1 and SG2 indicate that beam B5-C5 deformed in double curvature after the removal of column B5 in the first floor. Given the moment connections of beam B5-C5 with the columns at its ends, the double curvature deformed shape indicates the development of Vierendeel action in these beams.

A potentiometer (P5) was installed on the top of the second floor slab on the south side of column B5 to capture the elongation/shrinkage on the top surface of the slab. It is attached to the south face of column B5 and to the top of the slab 24” away from the column surface. The recording of this potentiometer (P5) is shown in Figure 3.79. As can be seen from the figure, the top of the 2nd floor slab close to column B5 experienced shortening (i.e. compressive strains) following the removal of column B5 in the first floor. This indicates positive bending at this location. This positive bending could be related to the double curvature deformation of the slab strip between columns B5 and B6.

**3.3.3.4 Load Redistribution**

After removal of column B5 in the first floor, the loads carried by this column are transferred to neighboring columns without having partial or total collapse in the structure. There were not any flexural cracks in the beams and slabs based on the visual inspection conducted after column removal. The structure stabilized after the top of the removed column displaced downward 0.25”. The elements in the vicinity of the removed column were capable of transferring the loads carried by the removed columns to the neighboring columns without any failure.
As explained in section 3.3.3.3, the recordings of strain gages SG1 and SG2 on beam B5-C5 in the second floor (see Figures 3.77 and 3.78) as well as the recording of the potentiometer P5 on the second floor slab along axis B between columns B5 and B6 (see Figures 3.79) indicates the development of Vierendeel action in the structure after column removal. In Vierendeel action, the loads are transferred to the neighboring columns through the shear forces developed in the elements as a result of their double curvature deformations.

To examine the change in the axial load in the columns above the removed one, a strain gage (SG3) was installed at the mid-height and mid-section (on the south face, see Figure 3.73) of fifth story column B5. Figure 3.80 shows the recorded change in the axial strain. Positive values show elongation or tensile strains (reduction in compressive strains). A close-up plot of the first 0.02 s along with the analytical results is also presented. The experimental data shows a sudden drop in the strain during the initial rising part (at 0.008 sec), which could be in part due to the initial pressure of explosion in the removed column and the corresponding axial stress (strain) waves traveling in the columns above confirming this would require further evaluation. The recorded change in the axial strain at the end of vibration (the permanent change of strain) was about 85 micro strain. The corresponding analytical change of strain is about 108 micro strain (change in the stress divided by the modulus of elasticity), which is about 27% more than the experimental value. The experimental data shows that the strain in the column was reduced to a value equal to the permanent strain in only about 0.01 s, which is comparable to the 0.012 s found analytically. That is, 0.01 s after the column removal, when joint B5 in the second floor had moved vertically by only about 0.013”, about 1/20th of the peak displacement (see Figure 3.75), the axial strain (force) in the fifth story column B5 was reduced to a value equal to its final (permanent) value. Following such a relatively fast reduction in the axial forces, all the floors above the removed column practically moved together.
3.3.4 Analytical Evaluation

3.3.4.1 Model Description

A three dimensional analytical model of the University of Arkansas Medical Center Dormitory was developed in SAP 2000 (2005) that uses the Finite Element Method. The columns and beams were modeled with 2-node Bernoulli beam elements. Beams were modeled with T sections accounting for the effect of the slab. Effective flange width on each side of the web was set equal to one fourth of the span length (ACI 318, 2005). Material nonlinearity was imposed to the model by using localized plastic hinges at the critical locations. The localized plastic hinges are assigned to the locations where yielding can occur (e.g. end of the elements and bar cut-off locations). The force-deformation relationships of plastic hinges are calculated by performing section analysis with the dimensions and reinforcement details obtained from structural drawings and material characteristics. Material characteristics used for concrete and steel were obtained from the tests performed on the samples obtained from the building. Geometric nonlinearity is also included in the analysis. A mass proportional damping ratio of $\xi=0.05$ in the first vertical vibration mode is used.

As can be seen in Figure 3.72, beam B5-C5 did not have top reinforcement beyond 43” away from the face of column C5. If the negative bending moment reaches the cracking moment at this section, strength of this section drops to practically zero. To account for this potential brittle failure, plastic hinges are assigned to the sections where top reinforcement ends. The capacities of the plastic hinges are set equal to the cracking moment capacity. As soon as the demand reaches the cracking moment the plastic hinges lose their strengths without any strain hardening.

The slab is modeled with two dimensional 4 node linear shell elements. SAP 2000 keeps the stiffness of the beam elements as well as shell elements constant during the analysis. To model the cracking in the beam and shell elements properly the iterative approach explained in Section 3.2.4.1 is followed.
Since element removal during an analysis is not readily available in SAP 2000 the procedure explained in Section 3.2.4.1 is used to obtain the response of the structure following removal of the column.

### 3.3.4.2 Global Displacements

Figure 3.81 shows the analytically obtained vertical displacement of Joint B5 in the second floor. It also shows the vertical displacement of the same joint recorded experimentally. Note that the experimental vertical displacement of joint B5 obtained from the potentiometer recordings reflects relative displacement of joint B5 with respect to joints B4 and B6 in the third floor where the other ends of the potentiometers were connected. To have a reasonable comparison between the analytical results and the experimental data, the average of the analytically estimated vertical downward movements of joints B4 and B5 in the third floor is added to the experimental displacement of the second floor joint B5 that is obtained from the potentiometer recordings. Analytical vertical displacement of Joint B5 shows a peak displacement of 0.431” at 0.07 seconds after column removal. Joint B5 stabilizes at about 0.35” of permanent vertical displacement about one second after the removal of the column. As one can see, the displacement obtained from the analytical model shows considerable vibration after peak displacement compared to experimental displacement.

Rayleigh damping was used for damping in the analysis of the structure and the mass and stiffness proportional coefficients are calculated based on the first two governing modes of vibration associated with the deformation of the structure after removal of the column. The damping ratio for these modes is set equal to 0.02. The reason for assuming a small damping ratio is that all nonstructural elements (partitions, walls) were removed from the structure prior to column removal. The only non structural elements existing in the structure were the exterior glass windows.

In spite of the fact that there were no nonstructural elements in the structure, the high damping observed in the experimental displacement histories can be in part due to the effect of the remaining rebars of the exploded column. The condition of column B5 after its explosion was examined visually. It is observed that vertical reinforcement of
column B5 stayed in place connecting the joints at the top and the bottom of the column while the concrete fell into pieces except for small portions close to the top and bottom of the column. The vertical rebars were bent outwards due to the high pressure of the explosion and the vertical displacement of joint B5 in the second floor while the transverse reinforcements were cut. Plastic deformation of the remaining bars could have added considerable damping to the structure. For this structure, the effect of the remaining bars was not further investigated. However, detailed evaluations about the effect of the remaining bars of the removed column on the response of the structure were carried out for the Baptist Memorial Hospital and presented in Section 3.4.4.5.

Figure 3.82 shows the analytically obtained vertical displacement of Joint B5 in the fifth floor along with the experimental vertical displacement of the same joint. Because of the same reason explained earlier for joint B5 in the second floor, the average of the analytically estimated vertical downward movements of joints B4 and B5 in the sixth floor is added to the experimental displacement of fifth floor joint B5 that is obtained from the potentiometer recordings. Analytical vertical displacement shows a peak of 0.326” at 0.075 seconds after column removal while the first peak of experimental vertical displacement was about 0.225” occurring at 0.136 seconds. The analytical model shows that joint B5 in the fifth floor stabilizes at about 0.241” of permanent vertical displacement about 1 second after column removal. The analytical permanent displacement overestimates the experimental permanent displacement (0.235”) by 3%. As in the second floor, the displacement obtained from the analytical model shows considerable vibration after peak displacement compared to experimental displacement. As mentioned earlier, this could be due to the effect of the remaining rebars of the exploded column. To see the effect of higher damping in the response of the structure, another analysis is performed with the damping ratio of 0.30. The vertical displacement of joint B5 in the fifth floor obtained from this analysis is plotted along with the experimental vertical displacement in figure 3.83. Even though the first peak displacement is reduced to 0.276, it still overestimates the experimental first peak displacement (0.225”) by 23%. The analytical permanent displacement of 0.233”, however, shows a good agreement with the experimental permanent displacement (0.235”).
As observed in the experimental results, the analytically estimated vertical displacement of joint B5 in the fifth floor is less than that in the second floor. The reason to explain this difference is the reduction in the axial forces of columns B5. Since the analytically estimated vertical displacements reflect the absolute values at the joints directly obtained from the analysis program, the vertical movements of the joints B4 and B6 do not have the effect they had for the experimental results. The reduction in the axial forces of B5 columns above the first floor after column removal caused these columns to elongate. As a result of the elongation of these columns, B5 joints in each floor experienced more displacement compared to the displacement in the upper floor.

3.3.4.3 Local Deformations

The schematic representation of the deformed shape of beams and columns between longitudinal axes B and C along transverse axis 5 for the second and third floors is shown in Figure 3.84. As can be seen from the figure, B5-C5 beams in all floors are deformed in double curvature which is consistent with the recordings of the strain gages on the second floor beam B5-C5. Note that the deformed shape presented in Figure 3.92 shows the state of the elements after all the vibrations in the structure are damped out.

Figure 3.85 shows the analytical bending moment diagram of the same elements shown in Figure 3.84. Similar to Figure 3.84, the moment diagrams show the permanent values after the structure stabilized. While the results correspond to a damping ratio of 0.02, the results for the larger damping ratio of 0.30 are close to the values shown.

Figure 3.86 shows the bending moment diagram of the same frame shown in figure 3.85 under dead loads. Note that the same scale factor was used to plot the bending moment diagrams in both Figure 3.85 and 3.86. As can be seen from Figure 3.86, the B5 end of the beam B5-C5 had negative bending moment (tension on the top) under dead loads. The same end, however, experienced positive bending moment (tension on the bottom) after removal of column B5 in the first floor. This is also consistent with the tensile strain recording of strain gage SG1 which was installed at the bottom of the B5 end of the beam B5-C5 (See Figure 3.77).
The C5 end of the beam B5-C5 in the second floor experiences considerably more (952%) negative bending moment after column removal. The analytical bending moment at the face of column increases to 2725 kip-in after column removal while it was only 286 kip-in under dead loading before column removal. The additional negative bending moment verifies the experimentally observed tensile strain recorded by strain gage SG2 which was attached to the top of the C5 end of the second floor beam B5-C5 (See Figure 3.78).

The direction of bending moments at the top and bottom of columns B5, which are required to be in equilibrium with the beam bending moments, changes after column removal. The slab bending moment along the faces of beam B5-C5 (particularly in the neighborhood of joint B5) also changes from a negative to a positive value after the column removal. Positive bending moment in the slab around joint B5 is also consistent with the recording of potentiometer P5 which was installed at the top of the second floor slab in the longitudinal direction (normal to the beam) over 24 in from the face of the column. Figure 3.79 shows the axial shortening of concrete at the top of the slab (compressive strain due to a positive bending moment).

Both deformed shapes and moment diagrams indicate the development of Vierendeel action in the frame shown in Figures 3.84 and 3.85 following the removal of the exterior column. Vierendeel action can be characterized by relative vertical displacement between beam ends and double curvature deformations of beams and columns. Such a deformed shape provides shear forces in beams to redistribute vertical loads, following the column removal. As joint B5 moved downwards after the column removal, the direction of the bending moments in all the elements connected to joint B5 changed due to moment connection of columns with beams and slabs. If the exterior connections were not moment connections, B5-C5 beams would deform as a cantilever with much larger bending moments at the faces of columns C5.

Analytical results show that after removal of column B5 in the first floor, cracking in the concrete was observed neither in B5-C5 beams nor in the slab. The tensile stresses in the concrete elements were lower than the modulus of rupture of the concrete. Note
that the top reinforcement of beam B5-C5 ends 43” away from the face of column C5. If the tensile stresses exceeded the tensile strength of the concrete at this location, the beam would lose almost all of its flexural strength after cracking. Similarly top reinforcement of the slab in the longitudinal direction cuts off 63” away from the face of the beams. The same type of brittle failure might have been observed at these locations if the cracks formed under negative bending moment. Besides the analytical results, no visible cracks were observed at the mentioned bar cut-off locations in beam B5-C5 and in the slab during the visual examination of the elements in the experimental program after column removal. Analytical results also showed that no yielding occurred in the structure following the column removal.

**3.3.4.4 Load Redistribution**

After removal of column B5 in the first floor the load carried by this column needs to be redistributed to the neighboring elements. In other words, the loads resisted by this column before column removal needs to find a new path to reach to the foundation of the building. After removal of the column, neighboring columns in the vicinity of the removed column are expected to take the loads that used to be carried by the removed one.

The axial forces in columns B4, B5, B6 and C5 in the first floor before and after removal of column B5 in the first floor are shown in Table 3.3. Table 3.3 also shows the differences in the axial loads of the columns. Note that positive values indicate an increase in the axial load. The total amount of increase in the axial load of columns C5, B4 and B6 adds up to 214 kips, 91% of the axial load carried by the removed column B5. The rest of the load is transferred to columns C4 and C6 through the slab. A small portion of the load of column B5 is expected to be carried by the remaining bars of the exploded column. The increase in the axial load of column C5 after column removal is 160 kips, 68% of the axial load carried by the removed column B5. That is, most of load carried by the removed column is transferred to column C5.

The loads are transferred to column C5 through the Vierendeel action of the B5-C5 beams in all floors with the interaction of B5 columns. As explained in Section
3.3.4.3, B5-C5 beams deformed in double curvature due to the moment connection with the columns and developed shear forces to transfer the loads on them.

Each column B4 and B6 carried 12% of the load that used to be carried by the removed column B5. The loads are transferred to these columns through the same mechanism that transferred the loads to column C5. Consider a slab strip running between column B4 and B5 (or B5 and B6). After column removal, this strip is deformed in double curvature due to the moment connections with columns B4 and B5 and developed shear forces to transfer the loads on it. Both the deformed shape and the moment values of the slab indicate the double curvature deformation. Therefore, the slab strips between columns B4 and B5 as well as between columns B5 and B6 behaved like beam B5-C5 and transferred the loads to columns B4 and B6 through the Vierendeel action. Furthermore, the same mechanism can be extended to the other longitudinal slab strips between beams B4-C4 and B5-C5 (and between beams B5-C5 and B6-C6). After column removal and deformation of beam B5-C5, the ends of these strips on Axis 5 moved downwards with beam B5-C5 while the ends on axes 4 and 6 stayed in place. Given the moment connections of these strips with the adjacent beams, they deformed in double curvature and transferred the loads to the adjacent beams through shear forces developed.

Since the stiffness of the B5-C5 beams is more than that of the slab strips mentioned above, considerable amount of the load (68%) is transferred by the C5-B5 beams of different floors (Vierendeel action of axis 5 frame), while the remaining portion is transferred by the one-way slabs to axes 4 and 6.

Figure 3.87 shows the axial forces in B5 columns over the height of the structure. The diagram shows the axial forces in the columns up to the 0.018 seconds after removal of column B5 in the first floor at every 0.003 seconds as well as at the time after the structure stabilized. The axial forces presented in Figure 3.87 were obtained from the analysis in which the damping ratio was assumed to be 0.02. Note that the presented values are closely valid for larger damping ratios.
At 0.003 seconds after the removal of column B5 in the first floor, the second floor column B5 experienced 23% reduction in its axial load. The ratio of reduction in the columns above the second floor decreases gradually over the height of the structure. At the same moment, the amount of reduction in the sixth floor column B5 was only 1% of the initial axial load in this column. The seventh floor and floors above did not experience any reduction at that time. That is, 0.003 seconds after column removal the axial stress wave propagated from the location of the removed column, but has not yet reached the seventh story. Even after 0.006 seconds after column removal the ninth and tenth floors have not been affected, yet the axial force in the second floor column reduced to about 50% of the initial value of the column. The axial forces in B5 columns in different floors shows fluctuations due to the vibration of the system and stabilizes at the values shown in the last part of the Figure 3.87. Columns B5 experience tensile forces during these fluctuations. The maximum tensile force in the columns is estimated at only about 35 kips. Note that column B5 in all floors have compressive force after the vibrations in the structure were damped out.

The axial force histories of the B5 columns above the removed column are plotted versus time for the first 0.02 seconds after column removal and presented in Figure 3.88. The values shown for each floor corresponds to the force at the bottom of the column at that floor. Negative values indicate the compressive axial force. Note that the same conclusions made above can be deduced from Figure 3.88 as well. As can be seen from Figure 3.88, the tenth floor column B5 experiences a drop in axial force for the first time at about 0.008 seconds after the column removal. At that time, column B5 in the second floor lost 67% of its initial axial force.

A validation of the analytical load redistribution can be made using the recordings of strain gages SG4 and SG5 (See Figure 3.73). As can be seen in Figure 3.73, strain gage SG4 was attached to column B5 in the fifth floor and strain gage SG5 was attached to column B6 in the first floor. Note that both strain gages were installed at the mid heights and at the center along the width of the columns to minimize the effect of the bending moment on the strain measurement. Analytical results show that the first floor columns B5 and B6 had the same axial load before column removal. Since the tributary
of these columns are exactly the same and the structure has a quite regular plan, it makes sense to have the same axial load for first floor columns B5 and B6. Strain gage SG5 measured 22 micro strain of permanent increase in the compressive strain of column B6 in the first floor. Based on the analytical results, 12% of the axial load of column B5 in the first floor is transferred to column B6 in the same floor after column removal. If 12% of the axial load of column B5, which is equal to the axial load in the column B6 before column removal, caused an increase of 22 micro strain in the compressive strain of the column, one can assume that the initial compressive strain was 183 micro strain before column removal. Given the vertical regularity of the structure, the axial load in the B5 columns are expected to vary linearly before column removal. Based on that, and assuming the initial strain in column B5 in the first floor was 183 micro strain, the initial strain in column B5 in the fifth floor can be estimated as 110 micro strain before column removal. Figure 3.87 shows the analytical axial force variation in the B5 columns in different floors after column removal. Note that the permanent axial load in the fifth floor column B5 is about 23% of the initial axial load in this column before column removal. 23% of the estimated initial strain of column B5 in the fifth floor (110 micro strains) gives 85 micro strain, which is equal to the measured change in the strain of column B5 in the fifth floor by strain gage SG3.

3.3.5 Potential Types of Failure and Further Load Redistribution

Two types of brittle flexural failures could have occurred in this structure. One was the formation of cracks at beam and slab sections where no top reinforcement existed to carry tensile forces. These would occur during negative flexural bending moments following concrete cracking. The other was potential bottom reinforcing bar pull-out (bond failure) due to lack of proper anchorage. If these brittle failures had taken place in the structure, it could have resulted in larger deformations in the structure and lead to further load redistribution.

Figures 3.89 and 3.90 show the reinforcement details in the second-floor slab and beam existing in the neighborhood of column B5. The same reinforcement pattern is used in other floors. As shown in Figure 3.72, beam B5-C5 has no top reinforcement beyond
43 in away from the face of column C5. If a flexural crack formed at this section due to increased negative bending moments following the column removal, there would be no beam top reinforcement to prevent the crack from opening. Note that the floor reinforcement parallel to the beam is #4@14”, located at the bottom of the slab. As can be seen in Figure 3.85, the negative bending moment in second floor beam B5-C5 at 43 in away from the face of column C5 is estimated at about 1270 kip-in, which is less than the beam cracking bending moment, 2070 kip-in. Figure 3.91 shows the experimentally recorded change in the axial strain at the top of beam B5-C5, at the location of top reinforcing bar cut off (the calculated initial strain was almost zero). The maximum and permanent recorded tensile strains are about 145 microstrain and 113 microstrain, respectively, which are less than the concrete tensile cracking strain. If however, the bending moment at the beam top-bar cut-off location was larger than the beam cracking bending moment (or the tensile strain was larger than the concrete tensile cracking strain), beam participation in transferring the loads (through Vierendeel action) would be compromised and the participation of the slab in the redistribution of the loads would become more significant.

Similarly, the slab top reinforcement normal to the beams is stopped 63” from the faces of the beams resulting in such slab sections being susceptible to formation of flexural cracks under negative bending moments. The analytically estimated bending moments at these locations did not exceed the cracking bending moment. In other words, the maximum calculated tensile stresses were smaller than the tensile strength of the concrete. Therefore, it is important to have reliable estimation of cracking bending moments of the elements as well as the internal force demands developed following column removal.

Sections with no top reinforcement in flexure are shown with thick dashed lines MNPO in Figure 3.89. These sections include beam B5-C5 at 43 in away from column C5 and the extension of this section into the slab (line NP) as well as the slab section at 63 in away from the faces of axes 4 and 6 beams (lines MN and OP). If the tensile strength of the concrete had been smaller such that the maximum applied bending moment at the cut off location of the top reinforcement in beam B5-C5 were larger than
the cracking bending moment, such section would have cracked. In fact, if the modulus of rupture of the concrete had been equal to the values estimated based on concrete compressive strength using ACI-318 (2005), \( f_{r} = 383 \text{ psi} \), cracks would have formed at the beam top reinforcement cut off locations. While beam top reinforcement was continued with #3 bars, the structural drawings as well as physical inspection show that the bar overlap is only about 2 in. Such a splice would fail at a low bar tensile stress. Note that there was no reinforcement at the top of the slab parallel to the beams. Formation of cracks at the cut off location of top bars would have resulted in a reduction of loads transferred through Vierendeel action along axis 5 and the slab would need to participate further in load redistribution. Such redistribution could have potentially resulted in formation of cracks at the slab top bar cut-off locations (lines MN and OP in Figure 3.89). If such cracks formed, then the vertical deformations would increase and another load transfer mechanism could develop through the slab Catenary action between axes 4 and 6.

As can be seen in figure 3.72, the bottom rebars of beam B5-C5 are not well anchored into the columns. The bottom reinforcement of beam B5-C5 extends only about 15 in into column B5. Note that the moment demand at the B5 end of the beam B5-C5 is changed to positive after column removal and the bottom rebars became susceptible to pull out. Beam B5-C5 had negative bending moment before column removal and there was not any risk for a potential pull out.

Similarly, the bottom rebars of the slabs are not well anchored into the beams (See Figure 3.71). The slab bottom reinforcement extends into the beams by only about 8 in and needs to be evaluated for potential pull out when subjected to positive bending moments following the column removal.

In order to verify the drawings, the rebars were exposed at representative locations away from the location of the column to be removed. Figure 3.90 shows top view of one of those locations. Figure 3.90 shows the continuous slab top reinforcement and the beam top straight and bent bars, all of which were in full agreement with the available drawings of the building. As shown in the figure, a 4” gap between the slab
bottom reinforcing bars at the axis of a typical 20” wide beam is observed at the exposed location which verifies the 8” extension of slab bottom rebars into the beams.

Potential locations under the risk of bar pull out are shown in Figure 3.89. Reinforcing bar pull-outs (bond failure) could have occurred under positive bending moment in beam C5-B5 at the face of column B5 (Line SS’ in Figure 3.89) and in the slab flexural reinforcement at the face of beam B5-C5 (Lines QR and Q’R’ in Figure 3.89). Bond failure at these locations would have resulted in larger vertical displacements and further redistribution of the loads. If the anchorage of the beam reinforcement had failed, the Vierendeel action would have further weakened and if the anchorage of slab reinforcement had failed, potential resistance of the slab through Catenary action would have reduced. In the latter case, the slab top reinforcement would have initially contributed in developing Catenary action. However, the top rebars would have torn out of the thin cover at the top, which would have resulted in loss of Catenary action.

3.3.6 Effects of Additional Gravity Loads

As mentioned before, all nonstructural elements including partitions, plumbing, furniture and mechanical systems were removed from the building prior to implosion. Only beams, columns and slabs were present in the structure. Therefore, the loading was only due to the self weight of the structural elements (i.e. columns, beams, and slab). There was no live load applied to the system.

To see the behavior of the structure in the case of the removal of the column B5 when the structure was functional, analysis of the structure is repeated by applying additional gravity loads to the system. One can assume that the following additional gravity loads affected the structure when it was functional: 1) An arbitrary-point-in-time (sustained) live load of 10 psf; 2) A partition dead load of 20 psf; and 3) A mechanical system load of 5 psf. These would add 35 psf to the gravity loads.

Considering the additional loads above, a new analysis is performed and permanent downward displacement of joint B5 in the second floor is found to be about 0.47”, 88% more than the displacement obtained from the analysis without considering
the additional loads. Note that still no yielding occurred in the structure and no cracking
was found at the beam top-bar cut-off locations. The loads are again transferred to the
neighboring columns through Vierendeel action of the B5-C5 beams with the
participation of the slab.

### 3.3.7 Summary

The progressive collapse resistance of the University of Arkansas Medical Center
Dormitory following the removal (explosion) of a first floor exterior column is evaluated
experimentally as well as analytically. Both experimental data and analytical results show
that the 10-story RC structure resisted progressive collapse without partial or full
collapse.

The maximum permanent vertical displacement recorded was about 0.25” at the
joint above the removed column in the second floor following the explosion of the first
floor exterior column. The corresponding measured permanent vertical displacement in
the fifth floor was 0.235”, 6% less than the permanent vertical displacement of the second
floor. The difference between the vertical displacements of the same joint in different
floors is mainly due to the axial deformations of the columns following the column
removal. For the given configuration of the potentiometers used to capture the
displacements (see figure 3.73), the elongation of the columns above the removed column
due to the loss of their axial compressive forces and the shortening of the neighboring
columns due to the additional compressive axial forces as a result of load redistribution
after column removal lead to smaller measured deformations of the beams and slabs in
higher floors compared to those of the lower floors. The same deformation pattern is also
observed in the analytical results. The experimental permanent displacements are closely
captured with the analytical model of the structure. Note that the difference in the vertical
displacements of different floors in the analytical results was due to the elongation of the
columns above the removed one due to the loss of their axial compressive forces.

Development of Vierendeel action in the transverse frame whose exterior column
was removed is identified as the major mechanism for the redistribution of the loads in
this structure. That is, the B5-C5 beams are deformed in double curvature due to the
moment connections with the columns and in turn developed shear forces to transfer the loads to columns C5. Almost 70% of the load that was carried by the removed column B5 is transferred to column C5 (see Figure 3.69) through the Vierendeel action of B5-C5 beams. Double curvature deformation of the second floor beam B5-C5 and the slab along axis B was captured experimentally through the strain gage and potentiometer recordings installed on these elements. The same type of deformation of the elements was captured analytically as well.

After column removal, the deformation of the beams and slabs corresponding to Vierendeel action caused change in the direction of the bending moments of the elements in the vicinity of the removed column. The change in the direction of bending moments could have resulted in a brittle type of failure at the locations where the bottom reinforcements of beams and slabs were not well anchored into the columns and adjacent beams, respectively. For instance, following the column removal, due to Vierendeel action the bending moments in B5-C5 beams and the bending moments in slabs in both longitudinal and transverse directions at the vicinity of joint B5 change from negative to positive. In this structure, no anchorage failure was observed. In general, however, the change in the direction of bending moments and in turn the tensile stress applied to bottom reinforcement requires careful consideration of potential anchorage failure. Considering the significant contribution of Vierendeel action in resisting progressive collapse, the importance of proper anchorage of a portion of beam bottom reinforcement into exterior joints is emphasized.

Furthermore, after column removal the negative bending moment demands went up considerably compared to values before column removal at the locations where there was no top rebar to take the negative bending moment in beams and slabs after cracking of concrete. Both experimental and analytical results show that there was no cracking in concrete after column removal. The analytical results show that tensile strains at the locations where top bars were cut-off were smaller than the concrete cracking strain. If the maximum tensile stresses at these sections were larger than the tensile strength of concrete, brittle flexural failure would have occurred at such sections. Such failure would have reduced the strength and stiffness associated with the Vierendeel action, leading to
larger vertical deformations and higher participation of slabs in the redistribution of loads. The small vertical displacement of the structure after column removal is in part due to the high modulus of rupture of concrete.

The experimental data on the building deformations suggest large damping effects in the system. This damping in part may be due to the dissipation of energy associated with large plastic deformations of the bowed out flexural reinforcing bars of the removed column.

The response of the structure under additional dead loads (i.e. partitions and mechanical systems) as well as an arbitrary-point-in-time (sustained) live load is analytically studied. The analytical results show that the permanent vertical displacement of the joint B5 in the second floor, the maximum displacement in the structure increased by about 88% without leading to progressive collapse. Note that still no yielding occurred in the structure and no cracking was found at the beam top-bar cut-off locations.

3.4 Baptist Memorial Hospital

3.4.1 Building Characteristics

The Baptist Memorial Hospital was located in Memphis, TN (See Figure 3.92). The building consisted of four wings connected to a core. The two North wings were constructed in 1956. The core and the South wings were completed in 1967. The wings and the core were 20 and 22 stories high, respectively, which were separated by expansion joints. Therefore, each part can be considered separately. Figure 3.93 shows the ground floor plan of the North-East wing. The lateral resisting system of the structure in the transverse (North-South) direction consisted of a structural wall along axis 7 and a frame with deep beams along axis 1. The lateral resisting system in the longitudinal (East-West) direction was made of RC frames. The floor system was one-way RC slab supported by longitudinal beams. The slab in the hallway (between Axes B and C) was 6 in (0.152 m) thick. In other areas, the slab thickness was 7 in (0.178 m).

The building was scheduled to be demolished by implosion. As a part of the demolition process, the partitions as well as the exterior walls were removed from the
first, third, sixth, tenth, fourteenth, and seventeenth floors. There were no live loads in the building other than some furniture left in the hospital.

Mechanical characteristics of the concrete and steel were calculated by performing tests on the samples obtained from the building. The concrete compressive strength was estimated as 4100 psi based on three tests performed on the samples from beams. The unit weight of the concrete was 146 lb/ft³. Yield and ultimate stresses for the steel used in the beams are estimated as 62 ksi and 87 ksi respectively based on tension tests of two samples from #5 and two samples from 1 in square reinforcing bars from the beams. The ultimate tensile strain was measured at 0.15. Yield and ultimate stresses for the steel used in the columns are estimated as 70 ksi and 98 ksi, respectively, with an ultimate tensile strain of 0.13 based on tension tests of two samples from #11 reinforcing bars from the columns.

The structural drawings that show the reinforcement details of the structural elements were available at the time of implosion. The reinforcement details of the axis C beams between the axes 2 and 4 in the longitudinal direction are shown in Figure 3.94. Note that second floor beams C2-C3 and C3-C4 are the beams that are bridging over the removed column C3 in the first floor. These beams had the same reinforcement detail in the second, third, and fourth floor.

3.4.2 Initial Damage Scenario

The response of the North-East wing of the Baptist Memorial Hospital following a sudden removal (explosion) of interior column C3 in the first floor is evaluated. As mentioned before, the wings and the core of the structure were separated by expansion joints and therefore each of them can be considered as an individual structure unless the deformations at the expansion joints affect the adjacent part. The removed column is indicated with a red circle in the first floor plan of the North-East wing (See Figure 3.93). Column removal is achieved by explosion of the column. Explosives were inserted to predrilled holes in the column by demolition contractors. The explosion of column C3 was not a part of the implosion. Therefore, no other column exploded in the vicinity of the removed column after column C3 exploded.
3.4.3 Experimental Evaluation

3.4.3.1 Sensors

In the experimental evaluation of the Baptist Memorial Hospital, potentiometers are used to capture the global displacements of the structure and concrete strain gages are used to measure changes in strains. Figure 3.95 shows Frame C (see Figure 3.93) between axes 2 and 4. The dimensions of the beams and columns as well as the heights of floors and the spans are shown. The locations of the sensors are also shown. 4 potentiometers used to capture vertical displacements of joints C3 in the second and seventh floors are shown in Figure 3.95. Also, two strain gages (labeled SG1 and SG2) were used to measure the change in the axial strains of the second and seventh floor columns C3, above the removed column.

The concrete strain gages used in the Baptist Memorial Hospital were 3.5 in long, having a maximum strain limit of ± 0.02. The sampling rate used in the experiment was 1000 Hz. The potentiometers used in the building had a 4” displacement capacity. They had a resolution (repeatability) of about 0.0004 in (0.01 mm) and a maximum operational speed of about 40 in/s (1.0 m/s).

3.4.3.2 Global Displacements

The vertical displacement of joint C3 in the second floor is calculated utilizing the recordings of two potentiometers (P1 and P2, see Figure 3.95) that are connected to this joint from points on the second floor columns C2 and C4, 8 ft above the floor. The locations of the potentiometers are shown in Figure 3.95. Figure 3.96 shows the vertical displacement of joint C3 in the second floor that is calculated by averaging vertical components of potentiometers P1 and P2 recordings. As can be seen from Figure 3.96, joint C3 had a peak displacement of 0.382”, 0.026 seconds after column C3 in the first floor was removed. Following a small vibration, it stabilizes at a displacement of 0.34” after about 0.15 seconds.

Note that the vertical displacement of Joint C3 in the second floor obtained from the recordings of potentiometers P1 and P2 is not the absolute vertical displacement of
that joint. The readings of potentiometers P1 and P2 reflects the relative movement of joint C3 with respect to the points on columns C2 and C4 where the other ends of potentiometers P1 and P2 were attached. To find the absolute displacement, one needs to consider the vertical displacements of those points on columns C2 and C4 after the column removal. After removal of column C3 in the first floor, some portion of the load that used to be carried by column C3 was transferred to columns C2 and C4 and the axial force in these columns increased. As a result, these columns experienced additional compressive strains. Therefore the points on columns C2 and C4 where the other ends of potentiometers P1 and P2 were attached moved downwards after column removal due to additional compressive strains. However, the vertical displacements of these points are small compared to the vertical displacement of joint C3 in the second floor. Note that the same issue was pointed out in the experimental evaluation of the response of University of Arkansas Medical Center Dormitory for progressive collapse. Similar configuration of instrumentation was used to capture the vertical displacement of the joint above the removed column. It was found out that considering the shortening of the neighboring columns that the other ends of the potentiometers were attached to, increased the measured displacement from 0.25” to 0.26”.

The vertical displacement of joint C3 in the seventh floor is presented in Figure 3.97. Similar to the vertical displacement of joint C3 in the second floor, the vertical displacement of joint C3 in the seventh floor is estimated utilizing the recordings of two potentiometers, P3 and P4, that are connected to this joint from points on the seventh floor columns C2 and C4, which were 8 ft above the floor. The locations of the potentiometers (P3 and P4) are shown in Figure 3.94. As can be seen from Figure 3.97, joint C3 in the seventh floor had a first peak displacement of 0.228” at 0.064 sec after column C3 in the first floor was removed. Without considerable vibration, it stabilizes at a displacement of 0.20” after about 0.4 seconds.

Note that the vertical displacement of joint C3 in the seventh floor obtained from the recordings of potentiometers P3 and P4 is less than the vertical displacement of joint C3 in the second floor obtained from the recordings of potentiometers P1 and P2. That can be explained by the elongation of columns C3 above the removed column as a result
of the loss in their compressive axial forces. After removal of column C3 in the first floor, column C3 in the third floor lost its support and unbalanced forces developed at joint C3 in the second floor. These unbalanced forces caused the downward movement of this joint. As joint C3 moved downward while joint C3 in the third floor had not experienced any unbalanced force, column C3 in the second floor elongated and the compressive strains on this column reduced. As a result, the axial force dropped and caused unbalanced forces at joint C3 in the third floor. This chain action continued up to the roof. At the end, column C3 in all floors elongated.

Another reason for the difference is that, as mentioned above, the axial compressive forces in the columns that are in the vicinity of the removed column (e.g. columns C2 and C4) increased since the loads that were carried by the removed column were transferred to these columns. In turn, the compressive strains in these columns increased as a result of the additional axial compressive force. The joints C2 and C4 in all floors moved downwards. Since the displacement of joints C2 and C4 adds up in the upper floors, the points on the second floor columns C2 and C4 that the other ends of potentiometers P1 and P2 attached moved downwards less than those in the seventh floor. The combination of these two reasons explained above caused the potentiometer recordings to show less vertical displacement for the joint C3 in the seventh floor than the joint C3 in the second floor.

3.4.3.3 Variation of Column Strains above Removed Column

As shown in Figure 3.95, two strain gages were installed to the second and seventh floor columns C3 to capture the strain variation in these two floors after column removal. Note that both strain gages were attached at mid-height and at the center of the column width to minimize the effect of bending moment on the strain recording. Figure 3.98 represents the recordings of these two strain gages. Note that strain gage recordings do not represent the absolute strain on the surface that the strain gage is attached to; instead, they show the change in the strain on the surface after column removal. Note, again, that positive change in the recording indicates tensile strain on the surface.
The recording of strain gage SG1 shows that the permanent change in the axial strain of second floor column C3 was 290 microstrain. The change in the axial strain of the seventh floor column had a permanent value of about 150 microstrain. Tensile axial strains are due to the loss of compressive force in the columns above the removed column. Therefore, one can expect more increase in the tensile strain (loss of more compressive force) in the lower floors than the increase in the tensile strain (loss of compressive force) in the upper floors since, conventionally, the lower floor columns carry more compressive force than the upper floors.

Strain gage SG1 recorded 238 microstrain in the second floor column five milliseconds after the column removal, 82% of the permanent change in the strain. The change in the strain of the seventh floor column C3 at that time was only 22 microstrain, 15% of the permanent change in the strain. The reason behind the delay in the change of the strain (i.e. reduction of the compressive axial force) of seventh floor column C3 is that five milliseconds after column removal the peak of the axial strain wave had not yet reached the seventh floor column.

Five milliseconds after column removal, joint C3 in the second floor moved downwards only 0.035”, about 10% of the maximum second floor displacement (see Figure 3.96). In other words, at the time the second floor column (above the removed column) had experienced about 82% of the total increase in its strain, the vertical deformation of joint C3 had only reached 10% of the total displacement. That is the axial forces of columns above the removed column dropped significantly faster than the vertical displacement of the structure. This was due to the higher speed of axial wave propagation compared to that of flexural wave propagation.

3.4.4 Analytical Evaluation

3.4.4.1 Model Description

A three dimensional finite element model of the North-East wing of the Baptist Memorial Hospital was developed in SAP 2000 (2005). The columns and beams were modeled with 2 node Bernoulli beam elements. Beams were modeled with T or L
sections accounting for the effect of the slab. Effective flange width on each side of the web was set equal to one fourth of the span length (ACI 318, 2005). The floors were modeled by two dimensional 4 node linear shell elements. Material nonlinearity was imposed to the model by using localized plastic hinges at the critical locations. The localized plastic hinges are assigned to the locations where yielding can occur (e.g. end of the elements and bar cut-off locations). The force-deformation relationships of plastic hinges are calculated by performing section analysis with the dimensions and reinforcement details obtained from structural drawings and material characteristics. Material characteristics used for concrete and steel were obtained from the tests performed on the samples obtained from the building. It is assumed that plastic hinge length is equal to half of the section depth. The modulus of elasticity of concrete is estimated at 3650 ksi. The modulus of elasticity of steel is set equal to 29000 ksi. Geometric nonlinearity is accounted for in the analysis. A mass proportional damping ratio of $\xi=0.05$ in the first vertical vibration mode is used.

As a part of the demolition, the infill walls and all other nonstructural elements are removed from the first, third, sixth, tenth, fourteenth, and seventeenth floors (see Figure 3.95). The second and fourth floors had hollow clay infill walls. Since the fifth floor was a mechanical floor, there were not any infill walls on that floor. Above the seventh floor all floors had dry walls except the tenth, fourteenth, and seventeenth floors. Neglecting the structural contribution of the dry walls, the clay walls are modeled using shell elements. The mechanical characteristics of the infill walls are obtained from FEMA 356 (2000). The infill walls are assumed to be in good quality based on visual inspection on the infill walls existing in the building. FEMA 356 (2000) suggests a modulus of elasticity of 644 ksi and a tensile strength of 26 psi for infill walls in good quality.

As mentioned before, SAP2000 keeps the stiffness of the beam elements as well as the shell elements constant during the analysis. To account for the cracking of the elements properly an iterative approach is followed:
i) First, all elements are assumed to be uncracked and have full flexural stiffness. A nonlinear time history analysis is run simulating column removal (Analytical column removal is explained later).

ii) The bending moment demands of the beam elements as well as the slab shell elements are compared with the cracking moments. Using the measured concrete compressive strength and based on ACI-318, the modulus of rupture of concrete in this evaluation is set equal to 0.48 ksi.

iii) The flexural stiffness of the beam and slab regions that have bending moment exceeding the cracking moment are reduced with the coefficients of 0.35 as suggested in ACI-318 (2005). No cracks are formed in the columns. This step is executed in a progressive fashion because of the fact that having smaller flexural stiffness would cause smaller bending moment demands in the next analysis (see Section 5.2.1).

iv) Another nonlinear time history analysis is run with the new model. The bending moments are again compared. The elements that have bending moment demand more than cracking moment are determined and flexural stiffness of those are reduced.

v) Step (iv) is repeated until all cracked regions are appropriately modeled.

Note that for the shell elements, the moment of inertia is reduced only in the proper direction. Note that yielding in slab cannot be modeled with shell elements. However, the bending moments in the slab shell elements were less than the yield moments allowing the use of linear shell elements. Also no cracks are formed under negative bending moments at the beam top reinforcement cut-off locations.

Cracking in the infill walls (modeled by shell elements) is also accounted for using the same procedure to model the cracked regions in the beams and slabs. Along with the beams and slabs, tensile stresses in the shell elements that model the infill walls are compared with the estimated tensile strength of the infill walls. Elements are separated at their interfaces where the developed tensile stresses are larger than the tensile strength. The building is reanalyzed and moments in beams and slabs and tensile
stresses in the infill walls are re-evaluated. This procedure is continued until all the cracked regions are properly identified and modeled.

Since element removal during an analysis is not readily available in SAP 2000, a method similar to the procedure explained in Section 3.2.4.1 is used to obtain the response of the structure following removal of the column analytically. Different from the other structures presented in this chapter, the remaining reinforcements of the exploded column are also included in the model. The steps of the procedure that is followed to account for the effect of the remaining rebars on the response of the structure are explained and visualized in Table 3.4.

The exploded column C3 in the first floor had 12#11 longitudinal deformed rebars. Figure 3.99 shows the removed column before and after the explosion. The high outward pressure of the explosion bent the bars outward. Following this initial deformation, the vertical downward movement of the top joint of the removed column increased the bending of the bars due to P-Delta effect. The outwards deformation of the bars can be clearly seen in the Figure 3.99 (b). The exposed length of bars following the explosion was about 90 in.

A schematic representation of column steel bar models and their connectivity is shown in Figure 3.100. Each column rebar is modeled with a beam element having localized plastic hinges. As can be seen, each rebar is connected with rigid elements to the column center line. Note that only four bars are shown in Figure 3.100 for clarity. In the model, a total of 12 beam elements are used to model the steel bars of column C3 as in reality.

The initial amount of outward deformation of the rebars was not known. The structure is analyzed assuming two initial values for the out-of-plane deformations of 0.5” and 2” (four times different) at the center along the height of the exposed rebars. The results show that both analyses with initial out-of-plane deformations of 0.5” and 2” gave almost identical deformation histories of the structure. Furthermore, the axial force variations in the bars for the two assumed outward deformations were also similar. That is, both had the same peak value of about 160 kips and the axial forces were only about
10% different at the end of the dynamic response. In other words, the response of the structure was not sensitive to the initial outward deformation of the rebars within the range mentioned above. The analytical results presented in this section are obtained from the analysis with an out-of-plane deformation of 0.5”. The analytical residual out-of-plane deformation at the center along the height of exposed rebars was 4.8”, very close to the experimental value of 4.5”.

3.4.4.2 Global Displacements

Figure 3.101 shows the analytically obtained vertical displacement of Joint C3 in the second floor. It also shows the vertical displacement of same joint recorded experimentally. Analytical vertical displacement of Joint C3 shows a permanent displacement of 0.41” while the experimental permanent displacement was about 0.34”. As can be seen from the figure, the displacement obtained from the analytical model shows considerable vibration after peak displacement compared to the experimental displacement. Similarly, the analytical and experimental displacement history of joint C3 in the seventh floor is presented in Figure 3.102. As in the second floor, the analytical displacement has significant vibration after peak displacement compared to experimental displacement. Both figures 3.101 and 3.102 show that the analytical permanent displacements in the second and seventh floor overestimate the corresponding experimental data by about 17% and 10%, respectively. The overestimation of the displacements can be in part due to smaller values of estimated concrete modulus of elasticity and modulus of rupture as well as effective flange width of T beams.

When comparing the analytical and experimental vertical displacements for joint C3 in the second and seventh floors, one needs to keep in mind that the vertical displacements of the C3 joint on the second and seventh floors obtained from the recordings of potentiometers are not absolute vertical displacements of these joints as mentioned before. The potentiometers used to measure the displacements of joint C3 in the second and seventh floors reflects the relative displacement of these joints with respect to the joints that other ends of the potentiometers were connected to (joints C2 and C4 on the third and eighth floors, respectively). Due to the increase in the axial forces
of columns C2 and C4, the joints on these columns moved downward following the column removal. However, the vertical displacement of joints C2 and C4 on the third and eighth floors are not significant compared to the vertical displacement of joint C3 in the second and seventh floors, respectively. The average peak vertical displacement of joints C2 and C4 in the third and eighth floors are calculated analytically at about 0.044” and 0.017”, respectively. Each one of these values is reduced to about one half at the end of vibration. To have a reasonable comparison between the analytical results and the experimental data, the average of the analytically estimated vertical downward movements of joints C2 and C4 in the third and eighth floors are added to the experimental displacements of the second and seventh floor joint C3 that are obtained from the potentiometer recordings and then presented in Figure 3.103 and 3.104.

### 3.4.4.3 Local Deformations

Figure 3.105 shows the analytical deformed shape of beams and columns between transverse axes 2 and 4 along longitudinal axis C up to the tenth floor with a scale factor of 200. As can be seen from the figure, beams C2-C3 and C3-C4 in all floors are deformed in double curvature. Note that the deformed shape presented in Figure 3.105 is obtained after all the vibrations in the structure are damped out.

Analytical bending moment diagrams of the same elements whose deformed shapes are shown in Figure 3.105 are presented in Figure 3.106 (b). Similar to Figure 3.105, the moment diagrams show the permanent values after the structure stabilized. Figure 3.106 (a) shows the bending moment diagram of the same frame under dead loads. Note that the same scale factor was used to plot the bending moment diagrams in both Figure 3.106 (a) and (b). As can be seen from Figure 3.106 (a), the C3 ends of beams C2-C3 and C3-C4 in all floors had negative bending moment (tension on the top) under dead loads. The same ends experiences a considerable drop in negative moment demand after removal of column C3 in the first floor. The C3 ends of beams C2-C3 and C3-C4 in third and fifth floors changes to positive bending moment (tension on the bottom).

Similar to the change in the direction of bending moments at the C3 ends of beams C2-C3 and C3-C4, the direction of the slab bending moments showed a change. In
the vicinity of joint C3 slab bending moments were negative in both the transverse and longitudinal direction. After column removal positive bending moments developed in both directions around joint C3. Slab negative bending moments in the vicinity of columns C2 and C4 as well as B3 and D3 showed an increase after column removal.

As can be seen from the figure 3.106 (a) and (b), the bending moments in the C3 columns before and after column removal do not show a considerable change. In both cases, the column moments are small because of the symmetry of the structure and the loading. The bending moment of columns C2 and C4, however, shows considerable change after column removal. That is, the top and bottom ends of these columns need to be in equilibrium with the increased beam end moments at joints C2 and C4.

Deformed shapes as well as the moment diagrams indicate the development of Vierendeel action in the frame shown in Figures 3.105 and 3.106 following the removal of the first floor interior column C3. Vierendeel action can be characterized by relative vertical displacement between beam ends and double curvature deformations of beams and columns. Such a deformed shape provides shear forces in beams to redistribute vertical loads, following the column removal.

The axial forces in columns B3, C2, C3, C4 and D3 before and after removal of column C3 in the first floor are shown in Table 3.5. Table 3.5 also shows the differences in the axial loads of the columns. Note that negative values indicate an increase in the axial compressive load. Note also that the axial load shown for column C3 after column removal is the load carried by the remaining longitudinal reinforcement of the column. Note that for brevity, the slight increase in the axial forces of columns B2, D2, B4, and D4 are not shown in the table. The total amount of increase in the axial load of columns C2, C4, D3 and B3 adds up to 563 kips, 85% of the axial load previously carried by the removed column C3. The rest of the load is transferred to columns B2, D2, B4, and D4 through the slab. The results show that the slab transferred a considerable amount of load to column B3, which was closest to the removed column.

The loads are transferred to columns C2 and C4 through the Vierendeel action of beams C2-C3 and C3-C4 in all floors. As explained in Section 3.4.4.3, beams C2-C3 and
C3-C4 deformed in double curvature due to the moment connection with columns and developed shear forces to transfer the loads on them.

Columns B3 and D3 carried 30% and 15%, respectively, of the load that used to be carried by the removed column C3. The loads are transferred to these columns through the same mechanism that transferred the loads to columns C2 and C4. Consider slab strips running between column B3 and C3 as well as between column C3 and D3. After column removal, these strips deformed in double curvature due to the moment connections with columns as well as the beams and developed shear forces to transfer the loads. Both the deformed shape and the moment values of the slabs indicate the double curvature deformation. Therefore, the slab strips between columns B3 and C3 as well as between column C3 and D3 behaved like beams C2-C3 and C3-C4 and transferred the loads to columns B3 and D3 through Vierendeel action.

Since the length of the slab strip between the columns B3 and C3 is shorter than the length of the slab strip between the columns C3 and D3, the stiffness of the slab strip between the columns B3 and C3 is more than that of the slab strip between columns C3 and D3. Because of that, more gravity load was transferred to column B3 than column D3 after column removal.

The explanation of the behavior of the slab using the strip notion reduces the load transferring mechanism of the slab to a 1 dimensional problem; in reality, the deformation and the load transferring mechanism of the slab occurs in 2 dimensions. Most of the load transferred through the slab, however, was transferred on the strips mentioned above.

The axial force histories of columns C3 above the removed column for the first eight floors are shown in Figure 3.107 for the first 0.5 seconds after column removal. The values shown for each floor corresponds to the force at the bottom of the column at that floor. Negative values indicate the compressive axial force. Figure 3.107 also shows the axial force history of the remaining rebars of the removed column C3. As explained before, the longitudinal reinforcement of the removed column C3 were included in the model explicitly. After all dead loads were applied to the structure before column
removal, the longitudinal reinforcement of column C3 in the first floor was carrying about 20% of the axial load carried by column C3, which equals 160 kips as can be seen in Figure 3.107 before initiation of column removal (i.e. before 0 seconds). At the peak displacement of joint C3 in the second floor, the axial force in the rebars reduces to 110 kips. After all the vibrations are damped out in the structure, the axial load in the rebars stabilizes at around 95 kips, about 13% of the total axial load of column C3 before column removal, which was 754 kips. The response of the structure in the case of the complete removal of column C3 in the first floor including the rebars is also studied and presented in section 3.4.4.6.

Figure 3.108 shows the variation of axial force in the C3 columns above the removed column for the first eight stories. Figure 3.108 shows the axial force variation in these columns for the first 0.018 seconds after column removal at every 0.003 seconds. Furthermore, the axial force variation in the columns after the structure stabilized (final) is also shown.

As can be seen from Figures 3.107 and 3.108, after the structure stabilized, the fifth floor column C3 carries more axial compressive force than the columns in the floors below. Note that beams C2-C3 and C3-C4 in the fifth floor are deeper than the same beams in other floors (See Figure 3.95). In other words they have more stiffness and strength than the other beams that are bridging over the removed column along the height of the structure. Because of that, they have a larger contribution in the gravity load redistribution. Therefore, the fifth floor column is provided with a higher upward force at the bottom, compared to those for the lower floor columns. As a result, the axial compressive force in the fifth floor column at the end of vibration is larger than those of the columns below. However, the axial compressive forces in the fifth to the eighth floor columns at the end of vibration are almost equal. That is, each one of these floors transfers its gravity loads almost entirely on its own. The axial force in the C3 columns in the fifth to eight stories are the largest axial forces over the height of the structure. The axial force in the ninth floor column C3 is about 200 kips. The axial force in the C3 columns above the ninth floors gradually reduces towards the twentieth floor.
As can be seen from Figures 3.107 and 3.108, the least amount of axial force over the height of the structure develops in the second floor column C3 after column removal. It is almost equal to the axial force carried by the remaining rebars. The third and fourth floor C3 columns have almost the same axial force, which is more than the axial force of the second floor column C3. This is in part due to the fact that diagonal compressive struts form in the second floor infill walls, providing support for joint C3 in the third floor, leading to a larger column axial compressive force in the floor above.

Figure 3.108 shows that 0.003 seconds after column removal, the second floor column C3 experienced 37% reduction in its axial load. At 0.006 seconds after column removal the reduction in the second floor column C3 reached 62%. The ratio of reduction in the columns above the second floor decreases gradually over the height of the structure. The eighth floor and floors above did not experience any reduction at that time. That is to say, 0.006 seconds after column removal the axial stress wave propagated from the location of the removed column, but has not yet reached the eighth story. Note that the axial forces in the columns at 0.018 s are almost the same as those at the end of vibration.

In order to better characterize the load redistribution in the structure, the deformations of the beams over the height of the structure is evaluated below. The axial deformation of columns in this structure had a significant effect on the beam deformations and in turn on the load redistribution. After column C3 in the first floor was exploded, the axial compressive forces in C3 columns above the removed column reduced. The reduction in the compressive force in these columns resulted in additional tensile strains in these columns. As a result, column C3 in all floors elongated. This in turn led to a smaller vertical displacement of joint C3 in a upper floor compared to that of a lower floor.

After removal of column C3 in the first floor, some portion of the load that used to be carried by column C3 was redistributed to columns C2 and C4. Therefore, the axial compressive force in columns C2 and C4 in all floors increased. The increase in the axial compressive force in these columns caused additional compressive strains and in turn the
length of columns C2 and C4 in all floors shortened. Therefore, joints C2 and C4 in all floors moved downwards. Given the accumulation of deformations of these columns, the vertical displacements of the neighboring joints C2 and C4 in higher floors were more than those in lower floors.

Schematic representation of deformation of beams C2-C3 and C3-C4 bridging over the removed column for two stories, one in a lower floor and the other is in a upper floor are shown in Figure 3.109. The initial position of the two beams is shown with a dashed line representing the shape of the beams before column removal. The deformations of the beams before column removal are neglected. Note that the end displacements at joints C2 and C4 are different for two beams because of the reason explained above (i.e. accumulation of deformations of columns C2 and C4). Note also that the vertical displacement at the center of the beam that is in the lower floor is more than vertical displacement of the beam that is in the upper floor due to the elongation of columns C3 in all floors as a result of loss of axial compressive force. Relative vertical displacements of joint C3 with respect to joint C2 and C4 are also shown in the figure. As can be seen, such deformation for the lower floor beam is larger than that for the higher floor beam. In order to quantify the difference between the deformation of the lower and higher floors, the elongation of column C3 is evaluated. At the end of vibration, the total elongation of columns C3 from the second to the twentieth floor was about 0.21 in. Furthermore, the average shortening of columns C2 and C4 over the height of the structure was about 0.04 in. That is, the vertical deformation of the top floor beam was about 0.25” less than that in the second floor. This was more than half of both experimental and analytical vertical displacements of the second floor joint C3 at the end of vibration, which were 0.36” (considering the vertical movements of joints C2 and C4 in the third floor) and 0.40”, respectively.

As can be seen from Figure 3.109, the flexural deformation of the beam in the lower floor is more than the flexural deformation of the beam in the higher floors. Assuming the same element dimensions for the two beams shown in Figure 3.109, curvatures and, in turn, bending moments of the beam in lower floor are larger that those of the beam in the upper floor. Furthermore, because the lower floor beams in general are
stronger and stiffer, they tend to transfer larger shear forces to the neighboring columns. For instance, analytical results show that the shear forces transferred to joint C4 by the sixth and twentieth floor beams, none of which were directly connected to infill walls, were about 28 kips and 16 kips at the end of vibration, respectively.

The same phenomenon can be extended to the behavior of the slabs in different floors. As mentioned before the slab strip notion running between columns B3 and C3 as well as C3 and D3 can be used to describe the load transfer to columns B3 and D3. As in columns C2 and C4, some portion of the load that used to be carried by column C3 was transferred to columns B3 and D3 and axial compressive force in these columns increased. The axial compressive strains in these columns also went up causing the shortening of these columns in all floors. As a result, joints B3 and D3 in all floors moved downwards having more displacement in the upper floors because of the accumulation of the displacements. Therefore the relative displacement of joints C3 with respect to joints B3 and D4 decreases as the floor level increases. In other words, the slabs in lower floors experienced larger deformations. Given the same distance between columns B3 and C3 as well as C3 and D3 and the same slab thickness in all floors, the slabs in lower floors redistributed larger shear forces than those in the upper floors. Again both beams and slabs in the lower floors deformed more than the ones in the upper floors and in turn had more contribution in the load redistribution. Therefore, the floors above are in part supported by the floors below. In this structure, given the deep beam in the fifth floor and the existence of the infill walls in the second and fourth floors, even more support was provided for the higher floors by the lower floors of the structure.

It should be emphasized that the importance of the column axial deformations in the load redistribution of the structure over the height would have been smaller if the vertical displacement of the structure had been larger. The axial deformation of the columns above the removed one as well as the neighboring columns becomes negligible as the vertical displacement of the joints above the removed column increases. Furthermore, the nonlinear response of the beams and floor slabs becomes more important and the forces do not linearly increase with larger deformations. Note that in
this structure, no plastic hinges formed in the beams above the fifth floor and the floor shell elements remained elastic.

**3.4.4.5 Effect of Remaining Bars of Removed Column on Damping of Structure**

Figure 3.110 shows the total axial compressive force of the rebars in the removed column, up to the maximum vertical displacement of joint C3 in the second floor. The points associated with the initiation of the dynamic response and the first peak displacement of joint C3 in the second floor are shown in the figure. Following the initial lateral deformation of the bars due to the explosion (and the complete removal of the concrete), as joint C3 in the second floor starts to move downward, the axial force imposes bending moments in the bars (P-Delta effect), which in turn leads to larger out-of-plane deformation and formation of plastic regions. In order to evaluate the effects of the dissipated energy in these bars on the response of the structure, consider an equivalent single-degree-of-freedom (SDOF) system associated with the vertical displacement of joint C3 as described by Sasani and Sagiroglu (2008). The equivalent damping ratio $\xi$ for a SDOF system can be estimated as (Chopra, 2000)

$$\xi = \frac{E_D}{4\pi E_S}$$

Equation 3.1

where $E_D$ and $E_S$ are the dissipated and elastic restored energies, respectively.

The amount of the dissipated energy in the rebars within this downward movement is about 61 k-in. The restoring force in the structure during the downward movement varying from zero to the peak value corresponds to a restored energy of about 128 k-in. Because such downward movement can be considered as a quarter cycle vibration (the restoring force changes from zero to its peak value), the equivalent damping ratio due to the nonlinear response of the column C3 bars is about 0.15 (Equation 3.1).
3.4.4.6 Response of Structure with Additional Gravity Loads and Complete Column Removal

All nonstructural elements including partitions, plumbing, furniture and mechanical systems as well as the exterior walls were removed from the first, third, sixth, tenth, fourteenth, and seventeenth floors prior to implosion. To investigate the effects of these additional gravity loads on the building response, in addition to the exterior walls, a unit floor load of 30 psf is added to the model to account for the weights of the partitions, plumbing, furniture and mechanical systems in these floors. There was no live load applied to the system except some furniture left in the hospital. To see the behavior of the structure in case of the removal of column C3 when the structure was functional, an additional live load of 25 psf, which is equal to 50% of the design live load is added to all floors. Furthermore, the remaining bars of the removed column C3 in the first floor are not included in the new model simulating the complete removal of column.

Considering the additional loads above and complete column removal, a new analysis is performed and maximum downward displacement of joint C3 in the second floor is found to be about 0.85”, which is 70% more than the maximum displacement obtained from the analysis without considering the additional loads and complete column removal. Permanent displacement of the same joint increased about 60% to 0.64”.

3.4.5 Summary

The progressive collapse resistance of the North-East wing of the Baptist Memorial Hospital following the removal (explosion) of a first floor interior column is evaluated experimentally as well as analytically. Both experimental data and analytical results show that the 20-story RC structure resisted progressive collapse without partial or full collapse.

The absolute maximum and permanent vertical displacements recorded were about 0.42” and 0.36”, respectively at the second floor joint C3 following the explosion of the first floor interior column C3. The corresponding absolute measured permanent displacement in the seventh floor was 0.21”, about 60% of the permanent displacement
recorded in the second floor. The difference between the vertical displacements of the same joint in different floors is mainly due to the axial deformations of the columns following the column removal. Analytical studies also verify that the displacements of higher floors were smaller than the lower floors.

The effects of the axial deformations of the columns on the load redistribution of the structure following the column removal are described. The elongation of the columns above the removed column due to the loss of their axial compressive forces and the shortening of the neighboring columns due to the additional compressive axial forces as a result of load redistribution after column removal lead to smaller measured deformations of the beams and slabs in higher floors compared to those of the lower floors. The same deformation pattern is also observed in the analytical results. As a result, the beams and slabs in the lower floors had more contribution in the load redistribution than the beams and slabs in the upper floors.

The analytical results show that 85% of the load carried by column C3 before explosion is transferred to the neighboring columns B3, D3, C2 and C4. The loads are transferred in the longitudinal direction to columns C2 and C4 through the Vierendeel action of beams C2-C3 and C3-C4. The same mechanism is observed in the load redistribution through the slab in the transverse direction to columns B2 and B4. That is, the beams C2-C3 and C3-C4 as well as slab strips between columns B3 and C3 and also C3 and C4 deformed in double curvature due to the moment connections with the columns and in turn developed shear forces to transfer the loads to neighboring columns.

The experimental histories of the axial strains of the second and seventh floor columns C3 above the removed column and the vertical displacement recording of second floor joint C3 are presented to discuss the difference between the speed of axial wave propagation and flexural wave propagation. That is, 0.005 seconds after column removal the drop in the compressive strain on column C3 in the second floor reaches 82% of the permanent value while at the same moment joint C3 in the second floor moved downwards only 0.035”, only 1/10th of the peak displacement.
To see the effect of the remaining rebars of the exploded column on the building response, rebars were included in the analytical model. After explosion, it is assumed that only the concrete part of the column C3 in the first floors lost its load carrying capacity completely. Rebars of the column, however, stay in place but deformed outwards due to high pressure of the explosion. After the structure stabilized, the axial load carried by the remaining rebars is found out to be about 13% of the axial load of the removed column before explosion. It is demonstrated that the dissipated energy by the rebars corresponded to an additional effective damping ratio of about 0.15 in the first mode of vertical vibration.

Considering the additional dead loads (i.e., the inclusion of the removed exterior walls, partitions and mechanical systems) as well as half of the design live load with complete column removal, a new analysis is performed and maximum downward displacement of joint C3 in the second floor is found about 0.85”, which is 70% more than the maximum displacement obtained from the analysis without considering the additional loads and complete column removal. Permanent displacement of the same joint increased about 60% to 0.64”.

3.5 Summary

One way to assess progressive collapse resistance of a structure is to evaluate the response of the structure following removal of a load bearing element as suggested in GSA and DOD guidelines. Experimental as well as analytical evaluations of three different real structures with three different initial damage scenarios are studied and presented in this chapter. During the experimental program, the buildings that are scheduled to be demolished by implosion are used to obtain experimental data. The experimental element removal is achieved by the explosion of the columns.

First, the response of Hotel San Diego in San Diego, CA, which was a 6-story RC structure subjected to removal of two exterior columns in its first floor, is presented. The structure resisted progressive collapse with maximum vertical displacements of 0.242” at the top of the removed columns. The dominant progressive collapse resisting mechanism for this structure is found out to be the bi-directional Vierendeel action beams in the
longitudinal and transverse directions. Additionally, some of the load that used to be carried by the removed columns were redistributed through the joist floor and the infill walls that were present on the top of the spandrel beams. Two types of potential brittle failure related to bar pull out and cracking of the beam sections lacking tensile reinforcement are described.

Second, the response of the University of Arkansas Dormitory in Little Rock, AR, which was a 10-story RC structure, is studied following removal of an exterior column in its first floor. In spite of the fact that two potential brittle failures could have happened in this structure, such as bar pull out due to lack of anchorage and cracking of the beam section that had no tensile reinforcement, the displacements and deformations in the structure after column removal were small, not causing any failure. The maximum and permanent recorded vertical displacements were equal, 0.25” at the top of the removed column. An analytical model of the structure captured the measured displacements well. The dominant load redistribution mechanism for this structure was the Vierendeel action of the transverse beams with the participation of the slab.

Finally, the response of the Baptist Memorial Hospital in Memphis, TN, which was a 20-story RC structure subjected to removal of an interior column in its first floor, is studied. The structure resisted progressive collapse without any partial or full failure. The maximum and permanent vertical displacements measured at the top of the removed column were 0.38” and 0.34”, respectively. An analytical model of the structure captured the measured displacements reasonably well. The load that used to be carried by the removed column is transferred to the neighboring columns without any failure through Vierendeel action of beams in the longitudinal direction and slabs in transverse direction. The effect of the change in the column axial forces (and as a result, strains) on the deformation of the beams in different floors and, in turn, on load redistribution is described. The effect of the remaining bars of the exploded columns in the experimental evaluation on the damping of the structure is also investigated.

In all the structures evaluated experimentally and analytically in this chapter, Vierendeel action is found to be the governing progressive collapse mechanism.
Vierendeel action can be described as relative vertical displacement between beam ends (or slab strips) and double curvature deformations of beams and columns. Shear forces developed in beams (or slab strips) as a result of such a deformed shape redistributed the gravity loads to the neighboring columns.

In the evaluated structures, three types of brittle failure risks have arisen as a result of development of Vierendeel action. Bending moment demand in the beams and slabs at their removed column sides changes to positive moment from negative moment due to relative end displacement and double curvature deformation. In case of the lack of bottom reinforcement at those sections, which is not needed considering only the dead loads, brittle flexural failure may be observed if the moment demands at such section reach the cracking moment capacity. Note that if the lateral loads are considered in the design, then beam end sections are likely to have bottom reinforcement. The amount of bottom reinforcement at the end regions of the beams depends on the lateral load resistance of the structure.

In the case of having bottom reinforcement at such sections, development of the full strength of the section depends on the existence of proper anchorage of the bottom reinforcement. Again, it is not an issue not having a proper anchorage of the bottom reinforcement into the column when only dead loads are considered. However, after column removal and the development of positive bending moment in the beams (or slabs) at the face of the removed column, the bottom bars become vulnerable to bar pull out due to potential lack of anchorage.

Another type of brittle failure in beams (or slabs) associated with the development of Vierendeel action could happen when the amplified negative bending moment demand in the beams (or slabs) exceeds the capacity of the top bar cut off locations. If the minimum reinforcement was not provided beyond such locations, brittle flexural failure could happen when the bending moment reaches the cracking bending moment capacity. The modulus of rupture of the concrete plays an important role in the behavior of such sections. A high modulus of rupture of the concrete can avert such a brittle failure.
In all the structures evaluated in this chapter, Vierendeel action was adequate to transfer the loads that used to be carried by the removed column(s) with the participation of the slabs and infill walls (if there was any). The performance of the Vierendeel action in the redistribution of the loads was not disrupted due to the potential brittle failures mentioned above. The loads are redistributed in all cases without having large deformations in the structures. If one or more of the brittle failures mentioned above happened, the Vierendeel action would weaken and in turn a new load redistribution mechanism could come to play, such as Catenary action. Note that the development of Catenary action for the Baptist Memorial Hospital and the University of Arkansas dormitory building was possible since the beams or slabs bridging over the removed column were continuous at least in one direction. For Hotel San Diego, however, if the Vierendeel action lost its contribution in load redistribution, Catenary action could not have formed because of the given geometry and the locations of the removed columns.

Note that a structure needs to experience large displacements for Catenary action to develop and transfer the loads to the neighboring columns through the tensile forces in the beams (or slabs) bridging over the removed column. Development of Catenary action after the vanishing of the Vierendeel action in a small scale 2-dimensional RC frame structure is presented in Chapter 4. Although the participation of the Catenary action in load redistribution existed before, the performance of the Catenary action for the frame presented in chapter 4 became significant after about 8” of vertical displacement during the testing of the frame under monotonically increased vertical displacement applied to the top of the removed column. Note that the frame tested during the experimental program was a 1/8th scaled model. Therefore, 8” of vertical displacement in the scaled model corresponds to 64” of vertical displacement in a full scale structure.

The difference between the speeds of axial wave propagation and flexural wave propagation is discussed. That is, the columns above the removed column lose their axial forces within a few milliseconds after column removal while it takes much more time for the structure to deform and in turn for the loads to be transferred to the neighboring columns.
After column removal, the columns above the removed one lose their axial compressive forces and elongate. The neighboring columns, however, experience compressive strains due to the additional compressive forces as a result of the load redistribution. As a result of combination of these two facts, the beams and the floor systems in the lower floors participate in load redistribution more than those in the upper floors. The difference between the contribution of the lower and upper floors in load redistribution due to the change in the column lengths becomes considerable when the maximum displacement above the removed column(s) is small.

The effect of the infill walls on the stiffness and load redistribution of the structure were found to be considerable, especially when the displacements were small and infill walls do not have significant damage. In the progressive collapse evaluation of Hotel San Diego, ignoring the infill walls in the analytical modeling of the structure resulted in a 240% increase in the maximum displacement. Modeling the infill walls with the compressive strut method in FEMA 356 resulted in an under-estimation of the stiffness of the walls, especially under small displacements. For small displacements, 2-dimensional shell elements (including cracking) were found to be more appropriate to model the infill walls.

In the experimental program the columns that are selected to be removed are exploded by the demolition contractors. Due to the high air pressure of the explosion, the rebars of the columns bent out but did not break while the concrete fell apart. The effect of the remaining bars on the damping of the structure as well as their load carrying capacity is investigated. In the evaluation of Baptist Memorial Hospital, it is demonstrated that the dissipated energy by the rebars corresponded to an additional effective damping ratio of about 0.15 in the first mode of vertical vibration. Following column removal the axial load carried by the remaining rebars is found out to be about 13% of the axial load of the removed column before explosion.
Table 3.1 Axial forces in first floor columns before and after column removal (FEM)

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Table 3.2 Axial forces in first floor columns before and after column removal (AEM).

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<tr>
<td>Change</td>
<td>-55.2</td>
</tr>
</tbody>
</table>

Table 3.3 Axial forces in first floor columns before and after column removal

<table>
<thead>
<tr>
<th></th>
<th>Column Axial Forces (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>B4</td>
</tr>
<tr>
<td>Before</td>
<td>235</td>
</tr>
<tr>
<td>After</td>
<td>265</td>
</tr>
<tr>
<td>Change</td>
<td>30</td>
</tr>
</tbody>
</table>
Table 3.4 Column removal procedure.

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>The structure is analyzed under the gravity loads and the internal forces are calculated at Section A-A. Given the axial strain of column C3, the force transferred by the column steel bars is calculated. Note that the bending moment due to the gravity loads in the column prior to column removal is negligible and the column is primarily under an axial compressive force.</td>
</tr>
<tr>
<td>II</td>
<td>In order to model the loss of concrete in the column due to the explosion and allow the steel bars to deform independent of the concrete, another model of the structure is developed. In this model, the concrete portion of column C3 is removed and its contribution to the column end forces ($F_c$) is applied to the structure along with the gravity loads. Note that for clarity, only the axial force is shown in the figure. Each column rebar is modeled as a beam element with localized plastic hinges (only two steel bars are shown in the figure). Note that the internal forces and the deformed shape of the two models at the ends of steps I and II are identical.</td>
</tr>
<tr>
<td>III</td>
<td>Forces with equal magnitudes and opposite directions to the loads applied at the top of the column (due to the concrete) are applied to the structure, to model the loss of concrete due to the explosion. The time period for these forces to reach their full values is two milliseconds, which is based on previous studies on column removal.</td>
</tr>
<tr>
<td>IV</td>
<td>Within the same time period (two milliseconds), the reinforcing bars are deformed outwards as a result of the explosion (Explosives were inserted into predrilled holes in the column) and the dynamic response of structure is evaluated.</td>
</tr>
</tbody>
</table>
Table 3.5 Axial forces in first floor columns before and after column removal.

<table>
<thead>
<tr>
<th></th>
<th>Column Axial Forces (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C2</td>
</tr>
<tr>
<td>Before</td>
<td>-773</td>
</tr>
<tr>
<td>After</td>
<td>-915</td>
</tr>
<tr>
<td>Change</td>
<td>-142</td>
</tr>
</tbody>
</table>

*Axial force in first floor column C3 longitudinal rebars
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Chapter 4

Experimental and Analytical Evaluation of Response of an RC Frame to Loss of a Column

4.1 Introduction

The progressive collapse resistance of the exterior frame of a 3 story RC building following the instantaneous removal of a first floor column is evaluated experimentally as well as analytically. Figure 4.1 shows the 3 dimensional view of the building of which the exterior frame is considered to be evaluated. The exterior frame in question is highlighted in the figure. A 1/8th scaled model of the frame with all the reinforcement details and scaled concrete has been constructed and tested. After scaled loading is applied to the structure simulating the loading condition as in the 3 dimensional building (i.e. providing the same state of stress in the elements), the center first floor column is removed instantaneously and the response of the structure is monitored with a number of sensors.

After removal of the center first floor column, the frame resisted progressive collapse with maximum vertical displacement of 0.426” at the top of the removed column. Furthermore, to study the resisting mechanism(s) of the two dimensional frame to progressive collapse another test is performed by applying a monotonically increased displacement at the top of the removed column, following the column removal. The joint at the top of the center first floor column was pushed down a total of 16.25” and force deformation relationship as well as other deformations were monitored. The force deformation of the frame had a peak of 1981 lbs at 4.71” vertical displacement while there was not significant damage on the structure. The resisting force decreased to around 1300 lbs following fracture of the flexural reinforcement in the center two span beams and then increased up to 2117 lbs because of the development of Catenary action in the center two span beams.
4.1.1 Characteristics of Building of which Exterior Frame to be Evaluated

The building of which exterior frame to be tested is a 3-story RC structure and assumed to be located in Memphis, TN. The 3 dimensional view and the typical plan of the building are shown in Figure 4.1 and 4.2, respectively. The structure has a regular plan with typical span lengths in both directions. The exterior frame of the building on axis 1 is evaluated. As can be seen from the figure 4.2, the floor system is a one-way joist floor with a four-inch thick slab running in the North-South direction. Joists are sitting on the transverse beams. The height of the first floor is 16’ while the height of other floors is 13’. Figure 4.3 shows the elevation view of the frame evaluated. Beams and columns have typical dimensions of 16” by 18” and 16” by 16”, respectively, in all floors.

The lateral resisting system in the East-West direction consists of special moment frames (SMF). Note that ASCE/SEI 7-10 requires a SMF for seismic category D RC frames when the short-period design acceleration ($S_{DS}$) is greater than 0.5g for Category I and II buildings. The short-period design acceleration ($S_{DS}$) was 1.0g (Site class C in Memphis, TN). The response modification factor (R) and seismic response coefficient ($C_S$) used for calculation of the lateral loads were 8 and 0.125, respectively. The building is designed to satisfy the ACI integrity requirements (ACI 318 (2005), Sec. 7.13), and SMF requirements.

The concrete compressive strength ($f'_c$) and steel yield stress ($f_y$) used in the design of the structure were 4 ksi and 60 ksi, respectively. Note that these material characteristics are used for design of the building. The material characteristics used in the analytical evaluation of the two dimensional scaled frame are obtained from the tests performed on the concrete and steel samples of the constructed model and presented in the next section. Unit dead load of 100 psf (including partitions) and live load of 70 psf as well as distributed 100 lb/ft for the exterior nonstructural walls are used in the design of the structure.

Reinforcement details of first and second floor columns and second and third floor beams of the exterior frame are shown in Figure 4.4. Figure 4.5 shows the reinforcement details of third floor columns and roof beams of the exterior frame.
4.1.2 Material Characteristics of Two-Dimensional Scaled Frame

Mechanical characteristics of the concrete and steel of the constructed frame are found by performing tests on the steel and concrete samples. The concrete compressive strength was estimated as 3675 ksi based on the tests performed on the 1”x2” and 2”x4” cylinder samples. The unit weight of the concrete was 135 lb/ft³. Figures 4.6 and 4.7 show the stress-strain relationships for 1”x2” and 2”x4” cylinders respectively. Each plot is the average of two compression tests.

Deformed wires are used for the longitudinal reinforcement of the beams and columns. Figures 4.8 and 4.9 show the stress-strain relationships for the longitudinal wires with 0.11” and 0.135” diameters, respectively. Yield and ultimate stresses for the 0.11” steel wires are estimated as 62 ksi and 86 ksi respectively. Yield and ultimate stresses for the 0.135” steel wires are estimated as 55 ksi and 77 ksi respectively. Material characteristics are measured based on tension tests of two samples from 0.11” and two samples from 0.135” reinforcing bars. The ultimate tensile strain was measured at 0.24 and 0.27 for 0.11” and 0.135” steel wires, respectively. The transverse reinforcement were smooth wires with a nominal yield strength of 65 ksi.

4.2 Experimental Evaluation

4.2.1 Test Setup and Procedure

Figure 4.10 shows the 1/8th scaled two-dimensional model of the exterior frame along axis 1 of the building (see Figure 4.2). Note that the first story center column is made of glass. The glass column had almost the same axial stiffness of the first floor RC columns. The RC frame is fixed to the steel reaction frame at its foundation. The out of plane deformation of the frame is restrained at the second floor columns on the B and D axes with the supports shown in Figure 4.10.

The loads transferred to the exterior frame from the adjacent joists and the beams in the longitudinal direction are applied to the structure. The dead load and the 25% of the design live load is considered in the calculation of the weights to be applied prior to
test. The total weight transferred from each joist is calculated as 179 lbs. Figure 4.11 shows the frame with gravity loads applied.

The dimensions of the scaled model are shown in Figure 4.12. Reinforcement details of first and second floor columns and second and third floor beams of the scaled model of the frame are shown in Figure 4.13. Figure 4.14 shows the reinforcement details of third floor columns and roof beams of the scaled model.

Note that all the dimensions of the constructed model are calculated by dividing the dimensions of full scale building explained in 4.1.1 by a scale factor of 8. This scaling includes span lengths, floor heights, element dimensions, bar diameters as well as the aggregate size of the concrete. The applied loads, however, are calculated by dividing the loads of the full scale building by 64 ($8^2$) to maintain the same state of stress in the constructed scaled model.

The loads transferred from the joists which are running on axes A, B, D and E (see figure 4.2) were not applied to the frame. Note that these loads would be directly carried by the columns on the corresponding axes and will have insignificant effect on the behavior of the beams between axes B and D, which are bridging over the removed column especially for small deformations. The load transferred from the joists on axis C, however, was applied to the frame as shown in figure 4.11, that has a significant effect after removal of the center column in the first floor.

Note that for larger deformations, especially after development of Catenary action in the beams bridging over the removed column, the lateral stiffness of the neighboring frames would play an important role in the load carrying performance of the Catenary action as explained in the following sections. Therefore, the axial load in the neighboring columns would affect the flexural strength of these columns and in turn lateral stiffness of the neighboring frames. Furthermore, the p-delta effect of the axial load in the neighboring columns would be significant after these columns had lateral drift due to Catenary action of the beams bridging over the removed column. Again this phenomenon will be explained in the next sections.
The loads coming from the other joists were applied along the length of the beams where the joists were sitting. Note that the weights on the beams modeling the gravity loads were free to rotate at the locations of application not to apply any other force or moment other than a vertical force to the beams as the structure deforms.

The experimental program on the frame shown in figure 4.10 is executed in two phases. In the first phase, after applying the loads simulating the gravity loads (dead loads and 25% of the design live load) transferred from the joists (see Figure 4.2) to the structure, the first floor center column that was made of glass was broken suddenly. After removal of the first floor center column, the behavior of the frame was monitored with a number of sensors which will be explained in the next section. Recording of the sensors are continued until the structure stabilized.

Following the column removal, all weights applied to the frame are removed to conduct the second phase of the test. That is, a monotonically increased displacement was applied to the top of the removed column. Figure 4.15 shows the loading mechanism used in the second phase to restrain out-of-plane and in-plane movement as well as the rotation in all three axes of the top of the removed column. The displacement is applied to the joint through a steel plate that was connected to the longitudinal reinforcement of the column. A hook type load cell was connected to steel plate to measure the resisting force of the frame to applied displacement. A steel rope attached to the hook of the load cell was pulled with a manually controlled crank.

Second phase (static loading) of the experiment is conducted in several steps. The test is stopped several times at certain displacements to report the condition of the frame. Furthermore, test is also stopped when a significant damage is observed such as bar rupturing.

4.2.2 Instrumentation

The dynamic response of the frame to removal of the first floor center column as well as response of the frame during the displacement controlled loading is monitored with several sensors. Potentiometers are used to measure the vertical and horizontal
displacements of the frame while concrete strain gages are used to measure change in the strain of the columns as well as the beams. Figure 4.16 shows the location of the potentiometers used in the experimental program.

Strain gages are attached to all first floor columns and columns above the removed column as well as the critical locations of the beams such as beam ends and reinforcement cut locations. The location of the strain gages are presented in Figure 4.17. Note that a strain gage is also attached to the glass column to determine the time when the column is removed. As mentioned before, a hook type load cell is attached to the top of the removed column to measure the resisting force of the frame to applied vertical displacement.

4.2.3 Results

4.2.3.1 Dynamic Part (Column Removal)

Following the removal of the first floor center column C1-C2 (see Figure 4.12), the 1/8th scaled model resisted progressive collapse. There was not significant damage other than flexural cracks formed at the end regions of the beams between axes B and D (see Figure 4.12). Neither bar ruptures nor shear failures are observed in beams and columns. Figure 4.18 shows the recording of the strain gage that was installed on the glass column. As can be seen from the figure, duration of the column removal was around 0.002 sec.

Figure 4.19 shows the vertical displacement of joint C2 in second floor (top of the removed column) that was recorded by potentiometers P1 and P2 (see Figure 4.16). Note that the column was removed at 0 sec. Recorded data shows that both potentiometers recorded the same displacement history. Figure 4.19 shows that joint C2 in the second floor had a peak displacement of 0.426” 0.093 sec after the center column in the first floor was removed. Following some vibration, it stabilizes at a permanent displacement of 0.397” at about 1.25 seconds.

Similarly, Figure 4.20 shows the vertical displacement history of joint C4 in the roof that was recorded by potentiometer P3 (see Figure 4.16). Joint C4 in the roof had a
peak vertical displacement of about 0.42”. The vertical movement of the joint stabilizes at about 1.25 seconds with a permanent displacement of 0.39”.

As can be seen from Figure 4.19 and 4.20, joint C2 in the second floor and joint C4 in the roof had almost the same displacement history. The peak and permanent vertical displacement of joint C2 in the second floor were 0.006” and 0.007” more than those of joint C4 in the roof, respectively. The difference between the vertical displacements of these two joints is due to the elongation of columns C2-C3 and C3-C4. After removal of the center column in the first floor, column C2-C3 in the second floor lost its support and an unbalanced vertical force developed at joint C2 in the second floor. This unbalanced force caused the downward movement of this joint. As the joint C2 moved downward while joint C3 on the third floor had not experienced any unbalanced force, column C2-C3 on the second floor elongated and the compressive strains on these columns reduced. As a result axial force dropped and caused an unbalanced vertical force at joint C3 in the third floor. The same sequence repeated in this floor. As a result, joint C4 in the roof experienced less vertical movement than joint C2 in the second floor because of the elongation of columns C2-C3 in the second floor and C3-C4 on the third floor.

Figure 4.21 shows the strain recording of strain gage SG-30 on the third floor column C3-C4 (see Figure 4.17). Note that recordings of the strain gages do not show the absolute strain; instead, they reflect the change in the strain on the surface that the strain gage is installed. The strain gage on the third floor column C3-C4 above the removed column shows an increase in the tensile strain (i.e. reduction in the compressive strain) indicating the elongation of the column due to the loss of axial compressive force. Note that another strain gage was also installed on the 2nd floor column C2-C3 right above the removed column but data is not available due to malfunctioning of the strain gage.

As explained in 3.4.4.4, the beams in the lower floors had more contribution in the load redistribution than the beams in the upper floors because of the elongation of the columns above the removed column due to the loss of their axial compressive forces and the shortening of the neighboring columns due to the additional compressive axial forces.
as a result of load redistribution after column removal. The distinction in behavior of the beams bridging over the removed columns in different floors was discussed in detail in 3.4.4.4.

Shortening of the columns in the adjacent axes to the removed column can be seen from the vertical displacement histories of joints B4 and D4 on the roof. Figure 4.22 and 4.23 shows the vertical displacement histories of these joints. As can be seen from these figures, joints B4 and D4 on the roof moved downwards after removal of the center column in the first floor. Joint B4 had a peak vertical displacement of 0.007” and a permanent vertical displacement of about 0.0057”. Joint D4 had a peak vertical displacement of 0.0055 and permanent vertical displacement of about 0.004”.

The downward movement of these joints indicates the additional compressive strain (i.e. additional compressive force) in the columns on the B and D axes due to the redistribution of the loads after column removal. Note that Joint B4 and D4 move upwards at the very beginning after column removal, then they move downwards. The transient upward movement of these joints is due to the propagation of the deflection wave that starts with sudden downward movement of joint C4 while permanent downward movement of these joints is due to additional compressive strains in axes B and D columns.

The additional compressive strain in the first floor axes B and D columns (i.e. shortening of the first floor axes B and D columns) is also observed in the recordings of strain gages SG-3 and SG-4 on column B1-B2 and strain gages SG-5 and SG-6 on column D1-D2 in the first floor (see Figure 4.17). Figures 4.24 and 4.25 show change in the strains of these two columns. Note that the change in the strain in each column is calculated taking the average of two strain gages that were installed on these columns at the same height. As can be seen from figures 4.24 and 4.25, both columns had the same peak increase of 100 micro strain in compression and almost the same permanent increase of around 70 micro strain in compression.

Note that neighboring columns on both sides of the removed column experienced the same amount of increase in the compressive strain. In other words, the amount of load
transferred to columns on both sides of the removed column was equal. Therefore, as expected from a symmetrical structure, the load redistribution was symmetrical after column removal.

Figure 4.26 and 4.27 show the recordings of strain gages SG-2 and SG-7 that were installed on columns A1-A2 and E1-E2, respectively. Both strain gages show an increase in the tensile strain of these columns indicating the reduction in the compressive force of these exterior columns. Similar phenomena are observed in the evaluated buildings discussed in Chapter 3. That is, the columns in the distant axes beyond the neighboring columns of the removed one experienced loss of compressive force due to the new deformed shape of the structure after column removal. The compressive strain of column A1-A2 experienced a peak reduction of 30 micro strain while the permanent reduction was 22 micro strain. Column E1-E2 experienced a peak reduction of 38 micro strain in its compressive strain while the permanent change was 32 micro strain.

Figure 4.28 shows the recording of strain gage SG-30 (see figure 4.17) for the first 0.1 seconds after column removal. As can be seen from figure 4.28, the increase in the tensile strain (i.e. reduction in the compressive strain or loss of compressive force) in column C3-C4 above the removed column reaches a value that is close to the permanent value in only about 0.021 seconds. After some fluctuation it stabilizes around 0.000015 strain. Note that the strain history has peak just after the column removal. As it could be due to axial wave in the column after explosion or a temporary reading error in the strain gage, it needs further detailed investigation (in future experimental studies) to clarify such a temporary intense change in the column strain above the removed one.

Figure 4.29 and 4.30 show the change in the strain of the first floor columns B1-B2 and D1-D2 for the first 0.1 seconds after column removal which is the same range of time period used in the figure 4.28. The strain histories of these columns are calculated by taking the average of recordings from two strain gages that were attached to each of these columns (see Figure 4.17). Comparing Figures 4.29 and 4.30 with figure 4.28, it can be deduced that the load transferred to columns B1-B2 and D1-D2 by the time the reduction in the compressive strain (i.e. axial load) of column C3-C4 reached its
permanent value was negligible. This difference in the response can be explained by the fact that the reduction in the compressive strain of column C3-C4 in the second floor is due to axial wave propagation, while the increase in the compressive strains of columns B1-B2 and D1-D2 is due to flexural wave propagation that requires the structure to deform and redistribute the gravity loads.

The difference between the axial and flexural wave propagation can also be seen in the deformation of the structure at the time when the column above the removed column had lost its compressive strain. The vertical displacement history of joint C2 just above the removed column is plotted for the same range of time period in Figure 4.31. When the change in strain in the column above the removed one reaches to permanent value, the vertical displacement of joint C2 in the second floor was only 0.0022”, about 1% of the peak displacement of that joint. That is, joint C2 in the second floor practically starts to move downwards when the column in the second floor experiences permanent loss in its axial load. After that, it takes 0.072 seconds for joint C2 to reach its peak displacement, which is 35 times longer than the time for the column above the removed one to reach the permanent strain.

The same phenomenon that the columns above the removed column experiences loss in the axial load much faster than the deformation of the joint above the removed column is also observed in full scale buildings and was discussed in Chapter 3. The difference between the times that takes the structure to deform and the columns above the removed one to lose their axial force is because of the difference in the speed of axial and flexural wave propagation.

Figure 4.32 shows the permanent change in strain recorded by the strain gages from SG-8 to SG-29 (see figure 4.17) that were installed on the beams between axes B and D. Beams bridging over the removed column showed relatively symmetrical response following removal of the center column in the first floor. The only considerable difference observed was between strain gages SG-22 and SG-29 that were installed at the bottoms of the B2 end of beam B2-C2 and the D2 end of beam C2-D2 on the 2nd floor.
(see figure 4.16). The difference in the recordings of these strain gages could be a result of unsymmetrical crack pattern in the beams after column removal.

All strain recordings show that after removal of the center column in the first floor the beams in the vicinity of the removed column axis experienced positive bending moment. The other ends of the beams on the faces of the neighboring columns on axes B and D experienced negative bending moment. Before column removal these beams had a negative bending moment at the faces of the column on the C axis. However, after column removal, the direction of the bending moment of beams in the vicinity of the removed column changed from negative to positive. The same phenomenon is observed in the experimental and analytical evaluation of the full scale buildings subjected to column removal and discussed in the Chapter 3.

The positive bending at the Axis C ends and negative bending at the Axis B and Axis D ends of the beams between axes B and D indicates double curvature deformation of these beams. That is, the loads on these beams are transferred to the columns on axes B and D through the shear forces developed in the beams as a result of their double curvature bending. The performance of the flexural behavior of the beams on the redistribution of the loads to neighboring columns depends on capability of the beam ends to develop the required bending moment and in turn shear forces to transfer the loads. Especially, the ends of the beams on the axis of the removed column (in this case, Axis C) play a critical role since the bending moment at these locations changes from negative to positive. If the ends of the beams at these locations were not able to provide the required positive bending moments due to the lack of bottom reinforcement (or insufficient anchorage in the case of discontinuity in the reinforcement), then these sections would lose their flexural capacities and the load transfer mechanism through the flexural behavior of the beams would vanish. In that case, Catenary action could potentially develop and transfer the loads to the neighboring columns through the axial force of the beams. During the dynamic test, none of the beam sections lost their flexural strengths and the Catenary action did not develop. Note that the building whose exterior frame was tested is designed to satisfy the ACI integrity requirements and Special Moment Frame requirements.
The development of Catenary action is clearly observed in the static part of the experiment and discussed in the next section. After all critical locations on the beams lost their flexural strengths due to the rupture of the tensile reinforcement at those sections, the applied load at the top of the removed column is transferred through the axial force in the beams bridging over the removed columns.

The Catenary action of the beams in the static part of the experiment has participated considerably in the resisting force of the frame to the applied displacement especially as the deformation of the structure increased. However, in the dynamic part of the experiment, the deformation of the beams were not large enough for the Catenary action to develop and transfer the loads. Therefore, the performance of a potential Catenary action mechanism in the case of sudden removal of column where the dynamic effects are in place is not available to evaluate. Further experimental and analytical research is needed to assess the performance of the Catenary action in two and three dimensional frame structures that are subjected to removal of one or more load bearing elements.

Following the removal of column C1-C2 in the first floor, the beams between axes B and C and the beams between axes C and D merged and behaved as single beams that were supported at axes B and D having a double span length. As these beams with double span length deform, they impose negligible effect on the columns on axes C. That is only because the beams between axes B and D are symmetric along the axis of the removed column (Axis C).

If the beams that are bridging over the removed column were not symmetric along the axis of the removed column (i.e. they had different span lengths), then for a given displacement at the top of the removed column, the bending moment demand at the ends of the beams on each side of the axis of the removed column would be different. As the beams on both sides of the axis of removed column deform as a single beam after column removal, they would impose a rotation to the top and bottom ends of the columns above the removed column. These rotations would cause the columns to deform in double curvature and bending moments would develop at the top and bottom ends of the
columns. The difference in bending moments of the beams on each side of the axis of the removed column would be equal to the moments of the column ends that are connected to the joint, satisfying the moment equilibrium at the joint. An analytical model of a representative frame is developed to demonstrate this phenomenon. Figure 4.33 (a) shows a two dimensional frame structure similar to tested frame but having a span length on one side of the axis of the removed column being twice the span length of other bays. The exaggerated deformed shape of the frame and the bending moment diagram of the elements after column removal is shown in Figure 4.33(b) and Figure 4.33(c), respectively. Both deformed shape and moment diagram of the frame show the double curvature bending of the columns above the removed column.

The mechanism by which the beams bridging over the removed column and the columns that are connected to these beams on the top of the removed column (in case of unsymmetrical configuration) deform in double curvature and transfer the loads to the neighboring columns is called Vierendeel action. Unlike the Catenary action, Vierendeel action can develop and transfer the loads to the neighboring columns even if the removed column is a corner column and the beams transferring the loads are not continuous beyond the removed column. The load redistribution mechanism of the Hotel San Diego building following the removal of a corner and penultimate column is a good example of the development of Vierendeel action in such a case (See Section 3.2). In the case of the two dimensional frame tested, there was no considerable demand on the columns above the removed column due to symmetric response of the beams on the both sides of the axis of the removed column.

**Beam Growth**

Following the removal of column C1-C2 in the first floor, the beams between axes B and D elongated. The elongations of the beams are captured with the potentiometers that were horizontally connected to the joints on axes B and D. After all vibrations are damped out, the elongation of both beams B2-D2 in the second floor and B3-D3 in the third floor are measured at 0.027” while that of the beam B4-D4 in the roof was 0.032”.
The beam growth is the result of the post cracking deformations in the beam sections along the length of the beam. As explained in detail in Chapter 5, in a RC section under bending, the neutral axis passes at the geometrical center of the section before concrete cracks. After cracking the neutral axis moves towards the compressive side causing tensile strains at the center of the section. The integration of the tensile strains at the center of beam sections over the length of the beam results in an elongation of the beam (beam growth).

If the beam is free to move horizontally at its one or both ends, then the length of the beam would increase without causing any internal force in the beam. However, if the ends of the beam are fixed (or partially fixed) in the horizontal direction, then restrained axial deformation causes reaction forces at the ends of the beam and in turn causes axial compressive force along the beam. The amount of reaction force would be maximized if the ends of the beam were fully fixed where there would not be any movement at beam ends. If the ends are partially fixed, then the displacement of the ends of the beam (i.e. elongation of the beam) and the axial compressive force that would develop in the beam will depend on the extent of fixity at the beam ends.

In this context, the elements in a frame structure can be considered as partially fixed at their ends and the amount of fixity comes from the stiffness of the other elements in the corresponding degree of freedom. The reason for beam B4-D4 on the roof having more elongation than the second and third floor beams B2-D2 and B3-D3 can be explained with the fact that the lateral restrain at the ends of beam B4-D4 is less than the lateral restrain at the ends of the beams B2-D2 and B3-D3. The lateral restrain at the end of the beam B4-D4 comes from the third floor columns below the roof level only while the lateral restrain for the beams in the second and third floor is provided by the columns above and below the corresponding floor level.

Strain gages SG-12, SG-18 and SG-27 are located at the bottom of the sections which can be considered close to the inflection point (i.e. moment demand is expected to be close to zero) (See Figure 4.17). Given the approximately same deflection of joints C2, C3 and C4 and considering the small deformations in the beams between axes B and
D, the less compressive strain measured at the roof beam with SG-12 than those at the second and third floor beams measured with SG-18 and SG-27 (See Figure 4.32), respectively, can be explained with the same reason stated above. That is, the axial compressive force developed in the second and third floor beams due to beam growth was larger than that in the roof beam since the amount of restrain at the ends of the second and third floor beams were more than the amount of restrain at the ends of the roof beam.

The readings of the potentiometers that are connected to the joints on Axes A, B, D and E to measure the horizontal displacements showed that the joints on Axes A and B had almost the same horizontal displacement at each floor level, which is also valid for the joints on axes D and E at each floor level. That is, the columns and beams between axes A and B and also the columns and beams between axes D and E act together to resist the elongation of the beams between axes B and D.

4.2.3.2. Static Test

In the second phase of the experiment, monotonically increased downward displacement was applied to the joint at the top of the removed column after all weights used in the first phase of the test were removed. The resisting force of the structure to applied displacement at joint C2 (the top joint of the removed column) is measured with a load cell. Figure 4.34 shows the resisting force (i.e. applied force) versus applied displacement at joint C2.

4.2.3.2.1. Force-Deformation relationship

The force deformation curve of the frame (See Figure 4.34) can be divided into four regions that are defined below. In the first region, the resisting force increases very sharply as the vertical displacement at joint C2 increases up to 0.36” after 0.36” vertical displacement, the slope of the force deformation curve decreases. The resisting force reaches its first peak of 1822 lb at 1.06” vertical displacement. In the second region, first the resisting force gradually reduces from 1822 to 1753 lb as the vertical displacement at joint C2 increases from 1.25” to 1.94”. Then, it again increases to 1980 lb at 4.7”. The
third region starts with a sudden reduction of the resisting force at 4.86” vertical displacement, which is the result of the first bar rupture occurring at the bar cut-off location of beam C3-D3 in the third floor close to axis D. Figure 4.51 shows the state of the section which first bar rupture occurred. The consequent bar ruptures occurred at different sections of the beams between axes B and D up to 9” of vertical displacement, causing a significant drop in the resisting force. The resisting force measured at 9” vertical displacement is around 1325 lb. In the fourth and the last region of the force deformation curve, the resisting force again starts to increase as a result of enhanced contribution of Catenary action of the beams between axes B and D although additional bar ruptures cause drops in the force. The resisting force reaches 2118 lb at 16.25” of vertical displacement which is the maximum displacement applied during the second phase (static part) of the test. The deformed shapes of the frame at various displacements during the experiment are shown in figure 4.35 to figure 4.49.

4.2.3.2.2. Load Transfer Mechanisms

**Vierendeel Action**

As in the dynamic phase of the test, the load transferring mechanism of the frame in early stages of the static phase was through the shear forces developed in the beams as a result of their double curvature bending. The decrease in the slope of the force deformation curve after 0.36” vertical displacement can be related to yielding of the rebars at the maximum demand locations.

The first concrete crushing occurred at the bottom of the bar cut-off location of beam C3-D3 at a vertical displacement of 1.1”. The concrete crushing in other critical sections of the beams between axes B and D where the demand/capacity ratios were maximum continued up to 3” vertical displacement. The order of the sections that concrete crushed and the corresponding vertical displacements are shown in Figure 4.50. Due to the crushing of concrete, the critical sections experienced a loss in their flexural strengths which caused a reduction in the resisting force of the frame to the applied displacement. However, crushing of the concrete did not cause the sections lose all of their flexural strength and in turn the resisting force capacity of the frame through the
flexural behavior of the beams between axes B and D did not drop significantly. The resisting force at 3” vertical displacement, when the crushing of concrete occurred at all critical sections, was around 1753 lb which is only 69 lb less than the peak resisting force of 1822 lb.

As in the dynamic phase, the resisting mechanism observed in the static phase was Vierendeel action. Vierendeel action can be characterized as the double curvature deformation of the beams bridging over the removed column, and redistribution of the loads to the neighboring columns through the shear forces develop in the bridging beams. Depending on the location of the removed column and the structural configuration, the beams may interact with the columns above the removed column. In the case of the frame tested, because of the symmetry of the structure there was no considerable demand on columns due to the interaction with bridging beams between axes B and D.

The first bar rupture occurred at 4.86” vertical displacement resulted in a drop of around 300 lb in the resisting force (See Figure 4.34). Figure 4.42 shows the deformed shape of the frame at this level of displacement and the location of the first bar rupture. Bar ruptured occurred at top rebar cut-off location. After bar cut-off two rebars at the top of section were continuous (See Section c-c in Figure 4.13). Given the fact that an open crack formed and the concrete was crushed at the section before the bar rupture (See Figure 4.40 for the state of the section at 3” vertical displacement), almost all flexural strength of the section was provided by bottom and top rebars. By losing one of two top rebars, the moment capacity of the section and in turn demand dropped almost in half. As a result, the shear force transferred through beam C3-D3 dropped. Since the resisting force is the sum of the shear forces transferred through the flexural behavior of the beams and vertical component of the axial forces in the beams, the drop in the shear force transferred by beam C3-D3 caused a drop in the resisting force of the frame.

As the vertical displacement increased beyond 4.86”, other sections which reached the maximum demand/capacity ratio experienced rebar ruptures. In the force deformation curve of the frame (See Figure 4.34), starting from the first bar rupture at 4.86” of vertical displacement up to around 9” of vertical displacement, each bar rupture
occurring in the beams between axes B and D caused a drop in the resisting force which can be clearly seen in Figure 4.34. Note that the resisting force shows an increases following each drop related to bar rupture. Given the fact that the concrete at all critical beam sections (the sections which have the maximum demand/capacity ratio) had crushed before the first rupture and the bar ruptures reduce the amount of shear force through the flexural behavior of beams, the increase in the resisting force indicates another resisting mechanism developed in the frame, which is called Catenary action.

**Catenary Action**

Catenary action can be characterized as redistribution of the loads to the neighboring columns through the axial tensile forces developed in the bridging beams. The contribution of Catenary action in the load redistribution increases as the slope of the bridging beams increases (i.e. the displacement of the joints above the removed column increases) since the vertical component of the axial force in the beam would be larger as the slope increase.

While the vertical component of the axial tensile force in the beams between axes B and D contributes to the resisting force of the frame against the applied vertical displacement, the horizontal component of the axial force applies lateral forces to the joints on axes B and D that pull these joints towards axis C. Figures 4.52, 4.53 and 4.54 show the horizontal displacements of the joints on axes B and D at the second, third floor and roof level up to 6” of vertical displacement, respectively. Note that the joints on axes B and D moved outwards from the beginning of the static test up to 1.5” of vertical displacement. After almost 1.5” of vertical displacement, the joints in all floor levels stopped moving outwards and started to move inwards. The change in the direction of the displacement was the result of the balance of the tensile force in the beam due to Catenary action and the compressive force in the beam due to beam growth. After 1.5” of vertical displacement the axial force in the beam turns from compression (due to beam growth) to tension (due to Catenary action). As result of tensile force in the beams between axes B and D the ends on the axes B and D moved inwards.
Figure 4.40 to 4.42 show that the joints on axes B and D in all floor levels moved outwards up to 1.5” of vertical displacement which indicates the elongation of the beams between axes B and D (Beam growth). As observed in the behavior of the beams after column removal in the dynamic phase of the test, the beam growth is due to post cracking deformations in the beam sections along the length of the beam. Note that the inward movement of the joints on axes B and D after 1.5” of vertical displacement (See Figure 4.52 to 4.54) does not mean the shrinkage of the beams between axes B and D since the deformations in the beam sections along the length of the beam did not drop.

Another indication of the Catenary action is the cracks formed in the beams. Figure 4.55 shows the axial cracks in beam B3-C3 at 9” of vertical displacement.

The performance of the Catenary action depends on essentially two factors: the axial tensile force capacity of the beams bridging over the removed column and the capacity of the rest of the structure to resist the tensile forces formed at the beam ends. In the case of the two dimensional frame tested, the resisting capacity of the frame to the tensile forces at the ends of the beams was provided by the lateral stiffness of the frame parts between axes A and B and Axes D and E. Since the horizontal displacement of the joints on axes B and D in all three floor levels were close, the relative horizontal displacement (story drift) was maximum between the foundation level and second floor level. Therefore the demand in the first floor columns and the second floor beams were higher than the elements in upper levels. The force and deformation capacity of these elements (first floor columns and second floor beams) was controlling the resisting capacity of the frame to the tensile forces at the ends of the beams. The deformed shape of the frame at 16.25” of vertical displacement (end of the static test) (See Figure 4.49) shows that the lateral deformation of the frame parts between axes A and B and Axes D and E was the result of the end rotations of the first floor columns. Figure 4.56 shows the deformation at the top and bottom of the first floor column B1-B2 at the same vertical displacement (16.25”). It can be deduced that the stiffness, strength and ductility of the first floor columns had a significant effect on the lateral stiffness of the adjacent frames that the beams bridging over the removed column were connected and consequently on the performance of the Catenary action.
Note that in the case of a three-dimensional structure with slab, the in-plane stiffness of the slab would provide considerable restraint to the bridging beams when the tensile forces develop due to Catenary action as well as compressive forces developed due to beam growth. The diaphragm effect of the slab on the beams bridging over the removed column is discussed in Chapter 5.

As can be seen from Figure 4.49, the inwards displacement of the joints at the B and D ends of the bridging beams between axes B and D in the roof level was less than those in the second and third floor levels. That is because of the fact that the beams bridging over the removed column in roof level had a different deformation pattern than those in the second and third floor levels. For all three floor levels, most of the end displacements of the beams at their ends on the axis C were practically coming from the plastic rotations at the failed sections along the length of the beams. The failed sections were located at the ends of the beam on the roof level. In the second and third floor, different from the roof level, the failed sections were located at axis C ends and bar cut-off locations close to Axis B and D. Given the fact that all floors had almost same displacement at the joints on axis C, the shorter length between the failed sections (hinges) of the second and third floor beams resulted in larger horizontal inwards displacement at the bar-cut-off locations.

4.2.3.2.3. Performance of Joint on Roof in Catenary action

Figure 4.57 shows the state of the joint C4 in the roof level at 8.5” of vertical displacement. As can be seen from the figure, the bottom reinforcement of the beams on both sides of the joint are ruptured at this level of displacement. As explained above, after beams B4-C4 and C4-D4 lost their flexural strength at their ends, the only load transfer mechanism was the Catenary action, which is through the axial force in the beams. The load in column C3-C4 below the roof joint C4 would be transferred to the beams B4-C4 and C4-D4 passing through only the top reinforcement of the beams. If the transverse reinforcement of the beam was not continued in the joint and the ends of the column longitudinal rebars were not bent, then the top reinforcement could tear out of the concrete cover and disconnect from the joint. In such a case, the contribution of the
bridging beams on the roof level would vanish resulting additional demands in the other story beams. Note that during the experiment the top reinforcement of the beams in the joint stayed in place without tearing out of the concrete cover and transferred the loads.

### 4.3 Analytical Evaluation

The analytical evaluation of the two dimensional frame structure described in Section 4.1 is carried out utilizing two analytical models that are developed using SAP2000 (2005) and Perform-3D (2006) finite element analysis software packages. As in the experimental program, the behavior of the frame is evaluated for two loading cases; (a) dynamic behavior of the frame following column removal and (b) the static behavior of the frame to the monotonically applied downward displacement at the top of removed column.

#### 4.3.1 Analytical Evaluation of Dynamic Test using SAP2000

Figure 4.58 shows the analytical model of the frame developed in SAP2000. Note that the removed column is excluded in the model. The beams and columns are modeled using two-node Bernoulli beam elements. The rigid zones are assigned to the end of the elements at beam-column intersections. The lengths of the rigid zones are calculated based on the dimensions of the adjacent beams and columns at the joints. The material nonlinearity in the frame is imposed through the localized plastic hinges that are assigned to the critical locations. Geometric nonlinearity is also included in the analyses accounting for P-Delta effects and large displacements.

The plastic hinges are assigned to the critical locations where the yielding is expected such as ends of the beams and columns and bar cut-off locations. The plastic hinges used in the analytical model are flexural hinges. The characteristics of the plastic hinges (moment-rotation relationships) are calculated based on the moment–curvature analysis of each critical section and assumed plastic hinge length. The moment curvature analyses of the beam sections are performed assuming the axial load in the section is zero. The Mander model (Mander et al., 1988) for unconfined concrete and confined
concrete and stress–strain model with parabolic strain hardening for steel are implemented in moment–curvature analyses.

The stress-strain curves for the concrete and steel to define material properties in the analytical model are obtained from the material tests conducted using at least two concrete and steel samples. Figures 4.6 and 4.7 show the stress-strain curves of the concrete used in the construction of the frame. Two different sizes of reinforcement having a diameter of 0.11-inch and 0.135-inch are used in the construction and had different material characteristics. Figures 4.8 and 4.9 show the stress-strain curves of these reinforcements.

The bottom joints of the first floor columns are assumed to be fixed supports. The frame is analyzed considering only the degree of freedoms in the plane of the frame (Two translation and one rotation degrees of freedom). Note that the frame was restrained to prevent out of plane deformations in the experimental program.

The mass of the structure includes both self mass of the frame and the mass associated with the point loads acting on beams and center columns. Rayleigh damping is used to model damping in the structure. The mass proportional ($\alpha$) and stiffness proportional ($\beta$) coefficients are calculated based on the periods of first two governing modes of vibration of the frame ($T_1=0.099$ sec and $T_2=0.036$ sec) and assuming 5% damping ratio associated with these periods.

Since element removal during an analysis is not readily available in SAP 2000, the following procedure is used to obtain the response of the structure following removal of the first floor center column:

i) The frame is analyzed under gravity loads (self-weight and the point loads applied to the beams and center columns that represent the loads transferred from the joists perpendicular to the frame). Note that the model used for this analysis includes the removed column. The internal forces at the top end of removed column are obtained.

ii) A new model is developed by deleting removed column from the previous model. Again the structure is analyzed under gravity loads. Along with the gravity loads,
the internal forces obtained for the removed column in the previous step are applied to the top of the removed column as external loads simulating the effect of the removed column that does not exist in the model. Note that the results of this model and the model analyzed in (i) are identical.

iii) After gravity loads and end forces of removed columns are applied to the structure as explained in (ii), the external forces that have same magnitude but in opposite direction of the end forces applied along with the dead loads are applied to the structure in one millisecond and a nonlinear time history analysis is run. By applying the external forces that have same magnitudes but opposite directions with the end forces that are applied along with the dead loads, the effect of the column that is simulated by applying their end forces found in (i) is simply cancelled out.

The results of the dynamic analysis show that when the first floor center column C1-C2 is removed, the loads carried by the removed column are redistributed to the neighboring elements through the beams between axes B and D (bridging over the removed column) without a partial or full collapse. The maximum deformation in the structure is limited to 0.505” at the top of the removed column. The structure stabilizes having a permanent deformation of 0.492” at the same joint. Both ends of all beams between axes B and D in all floors experienced yielding while other elements remained elastic. The plastic deformations in the yielded sections were far from being close to the deformation limits of the corresponding sections. Figure 4.59 shows the deformed shape of the frame at the end of the analysis when all the vibrations were damped out with a scale factor of 1. The yielded sections at the ends of the beams are also shown in Figure 4.59 with thick green dots.

Figure 4.60 shows the analytically obtained vertical displacement of Joint C2 after removal of the first floor center column. The figure also shows the vertical displacement of the same joint recorded experimentally. Analytical vertical displacement of Joint C2 shows a peak displacement of 0.505”, 0.134 seconds after column removal while the peak vertical displacement of the same joint in the experimental program was 0.425” at 0.098 seconds. The analytical vertical displacement of joint C2 stabilizes at about 0.492” of
permanent vertical displacement at around 1 second after removal of column while the experimental permanent vertical displacement is around 0.395”. As one can see, while the displacement obtained from the analytical model damped out considerably at 1 second, it takes more time for the experimental displacement to stabilize. It indicates the damping of the frame is less than the damping used in the analytical model which was 5% of the critical damping.

Figure 4.61 shows the analytical vertical displacement of Joint C4 on the roof after removal of the first floor center column along with the experimental vertical displacement of the same joint. The analytical vertical displacement of Joint C4 shows a peak displacement of 0.503” at 0.134 seconds after column removal while the peak vertical displacement of the same joint in the experimental program was 0.420” at 0.098 seconds. The joint C4 in the roof stabilizes at about 0.490” permanent vertical displacement at around 1 second after removal of column while the experimental permanent vertical displacement is around 0.390”. At that time it has not been damped out yet again indicating less damping in the frame than that used in the analytical model (5% of the critical damping).

As observed in the experimental data, the analytical results obtained from the dynamic analysis performed in SAP2000 also shows that the 2nd floor joint C2 had peak and permanent displacements more than those of the roof joint C4. As explained previously in the 4.2.3.1, that is because of the elongation of the columns above the removed column due to their loss of axial compressive force following column removal. Figure 4.62 and 4.63 show the axial force histories at the bottom ends of columns C2-C3 in the second floor and C3-C4 in the third floor. The axial force in the 2nd floor column C2-C3 drops to 0.27 kips from 1.25 kips while the force in the 3rd floor column C3-C4 drops to 0.27 kips from 0.62 kips.

Note that the external loads representing the loads that were supposed to be transferred through the beams in the perpendicular direction to the joints in axis-C are applied to the frame along the length of axis C columns (See Figure 4.11) in the experimental program. The same load pattern is used in the analytical model of the frame.
Figure 4.64 shows the axial force diagram of the axis C columns above the removed column before column removal and after column removal (after all vibrations are damped out). As can be seen from the figure, while the axial force at the top ends of the columns is almost zero after column removal, both columns in the second and third floor have an axial force at their bottom ends because of the loads that are directly applied to the nodes along the length of the columns. It can be deduced from the axial force diagram of the columns after column removal that the load on the 2nd floor column is transferred to the neighboring columns through 2nd floor beams and the load on the 3rd floor column is transferred to the neighboring columns through 3rd floor beams only. Almost zero axial force on the top of the third floor column C3-C4 indicates that there was no load transfer between the beams on the roof and the column C3-C4. In other words, the loads acting to the beams BC and CD on the roof are transferred to the neighboring columns through only the beams BC and CD on the roof independently. Again, almost zero axial force on the top of the second floor column C2C3 indicates that there is not any load transfer between the third floor beams BC and CD and second floor column C2C3. Therefore, the loads acting to the 3rd floor beams BC and CD and 3rd floor column C3C4 are transferred to the neighboring columns through only the beams BC and CD on the 3rd floor. Similarly, the loads acting to the 2nd floor beams BC and CD and 2nd floor column C2C3 are transferred to the neighboring columns through only the beams BC and CD on the 2nd floor. In summary, each floor transferred the loads on itself without any load transfer between adjacent floors. That can be explained by the fact that the beams in all three floors had the same stiffness. If the beams in one of three floors had more stiffness than the others then the beams in that floor would have participated in carrying the loads of the other floors as well. This fact is observed in the evaluation of the Baptist Memorial Hospital in Memphis (see Section 3.4), where the 5th floor beams was deeper than the typical beams in other floors. As explained in Section 3.4, the beams in the 5th floor provided support for the upper floor columns and carried the loads coming from the upper floors through the axial force in those columns.

Figures 4.65 and 4.66 show the axial force histories of the 2nd floor column C2C3 and 3rd floor column C3C4 (at their top ends) for the first 0.1 seconds after column removal. As can be seen from the figures, the reduction in the compressive force in the
2nd floor column C2-C3 (just above the removed column) takes only about 0.0025 seconds and it takes about 0.006 seconds for the 3rd floor column C3C4 to reach a value close to the permanent change in the axial force. Axial force in both floors shows some variations after the large reduction.

Figures 4.67 and 4.68 show the axial force histories of the 1st floor columns B1B2 and D1D2 for the first 0.1 seconds after column removal which is the same range of time used in figures 4.65 to 4.66. Comparing figures 4.65 and 4.66 with figures 4.67 and 4.68, it can be inferred that there was no load transferred to columns B1B2 and D1D2 by the time when the reduction in the axial force of column C2C3 reached a value around its permanent value, 0.0025 sec after column removal. This difference in the response can be explained by the fact that the reduction in the axial compressive force in the 2nd floor column C2C3 is due to axial wave propagation, while the increase in the axial compressive force in the 1st floor columns B1B2 and D1D2 is due to flexural wave propagation that requires the structure to deform and redistribute the gravity loads.

The difference between the axial and flexural wave propagation can also be seen in the deformation of the structure at the time the column above the removed column had lost its compressive axial force. The analytical vertical displacement history of joint C2 just above the removed column is plotted for the same range of time (for the first 0.1 seconds after column removal) in Figure 4.69. At the time that the change in the axial compressive force in the 2nd floor column C2-C3 above the removed column reaches a value around its permanent value, the vertical displacement of joint C2 in the second floor was only 0.0042”, only about 0.8 % of the peak displacement of that joint. That is, joint C2 in the second floor practically starts to move downwards when the column in the second floor experiences permanent loss in its axial load. After that, it takes 0.132 seconds for joint C2 to reach its peak displacement which is 52 times longer than the time for the column above the removed one to reach its permanent axial force.

The same phenomenon that the columns above the removed column experience loss in their axial force much faster than the vertical deformations above the removed
column is also observed in experimental evaluation of the frame (through the strain-gage recordings on the columns) and was discussed in 4.2.3.1.

Note that the axial compressive force in columns B1-B2 and D1-D2 show reduction at the very beginning (See Figure 4.67 and 4.68). The transient decrease in the axial compressive force in these columns is due to upward movement of these joints that is the result of the propagation of first deflection wave that starts with sudden downward movement of joint C4 while permanent increase in the axial compressive force is due to additional gravity loads transferred to these columns through the beams bridging over the removed column. This transient effect of the first deflection wave is also observed in the vertical displacements of the joints on axes B and D which caused these joints to move upwards at the very beginning. Permanent vertical displacements of the joint were downwards due to the shortening of the axes B and D columns as a result of additional compressive strain.

Figure 4.70 shows the bending moment diagram of the critical beams before and after column removal (at the time that vibrations in the frame are damped out). As it can be seen in the figure, after removal of the center column in the first floor, the beam ends in the vicinity of the columns above the removed column (axis C ends of the beams) experienced positive bending moment while the bending moment demand at these ends of the beams between axes B and D were negative prior the removal of the column. The other ends of the beams on the faces of the neighboring columns on the axes B and D experienced additional negative bending moment after column removal. The change in the direction of the bending moment of the beams at their ends along the axes of the removed column and the increase in the negative bending at their other ends are also observed in the experimental evaluation of the frame (through the strain-gage recordings on the beam ends) and discussed in 4.2.3.1.

The positive bending at the Axis C ends and negative bending at the Axis B and D ends of the beams between axes B and D indicates double curvature deformation of these beams. That is, the loads on these beams are transferred to the columns on axes B and D through the shear forces developed in the beams as a result of their double curvature
bending. Figure 4.71 shows the exaggerated deformed shape of the frame which the double curvature of the beams can be clearly seen.

As mentioned in the experimental evaluation of the frame tested, the mechanism through which the beams bridging over the removed column deform in double curvature and transfer the loads to the neighboring columns is called Vierendeel action. Following the removal of column C1-C2 in the first floor, the beams between axes B and C and the beams between axes C and D merged and behaved as single beams that were supported at axes B and D having a double span length. As these beams with double span lengths deform, they imposed negligible effect on the columns on axes C since the beams between axes B and D are symmetric along the axis of the removed column (Axis C). Therefore, the contribution of the columns in the Vierendeel action is canceled out.

Figure 4.72 shows the horizontal displacement histories of the joints on axes B and D (the ends of beams BD) in all three floors. Positive direction for the horizontal displacements in the frame is also shown in the figure. As can be deduced from the figure, the joints on both axes B and D moved inwards immediately after column removal.

For such a frame that is evaluated in this chapter, the horizontal displacements of the joints at the ends of the beams, which are bridging over the removed column, are determined by (a) beam growth and (b) geometric compatibility. As explained before, beam growth is the result of the post cracking deformations in the beam sections along the length of the beam. In a RC section under bending, the neutral axis passes at the geometrical center of the section before concrete cracks. After cracking the neutral axis moves towards the compressive side causing tensile strains at the center of the section. The accumulation of the tensile strains at center of beam sections along the length of the beam results in an elongation of the beam (beam growth). As a result of the beam growth, the ends of the beams tend to move outwards. However, the outward movement of the ends of the beam depends on the restrain at the ends. If the ends are partially restrained as in this frame, the ends move outwards but the compressive axial force would also develop in the beams due to its restrained elongation.
As the ends of the beams along the axis of the removed column moves downwards, the axial stiffness of the beams causes the other ends to pull inward due to geometric compatibility. Figure 4.73 illustrates the geometric compatibility of a representative beam where one end is subjected to a vertical displacement. Figure 4.74 shows the relationship between the horizontal inward movement of one end of a 28.5’ long beam (typical beam length in the frame) and the vertical displacement at the other end that is monotonically increased. Note that the relationship between horizontal movement of node-J due to the geometric compatibility and vertical displacement of node-I is not linear. It is rather 2\textsuperscript{nd} order polynomial suggesting the effect of the geometric compatibility would be significant for larger displacements. In the calculation of the horizontal displacements shown in Figure 4.74, the representative beam is assumed to have infinite axial stiffness. In reality, the horizontal displacement of the other end would be less due to the axial tensile deformation of the beam.

The resultant horizontal displacement of the ends of the beams that are bridging over the removed column would be the summation of these two effects mentioned above. For small vertical displacements, the contribution of the geometric compatibility would be insignificant compared to the effect of the beam growth. However, in large displacements, the effect of the geometric compatibility would overcome the effect of the beam growth. Therefore, for such a frame, which is evaluated in this chapter, and for small deformations, the ends of the beam are expected to move outwards as a governing effect of the beam growth. As the vertical displacement above the removed column increases, the effect of the geometric compatibility would be equal to the effect of the beam growth first (causing a stop in the horizontal displacements of the beam ends), and then would overcome the effect of the beam growth (causing the beam ends moving inwards).

As mentioned above and can be seen in Figure 4.72, the horizontal displacement histories of the joints on axes B and D (the ends of the beams BD) in all floors shows inward movement starting at the beginning of the dynamic analysis after column removal. The reason for such behavior is that the beam growth is not captured in the analysis. The beams are modeled with two node frame elements with localized plastic
hinges that accounts for the plastic deformation only in rotational degree of freedom and the strain variation over the sections is not tracked. Therefore beam growth is not captured during the analysis. The axial force due to the beam growth and the restraints at the ends of the beams did not develop. Furthermore, the effect of the axial load on the flexural behavior of the beam sections has not been accounted for. Beam growth and its effect on the flexural behavior of the beams through the axial force developed in the beams can be included in the analysis if the strain over the section is tracked. Using fiber section allows tracking of the strain variation over the beam sections and therefore includes the effect of the beam growth in the analytical evaluation of the structure. In the analytical evaluation of the frame using Perform-3D, the elements are modeled using fiber sections. The results of this analysis are presented in the following sections.

4.3.2 Analytical Evaluation of Dynamic Test using Perform-3D

Figure 4.75 shows the analytical model of the frame developed in Perform-3D with first floor column on the center axis removed. Different than the analytical model developed in SAP2000, the beams and columns are modeled using Bernoulli beam elements that have fiber sections. The rigid zones are assigned to the end of the elements at beam-column intersections. The lengths of the rigid zones are calculated based on the dimensions of the adjacent beams and columns. P-Delta effect is also included in the analyses.

The nonlinearity in the frame is imposed through nonlinear material behavior that is assigned to the concrete and steel fibers that form sections. The material characteristics for the concrete and steel are obtained by linearizing the stress-strain curves from the material tests conducted on concrete and steel samples (see figures 4.6 to 4.9). The elements are defined having fiber sections along entire length of the elements. The cracking of the concrete in the fiber sections is automatically accounted for since the concrete fibers are defined with a tensile strength. Another advantage of using fiber sections in the elements is that the axial force bending moment interaction is automatically accounted for. As it is shown later, the axial force bending moment
interaction becomes noticeable when the axial compressive force develops in the beams as a result of beam growth.

Other modeling parameters such as end conditions of the elements, mass pattern of the structure, the restrain to prevent out of plane deformations, and the damping of the structure are defined the same as in the analytical model developed in SAP2000 (See Section 4.3.1)

Similar to SAP2000, element removal during an analysis is not readily available in Perform-3D. Therefore, the same procedure is used to simulate the removal of first floor center column as used in SAP2000 (See Section 4.3.1).

The results of the dynamic analysis conducted in Perform-3D show that when the first floor center column C1-C2 is removed, the loads carried by the removed column are redistributed to the neighboring elements without a partial or full collapse. The maximum deformation in the structure is limited to 0.395” at the top of the removed column. The structure stabilizes having a permanent deformation of 0.359” at the same joint. Figure 4.76 shows the deformed shape of the frame at the end of the analysis when all the vibrations were damped out with a scale factor of 1.

Both ends of all beams between axes B and D in all floors experienced yielding. For example, Figure 4.77 shows the moment curvature history of the section at the axis B end of roof beam B4-C4 after column removal. The same plot includes the moment-curvature relationship of the section calculated independently without considering any axial load on the section. As can be seen from the plots, the plastic deformation in the yielded section was far from the deformation limits of the corresponding sections. Note that the bending moment demand of the section at yielding during the dynamic analysis is more than the yielding moment of the section calculated by the moment curvature analysis of the section. The reason for the difference is the effect of the axial compressive force develops in the beam during the dynamic analysis. Figure 4.78 shows the axial force at the section versus the curvature of the section. The axial compressive force developed in the beam during the dynamic analysis is a result of the elongation of the beam (beam growth) which is due to post cracking deformations in the section and the
horizontal displacement restrain provided by the adjacent elements to the elongation of
the beam. Even though the axial force developed in the beam is only 2% of the axial load
capacity of the section, it increases the yielding moment of the section by almost 10%.
Note that this phenomenon was not observed in the results of the dynamic analysis
performed in SAP2000 since the beam growth cannot be captured using two node
elements with flexure only plastic hinges.

Figure 4.79 shows the analytically obtained vertical displacements of Joint C2
after removal of the first floor center column obtained from the two models developed in
SAP2000 and Perform-3D. The figure also shows the vertical displacement of same joint
recorded experimentally. Analytical vertical displacement of Joint C2 obtained from the
Perform-3D model shows a peak displacement of 0.395” at 0.096 seconds after column
removal while the peak vertical displacement of the same joint in the experimental
program was 0.425” at 0.098 seconds. Based on the Perform-3D results, the joint C2
stabilizes at about 0.359” of permanent vertical displacement at around 1 second after
removal of column while the experimental permanent vertical displacement is around
0.395”.

Similar to figure 4.79, figure 4.80 shows the analytical vertical displacement of
Joint C4 in the roof after removal of the first floor center column. The results obtained
from both SAP2000 and Perform-3D models along with the experimental vertical
displacement of the same joint are presented in the same figure. Based on the Perform-3D
results, the analytical vertical displacement of Joint C4 shows a peak displacement of
0.394” at 0.095 seconds after column removal while the peak vertical displacement of the
same joint in the experimental program was 0.420” at 0.092 seconds. Joint C4 in the roof
stabilizes at about 0.357” of permanent vertical displacement around 1 second after
removal of column while the experimental permanent vertical displacement is around
0.390” at that time.

As one can see from figure 4.79 and 4.80, while the displacement response
obtained from the Perform-3D model damped out considerably at around 1 second, it
takes more time for the experimental displacement to stabilize. As also observed in the
SAP2000 results, it indicates the damping of the frame is less than the damping used in the analytical model developed in Perform-3D, which was 5% of the critical damping.

Figures 4.79 and 4.80 show that the vertical displacement demands above the removed column obtained from the SAP2000 analysis using 2-node beam elements with localized plasticity (accounts for the nonlinear behavior only in rotational degree of freedom) to model the beams and columns are more than the vertical displacement demands obtained from the Perform-3D analysis that uses fiber sections to model the beams and columns. Having all other parameters that are used in the modeling phase (dimensions, material properties, loading pattern, mass pattern, rigid zones, damping, end conditions, geometric nonlinearity, etc.) the same for both models developed in SAP2000 and Perform-3D, the only difference in the displacement response of two models can be related to the distinction between the performances of using 2-node beam elements with localized plasticity (flexural) and using elements with fiber sections. As shown above, being able to capture the beam growth by using the fiber section, and consequently capturing the axial force related to beam growth and having axial force bending moment interaction at the sections during the analysis caused the sections in the model which used fiber sections (Perform-3D) to be stronger than the other model (SAP2000). Thus, the peak and permanent displacements of the model that used fiber sections (Perform-3D model) were smaller than those of the model which used 2-node beam elements with localized plasticity (flexural) (SAP2000 model).

Figure 4.81 shows the horizontal displacement histories of the joints on axes B and D (the ends of the beams BD) in all three floors. Positive direction for the horizontal displacements in the frame is also shown in the figure. As can be seen from the figure, the joints on both axes B and D move outwards after column removal.

As stated in section 4.3.1.1 for the frame that is evaluated in this chapter, the horizontal displacements of the joints at their axis B and axis D ends of the beams which are bridging over the removed column are determined by the elongation of the beam, (beam growth) which is a result of the post-cracking deformations in the beam sections along the length of the beam, and the geometric compatibility of the frame. While the
beam growth causes the ends of the beams which are bridging over the removed column to move outwards, the geometric compatibility causes the ends of the beams to move inwards due to the axial stiffness of the beams as the other ends of the beams along the axes of the removed column moves downwards (See the illustration shown in Figure 4.73 for the effect of geometric compatibility).

As can be seen from Figure 4.81, the joints on both axes B and D move outwards after column removal, indicating that the elongation of the beams (beam growth) was governing over the effect of the geometric compatibility. For the 0.359” of permanent vertical displacement at top of the removed column, the graph shown in Figure 4.74, which relates the expected horizontal displacement of one end of a representative beam that is 28.5”-long (with infinite axial stiffness) for a given vertical displacement at the other end, suggests 0.0023” inward horizontal displacement due to geometric compatibility. The resultant outwards movements of the joints on the B and D axes were around 0.021”, almost ten times larger than the expected inward horizontal displacement due to geometric compatibility. Thus, the effect of the inward movements of the joints on axes B and D due to geometric compatibility was insignificant compared to the outwards movement of the same joints due to beam growth. If the vertical displacement above the removed column reached larger values, the effect of the geometric compatibility could be comparable with the effect of the beam growth. When their effect became equal, the horizontal displacements of the beam ends would first stop, and then the beam ends would move inwards as a governing result of the geometric compatibility.

The effect of using fiber sections to model the elements in the analytical evaluation of the two dimensional frame presented in this chapter can be summarized by the following: using the fiber sections made it possible to track the strain variation over the sections of the beam elements and also the change in the strain at the center of the sections which defines the change in the length of the elements. Thus the beam growth is captured in the analysis. As a result of beam growth and the restraints at the ends of the beams, compressive axial forces developed in the beams. The axial load affected the flexural behavior of the beam sections providing extra flexural strength compared to the flexural capacity of the beams without an axial force. The same argument is also valid for
the behavior of the columns, especially neighboring columns which the loads of the removed column are transferred to. Additional axial force would affect the columns flexural behavior as in the beams. Note that columns remained elastic during the dynamic analysis. However, they are expected to have inelastic behavior in the static part of the evaluation of the frame where a monotonically increased displacement is applied to the top of the removed column.

### 4.3.3 Analytical Evaluation of Static Test using CSI Perform-Collapse

In the static part of the analytical evaluation of the two dimensional frame, the results of the analysis performed in CSI Perform-Collapse are presented. Perform-3D is not used since it does not account for the large displacements. That is, all equilibrium equations are considered in the undeformed configuration of the structure during the analysis. Given the displacements and deformations of the frame structure observed in the static phase of the experimental program (see section 4.2.3.2), equilibrium equations must be written in the deformed configuration of the structure to have meaningful results. Therefore, CSI Perform-Collapse, which can account for the large displacements, is used instead of Perform-3D. SAP2000 can also account for large displacements. However, the results are not available due to convergence problems faced during the nonlinear static analysis.

The model used in CSI Perform-Collapse is same as the model used in Perform-3D. That is, beam and columns are modeled with fiber sections. Each rebar in the section is modeled with a separate fiber and appropriate number of fibers used to model confined and unconfined concrete in the section. Both concrete and steel fibers have the material characteristics of the concrete and steel used in the experimental program. The Mander model is used to define the confined concrete in the material characteristics. Rigid zones are assigned to the beam and column intersections. The element length is selected as 9", which is the half of the beam depths. The effects of the element length on the results are discussed in the following sections. As in the experimental study, prior to static analysis the loads used in the dynamic analysis are removed from the structure. Displacement controlled multi-step nonlinear static analysis is defined to analyze the response of the
structure to monotonically increased downward displacement applied to joint C2 (see Figure 4.12) at top of the removed column. This analysis is similar to a pushover analysis with the difference being that the load is applied to only one node and it is in the vertical direction (downwards).

Different than the dynamic analysis, symmetry of the frame is utilized and only half of the frame is modeled (the elements between axes A and C) to perform nonlinear static analysis to reduce the computational cost and to avoid potential convergence problems. For example, in a stage that a bar rupture would occur in the structure, if a full model was used, there would be two sections at the same time that would experience strength loss due to the symmetry. The drastic change in the force deformation relationship of the two sections at the same time could potentially make having convergence in the corresponding time (displacement) step harder. Using the half of the model is intended to avoid such computational difficulties. Appropriate restraints are assigned to the joints on the axis C to include the effect of the other half of the frame in the analysis. That is, the nodes have vertical translational degree of freedom, horizontal translational and rotational degree of freedoms of the joints on Axis C restrained. Note that while presenting the results, the applied force is multiplied by 2 and used to plot the force deformation relationship of the structure. Analytical deformed shapes of the frame are obtained by adding the other half of the frame like a mirror.

### 4.3.3.1. Force-Deformation relationship

Figure 4.82 shows the resisting force (i.e. applied force) versus applied displacement at the top of the removed column (Joint C2) obtained analytically as well as experimentally. As one can see, analytical and experimental force deformation curves show a generally similar pattern. That is, the resisting force goes up very sharply as the vertical displacement at joint C2 increases at the very beginning of the analysis where the elements are in their elastic regions. Then the slope of the force displacement curve reduces due to the yielding of the rebars and concrete crushing. Then, after a certain displacement, bar ruptures cause sharp drops in the resisting force. However, the resisting force of the frame continues to increase after each bar rupture. The resisting force
experiences its lowest value around 8 inches of vertical displacement and then shows increase due to the extended contribution of Catenary action which is explained in the following sections.

One significant reason for the difference between analytical and experimental resisting force versus displacement curves is that the sequence of the bar ruptures in the two cases are different. While all tensile rebars in a section rupture at the same time in the analytical model, in the experimental study, however, there was a time (vertical displacement) lag observed between the rupture of the tensile rebars of the sections. Furthermore, because of the symmetry, two sections on each side of the symmetry line of the frame (Axis C) lost their flexural strength at the same time in the analytical model. Again in the experimental program the sequence of the bar ruptures were not perfectly symmetric. That can be explained with the imperfections in the tested frame. The difference in the rupture strain of the rebars used in a section could show slight difference, causing the time lag between different rebars. In the analytical results, since two sections lose their flexural strength at the same time (due to symmetry) and each section lost all tensile reinforcement at the same time, the correlated drop in the resisting force is sharper than the experimentally observed drops which are usually correlated with rupture of one rebar at a time.

The analytical force displacement curve reaches 1770 lb of resisting force at almost 0.4” vertical displacement. Between 0.4” and 0.85” of vertical displacements, the resisting force experiences a decrease of around 30 lb. After 0.85” of vertical displacement, the resisting force increases from 1740 lb to 2170 lb. After reaching 2170 lb at 4.3” vertical displacement, the first bar rupture causes 400 lb drop in the resisting force. Drop in the resisting force continues up to 7.9” of vertical displacement due to following bar ruptures reducing the resisting force to almost 900 lb. Between 7.9” and 13.8” of vertical displacements, no bar ruptures are observed and the resisting force goes up to 1770 lb. The bar rupture at 13.8” of vertical displacement causes a drop of almost 400 lb in resisting force. Beyond 13.8” of vertical displacement, the resisting force continues to increase up to 1850 lb at 19” of vertical displacement.
Analytically obtained deformed shapes of the frame at various displacements are shown in Figures 4.83 to 4.97. Note that the displacements at which the analytical deformed shapes of the frame are presented in Figure 4.83 to 4.97 are selected to be compatible with the displacements that the experimental deformed shapes of the frame presented in Figure 4.35 to 4.49.

The sequence of the bar ruptures in the analytical model is summarized below. Note that each row corresponds to two beam cross section on each side of the symmetry line of the frame (Axis C) due to symmetry.

<table>
<thead>
<tr>
<th>Order</th>
<th>Location</th>
<th>Vertical Displacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3rd Floor beam - Bar cut-off location (2#7) on axis B side</td>
<td>4.37”</td>
</tr>
<tr>
<td>2</td>
<td>2nd Floor beam - Axis C end</td>
<td>5.08”</td>
</tr>
<tr>
<td>3</td>
<td>3rd Floor beam - Axis C end</td>
<td>6.45”</td>
</tr>
<tr>
<td>4</td>
<td>Roof beam - Axis C end</td>
<td>6.56”</td>
</tr>
<tr>
<td>5</td>
<td>2nd Floor beam - Bar cut-off location (2#7) on axis B side</td>
<td>7.90”</td>
</tr>
<tr>
<td>6</td>
<td>Roof beam - Bar cut-off location (2#7) on axis B side</td>
<td>13.83”</td>
</tr>
</tbody>
</table>

4.3.3.2. Load Transfer Mechanisms

As discussed in the experimental evaluation of the frame, two load transfer mechanisms developed, resisting the applied displacement; Vierendeel Action and Catenary Action. Before the beams between axes B and C lost their flexural strengths and when the displacements were comparatively small, Vierendeel action was the mechanism by which most of the resisting force was developed. That is, the force applied to joint C2 is transferred to the neighboring columns through the shear forces developed in the beams between axes B and D that are deformed in double curvature.

The contribution of the Vierendeel action in the resistance of the frame depends on the flexural capacities of the beams. The critical locations are the beam sections where
demand/capacity ratios are large (e.g. beam ends and bar cut-off locations). As the critical locations lose their flexural strength due to bar ruptures, the moment demands at the ends of the beams would drop and in turn the shear forces developed in the beams would reduce. Figure 4.98 shows the shear force histories at the center of the BC beams in all three floors versus vertical displacement at joint C2. Comparing with the bar rupture sequence presented in the previous section, one can clearly see that bar rupture and in turn loss in the flexural strength of a beam in a floor causes reduction in the shear forces developed in that floor. The reduction in the shear forces in the beams between axes B and D can be seen directly as a reduction in the resisting force of the frame (See Figure 4.82).

The decrease in the slope of the force deformation curve starting at 0.17” of vertical displacement can be related to yielding of the rebars at the maximum demand locations. Figure 4.99 shows the moment history of the critical beam sections in all three floors (the sections where the bar rupture occurred) versus vertical displacement at joint C2 for the first 1” of vertical displacement. Note that all critical sections experienced yielding between 0.17” and 0.32” of vertical displacement. As the moment demands in the critical sections lose their rate of increase per applied vertical displacement, shear forces that are the result of the bending moments in the beams also lose their rate of increase per applied vertical displacement and in turn the slope of the resisting force curve decreases.

Figure 4.100 shows the shear force histories at the center of the BC beams in all three floors versus vertical displacement at joint C2 for the first 6” of vertical displacement. The figure also shows the resisting force for the same range of vertical displacement. After the yielding of rebars at critical beam sections in all three floors, shear forces and in turn resisting force did not increase up to around 0.9” of vertical displacement. Between 0.9” and 2.9” of vertical displacement, the shear forces continued to increase due to strain hardening of the rebars and in turn, the resisting force increased from 1740 to 1970 lbs. Note that between 2.9” and 4.3” (first bar rupture) of vertical displacements, the shear force in the roof beam shows an increase of 20.3 lb and the shear force in the 2\textsuperscript{nd} floor beam shows a decrease of 37.5 lb while the shear force in the 3\textsuperscript{rd}
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floor beam shows a decrease of 6.25 lb. The summation of the change in the shear forces in all three floor beams between 2.9” and 4.3” vertical displacements is around a drop of 23 lb. However, the resisting force of the frame showed an increase of almost 197 lb between corresponding displacements. Considering the fact that only the half of the frame is modeled and the shear forces would be twice in the full frame case, there is a considerable difference between the increase in the resisting force and the decrease in the shear forces in the beams. The difference between the increase in the resisting force and the decrease in the shear forces in the beams indicates another resisting mechanism other than the Vierendeel action which relies on the shear forces developed in the beams through double curvature deformation. The mechanism causing the increase in the resisting force of the frame is nothing but the Catenary action (cable like action) which relies on the axial tensile force in the beams as a result of geometric compatibility at large displacements.

Figure 4.101 shows the axial force histories of the beams at their center versus the applied vertical displacement at joint C2 for the first 6” of vertical displacement (similar to figure 4.100). The Figure also shows the resisting force for the same range of vertical displacement. Note that 2nd and 3rd floor columns start to experience axial tensile force after around 2.9” of vertical displacement and they increase continuously up to 4.3” reaching 487 lb in the 2nd floor and 228 lb in the 3rd floor. The axial forces of the roof beams remain on the compressive side because of the difference between the deformation patterns of the roof with the other two floors.

Figure 4.102 shows the analytical deformed shape of the frame at 4.3” of vertical displacement. Note that the deformations in the 2nd and 3rd floor BC beams are concentrated in the bar cut-off locations where the top bars are reduced to 2#7 (See Section-c in Figure 4.13) and at the axis C ends of the beams. In the roof beam, however, the deformations are concentrated at the ends of the beam on axis B and C. Given the fact that the axis C ends of all three floors beams have practically the same vertical downward displacement and ignoring the deformation of the other sections, the horizontal movement of the axis B ends of the beams due to geometric compatibility (as explained in section 4.3.1.1) is a function of the distance between the sections where the
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deformations (rotations) are concentrated. Since that distance is shorter in the 2nd and 3rd floor beams (the distance between bar cut-off location and axis C end) than the roof beam (practically the length of the beam), the horizontal inwards end movements in the 2nd and 3rd floors due to geometric compatibility is more than that in the roof assuming the same vertical displacement for all three floors. The difference in the horizontal movements of Joint B4 and Joint B3 imposes compressive force in the roof beam through the 3rd floor columns. Because of that reason, the roof beam between axes B and D experiences compressive force. A similar deformed shape is also observed in the experimental program showing that the horizontal inwards movement of the ends of the beams between axes B and D in the roof was smaller than that in the 2nd and 3rd floor due to the difference in the location of the sections where the deformations are concentrated.

Note that even though the axial force in the second and third floor beams after around 3” of vertical displacement is a tensile force, they were on the compressive side before that level of displacement. As one can see from figure 4.101, the axial compressive force in these beams increases up to around 1.6” of vertical displacement and then decreases and goes to the tensile side at around 3” of vertical displacement. The compressive force observed in these beams in the early stages of analysis is nothing but the effect of the beam growth.

Figure 4.103 shows the horizontal displacements of joints B2, B3 and B4 (see Figure 4.12) versus applied vertical displacement at joint C2. The joints in all three floors show an outward displacement in the beginning of the analysis. At around 1.7” of vertical displacement, outwards horizontal movement of the joints stops and changes its direction to inwards movement. As mentioned in the 4.3.1.1, the horizontal movement of the joints on the axis B is the result of two sources; the elongation of the beam, (beam growth) which is the result of the post cracking deformations in the beam sections along the length of the beam, and the geometric compatibility of the frame. Both sources exist from the beginning until the end of the analysis. However, the resulting horizontal displacement depends on the amount of vertical displacement (i.e. deformation level of the frame). In the early stages of vertical displacement, the effect of the geometric compatibility which causes the joints on the B axis move inwards is comparatively
smaller than the effect of the beam growth which forces the joints on the axis B move outwards. Therefore, the joints on axis B moved outward at the beginning of the analysis (up to around 1.7” of vertical displacement). Note that the analytical vertical displacement at which the joints on axis B change their direction shows agreement with the experimental data, which was around 1.5” of vertical displacement. The effects of beam growth and geometric compatibility reach a balance state at around 1.7” of vertical displacement causing the horizontal displacements of joints on axis B to stop. Beyond 1.7” of vertical displacement the effect of the geometric compatibility overcomes that of beam growth and joints on axis B start to move inwards. As the vertical displacement increases, the contribution of the geometric compatibility amplifies the horizontal displacements of the joints on axis B which can be seen from figure 4.103.

The axial compressive force developed in the early stages of the static analysis is the combined result of the elongation of the beam (beam growth) that is due to post-cracking deformations in the sections and the restrain provided by the adjacent elements to the elongation of the beam. Beam growth is also observed in the dynamic analysis. Note that the beam growth is a result of the tensile strains at the center of the sections after cracking of the concrete. Therefore, to be able to capture beam growth, sections need to be modeled in such a way that the strain variation over the height of the section can be captured. Using fiber section in the model developed in CSI Perform-Collapse allows to track the strains at the center of the section and therefore to capture the beam growth.

As a resisting mechanism, Vierendeel action in the frame exists since the beginning of the analysis until the beams lose their flexural strengths and become unable to develop shear forces. The contribution of Catenary action as a resisting mechanism increases as the slope of the bridging beams increase (i.e. the displacement of the joints on axis C increase) since the vertical component of the axial force in the beam would be larger as the slope increases.
4.3.3.3. Effect of Element Length Used in Model

To evaluate the effect of the element length on the issues that will be mentioned in Section 5.2, three different analytical models are developed using three different element lengths: 3”, 6” and 9” and analyzed under monotonically increased vertical displacement at joint C2. The force displacement relationships of the three different models of the frame are presented in Figures 4.104 to 4.106.

As one can clearly see, having longer element length increased the vertical displacement at which the first bar rupture occurs (first section loses its flexural strength). While the first bar rupture occurs in the model that element lengths were 3” at 2.83” of vertical displacement, the first bar rupture occurs in the models that element lengths were 6” and 9” at 3.35” and 4.37” of vertical displacements, respectively. Consequently, the vertical displacements that the following bar ruptures occurred were also increased as the element length used in the model increased (see Figure 4.104 to 4.106).

In summary, the analytical model developed in CSI Perform-Collapse captured the behavior of the frame tested under monotonically increased vertical displacement at joint C4 at the top of the removed column reasonably well (see figure 4.82). Bar ruptures in the beam sections are captured, with a difference that all the rebars of a section fractured simultaneously at both sides of the symmetry line at the same time. Therefore, the drops associated with the analytical bar ruptures were more severe than the drops in the experimental result that are usually associated with one bar rupture at a time. Analytical and experimental deformed shapes of the frame also show agreement at various displacements. Two resisting mechanisms are observed in the frame; Vierendeel action and Catenary action. While the Vierendeel action lost its contribution in the resisting mechanism of the frame as the beams lose their flexural strengths at critical locations and become unable to develop shear forces, Catenary action provided resisting force through the axial forces developed in the beams as a result of axial stiffness of the beams and lateral stiffness of the neighboring elements between axes A and B (and axes D and E). Beam growth and consequential horizontal outwards movements of beam ends
in the early stages of the analysis were also captured. The effects of the element length used in the model on the nonlinear analysis results were also investigated.

### 4.4 Summary

The 1/8\textsuperscript{th} scaled model of the exterior frame of a 3 story RC building is constructed and tested under instantaneous removal of the first floor center column. The response of the structure is monitored with a number of sensors. The structure resisted progressive collapse with a peak displacement of 0.426" at the top of the removed column. Other than flexural cracks formed at the ends of the beam that are bridging over the removed column, there was not any damage in the structure. Two analytical models of the frame were developed using two different types of plastic hinge. The model that the beams and columns are modeled with fiber sections that allows to capture the beam growth (and related axial compressive force in the beams) and accounts for axial force moment interaction is found to be capturing the behavior better than the one that the elements are modeled with two-node beam elements with localized flexure only plastic hinges that accounts for flexural nonlinear behavior without axial force-moment interaction. The resisting mechanism of the frame is characterized as Vierendeel action, which relies on the double curvature bending of the beams which are bridging over the removed column and the redistribution of loads to the neighboring elements through shear forces in the beams. The difference between axial and flexural wave propagation in the frame is discussed.

In the second phase of the experiment, the structure was subjected to a monotonically increasing displacement at the top of the removed column to further study the resisting mechanism(s). Again, the response of the structure is monitored with a number of sensors. An analytical model of the frame in which the elements are modeled with fiber sections is analyzed with the same type of loading and the results are compared with experimental data. The experimental force deformation relationship of the frame was captured reasonably well. Two load transfer mechanisms, Vierendeel action and Catenary action, are observed. Bar ruptures occurred in the beams (as the vertical displacement increases) reduced the performance of Vierendeel action. Contribution of Catenary action, however, increased the resisting force as the vertical displacement
increased and, in turn, slope of the beams increased. The outwards movement due to beam growth and inwards movement due to geometric compatibility of the ends of the bridging beams are discussed. Finally, the effects of the element length used in the model on the analysis results were also investigated.
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Figure 4.2 Typical plan of building of which exterior frame to be evaluated
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Figure 4.3 Elevation view of frame evaluated

Figure 4.4 Reinforcement details of first and second floor columns and second and third floor beams of exterior frame (building)
Figure 4.5 Reinforcement details of third floor columns and roof beam of exterior frame (building)

Figure 4.6 Concrete stress-strain relationship for 1x2-inch cylinders
Figure 4.7 Concrete stress-strain relationship for 2x4-inch cylinders

Figure 4.8 Stress-strain relationship for 0.110 inch diameter deformed wire
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Figure 4.10 1/8th scaled two-dimensional model of exterior frame along axis 1 of building
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Figure 4.12 Dimensions of 1/8\textsuperscript{th} scaled model
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Figure 4.14 Reinforcement details of third floor columns and roof beams of scaled model
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Figure 4.18 Recording of strain gage that was installed on glass column (SG1)
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Figure 4.20 Vertical displacement of joint C4 in roof
Figure 4.21 Strain recording of strain gage SG-30 on third floor column C3-C4

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Figure 4.24 Strain recordings of strain gages SG-3 and SG-4 on first floor column B1-B2
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Figure 4.26 Strain recording of strain gage SG-2 on first floor column A1-A2
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Figure 4.27 Strain recording of the strain gage SG-7 on the first floor column E1-E2

Figure 4.28 Strain recording of the strain gage SG-30 on the third floor column C3-C4 (for the first 0.1 second)
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Figure 4.30 Strain recordings of strain gages SG-5 and SG-6 on first floor column D1-D2
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Figure 4.32 Permanent changes in strains recorded by strain gages from SG-8 to SG-29
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Figure 4.34 Resisting force (i.e. applied force) versus applied displacement relationship of frame

Figure 4.35 Experimental deformed shape of frame at 0" of vertical displacement
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Figure 4.36 Experimental deformed shape of frame at 0.45" of vertical displacement

Figure 4.37 Experimental deformed shape of frame at 1.1" of vertical displacement
Figure 4.38 Experimental deformed shape of frame at 1.5" of vertical displacement

Figure 4.39 Experimental deformed shape of frame at 2" of vertical displacement
Figure 4.40 Experimental deformed shape of frame at 3" of vertical displacement

Figure 4.41 Experimental deformed shape of frame at 4" of vertical displacement
Figure 4.42 Experimental deformed shape of frame at 4.9" of vertical displacement

Figure 4.43 Experimental deformed shape of frame at 5.3" of vertical displacement
Figure 4.44 Experimental deformed shape of frame at 5.8" of vertical displacement

Figure 4.45 Experimental deformed shape of frame at 6.1" of vertical displacement
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Figure 4.47 Experimental deformed shape of frame at 9" of vertical displacement
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Figure 4.49 Experimental deformed shape of frame at 16.25" of vertical displacement
Figure 4.50 Order of sections that concrete crushed (in circles) and corresponding vertical displacements

Figure 4.51 State of section which first bar rupture occurred at 4.86” vertical displacement
Figure 4.52 Experimental horizontal displacements of joint B2 and D2 in the second floor

Figure 4.53 Experimental horizontal displacements of joint B3 and D3 in the third floor
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Figure 4.55 Axial cracks formed (pointed out with arrows) in beam B3-C3 at 9” of vertical displacement
Figure 4.56 State of first floor column B1-B2 at 16.25” of vertical displacement

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Figure 4.59 Deformed shape of frame at end of the dynamic analysis with a scale factor of 1 (Thick dots indicates yielded sections)
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Figure 4.63 Axial force at bottom end of column C3-C4 in third floor
Figure 4.64 Axial force diagram of axis C columns (a) before and (b) after column removal

Figure 4.65 Axial force at top end of 2nd floor column C2-C3 (for first 0.1 seconds after column removal)
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Figure 4.67 Axial force of 1st floor column B1-B2 (for first 0.1 seconds after column removal)
Figure 4.68 Axial force of 1st floor column D1-D2 (for first 0.1 seconds after column removal)

Figure 4.69 Vertical displacement of joint C2 in second floor (for the first 0.1 second).
Figure 4.70 Bending moment diagram of elements of frame (a) before and (b) after column removal.

Figure 4.71 Exaggerated deformed shape of frame.
Figure 4.72 Horizontal displacement histories of joints on axes B and D in all three floors. (Positive direction is shown in figure)

Figure 4.73 Illustration of geometric compatibility.

\[ x = L - \sqrt{L^2 - \Delta^2} \]
Figure 4.74 Relationship between horizontal inward movement and vertical displacement of a representative beam.

Figure 4.75 Analytical model of frame developed in Perform-3D.
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Figure 4.77 Moment curvature history of section at axis B end of roof beam B4-C4.
Figure 4.78 Axial force versus curvature of section at axis B end of roof beam B4-C4.

Figure 4.79 Experimental and analytical (SAP2000 and Perform-3D) vertical displacements of Joint C2.
Figure 4.80 Experimental and analytical (SAP2000 and Perform-3D) vertical displacements of Joint C4.

Figure 4.81 Horizontal displacement histories of joints on axes B and D in all three floors.
Figure 4.82 Experimental and analytical (CSI Perform-Collapse) resisting force versus applied displacement relationship of frame.

Figure 4.83 Analytical deformed shape of frame at 0" of vertical displacement.
Figure 4.84 Analytical deformed shape of frame at 0.45" of vertical displacement.

Figure 4.85 Analytical deformed shape of frame at 1.1" of vertical displacement.
Figure 4.86 Analytical deformed shape of frame at 1.5" of vertical displacement.

Figure 4.87 Analytical deformed shape of frame at 2" of vertical displacement.
Figure 4.88 Analytical deformed shape of frame at 3" of vertical displacement.

Figure 4.89 Analytical deformed shape of frame at 4" of vertical displacement.
Figure 4.90 Analytical deformed shape of frame at 4.9" of vertical displacement.

Figure 4.91 Analytical deformed shape of frame at 5.3" of vertical displacement.
Figure 4.92 Analytical deformed shape of frame at 5.8" of vertical displacement.

Figure 4.93 Analytical deformed shape of frame at 6.1" of vertical displacement.
Figure 4.94 Analytical deformed shape of frame at 8.5" of vertical displacement.

Figure 4.95 Analytical deformed shape of frame at 9" of vertical displacement.
Figure 4.96 Analytical deformed shape of frame at 12" of vertical displacement.

Figure 4.97 Analytical deformed shape of frame at 16.25" of vertical displacement.
Figure 4.98 Shear force histories at center of BC beams in all three floors versus vertical displacement at joint C2.

Figure 4.99 Moment histories of critical beam sections versus vertical displacement at joint C2 (for first 1" of vertical displacement).
Figure 4.100 Shear force histories at center of BC beams in all three floor and resisting force of frame versus vertical displacement at joint C2 (for first 6" of vertical displacement).

Figure 4.101 Axial force histories of beams at their center versus applied vertical displacement at joint C2 (for first 6" of vertical displacement).
Figure 4.102 Analytical deformed shape of frame at 4.3” of vertical displacement.

Figure 4.103 Analytical horizontal displacements of Joints B2, B3 and B4 versus applied vertical displacement at joint C2.
Figure 4.104 Analytical force displacement relationships of frame with element length of 3".

Figure 4.105 Analytical force displacement relationships of frame with element length of 6".
Figure 4.106 Analytical force displacement relationships of frame with element length of 9".
Chapter 5

Modeling Issues and Assumptions

5.1 Introduction

Modeling and analyzing structures mathematically involves various assumptions. It is obvious that a structure cannot be modeled with all the details and imperfections it has. Even if it were possible to model a structure in a way that many details would be included, such a model would not be optimum if the response of the structure could be obtained from a simpler model. Therefore, logical assumptions are needed to be made to make the model as simple as possible yet capable of capturing the behavior reasonably well.

Depending on the nature of the analysis and the behavior of the structure, the modeling techniques used and assumptions made need to be verified such that the model reflects reasonable and meaningful behavior. In a progressive collapse evaluation of a structure, compared to conventional design practices, the structural and nonstructural members may experience demands beyond their capacities causing partial or full collapse of the structure during the analysis. Given the dynamic nature of progressive collapse, the severity as well as direction of the demands on a member may change suddenly as a result of failure of other members.

The potential damages or failures that the structure may experience should have been foreseen and properly included in the model. Otherwise, the model of the structure would not be realistic, and the analysis results would be null. Therefore, the elements in the analytical model should be capable of reflecting the potential damage or failure mechanisms. This requires experience and good understanding of the structural behavior under the conditions that can cause collapse of the structure. Similar to the nature of the problem, modeling can be performed in a progressive manner that is updating the model
with the expected failure mechanisms, which are foreseen by understanding of preliminary analyses.

There is not a unique way to model a structural or non-structural element in the analytical evaluation of a structure. Each method used to model an element may have its own strengths and shortcomings. The selection of the element type or the method of modeling should be made consciously such that the type of the element or the method would be able to represent the behavior realistically.

The limitations of the software or the shortcomings of element types used to model a structure and evaluate its response to progressive collapse can be overcome using alternative techniques. For instance, the cracking of concrete cannot be automatically modeled if the elements are modeled with 2-node beam elements. However, the reduction in the flexural stiffness of the elements due to cracking can be modeled by following the iterative procedure as explained in 5.2.1. Element removal during a dynamic analysis is not readily available in all software. Again, this can be achieved by using an alternative method as explained in 5.5.2.

The third chapter of this dissertation is about experimental and analytical evaluation of the response of actual buildings to loss of column(s) and the fourth chapter presents experimental and analytical evaluation of the response of an RC frame to the loss of a column. All the structures studied in this dissertation are modeled with the Finite Element Method. Two programs namely, SAP2000, Perform-3D and CSI CSI Perform-Collapse are used to develop Finite Element Models of the structures. Only Hotel San Diego (See section 3.2) was modeled by ELS (Extreme loading for Structures) which uses the Applied Element Method in addition to the Finite Element Method. The responses of the models are compared with the experimental data. The modeling techniques used during the analytical evaluation of the structures presented in Chapters 3 and 4 and related assumptions along with the issues faced are presented in this chapter.
5.2 The Issues Related to Modeling Beams and Columns

5.2.1 Cracking in Beams and Columns

Cracking occurs in a RC section when the maximum tensile stress reaches the tensile strength of concrete. The section behaves linearly elastic up to cracking. For a section under flexural behavior, cracking causes a drop in the moment capacity but more importantly causes reduction in the section stiffness. ACI suggests using 35% of the uncracked flexural stiffness for cracked beam elements, and 70% of uncracked flexural stiffness for cracked column elements. The cracking can be automatically accounted for if the element used to model concrete is able to track the stress (or strains) during the analysis, and follow the constitutive law of the material. Otherwise, alternative procedures can be utilized as explained in the following.

The structures evaluated in this dissertation are modeled by using 3 different element types which are available in the software packages used. Beam elements with fiber sections are used in Perform-3D and CSI Perform-Collapse. Two-node linear beam elements with localized plastic hinges are used in SAP2000. And an assemblage of small elements that are connected through a set of normal and shear springs located at contact points distributed along the element faces are used in ELS. Perform-3D, CSI Perform-Collapse and SAP2000 use the Finite Element Method (FEM) while ELS uses the Applied Element Method (APM).

When the beams and columns are modeled with fiber sections, as in Perform-3D and CSI Perform-Collapse, or with an assemblage of small elements that are connected through a set of normal and shear springs, as in ELS, the stresses in the fibers (or in springs) are calculated in each time step (or in each incremental step in a static analysis). When the stress in a fiber (or in a spring) that models the concrete reaches the tensile strength of concrete, that fiber (or spring) loses its tensile strength. Therefore, the cracking of the concrete is automatically accounted for.

When linear two-node Bernoulli beam elements are used to model beams and columns, as in SAP 2000, these elements behave linearly. In progressive collapse
evaluation of a structure, after selected columns are removed from the structure, some beam and column sections can likely develop moments that are higher than the cracking moment capacities. Sections lose some flexural stiffness if they crack. Depending on the reinforcement used in the beam or column, cracks may extend over a length. This means some parts of the element would have less stiffness compared to uncracked parts. Using two-node Bernoulli beam elements, as in SAP2000, does not allow tracking the cracking in the elements and assign reduced stiffness to those which cracked. Ignoring this fact in the modeling phase would make the model have overestimated stiffness. The procedure below is used to account for cracking of the elements and related loss of stiffness when the cracking in the beams are not automatically accounted for.

i) First, all elements are assumed to be uncracked and have full flexural stiffness. All dead loads on the structure are applied statically. And then a time history analysis is run that simulates column removal.

ii) The bending moments of the beam elements are compared with the cracking moments for each beam section.

iii) The flexural stiffness of the regions that have bending moment exceeding the cracking moment are reduced with the coefficients of 0.35 and 0.7 for beams and columns respectively as suggested in ACI-318(2008). In this step, the fact that having smaller flexural stiffness would cause smaller bending moment demands in the next analysis has also been accounted for.

For example, consider the beam shown in Figure 5.1 that belongs to a structure that is subjected to a column removal. Figure 5.1 (a) represents the bending moment diagram of the beam after dead loads are applied. Figure 5.1 (b) shows the maximum (envelope) bending moment diagram of the beam after selected columns are removed from the structure. These bending moment diagrams are obtained from an analysis that all elements are assumed uncracked and have full flexural stiffness during the whole analysis. Consider again that the cracking moment is 420 kip-in for that beam. Note that the beam has 10 sub elements along its length. Comparing the bending moment demands along the beam with the cracking moment, one can conclude that 2 sub elements from the
I end and 3 sub elements from the J end of the beam reached the cracking moment and they cracked. It is obvious that the all of these elements did not reach the cracking moment at the same time. Based on the shape of the moment diagram, the first elements from the I and J end reached the cracking moment before the other sub elements. The cracking of the first elements would cause a stiffness reduction in that element and that reduction would affect the results in the next steps of the analysis. Therefore, a progressive approach would be meaningful to follow. Instead of reducing the flexural stiffness of all elements that have a moment demand in the first analysis higher than the cracking moment, starting to reduce the flexural stiffness of the elements that have the highest demand would give a chance to see if the adjacent elements will reach the cracking moment or not after the first elements are cracked. Figure 5.1 (c) shows the resulting bending moment diagram of the same beam after flexural stiffness of the first element from both ends is reduced. The sub elements that have the reduced flexural stiffness are shown in red. Note that the 3rd element from the J end now has a bending moment demand which is less than the cracking moment. The second elements from both ends, however, still have moment demand higher than the cracking moment. Therefore, their flexural stiffness needs to be reduced in the next step. Note that the best way would be having the program track all elements that reached the cracking moment and reduce their flexural stiffness during the analysis. Due to limitations, such an approach is utilized to model the cracked regions as accurately as possible.

iv) The modified structural model with some sections having reduced moment of inertia is re-analyzed under the load case used in Step (i). The bending moments are again compared and the sections that have bending moment demands more than their cracking moments are determined and flexural stiffness of those are reduced with the logic explained in (iii).

v) Step (iv) is repeated until all cracked regions are appropriately modeled.

One issue that needs to be considered while following the approach explained above is the number of sub-elements that the beams and column have along their lengths. In a structural model, beams usually consist of a number of segments, for instance,
because they have common joints with shell or beam elements that model the slab. Columns, however, usually consist of one single element. One should pay attention to the number of elements needed to define the cracked regions properly. Consider a beam element that has two sub segments with equal length. Consider again that the moment demand reached to cracking moment capacity at one of its ends. Assigning the reduced moment of inertia to the element that has the moment demand larger than the cracking moment would make half of the beam cracked which in reality is not the case. So having enough sub segments along an element is necessary to not underestimate the stiffness of the element due to cracking.

Another issue with the procedure mentioned above is that all elements which are expected to be cracked based on the initial analysis results would have a reduced moment of inertia at the beginning of the next analysis. In reality, however, all elements would be uncracked and would have full moment of inertia if they have not been cracked yet due to dead loads prior to column removal. After column removal, elements start to deform and cracking occurs progressively in the structure. It is obvious that cracking in one location and related stiffness reduction would affect the behavior of other elements. In other words, the stiffness of the structure would change as the cracking occurs in different locations. Using fiber section to model elements would account for this fact, since the stresses in the fibers are checked every time step during the analysis and the cracking of the elements and related stiffness reduction occurs during the analysis.

In determination of the cracked zones in the elements and in the process of reducing the moment of inertia of the ones which are expected to be cracked, all the structural (e.g. beams, columns, floor system) and non-structural (infill walls) elements should be considered together since all structural and non-structural elements are in interaction with each other during the analysis. In other words, cracking in one element at a certain time step would affect the demands on the other elements in the next time step since the stiffness of the structure changed.

Among the evaluated structures, some of them (e.g. Hotel San Diego, University of Arkansas Dormitory) are found to have some beams that do not have continuous
reinforcement in the regions where the demand due to the loads that the structure is designed for does not require any reinforcement. The structural code at the time of the construction did not require continuous reinforcement for those beams.

For example, Figure 5.2 (a) shows the reinforcement detail of one of the critical beams of Hotel San Diego. Figures 5.2 (b) and (c) show the bending moment diagram of the same beam before and after column removal. As can be seen from Figure 5.2 (b) the bending moment demands due to dead loads do not require top reinforcement on the mid region of the beam. However after removal of columns, the bending moment diagram developed in the beam (See Figure 5.2 (c)) is completely different from the bending moment diagram due to dead loads (See Figure 5.2 (b)). Note that if the negative bending moment demand reaches the cracking moment capacity at the sections that do not have top reinforcements single cracks will form and will not be able to extend. Since there is no reinforcement to resist the axial strains, the section would lose practically all of its flexural strength after cracking of concrete.

In the models developed in Perform-3D, CSI Perform-Collapse and ELS, since the sections are made of an array of distributed fibers and springs, respectively, the issue explained above is automatically taken care of. In the models developed in SAP 2000, however, since the beams are made of 2-node linear Bernoulli beam elements, localized nonlinear plastic hinges are assigned to the critical locations where such a brittle failure can occur. The hinges behave linearly until the bending moment demand at the hinge location reaches the defined cracking moment value. After the moment exceeds the cracking moment the hinge loses its flexural strength.

5.2.2 Yielding in Beams and Columns

Material nonlinearity can be included in the analytical model of a frame structure generally in two ways; distributed and localized plasticity. In the modeling of the structures presented in Chapter 4 and 5, distributed plasticity is used in the modeling with Perform-3D, CSI Perform-Collapse and ELS. The beam and column sections behave nonlinearly since the sections are made of an array of distributed fibers or springs, respectively, and each fiber (or spring), either concrete or steel, follows the nonlinear
force deformation relationship that is assigned to that fiber (or spring). In Perform-3D, CSI Perform-Collapse and ELS, multi-linear force deformation relationships for the concrete and steel are entered based on the material tests results performed on the samples that are taken from the real structure.

In SAP 2000, however, the yielding of reinforcement is modeled through localized plastic hinges. The force-deformation of the section that the plastic hinge is assigned is calculated by performing a section analysis. The geometry of the section and the material characteristic of concrete and steel obtained from the material test results are used to calculate the force-deformation relationship of the section. Then the simplified multi-linear force-deformation relationship is assigned to the plastic hinge. The plastic hinges are assigned to the critical locations where yielding can occur including the end of the beams and columns as well as the bar cut-off locations.

When the distributed plasticity (as in Perform-3D, CSI Perform-Collapse and ELS) is used in the modeling of the structures which are presented in the third and fourth chapters of this dissertation, the axial force – moment interaction is automatically accounted for. However, when the localized plastic hinges are used to model the yielding in beams (as in SAP 2000), flexural plastic hinges can be used with or without axial force – moment interaction. In the modeling of the building structures that are also studied experimentally (see Chapter 3), flexural plastic hinges without axial force – moment interaction are used in the beams. The reason for this assumption was that the maximum vertical displacements were limited to small values in all the structures evaluated experimentally without causing tensile forces in the beams bridging over the removed column. Note that one may expect to observe compression forces in the beams bridging over the removed column due to beam growth in small displacements and have an interaction with the moment in the beam sections. But since the beam growth cannot be captured with 2-node beam elements, the usage of the plastic hinges with axial force – moment interaction would not be useful.
5.2.2.1 Effect of Element Length Used in Model in case of Yielding

When distributed plasticity is used to include the material nonlinearity in the model, length of the elements have an effect on the results. For such a beam as shown in Figure 5.3 the rotations at the nodes would be the product of curvature and the tributary length of the node. Therefore having more tributary length would cause more rotation for a specific node. Consider an applied displacement to the tip of the beam shown in Figure 5.3. Also consider a moment-rotation relationship very close to elastic perfectly plastic (i.e. very little slope after yielding) as shown in Figure 5.4(b). End deflection is the result of summation of the rotations at each joint. The rotation in each joint which is the product of curvature and tributary length is proportional with the moment demand at that point before yielding of the section at the support (where the maximum moment demand is). Since all the nodes are on the first linear branch of the moment rotation relationship and since the moment diagram is triangular (linearly increasing between nodes) (shown with (a) in Figure 5.4(a)), the rotations at the nodes are linearly increasing from tip of the beam to the support (triangular) (same as moment diagram).

After yielding at the support, the small amount of increase in the moment would cause significant curvature at the support compared to the rotation at the next node (shown with (b) and (c) in Figure 5.4(a)). Since the next node has not yielded yet, the deformation at the next node (and at the nodes beyond) would increase relatively quickly (with the initial slope) while the curvatures at the yielded section would increase relatively slowly with the second slope which is much shallower than the initial slope.

The issue explained above is the effect of the assumed plastic hinge length (Lp) on the results. The rotation will be the result of curvature at the node times the tributary length. So the length of the element should be chosen equal to the assumed plastic hinge length (Lp).

5.2.3 Bar Rupture in Beams and Columns

If beam or column section under flexure had a demand that causes the strain in a reinforcement bar to exceed the ultimate strain capacity of the steel used, then the rebar
would rupture. In the context of progressive collapse, bar rupture is an event that is likely to occur. Therefore, the analytical model should include this fact and related flexural capacity reduction in potential locations to be able to provide reasonable results if the preliminary analyses shows that the demands in the elements are in a level that can lead to bar rupture. Although the bar rupture was not observed in the experimental and analytical progressive collapse evaluation of the buildings presented in the third chapter of this dissertation, a severe number of bar ruptures occurred in the experimental evaluation of a two dimensional scaled frame that was subjected to monotonically applied vertical displacement at the top of the center first floor column (removed column in the dynamic phase, see Chapter 4). The analytical results also showed that all of the critical sections lost their tensile reinforcements as the structure deformed as a result of applied displacement. Although the sequence in the analytical model was not completely in agreement with the experimental results, in general, experimental and analytical results beyond the first bar rupture were comparable.

5.2.3.1 Effect of Element Length Used in Model in case of Bar Rupture

When distributed plasticity is used to include the material nonlinearity in the model, the element length has also an effect on the post-yielding behavior of the structures. Again, consider the beam shown in Figure 5.3. End displacement of the beam is dependent on the rotations at the nodes. After yielding (especially for the force deformation relationships that are close to elastic-perfectly plastic), the rotations are more concentrated on the yielded sections (i.e. most of the deformation comes from rotation at the yielded section as explained above). If the element sizes were reduced, then for the same displacement (e.g. same rotation) the curvature at the yielded section must be more to provide corresponding rotation. Then the moment would be higher. Therefore, when shorter lengths are used, sections would reach their capacity sooner in terms of end displacement. This phenomenon is observed in the analytical evaluation of the 2-dimensional 3-story 4 bay frame which was subjected to a monotonically increased vertical displacement at the top of the removed first floor center column (See Chapter 4). Three analytical models of the frame are developed using three different element sizes and the results are compared. Figure 4.76 to 4.78 show the force deformation
relationships of the frame for three different models. As can be clearly seen from the figures, using smaller element size caused the first bar rupture in the frame to happen at a smaller vertical displacement at the top of the removed column.

Another issue with the element length is that when a section reaches its capacity and loses moment, the adjacent sections would lose their moment too (because of equilibrium) (moment diagram would change). This would cause unloading in the adjacent sections and the curvatures and the rotations would decrease. This would increase the contribution of the section which lost its strength in the deformation of the structure. This issue is schematically illustrated in Figure 5.5 where the red point corresponds to a state of the section that loses its strength and the green point corresponds to a state of the adjacent section. Therefore, the length of the element that loses its strength would affect post-failure deformations in the structure.

5.2.4 Effect of Beam Growth

Beam growth can be defined as the elongation of the beam as a result of the post cracking deformations in the sections along the length of the beam. Consider the fixed supported beam shown in Figure 5.6. As the vertical downward displacement is applied at the mid span of the beam, the beam will start to deform and the curvatures in the sections will increase. The Figure 5.7 (a) shows the schematic strain distribution in cross section A (see Figure 5.6), which experiences a negative bending moment, at a certain level of vertical displacement. Assume that at this level of deformation the concrete has not cracked yet and the neutral axis is at the center of the section. After the concrete cracks the neutral axis would move downwards (towards to the compressive side) for this section as shown in Figure 5.7 (b). Note that at this level of deformation, the center of the section experiences tensile strains while the strain was zero at the center of the section before cracking. Note that the same argument would be valid for a section that experiences positive bending moment such as Section B (see Figure 5.6). The neutral axis would move towards to compression side from the geometrical center and the center of the section would experience tensile strains after concrete cracks. As a result of the tensile strains developed at the center of the sections, the beam would grow (elongate).
However, in the case of fixed supported beam, the end supports would prevent the elongation and in turn axial compressive forces would develop in the beam. Note that if the beam were simply supported then there would not be any compressive force in the beam due to beam growth since the end with a roller would move and let the beam elongate. For the fixed ended beam, after a certain amount of vertical displacement, the change in the geometry of the beam would cause tensile forces which is also known as Catenary action and overcome the compression force due to beam growth.

To capture the beam growth in an analytical model, the beam cross section needs to be modeled in such a way that the variation of the strain over the height of the section due to both flexural and axial deformations can be captured. Note that it is not possible to capture the beam growth if the beam is modeled with 2-node linear Bernoulli beam elements with flexure only plastic hinges.

The axial compressive force developed in the beam due to beam growth would interact with the moment and may increase or decrease the flexural strength of the beam. Figure 5.8 shows a 2-dimensional frame structure analytically modeled in Perform 3D using fiber sections. A monotonic downward displacement is applied at joint A. The maximum analytically estimated axial compressive force developed in the beam due to beam growth is found to be 0.26 kips which is 0.7 % of the axial load capacity of the section. Figure 5.9 shows the Axial Force – Moment (P-M) interaction curve of Section A. As shown in Figure 5.9, moment capacity of the section is 3.03 kip·in for zero axial force while it increases to 3.26 kip·in (8 % increase in the moment capacity) when 0.26 kips of axial compressive force exists on the section.

Another effect of the compressive force developed in the beam due to beam growth is that it imposes additional bending moment to the beam. Figure 5.10 shows the internal forces developed at the BC end of the beam shown in Figure 5.6 on the deformed shape schematically. The moment developed at the ends of the beam must be in equilibrium with the end shear and axial forces. Therefore, the end moments can be written as:

\[ M_B = M_C = V \times L + N \times d \]  
\[ \text{Eq. 5.1} \]
As can be seen in Eq. 5.1, some portion of the end moment is due to the axial load in the beam developed as a result of beam growth. Note that for the axial load to have P-Δ effect on the moment, the geometric nonlinearity needs to be included in the analysis.

### 5.2.5 Effect of Axial Tensile Force under Large Displacements on Flexural Behavior of Beams

After the axial force in the beams bridging over the removed column turns to tension, the beam sections may crack if the tensile stresses in the section due to tensile axial force exceed the tensile strength of concrete. This fact has been accounted for in the models use fiber sections (or with springs as in the Applied Element Method).

Moreover, from the flexural behavior point of view, tensile axial force developed in the beam would reduce the moment capacities of the sections due to Axial Force - Moment interaction. Again, this fact would be accounted for when the beam is modeled with fiber sections (or with springs as in the Applied Element Method). In such a model that the beams are modeled with 2-node linear beam elements and the yielding under flexure is modeled with localized plastic hinges, one can use flexural plastic hinges that can account for the P-M (Axial Force-Moment) interaction. Note that the geometric nonlinearity that accounts for large displacements (i.e. the equilibrium equations are written in the deformed configuration) needs to be included in the analysis to observe axial forces in the beam elements.

Note that the same type of plastic hinges could have been used to account for the effect of the compressive force developed in the beam due to beam growth on the moment capacities of the section. However, though linear beam elements can capture P-delta effects due to geometric nonlinearity, the beam growth cannot be captured with 2-node linear beam elements.

As discussed previously, the demands in the beams that are bridging over the removed column(s) (i.e. the beams in which the Catenary action forms) may cause bar ruptures in the critical sections. Consider the frame structure presented in Chapter 4. Bar rupture and related flexural capacity reduction in the section level causes the reduction in
the contribution of the Vierendeel action in the system level resistance of the structure. Following bar rupture, sections lose their flexural capacity and practically become hinges. The performance of the Catenary action depends on the axial force capacity of such sections which is practically the axial force capacity of the remaining rebars. It is possible to observe rupture in those bars as a result of the axial force due to Catenary action. Therefore, such a failure mechanism should be included in the model.

5.2.6 Effect of Modulus of Rupture of Concrete on Performance of Vierendeel Action of Beams under Special Circumstances

The modulus of rupture of concrete plays an important role in the Vierendeel action especially when the beams have sections with less than reinforcement. Consider the two dimensional frame structure shown in Figure 5.11 (a). The expected moment diagram of the beams between axes 2 and 4 under dead loads is shown in Figure 5.11 (b) schematically. Assume the beams do not have top reinforcement in the positive moment regions and do not have bottom reinforcement in the negative moment regions under dead loads as shown in Figure 5.11 (c). After the column HM was removed, the expected moment diagram of the beams bridging over the removed column is schematically shown in Figure 5.11 (d). As can be seen from the Figure 5.11 (d) some regions that do not have top reinforcement experienced negative bending moment while some regions without bottom reinforcement experienced positive bending moment after column removal. If the bending moment demand exceeds the cracking moment capacity for such sections, brittle failure may occur under flexure and these sections may lose their flexural strength. For such a structure shown in Figure 5.11, if both ends of beams BC, CD, GH and HI crack and lose their flexural strength, Vierendeel action of those beams with the interaction of column CH vanishes. Then a new mechanism called Catenary action that redistributes the loads through the axial tensile forces in the bridging beams over the removed column would form. However, if the cracking moment capacity of the beam sections at the regions with no reinforcement in the tensile zone were more than the maximum demand, then the loads could have been redistributed through Vierendeel action without any partial or full collapse.
For instance, in the evaluation of The University of Arkansas Medical Dormitory building in Little Rock, one exterior column in the first floor was removed from the structure as discussed in Section 3.3. The dominant mechanism in the structure is found to be Vierendeel action of transverse beams B5-C5 (see Figure 3.92). The maximum displacement at the joint above the removed column was limited to 0.25". The modulus of rupture of concrete played an important role to limit the maximum displacement to such a small value. As explained in the section 3.3.4.3, the direction of the moments in beam B5-C5 which is connected to the removed column (and the same beams in the upper floors), changed after removal of column. Vierendeel action of those beams with the interaction of columns above the removed one imposed negative bending moment to sections that do not have top reinforcement. Since the demand on that section was lower than the cracking moment capacity, the section did not crack and the vertical loads were redistributed through Vierendeel action. If the demand in those beam sections were larger than the cracking moment capacity, then the performance of Vierendeel action in the redistribution of loads would drop since there is no top reinforcement in the section to prevent the crack from opening. After such a brittle failure in the beam sections, the participation of the slabs in the redistribution of the loads would increase.

In the evaluation of such structures that do not have continuous reinforcement in their beams; the modulus of rupture of concrete may play an important role in the performance of Vierendeel action. Incorrect estimation of the modulus of rupture of concrete may cause significant variations in the results. Therefore, the modulus of rupture of concrete along with the other material characteristics should be estimated reliably. The estimation of the internal forces through the structural analysis should have the same reliability.

5.2.7 Bar Pull Out in Beams due to Lack of Anchorage

Another brittle type of failure that may occur in the beams is due to lack of anchorage of the beam longitudinal reinforcement into beam column joints. The same type of failure may occur in the slabs as well.
Consider the two dimensional frame structure shown in Figure 5.12. Assume that the anchorage of the bottom reinforcement of beams GH and KL into the exterior columns are not enough for the beam sections to develop full moment capacity at the face of the exterior columns (Sections S1 and S2, see Figure 5.12 (a)). It is obvious that under only dead loads the direction of expected bending moment at those sections would be negative and there is no possibility of bar pull out for the bottom reinforcement in these sections. Consider now that the column DH is removed from the structure. The expected deformed shape and the bending moment diagram of the structure is shown schematically in Figure 5.12 (b) and (c) respectively. After column removal, Sections 1 and 2 would experience positive bending moment due to formation of Vierendeel action. If the positive bending moment at these sections exceeds the cracking moment capacity (i.e. the tensile stress at the bottom of the sections reaches the tensile strength of concrete), the cracks will form at those sections on the column faces initiating from the bottom. After cracking of the concrete, the stresses in the bottom reinforcement will increase rapidly. At this stage, if the anchorage of the bottom reinforcement into the joint at the top of column HL failed, then the bottom reinforcement would be pulled out and the section would lose its flexural strength. Following the bar pull out, Sections 1 and 2 would behave like a hinge under flexure and the Vierendeel action of beams GH and CD with column DH would vanish. Then beams GH and CD would behave like cantilever beams and would deform under single curvature. This change of behavior of beams GH and CD would cause more negative bending moment at the G and K ends of beams GH and CD, respectively. If the negative bending moment capacities at those ends of the beam were enough to carry the increased demand, then the loads would be redistributed without collapse. Note that the change of deformation from double curvature to single curvature would cause additional demand at the bar cutoff locations near the G and C ends of beams GH and CD, respectively. Beams may fail at these bar cutoff locations even though the flexural capacities were enough to carry the demand at the face of the columns CG and GK where the maximum negative bending moment demand occurs.

If the structure was a three dimensional structure with a slab, other elements would participate to redistribute the loads after such a brittle type of failure that occurred
due to lack of anchorage. Depending on the capacities of those other elements, the structure may stabilize without any partial or full collapse.

In the evaluation of a structure with elements that may experience brittle failure due to bar pull out, this phenomenon should be included in the model. Ignoring such a failure in the elements may cause an improper estimation of the mechanism of the structure to redistribute the loads following removal of a load bearing element. Even though the model did not have a failure mechanism due to bar pull out in the preliminary analysis, the results needs to be evaluated carefully to identify the potential spots and the model should be updated accordingly.

5.3 The Issues Related to Modeling Slabs

In this dissertation, when the Applied Element Method is used to model a structure, all the structural and nonstructural elements including beams, columns, slabs as well as infill walls are modeled with the same type of element. That is, all the elements are made of rigid cuboids and the springs that connect the rigid cuboids to each other. All of the mechanical characteristics of connected elements (cuboids) are lumped to those springs. As with the beams and columns, the slabs are modeled with cuboids and springs between them. For the reinforcement of the slab, separate springs that represent the rebars are used in addition to the springs that represent the concrete between the rigid cuboids. The characteristics of the concrete springs are calculated based on the geometry and material properties of the adjacent cuboids. Slabs must be divided to enough small cuboids to model the behavior properly. Note that the cracking in the slabs is automatically accounted for through the concrete springs. That is, when the tensile stress in a concrete spring exceeds the specified tensile strength of concrete, the spring loses its tensile strength.

In Finite Element Models of the evaluated structures, which are developed with SAP 2000 and Perform-3D, two methods were available to model the slab. The first method is using two dimensional shell elements and the second method is using a grid of beam elements.
Cracking in the slab can be included in the model using both methods (e.g. shell elements or beam elements). For the cracking of the beam or shell elements that model the slab, one can follow the procedure explained in 5.2.1. The progressive procedure mentioned in 5.2.1 can be executed for beams, columns and slabs simultaneously. The cracking moment capacity of each mesh or strip should be calculated based on its sectional geometry. As mentioned in 5.2.1, the beam elements that model the slab strips should have enough sub-elements to model the cracked regions properly.

The linear shell elements used in the models developed in SAP 2000 and Perform-3D cannot model nonlinear behavior. If the slab is expected to behave linearly (i.e. no yielding is expected), then two dimensional linear shell elements can be used to model the slabs.

If the slab is expected to experience yielding, then a grid of rectangular beam elements, that can have nonlinear behavior through the localized plastic hinges, can be used to model the slab. A slab can be considered as a combination of several strips in both perpendicular directions. The height of all strips would be equal to the slab height while the widths of the strips would add up to total width of the slab in each direction.

For the yielding of the beam elements that model the slab strips, localized plastic hinges are used as in the beams and columns. The force deformation characteristics of the hinges are calculated through a section analysis based on the cross section of the strips. In the step of dividing the slab into the strips, the size, number and location of reinforcement that each strip will have are calculated and used in the section analysis. Localized plastic hinges are assigned to the locations where yielding can occur including the edges of the slabs and the bar cut-off locations.

There is a difference between the shear flow in the slab and that of the beam elements with rectangular sections modeling the slab. Because of this, the torsional stiffness is set equal to one-half of that of the gross sections (MacLeod, 1990).

One issue that needs to be considered while modeling the slab with a grid of beam elements is the weight and the mass of the slab. If the weight of the elements are
calculated and applied automatically by the program using the unit weight of the material and the geometry, then the weight of the slab would be counted twice since the volume of the strips in each perpendicular direction would be equal to the volume of the slab. Therefore, the unit weight of the material used for slab strips should be half of the slab’s unit weight if the weight is calculated and applied by the program. The same statement is valid for the mass of the slab.

5.3.1 Diaphragm Effect of Slab

The effect of the floor system on the constraint provided to the beams in both beam growth and the Catenary action is not ignorable. Therefore, the slab (floor system) shall be modeled in a way such that it provides the constraints reasonably to the beams. For instance, if the slab is modeled with a grid of beam elements, then grid must have enough strips in both perpendicular directions to model the constraint provided by the slab properly. Also when the slab is modeled by shell elements, it must have a mesh size small enough to model the distributed constraint on the beam provided by the slab.

In the cases of a two dimensional and three dimensional frame structure as shown in Figure 5.13 (a) and (b) respectively, the ends of the BD beams are neither free to move nor as rigid as a fixed support. In the case of a two dimensional frame, the support to the beam is provided by the lateral stiffness of adjacent frames ABKL and DEMN (See Figure 5.13 (a)).

In the case of a 3 dimensional structure, however, the supporting mechanism is more complex than the 2-dimensional frame structure. To simplify, consider a 3 dimensional frame structure without slabs as shown in Figure 5.14 (a). The exaggerated deformed shape of the structure after beam BD elongated is shown in Figure 5.14 (b). In this case, in addition to the lateral stiffness of adjacent frames ABFG and DEIJ, the flexural stiffness of the beams AK, BL, DN and EO about their minor axes would participate to prevent lateral end displacements of beam BD. Similar to vertical frames ABFG and DEIJ, horizontal frames ABKL and DENO would provide lateral support to beam BD as shown in Figure 5.14 (b). The stiffness of the elements beyond axis B also
has an effect on the lateral constraint provided to beam BD but their effect is ignorable compared to that of the elements up to axis B.

Figure 5.15 shows the same 3 dimensional frame structure with slabs. Note that beams and slab are cast together. The effect of the slab as a lateral support to beam BD can be considered in two ways. The first effect would be the distributed constraint of the slab on beam BD through the shear between beam BD and the adjacent slab BDNL. When beam BD wants to elongate, the shear stresses would develop at the face of the beam as shown in Figure 5.16. In plane stiffness of the slab would resist the elongation of beam BD and, in turn, in-plane tensile stresses would develop in the slab.

To demonstrate the second effect of the slab on resisting elongation of beam BD, consider the same structure but with a gap between beam BD and adjacent slab BDNL. In this case there is no direct connection between beam BD and the slab. The exaggerated deformed shape of the structure after beam BD elongated is shown in Figure 5.17. If the slab were made of a very flexible material the deformed shape would be as shown in Figure 5.17. However, in plane stiffness (diaphragm effect) of the slab would prevent this deformation and joints B and D would not move as much as in the case of without slab. In plane stiffness of the slab would participate to prevent the axial deformation of beam BD even it is not directly connected to the beam.

After a certain amount of displacement, the ends of beam BD would stop moving outwards and start to move inwards. Figure 5.18 shows the horizontal displacements of joints B and D. As can be seen from the figure, the joints B and D come to their initial locations at around 0.12 inch of vertical displacement at Joint C. Figure 5.19 shows the deformed shape of the frame at 0.12 inches of vertical displacement at Joint C with a scale factor of 50. As can be seen from the figure, even though joints B and D came to their initial locations, it does not mean that beam BD has shrunken. In contrast, since the deformation of the beam is increased as joint C moved downwards, the curvatures in the beam sections went up and the strains at the center of the sections increased. In other words, the beam continues to grow as it deforms. The reason that joints B and D moved
inwards is the geometric compatibility. As joint C moved downwards, the horizontal component of the force developed in beams BC and CD pulled joints B and D inward.

The same constraint mechanism would prevent the horizontal movement of joints B and D as in the beam growth. For a two dimensional frame structure as shown in Figure 5.13(a), the adjacent frames ABFG and DEIJ would provide a constraint. For a three dimensional structure, in addition to adjacent frames, the floor system and the beams perpendicular to the deformed beam would participate to prevent the lateral movement of the beam ends. The analysis should include geometric nonlinearity (P-Delta effect and large displacements) to account for the axial tensile force developed in the beam.

5.4 Modeling Infill Walls

Although there may be different methods available to model infill walls with different levels of complexity, three methods are utilized in this dissertation. They are: using two-dimensional shell elements and compressive struts in Finite Element Models, and using cuboids with the connecting springs in Applied Element Models.

In the Applied Element Method, infill walls are modeled with the same type of element (rigid cuboids with the connecting springs) that is used for beams, columns and slabs. The size of the cuboids is selected to be equal to the size of the bricks. Different from the beams and columns, the connection between the cuboids is modeled with another material that represents the mortar between the bricks. It is assumed that cracking occurs only at the interface of the bricks and the grout and not through the bricks. It is assumed that mortar loses all of its tensile strength when it cracks. The analysis program tracks the stresses developed in the springs that models the mortar and if they reach the tensile strength of the material they simply disconnect since the post-cracking strength is set to zero for mortar.

In finite element models developed in SAP 2000, the infill walls are modeled with two different methods. The first method is using two dimensional shell elements and the second method is using diagonal compressive struts as suggested in FEMA 356.
When the shell elements are used to model the infill walls, similar to the cuboids size used in applied element models, the size of the shell elements are set equal to the size of the bricks that infill walls are made of. The thickness of the shells is set equal to thickness of the bricks. Different from the Applied Element Model, the shell elements and nodes that connect the shell elements to each other and to the adjacent columns and beams behave linearly. In other words, the cracking of the infill walls are not automatically accounted for as in the Applied Element Method (through connecting springs that follow the constitutive law of mortar). A similar approach used to model the cracking in the beams is applied to the infill walls.

i) The building is analyzed having all the shell elements connected to each other and to the beams and columns.

ii) The tensile stresses developed in the shell elements are compared with the tensile strength of the infill wall. The shell elements are disconnected at the joints where the tensile stresses exceed the tensile strength of the infill wall.

iii) Then the new structure is re-analyzed and the stresses in the shell elements are re-checked and the needed modifications are made.

This procedure is repeated until the cracking in the infill walls are properly modeled. Note that the definition of the crack pattern in the infill wall is made along with the definition of the cracked regions in beams, column and slabs since they interact with each other.

The other method to model the infill walls is using the diagonal compressive struts as suggested in FEMA 356. The diagonal compressive struts that represent the in-plane stiffness of the infill wall would have the same thickness and modulus of elasticity with the infill wall it represents. The width of the diagonal compressive struts can be calculated as suggested in FEMA 356 by considering different parameters of the infill wall. These parameters are column height between centerlines of beams, height of infill panel, expected modulus of elasticity of frame material, expected modulus of elasticity of infill material, moment of inertia of column, length of infill panel, diagonal length of
infill panel, thickness of infill panel and equivalent strut, and the angle whose tangent is the infill height-to-length aspect ratio. The locations and the orientations of diagonal compressive struts are based on the expected deformation of the frame that the infill wall is in and the pattern of the infill wall. For instance, fully infilled frames can be modeled with a single compression strut while the infill walls with openings like windows and doors can be modeled with two or more diagonal compressive struts. Again the location and the orientation of the compressive diagonal struts in the infill walls with openings would be based on the location of opening(s) in the infill and the expected deformation that frame would experience.

For the analytical evaluation of the Hotel San Diego building (see Chapter 3), both methods explained above (i.e. two dimensional shell elements and diagonal compressive struts) are used to model the infill walls existing in the building. When the global displacements obtained from both models were compared with the experimental data, it is found out that the results obtained from the model with two dimensional shell element are in good agreement with the experimental results while the model that the infill walls were modeled with diagonal compressive struts overestimates global displacements of the structure. In other words, the diagonal compressive strut model underestimated the in-plane stiffness of the infill walls.

One important difference between using the shell elements and using diagonal compressive struts is that the tensile stresses developed in the infill wall after the frame starts to deform are ignored in the diagonal compressive strut model. By using diagonal compressive struts, the constraint that infill walls put on the frame is reduced to only the compressive zone that develops in the infilled frame.

Another difference between the two methods is that, the constraint of the infill wall on the surrounding beams and the columns is distributed through the joints when the two dimensional shell elements are used. In the other case, it is concentrated to only the ends of the diagonal compressive strut. For instance, when a single compression strut is used to model a fully infilled frame, the compressive forces applied to the frame by the compressive struts would be at the joints where the beam and columns connect. Neither
column nor beams would have distributed loading along their lengths due to the infill wall.

For all of the modeling methods of the infill walls (e.g. using compressive struts, shell elements, etc.) in the Finite Element Method and the Applied Element Method, the lower-bound and expected material properties of the infill walls are estimated using the tables 7-1 and 7-2 in FEMA 356 (2000). These properties include masonry compressive strength, elastic modulus in compression, flexural tensile strength, and masonry shear strength. FEMA 356 classifies the masonry infill walls as in good, fair and poor condition. The condition of the masonry infill walls can be estimated based on the visual examination as suggested in FEMA 356.

5.5 Other Modeling Issues

5.5.1 Geometric Nonlinearity

In progressive collapse evaluation of the structures studied in this dissertation, two mechanisms are found to be dominant in resisting progressive collapse, namely Vierendeel action and Catenary action. Figure 5.20 and 5.21 show schematically Vierendeel and Catenary action on representative beams, respectively.

Figure 5.20 (a) shows the deformed shape of a representative beam under Vierendeel action schematically. As shown in Figure 5.20 (a) Vierendeel action can be characterized as the double curvature deformation of the beams bridging over the removed column, and redistribution of the loads that used to be carried by the removed column to the neighboring columns through the shear forces developed in the bridging beams. Figure 5.20 (b) shows the forces developed in the beams due to Vierendeel action after they deformed in double curvature. Note that the axial force in the beam is shown as a compression force considering small deformations and the effect of the beam growth as explained in the previous sections. After a certain level of displacement, the axial force in the beam would turn to tension from compression. To capture the double curvature deformation of the beam and corresponding load redistribution through shear forces in the beam, the geometric nonlinearity does not need to be included in the analysis.
However, the effect of the axial force on the moment in the beam would be ignored if P-Delta effect is not accounted in the analysis.

Figure 5.21 (a) shows the deformed shape of a representative beam under Catenary action schematically. As the displacement of the joint above the removed column increases, the slope of the beam increases and the participation of the axial load in the beam would be more and more. Figure 5.21 (b) shows the forces developed in the beams after the axial force in the beam turned to tension from compression. Note that the axial tensile force due to change of geometry starts to develop as soon as the joint above the removed column moves downwards. Since the axial tensile force developed due to the change of geometry is much less than the axial compressive force developed due to beam growth and the constraints at the beam ends at the beginning, the resulting axial force in the beam would be compression. As the slope of the beam increases, the axial tensile force in the beam will increase and overcome the compressive force due to beam growth. To capture the axial tensile force in the beam due to change of geometry, the analysis should include the large displacements. Without including large displacements in the analysis, axial tensile force would not be observed even if the slope of the beams is 100%.

### 5.5.2 Column Removal Procedure

The removal of the columns in the real buildings during the experimental program is performed by exploding the columns. For practical purposes, the removal duration can be assumed as instantaneous in the physical removal of the column.

In the analytical studies, some structural analysis programs (e.g. CSI Perform-Collapse and ELS) have the ability to remove one or more elements from the structure and analyze the dynamic response following the element removal. In the structural analysis programs that cannot remove the elements automatically during the analysis, the following procedure is used to simulate the element removal:

i) The structure is analyzed under gravity loads (dead and live loads) with all the elements. The internal forces (one axial force, two shear forces, two bending moments
and torsion) at the top end of removed column are obtained. This is performed for all the removed columns if there is more than one column removed. If a full beam or part of a beam is exploded too, then the internal forces at the both sides of the damaged part of the beam are obtained.

ii) A new model is developed by deleting removed columns and the damaged part of the beams from the previous model. Again the structure is analyzed under the same gravity (dead and live) loads. In addition to gravity loads, the internal forces obtained at the top of the removed columns in the previous step as well as the internal forces at the ends of the exploded beam parts are applied to the structure as external loads simulating the effect of those columns and beam parts that are non-existent in the model. Note that the results of this model and the model analyzed in (i) are identical.

iii) After gravity loads and end forces of removed columns and the beam parts are applied to the structure as explained in (ii), the external forces that have same magnitude but opposite direction with the end forces that are applied along with the dead loads are applied to the structure in the first time step of the time history analysis and the dynamic response of the structure is calculated. By applying the external forces that have same magnitude but opposite direction with the end forces that are applied along with the dead loads, the effect of the columns and the beam parts that is simulated by applying their end forces found in (i) is simply cancelled out.

Note that in step (ii) the gravity loads and the end forces of the removed elements are applied simultaneously. In this approach, the structure is assumed to behave linearly. If the structure behaves nonlinearly under the dead loads then applying element end forces and the gravity loads simultaneously would give different results compared to the analysis when all the elements are in place. In general, structures do not show nonlinear behavior under dead loads and the approach explained above can be used.
5.5.3 Effect of Remaining Rebars of Removed Columns in Experimental Studies

As explained in Chapter 3, the progressive collapse potential of the several buildings is studied experimentally as well as analytically. All the buildings studied experimentally were scheduled to be demolished by implosion. As a part of the demolition process, one or more columns are removed from the structure by explosion. This is done by instantaneous ignition of the explosives that are located in predrilled holes in the columns.

Figure 5.22 (a) and (b) show one of the removed columns in the experimental program before and after explosion. As can be seen from Figure 5.22 (b), the longitudinal reinforcement of the exploded column is not completely gone although the concrete fell to pieces almost all over the height of the column. The vertical rebars of the removed column are bent outwards due to the pressure of the explosion and vertical deformation of the top joint, however they are not cut. Note that the transverse reinforcement of the column could not resist the outwards bending of the vertical rebars and the pressure of the explosion and they are cut.

The analytical studies on the remaining bars of the removed column in the Memphis Memorial Hospital in Memphis, TN (See Section 3.4.4.5) showed that after explosion of the column and the vibrations in the structure stabilized, 13% of the axial load that used to be carried by the removed column before explosion was found to be carried by the remaining bars.

In addition to load carrying capacity of the remaining bars, they provide additional damping to the structure as well. Again, based on the analytical studies on the remaining bars of the Memphis Memorial Hospital, the equivalent damping ratio due to the nonlinear response of the rebars of the removed column is found to be about 0.15.

The remaining bars of the exploded columns in the experimental program can be observed, and the conditions of them can be considered in their modeling (e.g. if they are cut or not, how much they displaced outwards, etc…). Therefore, the effect of the
remaining bars of the exploded columns can be included in the analytical model of the structure if it is studied also experimentally. Including the remaining bars in the modeling would increase the accuracy of the analytical model for verification purposes with the experimental data collected. However, in the evaluation of the progressive collapse potential of a structure, that is done analytically only, the effect of the remaining bars can be ignored; since the condition of the bars cannot be estimated after an initial damage occurs.

5.5.4 Effect of Direct Air Blast in case of a Real Threat

In case of an explosion, the air-blast shock wave is the primary damage mechanism. Damage due to the air blast may be divided into direct air-blast effect and progressive collapse. Direct air-blast effect causes damage by the high-intensity pressure that induces localized failure to nearby buildings. Such localized failure depends on the size of the explosion, its distance to the building, and the building characteristics. Damaged structures may confine the damage to the initially affected zone, otherwise the collapse may propagate. In this dissertation, potential progressive collapse of structures following initial local failure is studied while direct air blast effects are ignored.

5.5.5 Previous Load Experience of Building

In the evaluation of progressive collapse potential of a structure, its previous load experience might have an effect in the response of the structure to removal of one or more load bearing elements. If the structure had experienced an earthquake or wind loading that caused a nonlinear behavior in the structure, then the response of such a structure would be different from the same structure that has not experienced a nonlinear behavior in the past.

The behavior of the structure under dead and live loads can be estimated analytically since the vertical loads on the structure are known. Although yielding is not expected in the elements under dead and live loads, cracking can be imposed to the structure prior to column removal if dead and live loads are causing any cracking in the elements. However the extent of the cracked regions in the structure may be much more
if the structure has experienced a strong ground motion (or wind loading). Ignoring this would cause the over estimation of the stiffness of the structure.

Moreover, if the yielding has occurred in the elements in the vicinity of the removed columns due to previous loading experience of the structure (e.g. earthquake or wind loads), those section may have residual plastic deformations. Having residual plastic deformation would decrease the ultimate deformation capacity of the element. Depending on the residual deformation on the element accumulated, the element may fail soon after it goes beyond the elastic behavior.

If the structure that is evaluated for progressive collapse potential had elements with residual deformations, ignoring this issue in the analytical modeling may cause unrealistic conclusions. Again, it would not make much difference to ignore the previous load experience of the structure if the residual deformations are negligible compared to the ultimate deformation capacity of the elements.

If it is known that the structure had experienced a severe ground motion (or severe wind loading) one can include this issue in the analysis if the earthquake recording (or wind load recording) is available for the site of the structure. To include the effect of previous loading, one can use the following approach:

i) The structure would be analyzed under dead and live loads.

ii) After applying gravity loads to the structure, a nonlinear time history analysis would be run using the recording of the earthquake (or wind) that the structure experienced before. After the earthquake (or wind) loading is finished, analysis may be continued to give enough time to the structure to get to a rest position (the state that all the vibrations would be damped out).

iii) Continuing after the earthquake (or wind) loading, another nonlinear time history analysis the selected element(s) can be removed from the structure and the response of the structure can be obtained.
Note that (ii) and (iii) follows the previous analyses. In other words, at the beginning of (ii) and (iii), the initial state of the elements comes from the end of the analysis (i) and (ii), respectively.

The procedure explained above can be used if the structural analysis program has the ability to remove an element during the analysis. If the program does not have the capability of automatic element removal, then the procedure explained in Section 5.5.2 can be used with some modifications to simulate removal of elements such that the analysis would include the effect of a previous loading on the structure prior to element removal.

The procedure to simulate the removal of elements from the structure can be used as explained in Section 5.5.2 because of two assumptions. First, the gravity loads are applied to the structure monotonically and second, it is assumed that the structure behaves linearly under the gravity loads.

Both earthquake and wind loading imposes cyclic deformation in the elements. Therefore, the values of the internal forces at the ends of the removed elements would be continuously changing during an earthquake or wind loading. Moreover the elements (either removed elements or the other elements) can show a nonlinear behavior under earthquake or wind loadings which is different than the gravity loads. To account for the effects of these two facts, the following approach can be used:

i) The structure would be analyzed under dead and live loads with the all elements in place.

ii) After applying gravity loads to the structure, a nonlinear time history analysis would be run using the earthquake (or wind) recording provided. After the earthquake (or wind) loading is finished, analysis may be continued to give enough time to the structure to get to a rest position (the state that all the vibrations would be damped out). After nonlinear analysis is completed, the internal force histories (one axial force, two shear forces, two bending moments and torsion) at the top end of removed columns can be obtained. If a full beam or part of a beam is exploded in addition to removed columns,
then the internal force histories at the both sides of the damaged part of the beam would be obtained too.

iii) A new model is developed by deleting removed columns and the damaged part of the beams from the previous model. Again the structure is analyzed under same load used in (ii) (earthquake or wind). In addition to earthquake (or wind) load, the internal force histories obtained at the top of the removed columns in the previous step as well as the internal force histories at the ends of the damaged beam parts are applied to the structure as external loads simulating the effect of those columns and beam parts that are non-existent in the model. Note that the results of this model and the model analyzed in (i) are identical even though structure shows a nonlinear behavior.

iv) After earthquake (or wind) loads and end forces of removed columns and the beam parts are applied to the structure dynamically as explained in (iii), the external forces that are applied to the ends of the removed elements in the last step of the dynamic analysis explained in (iii) are applied with the same magnitude but opposite direction to the ends of the removed elements in the first time step of a nonlinear time history analysis and the dynamic response of the structure is calculated.

Note again that the analyses explained in (ii) and (iii) follow the gravity load analysis explained in (i). The analysis explained in (iv) follows the nonlinear time history analysis explained in (iii). In other words, at the beginning of an analysis case, the initial state of the elements comes from its preceding analysis.

5.5.6 Symmetric - Asymmetric response

The 4th chapter of this dissertation presents experimental and analytical evaluation of a two dimensional 3 story – 4 bay RC frame structure which is subjected to removal of a ground floor column. The experiment was conducted in two parts: the first part was the instantaneous removal of a center ground floor column which was made of glass while all vertical loads (dead and 25% of live load) were applied to the structure. The second part was applying a monotonic vertical displacement to the joint at the top of the removed column after the vertical loads are removed from the structure. The maximum vertical
displacement applied to the structure was 16.25” at the end of the 2nd phase. Figure 5.23 shows the deformed shape of the structure at the end of the experiment. As can be seen from the figure, the deformed shape of the structure is not perfectly symmetrical about the center vertical axis. Moreover, the sequence of the cracking and yielding in the elements during the experiment were not symmetrical. There were several bars fractured during the experiment. The progression of the bar fractures in the elements also occurred asymmetrically.

The reason for the unsymmetrical behavior is the imperfections in the frame. The concrete is not a perfectly homogenous material. For instance, the slight change in the water to cement ratio, or the density of the aggregates in the different parts of the frame would cause different strength in the concrete. Note that the frame used in the experiment was cast in one shot. The imperfections in the dimensions of the elements could be another source of the unsymmetrical behavior. The imperfections in the loading mechanism can also cause unsymmetrical response. As for the analytical results, however, the response of the analytical model was symmetrical because the imperfections mentioned above are not included in the model.

Figure 4.69 shows the resisting force (i.e. applied force) versus applied displacement at the top of the removed column (Joint C2) obtained analytically as well as experimentally. Although analytical and experimental force deformation curves show a generally similar pattern, the sequence of the bar ruptures and related force reductions have difference. In the analytical model, all tensile rebars in a cross section rupture at the same time. In contrast, there was a time (vertical displacement) lag observed between the rupture of the tensile rebars of the sections in the experimental study. Furthermore, because of the symmetry, two sections on each side of the symmetry line of the frame (Axis C) lose their flexural strength at the same time in the analytical model. Again in the experimental program the sequence of the bar ruptures were not perfectly symmetric. Again, these can be explained with the imperfections in the tested frame. In the analytical results, since two sections lose their flexural strength at the same time (due to symmetry) and each section loses all tensile reinforcement at the same time, the correlated drop in
the resisting force is sharper than the experimentally observed drops, which are usually correlated with rupture of one rebar at a time.

Figure 5.24 shows the 3/8\textsuperscript{th} scaled model of a two span beam which represents two continuous beams bridging over a removed column (Bazan, 2008). The beam was tested under increasing monotonic vertical displacement at its center in a test setup as shown in Figure 5.25. Although all the detailing of the beam was designed as symmetrical, the response of the beam at the end of the test was unsymmetrical. Especially after the bottom reinforcement at the right side of the center stub ruptured, the rotations at the center zone are concentrated at the section where the bottom reinforcement ruptured (See Figure 5.26).

The same reasons mentioned for the two dimensional frame are valid for the beam test. The imperfections in the material, construction and the loading caused the system to behave unsymmetrical. The unsymmetrical behavior on both sides of the center line of the beam caused the center stub to have rotation and horizontal displacement. In other words, the rotation and the horizontal displacement of the center stub (see Figure 5.26) can be explained as the result of the unsymmetrical response of the beam.

To consider the unsymmetrical behavior in the analytical modeling, rotation and the horizontal movement of the center stub is utilized to mimic the asymmetry. Note that the rotation and the horizontal movement of the center stub are measured along with many displacements and deformations in the beam through intense instrumentation during the experimental program. Two different analyses were run to compare the effect of unsymmetrical behavior on the performance of the beam (Bazan, 2008):

i) In the first analysis of the beam, the rotation as well as the horizontal displacement of the center stub obtained from the experimental measurements is applied to the structure as the vertical displacement at the center is monotonically increased.

ii) In the second analysis, the same model is analyzed under only the monotonically increased vertical displacement at the center of the beam. Since the model and the loading are symmetric this model behaved perfectly symmetric.
Figure 5.27 and 5.28 compares the applied force and displacement at the center of the beam for two cases mentioned above. As can be seen from the figure, both models show similar behavior up to the first peak of resistance force. After the first peak both models show a descending branch in their applied force vs. displacement curves. The force –displacement curves of both the models starts to differ from each other after the first peak in resisting force. They show different patterns until the bottom bars next to the center stub rupture in the model that experienced symmetric response. This occurs at around 10” of vertical displacement at the center. After that, the applied force –vertical displacement relationships of both models show again a similar pattern (Bazan, 2008).

If the evaluation of a structure would be performed only analytically, then the model cannot include the asymmetric response since there would not be any information about the direction and the magnitude of the asymmetry beforehand, to impose to the model as it is done in the beam experiment mentioned above. Therefore, the imperfections in the structure may cause asymmetric response which may not be captured with the analytical modeling. If the structure and the damage do not have symmetry (e.g. Hotel San Diego) then the results of the analytical model would not be affected by this fact.

5.6 Summary

The analytical evaluation of structures subjected to progressive collapse includes various assumptions and issues in the modeling phase. An optimum analytical model of a structure can be defined as a model that is as simple as possible yet able to reflect the behavior reasonably well.

For the structure to be able to reflect the behavior in the system level reasonably well, the elements in the analytical model should be capable of reflecting the potential damage or failure mechanisms. For a structure that is evaluated for progressive collapse, foreseeing all of the potential failure mechanisms may not be possible at the beginning of the modeling process. As the behavior of the structure is understood after the preliminary analysis are evaluated, the model can be updated by considering the potential failure mechanism in the model and by paying attention to the critical components in the model.
For instance, the level of accuracy in the estimation of the material characteristics may play a critical role under particular circumstances. As discussed in the section 5.2.6, in the progressive collapse evaluation of The University of Arkansas Dormitory building, the tensile strength of the concrete played a critical role on the performance of the resisting mechanism (which was Vierendeel action for the case discussed) after element removal and prevented failure of the Vierendeel action. If the tensile strength of concrete were lower and the Vieren0deel action vanished due to the brittle failure of the sections which did not have top reinforcement (see section 5.2.6) then the structure might have experienced large displacements and further load redistribution mechanisms.

In the modeling of structural and nonstructural elements of a building, different types of elements and methods can be used. The performance of the method selected should be checked and verified to make sure that it reflects behavior of the element realistically. For instance, the infill walls of a structure can be included in the analytical model in different ways. In the evaluation of Hotel San Diego, the modeling of the infill walls that were present on the spandrel beams of the structure is achieved by using two different methods (i.e using two dimensional shell elements as well as using compressive struts) and the results are discussed (see Section 3.2). When the results of the two models are compared with the experimental data, it is found out that the modeling infill walls with compressive strut method caused underestimation of the contribution of the infill walls to building stiffness. The results of the model with the infill walls modeled with two dimensional shell elements showed a good agreement with the experimental data. It should be mentioned that the difference observed between the performances of these two ways of modeling of the infill walls are considerable for the cases such that the maximum displacements and the deformation of the frames where infill walls exist are small. The 2-dimensional shell elements were able to provide the constraints through the tensile zones that develop in the infill wall and transferred load to the frame through the distributed joints over the beams and columns. If the maximum displacements were more, however, the difference between the results of the two models could be closer since the constraint of the infill walls on the frames due to the tensile zones would became insignificant due to extensive cracking and the contribution of the compressive zones would be dominant in the participation of the infill walls on the building stiffness.
Another example can be given for the case of beam growth. As mentioned in 5.2.4, the beams need to be modeled in a way such that the strain distribution over the height of the section can be captured before and after cracking of the concrete since the beam growth is the result of the tensile strains develops at the center of the beam sections following the cracking of the concrete. For instance, beam growth and related compressive axial forces cannot be captured using two node linear beam elements.

It is shown that some limitations of the element type or software used to model the structure and evaluate the response to element removal can be overcome using alternative techniques. For instance, including concrete cracking in the model when two node linear beam elements are used to model structural elements such as beams, columns and slabs is achieved by following a progressive procedure explained in 5.2.1. Similarly, the cracking of the infill walls modeled with two-dimensional linear shell elements is achieved by following the procedure explained in 5.4. The procedure explained in 5.5.2 made it possible to simulate sudden element removal in software packages in which it cannot be performed automatically.
Chapter 5: Modeling Issues and Assumptions

Figure 5.1 Determination of cracked regions of a beam

Figure 5.2 Reinforcement detail and bending moment diagrams of 2nd floor beam A3B3 of Hotel San Diego
Figure 5.3 A representative cantilever beam

Figure 5.4 Moment diagram of beam and Moment-Curvature relationship of its section

Figure 5.5 Unloading of adjacent section (green) when bar rupture occurs (red)
Figure 5.6 A representative fixed ended beam

Figure 5.7 Schematic strain variation in cross Section-A (see Figure 5.3)

Figure 5.8 A representative 2-dimensional frame to demonstrate beam growth of partially fixed ended beam
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Figure 5.16 Demonstration of distributed constraint of slab on bridging beams over a removed column

Figure 5.17 Demonstration of constraint of slab on bridging beams over a removed column in case of a gap between beam and slab
**Figure 5.18** Horizontal displacements of joints B and D (See Figure 5.10(a))

**Figure 5.19** Deformed shape of frame shown in Figure 5.10(a) at 0.12” vertical displacement
Figure 5.20 Demonstration of Vierendeel action

Figure 5.21 Demonstration of Catenary action
Figure 5.22 One of removed columns in experimental program (a) before and (b) after explosion.
Figure 5.23 Deformed shape of frame structure (See Chapter 4) at end of experiment

Figure 5.24 3/8th scaled model of a two span beam which represents two continuous beams bridging over a removed column (Bazan, 2008)
Figure 5.25 Beam test setup (Bazan, 2008).

Figure 5.26 Deformed shape of beam (center portion) (Bazan, 2008).
Figure 5.27 Force deformation of beam tested (unsymmetric loading) (Bazan, 2008).

Figure 5.28 Force deformation of beam tested (symmetric loading) (Bazan, 2008).
Chapter 6

Analytical Evaluation of Response of an RC Building under Different Initial Damage Scenarios

6.1 Introduction

As discussed in Chapter 2, the two current guidelines to evaluate the progressive collapse potential of existing and new designed buildings, “Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects” by GSA (2003) and “Unified Facilities Criteria (UFC): Design of Buildings to Resist Progressive Collapse” by DOD (2010a), provide a threat-independent approach in which the structure is required to be analyzed and evaluated for the case of an instantaneous loss of a primary vertical support (e.g. the removal of a column or a vertical load bearing element such as a shear wall). They provide suggestions for the location of elements to be removed on the plan of the structure and stories to be considered for element removal. In this chapter a seven story RC building is first designed and then analyzed under 15 different initial damage scenarios which are selected based on the suggestions in the guidelines mentioned above. The design parameters used and details of the structure as well as the initial damage scenarios used for the progressive collapse analyses are given in the following sections. The response of the structure for each initial damage scenario is investigated and load redistribution mechanisms are characterized. The performance of the structure to resist progressive collapse is discussed based on the results and comparisons.

The guidelines mentioned above allow the use of linear analysis techniques (e.g. linear static or linear dynamic) for the evaluation of progressive collapse potential of the structure in question. As also stated in the guidelines, however, the use of linear analysis
techniques is intended to determine the potential for progressive collapse (i.e., a high or low potential for progressive collapse), not to predict the response of the structure when it is subjected to the instantaneous removal of a primary vertical element. Therefore a nonlinear dynamic analysis technique in which the material and geometric nonlinearities are accounted for is used for all the progressive collapse analyses presented in this chapter. The details about the analytical modeling of the structure are explained in detail in the following sections.

6.2 Building Characteristics

The building evaluated in this Chapter is a newly designed 7 story RC structure. The building stands on a base area of 60 ft by 156 ft. The clear height of the first story is 11'-10” and that of the other floors is 9'-10”. The height of the first floor is designed to be 2 in more than that of the other floors to provide more open space at the entry level of the building. The building has 6 spans in longitudinal direction with an equal span length of 26 ft and two spans in transverse direction with an equal span length of 30 ft. The span lengths have ± 2 in flexibility for construction purposes. Figure 6.1 shows the typical floor plan and the typical size of the structural elements (beams, columns and joists). Elevation views of the Axis A and Axis 1 are shown in Figure 6.2. The size of the beams, columns, joists and slab thickness are kept same over the height of the structure to prevent changes in the formwork size and by doing so provides economical construction.

The floor system is a one way joist floor with a 4 in thick slab running in transverse direction supported by longitudinal beams. The 4 in slab provides 1.5 hour fire resistance. Joist ribs are 9 in wide and the spacing between joists is 66 in. The 66 in spacing is selected by considering the use of standard form dimensions to reduce the construction cost. The depth of the joists is 20 in, equal to the depth of the supporting longitudinal beams. The cross section of joists and transverse beams between Axes 1 and 2 can be seen in Figure 6.3. The reinforcement detail of a typical joist is also shown in figure 6.3. The same reinforcement is used for all joists in the structure.

All external beams are 18 in wide and 20 in deep. All internal beams are 22 in wide and 20 in deep. Having the same depth for external beams, internal beams and joists
allows forming the bottom of the entire floor system in one horizontal plane which reduces the installation costs for utilities, partitions, and ceilings (Alsamsam and Kamara, 2007). The depth of the beams and joists are selected based on the ACI requirements to control deflections (ACI 318, Item 9.5). All external columns have a cross section of 18 in by 18 in while all internal columns have a cross section of 22 in by 22 in. The widths of the beams are selected to be equal or greater than the column width in the direction that the beams are connected to provide economical formwork.

The building is designed according to ACI 318 (2011) and IBC (2009). The load combinations provided in ASCE/SEI 7-10 are implemented in the calculation of demands of the structural elements to be used in design. In the design of columns, live load reduction is accounted for per Section 4.8 of ASCE/SEI 7-10. The design lateral loads are calculated based on the International Building Code (2009). The equivalent lateral force procedure is used to calculate the demand of the structural elements due to earthquake. The seismic response coefficient used was 0.0534 considering ordinary moment frames as the lateral resisting system of the structure. Ordinary RC moment frames make up the structure’s lateral resisting system in both the longitudinal and transverse directions. The building does not have any shear walls.

The dead load on the structure consist of self weights of the beams, columns and a floor load of 100 lb/ft$^2$ which includes the weight of the slab, joists, partitions, ceiling etc. Live load is assumed to be 50 lb/ft$^2$. The uniformly distributed 100 lb/ft wall weight on the perimeter beams is also accounted for.

The concrete used in design is assumed to have a compressive strength of 5 ksi ($f'_c$) and has a unit weight of 150 lb/ft$^3$. The modulus of elasticity of the concrete is estimated to be 4286 ksi based on the unit weight and compressive strength of concrete (ACI 318, Item 8.5.1). The reinforcement used in the design of structural elements assumed to have a yield stress of 60 ksi ($f_y$).

The reinforcement pattern of the beams is changed in three levels: one pattern for the second and third floor, one pattern for the fourth and fifth floors and another pattern for the sixth floor and above. The dimensions of the beams are kept same over the height.
Figure 6.4 and 6.5 show the reinforcement detail of exterior longitudinal and exterior transverse beams. Similarly Figure 6.6 and 6.7 show the reinforcement detail of interior longitudinal and interior transverse beams.

Similar to beams, the dimensions of the columns are kept same over the height of the building but the reinforcement pattern is changed in three levels: one pattern for the first floor, one pattern for the second, third and fourth floors and another pattern for the fifth floor and above. Figure 6.8 shows the reinforcement detail of exterior and interior columns.

In the design of beams and joists, integrity requirements are accounted for per Section 7.13 of ACI 318. The requirements aim to improve the redundancy and ductility in the structure so that a potential damage to a major supporting element or an abnormal loading may be confined and will not propagate and result in partial or total collapse of the structure.

### 6.3 Selection of Initial Damage Scenarios

GSA guidelines (2003) classify the structures based on their configurations into two categories: typical and atypical structures. Typical structures are defined as having a relatively simple layout with no atypical structural configurations. Combination structures, vertical discontinuities, variations in bay size/extreme bay sizes, plan irregularities and closely spaced columns are considered as examples of possible atypical structural configurations. According to these definitions, the structure evaluated in this chapter falls into the “typical” structures category.

The GSA guidelines propose the following scenarios to be used in the assessment for progressive collapse of framed or flat plate structures with typical structural configurations and without an underground parking or uncontrolled public ground floor areas.

1. Instantaneous loss of a first floor external column located at or near the middle of the short side of the building.
2. Instantaneous loss of a first floor external column located at or near the middle of the long side of the building.

3. Instantaneous loss of a first floor external column located at the corner of the building.

The guidelines require all columns to be removed in the first story (above the grade) only.

The DOD guidelines provide the same locations of the columns to be removed for an initial damage scenario as the GSA guidelines in plan layout if there is not an underground parking or uncontrolled public ground floor areas. However, DOD guidelines require that the removal of the columns in the locations in the plan mentioned above be repeated at different elevation levels listed below:

1. First story above grade
2. Story directly below roof
3. Story at mid-height
4. Story above the location of a column splice or change in column size

Based on the initial damage scenarios suggested in both GSA and DOD guidelines, the following scenarios are selected and implemented in the progressive collapse evaluation of the structure explained in 6.2. Five different column locations, four exterior and one interior, are selected in the plan layout as follows:

1. Column C-4 (See Figure 6.9) (the middle of the long side of the building)
2. Column C-2 (See Figure 6.9) (Adjacent to the corner of the building)
3. Column C-1 (See Figure 6.9) (At the corner of the building)
4. Column B-1 (See Figure 6.9) (the middle of the short side of the building)
5. Column B-4 (See Figure 6.9) (interior)
The locations of the columns to be removed in the plan layout are all shown in the Figure 6.9. These five locations are repeated for the following story levels:

1. First floor (above the grade)
2. Fourth floor (mid-height of the building)
3. Seventh floor (directly below the roof)

In summary the building is analyzed under 15 different initial damage scenarios which are listed below and shown on the 3 dimensional view of the building in Figure 6.10:

1. **Scenario 1-1**: Removal of column C-4 in the 1st floor.
2. **Scenario 1-4**: Removal of column C-4 in the 4th floor.
3. **Scenario 1-7**: Removal of column C-4 in the 7th floor.
4. **Scenario 2-1**: Removal of column C-2 in the 1st floor.
5. **Scenario 2-4**: Removal of column C-2 in the 4th floor.
6. **Scenario 2-7**: Removal of column C-2 in the 7th floor.
7. **Scenario 3-1**: Removal of column C-1 in the 1st floor.
8. **Scenario 3-4**: Removal of column C-1 in the 4th floor.
9. **Scenario 3-7**: Removal of column C-1 in the 7th floor.
10. **Scenario 4-1**: Removal of column B-1 in the 1st floor.
11. **Scenario 4-4**: Removal of column B-1 in the 4th floor.
12. **Scenario 4-7**: Removal of column B-1 in the 7th floor.
13. **Scenario 5-1**: Removal of column B-4 in the 1st floor.
14. **Scenario 5-4**: Removal of column B-4 in the 4th floor.
15. **Scenario 5-7**: Removal of column B-4 in the 7th floor.

Scenario names are selected to show the location of the removed column in the plan layout and the floor it is removed from. For example, in the scenario name 1-4 first number “1” indicates the location of the column in the plan layout (column C4) (see
Figure 6.9), and the second number “4” indicates the floor number the column is removed (4th floor).

### 6.4 Modeling for Progressive Collapse Analysis

Three dimensional finite element models of the building are developed in SAP2000 (2011) and CSI Perform-Collapse (2007). CSI Perform-Collapse is used only for Scenarios 3-7 and 4-7 (the scenarios which lead to collapse) due to the better convergence achieved. All of the analyses for column removal are performed using nonlinear dynamic analysis technique. Given the fact that nonlinear analysis is a computationally demanding process, the analytical model of the building is modified for each scenario to optimize the analysis. That is for each case the elements in the vicinity of the removed column are modeled with nonlinear elements and the rest of the structure where no significant demand is expected after column removal is modeled with linear elements. The detailed explanation of the models for each scenario is explained in the following sections.

The types of the elements (e.g. linear / nonlinear) used to model the structural elements in a floor level for each scenario are shown in Figure 6.11. The locations of the removed columns on the plan are also shown. The different color pattern is used to point out the nonlinear elements. The patterns shown in Figure 6.11 are used in one floor below (if applicable) and all the floors above the removed column along the height of the building. Other floors are modeled with linear elements only. For instance, first and second floors are modeled completely with linear elements and the pattern shown in Figure 6.11 used for the third floor and up when the removed column was on the fourth floor. Similarly, the columns in one story below (if applicable) and all the stories above the removed column are modeled with nonlinear elements. On the plan, the columns located inside the extent of the red regions shown in Figure 6.11 are modeled with nonlinear elements. The types of the elements used in the linear and nonlinear regions are explained below.

In both software programs, the joist floor system is modeled with 2-node linear Bernoulli beam elements which represent the joist ribs and two dimensional 4-node linear
shell elements which represent the slab between the joist ribs in the linear regions. All beams and columns are also modeled with 2-node linear Bernoulli beam elements.

In the nonlinear regions, the floor system including the slab is modeled with a grid of beam elements as can be seen in Figure 6.11. In the transverse direction 7 beam elements per span are used to model the floor system. Three of the beam elements represent the joists and other four elements represent the portion of the slab between joists and beams. The elements representing the joist are modeled with T sections to account for the effect of the slab. The effective flange width is selected as four times the slab thickness as suggested in ACI 318 (2011). In the longitudinal direction, 4 beam elements with a rectangular section per span are used to model the slab. External beams are modeled with L sections while the internal beams are modeled with T sections to include the effect of slab. Similar to joists, the effective flange width is selected as four times the slab thickness on each side of the beams.

In SAP2000, the material nonlinearity was imposed to the analytical models by defining the cross-sections of the elements with fiber sections and assigning localized plastic hinges to these elements at the critical locations with a defined plastic hinge length. Similarly, in CSI Perform-Collapse, nonlinear beam elements with fiber section are used. While defining sections of joists, slab, beams and columns, each section is divided into a two dimensional array of axial fibers. Over the section, each fiber has a location, a tributary area and a material which has a specified constitutive law (stress-strain relationship). The appropriate material properties (unconfined concrete, confined concrete) are assigned to fibers based on the location of the fiber on the section. The steel rebars are also represented with separate fibers. Figure 6.12 shows the stress strain relationships of unconfined (cover) and confined concrete (core). Characteristics of the core concrete are calculated based on Mander et al. (1988). Figure 5 shows the stress strain relationship of the steel. Under large deformations, the compressive rebars of an RC section are susceptible to buckling, which occurs between two transverse hoops. Considering the large deformations (potentially) in progressive collapse analysis, the buckling of compressive rebars is accounted for. The model developed by Urmson and Mander (2012) for the compressive behavior of longitudinal reinforcing bars is used to
calculate the stress-strain relationship of steel under compression as shown in Figure 6.13.

The plastic hinges used in the analytical models are P-M2-M3 type hinges which accounts for the interaction between axial load and the bending moments. The axial force (P) and moments (M2 and M3) are calculated by integrating the axial stresses of the fibers over the section. Strength loss in a fiber hinge is determined by the strength loss in the underlying stress-strain curves. Given the fact that all the fibers in a cross section do not usually fail at the same time, it is more likely to exhibit gradual strength loss for a fiber hinge compare to the hinges with directly specified moment-rotation curve (SAP2000, 2011).

As mentioned in section 5.2.4, the beam growth, defined as the elongation of the beams as a result of the post cracking deformations in the beam sections along the length of the beam, has an effect on the flexural behavior of the beams especially those that are bridging over the removed columns. As stated in the same section, to be able to capture the beam growth and its effects in an analytical model, the beam cross sections need to be modeled in such a way that the variation of the strain over the height of the section due to both flexural and axial deformations can be captured. As a secondary effect, the compressive force develop in the beam due to beam growth imposes additional bending moment to the beam due to P-delta effect (See Chapter 5, Item 5.2.4). To be able to take account of this effect of beam growth in the analytical model, the geometric nonlinearity needs to be included in the analysis too.

Note that different from distributed plasticity, modeling beams with localized fiber hinges allows including the beam growth in the model only for a certain segment of the beam which is simply the plastic hinge length assigned to the hinge. To account for the beam growth over the entire length of the beam, fiber plastic hinges are distributed over the whole length of the beam. At the critical regions such as the ends of the beam and bar cut-off locations, the plastic hinge length is selected as half of the beam depth as is the spacing between the hinges. At these critical regions, at least three plastic hinges with a plastic hinge length and spacing of half of the beam depth were assigned. Apart
from these critical regions, the spacing between hinges varies and is usually more than half of the beam depth. However, it is ensured that enough plastic hinges are assigned to the beam along its length to approximate the distributed plasticity by using localized plastic hinges. The plastic hinge lengths of the hinges are selected based on their tributary lengths such that the whole length of the beam is covered. Note that the same modeling approach is used for the beam elements that model the floor system as well. In other words, the potential growth in the slab and joists after column removal are also included in the analyses.

In SAP2000, one issue that needs to be paid attention to while using fiber plastic hinges is that the fiber hinge imposes additional flexibility to the beam even in the elastic range. This additional flexibility is the product of curvature of the fiber section and the assigned plastic hinge length. Therefore the flexibility of the beam segment represented over the length of a specific fiber hinge is counted twice. Note that flexural (M3 only) hinges which are used in the analytical models of the other structures represented in the other chapters of this dissertation, have a perfectly rigid-plastic behavior and do not cause the problem mentioned above. Prior to reaching yielding, all deformation is linear and occurs in the frame element itself, not in the hinge. Plastic deformation beyond yielding occurs in the flexural hinge (M3 only) in addition to any elastic deformation that may occur in the frame element. The following solution is used to overcome this issue. The axial and flexural stiffness of the beam sections where distributed fiber hinges are used are increased by a large number by using section property modifiers in SAP2000. After yielding, the source of the axial and flexural flexibility of the beam is limited to fiber hinges which cover the entire length of the beam.

Since the elements which are expected to experience a noteworthy change in their demands after column removal are modeled with fiber sections, the cracking of the concrete in these elements is automatically accounted for. That is the stresses in the fibers are calculated in each time step of the analysis and when the stress in a concrete fiber reaches the tensile strength of concrete, that fiber loses its tensile strength.
A mass-proportional damping ratio of $\xi=0.05$ in the first vertical vibration mode is used in SAP2000. CSI Perform-Collapse does not include damping in the analysis. To evaluate the effect of damping, a concentrated viscous damper equivalent to the damping of the first mode with a damping ratio of $\xi=0.05$ is externally added to the model. The results, however, are almost identical to the analysis results of the structure without damping.

### 6.5 Loading Combinations to be used in the Column Removal analyses

The following gravity load combination is applied to the structure prior to column removal as suggested in GSA guidelines (GSA, 2003).

\[
\text{Load} = DL + 0.25LL
\]

Where, DL is dead load and LL is live load. The 0.25 load factor for live load is used to represent expected live load on the structure in an event of possible progressive collapse.

Since instantaneous element removal is not readily available in SAP2000, the procedure presented in 5.5.2 is used to simulate column removal. That is each model is analyzed under the gravity load combination mentioned above and the end forces of the column to be removed are calculated. After the selected column is removed from the analytical model, the structure is re-analyzed under the same gravity load combination. Different from the first gravity analysis, the column end forces are applied to the end joints of removed column simultaneously along with the gravity loads in a nonlinear static analysis. As a successor to this static analysis, a dynamic time history analysis is carried out by applying the opposite end forces of the removed column in 0.005 seconds. Time history analysis is continued for 1 second which gives enough time to the structure to reach an approximately stabilized state.

GSA guidelines suggest that the vertical supporting element should be removed instantaneously or, if it is not possible, over a time period that is no more than 1/10 of the period associated with the structural response mode for the vertical element removal. For
each model developed for each scenario, a separate modal analysis is carried out. The results show that the periods of the first dominant modes associated with the structural response mode for the vertical element removal vary between 1.878 and 2.579 seconds. Therefore the duration of 0.005 seconds, that is used to apply the opposite end forces of the removed element, satisfies the requirement stated in GSA guidelines.

6.6 Progressive Collapse Analysis Results

6.6.1 Global Response

Analysis results of the 15 column-removal scenarios show that the 7-story RC building was able to redistribute the loads to the neighboring elements of the removed column and resist progressive collapse for 13 of the 15 scenarios. The two scenarios in which the building could not reach an equilibrium state and resulted in collapse are Scenarios 3-7 and 4-7, which are the removal of top floor columns on the corner and the middle of the short edge of the building, respectively (See Figure 6.10). The maximum vertical displacements for the 13 scenarios that the structures resisted collapse were limited to 4.1 in. Figure 6.14 shows the displacement histories of joints at the top removed column for each of 13 scenarios. Table 6.1 summarizes the peak and permanent displacements of the joints at the top of the removed columns for each scenario. For all 13 scenarios, the time at the peak displacement is between 0.2 to 0.25 seconds after column removal and most of the vibration is damped out around one second after column removal.

6.6.2 Load Redistribution Mechanism

In this section, the load-resisting mechanisms of the 13 scenarios which do not lead to collapse are discussed. The behavior of the structure under Scenarios 3-7 and 4-7 will be presented in detail in the following sections separately. For each scenario, deformed shapes, as well as the internal force diagrams (e.g. moment, axial force, shear force and torsion) of the elements in the vicinity of the removed column before and after column removal are investigated to identify the load redistribution mechanisms developed following column removal. The dominant load resisting mechanism for all
scenarios is found to be the Vierendeel Frame Action of the frames in the longitudinal and transverse directions connected to the joints above the removed column as well as that of the joists along with the moment-axial force interaction of beams. It is discussed below and shown clearly that axial force developed in the beams due to beam growth increase the beams flexural capacity significantly, and in turn prevent the structure from collapsing.

In all 13 scenarios in which the structure reached an equilibrium state, the beams connected to the joints above the removed column are deformed in double curvature. This observed double curvature deformation is the characteristic of the Vierendeel Frame Action. Vierendeel Frame Action develops when there is a relative vertical displacement between beam ends causing double curvature deformations of beams with the interaction of columns. Such a deformed shape provides shear forces in beams (or floor elements) to redistribute gravity loads to the neighboring elements following column removal.

It is observed for all 13 scenarios that axial forces in the columns above the removed column drop to almost zero rapidly following the removal of the column. This fact indicates that there is not load transfer between floors through the columns above the removed column. In other words, the columns above the removed column do not push the floors down; rather each floor redistributes its own load.

Since the transverse beams connected to the joints above the removed column (for Scenarios 1-1, 1-4, 2-1, 2-4, 3-1 and 3-4) and the longitudinal beams connected to the joints above the removed column (for the Scenarios 3-1, 3-4, 4-1 and 4-4) are not continuous, these beams interacted with columns to form the Vierendeel Frame Action. Figure 6.15 shows the moment diagrams of exterior longitudinal beams and exterior columns of Axis C for Scenario 3-1. Moment interaction of beams with columns can be clearly seen in the figure. Note that the transverse beams connected to the joint above the removed column for Scenarios 1-7 and 2-7 are neither continuous over the removed column nor did they have a column to interact with. However, the torsional stiffness of exterior longitudinal beams in the perpendicular direction provides the transverse beams enough restraint to develop positive bending moment and in turn, they deform in double
curvature. Note that exterior longitudinal beams in Scenarios 1-7 and 2-7 do not crack under torsion due to small displacements and deformations.

All elements in the vicinity of the removed column participate in the load redistribution. The contribution of each element to the redistribution of loads can be estimated calculating the change in their shear forces at the edges shown in Figure 6.16 with thick dashed lines. The beams, joists and slabs participating in the load redistribution are highlighted with red-toned colors. The contribution of different types of elements (e.g. beams, joists and slabs) in the load redistribution for all scenarios is approximately the same. On average, 65-75% of the load is redistributed by the longitudinal and transverse beams that are directly connected to the joints above the removed column to the neighboring columns in all scenarios. Around 25% of the load is redistributed by the joists in the vicinity region to the beams that surround the vicinity region. The insignificant remainder of the load is transferred by the slabs. These percentages demonstrate that the dominant load transfer mechanisms observed in the structure are the Vierendeel Frame Action and axial force-moment interaction of longitudinal and transverse beams as well as that of the joists.

6.6.3 Beam Growth and its Effect on Flexural Behavior

The beams listed in Table 6.2 that are bridging over the removed column experienced axial compressive forces. The axial compressive forces in these beams indicate beam growth. Beam growth can be defined as the elongation of a beam as a result of the post-cracking deformations in the sections along the length of the beam. The axial compressive forces develop in the growing beam when it is restrained from elongation. For the beams listed in Table 6.2, the lateral support provided by the neighboring frames (where available) and the in-plane action of the floor provided restraint to the beam elongation.

As an example, the axial force diagram of the beams and the floor elements in the second floor for Scenario 1-1 is plotted in Figure 6.17 (a). Figure 6.17 (a) is representative for the axial force diagrams of the floors above the removed column for all three Scenarios 1-1, 1-4 and 1-7. The compressive force in the longitudinal exterior beam
and tensile axial force in the elements modeling the floor in the longitudinal direction can be clearly seen in the figure. As the beam tends to grow, in-plane action of the slab as well as the neighboring frames resist and as a result axial tensile forces develop in the slab in the direction parallel to the beam. Figure 6.17 (b) shows the axial force diagram of the beams and the floor elements on the fifth floor for Scenario 3-4. Note that even though the exterior longitudinal beam C1-C2 and exterior transverse beam B1-C1 are constrained by an adjacent frame only on one side, the restraint provided by the in-plane action of the floor is enough to develop axial compressive forces in the beams.

As the beams mentioned in Table 6.2 deform, due to the change of geometry, axial tensile forces are expected to develop in the beams (Catenary Action). However, for small displacements, the compressive axial forces due to beam growth govern over the tensile axial forces owing to change of geometry. In other words, Catenary Action could not form in the structure for the 13 scenarios discussed in this section since it requires the structure to experience large displacements and the displacements are small in the structure for the given scenarios (e.g. around 4 in at peak).

To evaluate the effect of the axial compressive forces developed in the beams due to beam growth on their flexural behavior, the moment values at the time of yielding for the beam sections which experienced yielding are compared with the yield moments of the same sections obtained from a section analysis without considering any axial load. It is noticed that the flexural performance of the beams are significantly improved due to the axial compressive force developed in beams through axial force-moment (P-M) interaction.

For instance, in Scenario 1-1, the section at the C4 end of beam C3-C4 in the sixth floor yields at a positive bending moment (tension at the bottom) of 2866 kip-in while the axial force on the section is 148 kips. The section analysis of the same section considering zero axial force results in a yield moment of 1425 kip-in. The compressive axial force on the section increases the yield moment from 1425 kip-in to 2866 kip-in (101% increase).
It can be deduced that the compressive axial force developed in the longitudinal exterior beams due to beam growth and the end constraint provided by the in-plane action of the floor system and by the neighboring frames, interacts with the moment and enhances the flexural behavior of the beams, and in turn improves the performance of the Vierendeel Action. Note that the positive effect of axial forces developed in the beams on their flexural behavior would be neglected if the modeling of the beam elements was not capable of capturing beam growth (e.g. using flexural only plastic hinges with linear beam elements).

### 6.6.4 Results of Scenario 3-7 (Removal of Top Floor Corner Column)

#### 6.6.4.1 Flexural Behavior

The results of a nonlinear dynamic analysis of the structure under Scenario 3-7 (See figure 6.10) show that the structure is not capable of redistributing the loads and fails to reach an equilibrium state. It is shown for the 13 scenarios that do not lead to collapse that beams connected to the joints above the removed column transferred most of the load (on average 65-75%) through Vierendeel Frame Action along with the effect of axial force-moment interaction in the beams. The analysis did not converge beyond 30.6 in of vertical displacement at corner joint C1 on the roof. CSI Perform-Collapse is based on an event-to-event strategy. The analysis is terminated when the displacement at corner joint C1 on the roof is 30.6 in since the number of events in a time step exceeded the maximum limit. At this level of displacement, the top rebars of transverse beam B1-C1 at the face of column B1 have already ruptured, having a maximum strain of 0.17. The bottom rebars of longitudinal beam C1-C2 have buckled under compression and lost almost all of their strength at the face of column C2, having a compressive strain of 0.14, while the maximum tensile rebar strain is around 0.07 (close to rupture strain). Also, severe confined concrete crushing occurred at both sections. Thus the two primary load transferring beams after column removal, exterior transverse beam B1-C1 and exterior longitudinal beam C1-C2 failed at their B1 and C2 ends, respectively. Figure 6.18 shows the displacement history of joint C1 on the roof (See Model A in the figure). The level of damage that occurred in the two primary load transferring elements along with the
increasing slope of the displacement-time history suggests that the structure is likely to collapse.

The bending moment diagram of longitudinal beam C1-C2 before column removal (under gravity loads) and 0.45 sec after column removal are shown in Figure 6.19 (a) and (b), respectively. The moment diagram of the beam at 0.45 sec indicates double curvature deformation. Note that the beam is neither continuous over joint C1 nor is it connected to a column at joint C1. The developed positive moment at its C1 end is due to the rotation restraint provided by the torsional stiffness of the transverse beam B1-C1 (See Figure 6.1). The torsional cracking in the transverse beam B1-C1 occurs 0.46 seconds after column removal. Beam B1-C1 is assumed to have only 5% of its uncracked torsional stiffness after cracking (Valipour and Foster, 2010). Figure 6.19 (c) shows the moment diagram of beam C1-C2 0.5 sec after column removal (around 0.05 sec after cracking of transverse beam B1-C1 under torsion). The flexural wave developed due to sudden drop in the moment at the C1 end of the beam can be seen in the figure. Figure 6.19 (d) shows the moment diagram of the beam 1.35 sec after column removal when the fluctuations due to the flexural wave related to the sudden drop have diminished. Comparing Figure 6.19 (b) and (d), after the transverse beam B1-C1 cracks under torsion, and in turn, the rotational restraint at the C1 end of beam C1-C2 is released, the beam behaves like a cantilever as opposed to functioning in Vierendeel Frame Action. This change imposes more demand at the C2 end of the beam and eventually results in failure of the end section. The change in the deformation pattern due to torsional cracking of the beam in the perpendicular direction as explained above is also valid for the transverse beam B1-C1.

To demonstrate the importance of Vierendeel Frame Action as a vital resisting mechanism, an additional scenario is considered that is the removal of corner C1 column from the sixth floor. After column removal, beams B1-C1 and C1-C2 on the seventh floor and roof with seventh floor column C1 form the Vierendeel Frame Action and redistribute the loads to neighboring columns. The maximum displacement in the structure is limited to only 3.2 in. Based on this, it can be deduced that the capability of structure to develop Vierendeel Frame Action is crucial in resisting progressive collapse.
Moreover, it is verified and important to mention that if the torsional cracking of the elements was mistakenly neglected in the analysis under Scenario 3-7, then the structure would be able to redistribute the load of removed column and stabilize. Although in such a case, the resistance of the structure to progressive collapse would be overestimated but it further demonstrates the importance of the Vierendeel Frame Action.

### 6.6.4.2 In-Plane Behavior

As the corner joint C1 moves downwards after column removal, axial forces develop in the transverse and longitudinal exterior beams as well as in the joists and slabs in the corner panel. The axial forces are the result of (a) beam growth, (b) geometric compatibility, (c) equilibrium. These will be explained in the following paragraphs.

Figure 6.20 shows the elements in corner panel B1-B2-C1-C2. Figure 6.21 (a) shows the axial force histories of the longitudinal exterior beam C1-C2 segments (See Figure 6.20) versus vertical displacement of joint C1. Each segment represents a portion of the beam between the joints where the joists and slab elements parallel to the joists connect to the beam. The plot can be divided into three regions for evaluation purposes. In the first region (between 0 and 0.4 in of vertical displacement) axial tensile forces develop in all exterior longitudinal beam segments. Note that up to about 0.4 in of vertical displacement no concrete cracking is observed along the beam. Therefore the only source of axial force in the beam is due to geometric compatibility. As the C1 end of the beam moves downwards after column removal, joint C1 tends to move inwards (towards C2) due to geometric compatibility. However, the horizontal movement of joint C1 is restrained by the transverse exterior beam B1-C1 and also by the floor elements in the perpendicular direction which provide resistance through shear forces about their minor axes (in plane action) and in turn tensile axial force develops along beam C1-C2.

After about 0.4 in of vertical displacement, concrete cracking in the beam causes the beam to grow. That is after cracking the neutral axis moves towards the compressive side causing tensile strains at the center of the section. At the yielded sections, the tensile strains at the center further increase. The integration of the tensile strain at the center of the section along the length of the beam element results in an elongation of the element.
As a result of the beam growth, the ends of a beam segment tend to move away from each other. Since the elongation of the beam segment is restrained by the adjacent beam segments and more importantly by the in-plane action of the floor system, they develop compressive axial forces.

Figure 6.22 shows the axial force of the exterior longitudinal beam segments as well as the slab segments on lines LS-A and LS-B, (See Figure 6.20) which are parallel to exterior longitudinal beam C1-C2, at 6.8 in of vertical displacement (at the peak axial compressive force in the beam). Only the first three segments from the C2 end of the beam experience elongation due to concrete cracking and yielding of tensile rebars. The amount of elongation in the first segment is around 0.06 in at peak. The axial compressive force in the beam segments beyond the first three segments from the C2 end, however, is due to the effect of the elongation of the first three segments. That is, even if a beam segment do not experience beam growth, which is the cause of the axial compressive force in the first three segments, they could also experience axial compressive forces as a result of resisting the growing adjacent segments.

As mentioned earlier, the floor system including slabs is modeled as a 2-dimensional grid of frame elements. Slab strip LS-A (See Figure 6.20) runs just next to the longitudinal exterior beam and it experiences tensile axial forces. Since the elements modeling the slab strip next to the beam (4 in deep) are more flexible than the exterior longitudinal beam elements (20 in deep) they do not experience beam growth as much as exterior longitudinal beam elements. Hence they provide axial restraint to the beam segments, which tend to elongate. The joists and slab elements in the transverse direction connecting the external longitudinal beam C1-C2 and slab strip LS-A provide the required strength to transfer the shear forces in their minor axes (in plane action). Slab strip LS-B also experiences axial tensile forces but its magnitudes are much smaller than slab strip LS-A because of the difference in distance to exterior longitudinal beam C1-C2.

Figure 6.21 (a) shows that the axial forces in the segments of the longitudinal exterior beam change to tension after about 9.5 in of vertical displacement. The primary reason is for the change of sign of the axial force in the beam is the increased effects of
larger vertical displacements and geometric compatibility. Another reason for the change of sign of the axial force is that the end section of the longitudinal exterior beam at the C2 end (the section which has the highest tensile strain at its center) experiences concrete crushing around 7 in of vertical displacement and the tensile strain at the center of the section drops considerably. In turn, elongation of the beam drops causing a decrease in the compressive axial force due to beam growth.

Axial tensile force in the beam reaches 183 kips at 30.6 in of vertical displacement. The vertical component of this force accounts for about 23% of the shear resisted at the C2 end. As is the case with catenary action, the large beam axial tensile forces at large displacements contributes to carrying shear forces along with the flexural behavior of the beams.

Figure 6.21 (b) shows the elongation of the first three exterior longitudinal beam segments (See Figure 6.20) versus time. Note that only the first three segments experienced elongation along the beam. Events such as concrete cracking, yielding of rebars and concrete crushing are labeled in the figure and listed below to relate to their effects on the beam growth:

1) Cracking of concrete in the first segment and initiation of beam growth;

2) Cracking of concrete in the second segment;

3) Cracking of concrete in the third segment;

4) Yielding of top rebar in the first segment (at the face of column C2);

5) In the first segment, the first layer of core concrete fibers reaches the peak stress (strength) of concrete (initiation of crushing);

6) In the first segment, the third layer of core concrete fibers reaches the peak stress (strength) of concrete;

7) Between (5) and (7), 8 out of 16 layers of core concrete fibers (half of the beam section depth) exceed the strain corresponding to the peak stress of concrete in the first
segment. At the peak elongation, the strain in the extreme bottom cover concrete fiber is around 0.085;

8) Yielding of top rebar in the third segment (at bar cut-off location); and

9) The ninth layer (out of 16) of core concrete fibers exceeds the peak stress of concrete in the first segment.

Accordingly, at the section level, the concrete cracking initiates the beam growth. Yielding boosts the tensile strains at the center and in turn, the amount of elongation of the corresponding segment further increases. Crushing of concrete in the section reduces the tensile strains at the center and causes the corresponding segment to start shrinking.

6.6.4.3 Effect of Slab Elevation in Analytical Modeling

In the analytical model, although the slabs are accounted for as the flange of the T and L beams and are modeled at the correct height, the frame elements modeling the slabs and the beams lie in the same horizontal plane. This model will be referred to as Model A in this section. There is an 8 inch height difference between the centers of the beams (20 in deep) and the slabs (4 in deep). To evaluate the effect of slab elevation in the analytical modeling, another model is developed and analyzed under Scenario 3-7 (Model B). In this model, slab elements lie in a plane that is 8 in higher than beams and the connection between slabs and beams (also joints) is achieved by using rigid vertical link elements.

Figure 6.18 compares the displacement history of joint C1 for Model A and B. Similar to Model A, the analysis results of Model B show that two primary load transferring beams B1-C1 and C1-C2 failed at their B1 and C2 ends, respectively (See Figure 1). In both sections, the bottom rebars buckled and the sections experienced excessive concrete crushing. Figure 6.23 compares the moment demands of both sections for Model A and B versus the vertical displacement of joint C1. Figure 6.24 compares the axial force in these two critical sections for Model A and B. As seen in the figure, both critical sections have higher axial compressive force in Model B compared to Model A. This can be explained with the fact that the beam tensile strains 8 in above the center line
are higher than those at the center line after cracking (under negative moment). As the beams crack and yield, they tend to grow. The neighboring slab elements provide in-plane resistance to the beam growth and in turn, axial compressive forces develop in the beams due to restrained elongation. Since the amount of elongation at 8 in above the center line is more than that at the center line, the axial compressive forces developed in the beams of Model B are larger than those in Model A. The axial tensile forces developed in the slab elements are also higher in Model B due to equilibrium. There is a sudden drop in the axial force in beam B1-C1 at about 2 in of vertical displacement of joint C1. Beam C1-C2 has a similar drop at about 3 in of vertical displacement of joint C1. These sudden drops are associated with the cracking of the neighboring slab elements and, in turn, reduction in their in-plane resistances. Note that the axial tensile forces in the slab elements in Model A are smaller such that they do not lead to cracking. The drops after the peak axial force are associated with the rapid reduction of the center tensile strain due to concrete crushing at the beam end section at C2.

Additional compressive force in the beams interacts with their flexural behavior. Figure 6.23 shows that the yield moments are higher in the case of Model B than Model A. The figure also shows that concrete crushing occurs at earlier displacements in Model B compared to Model A. Accordingly, the larger axial compressive force in the beams due to augmented restraint of the elevated slab on the beam growth resulted in stronger but less ductile behavior of the beams. It can also be seen in the displacement response as shown in Figure 6.18 that the Model B is stronger than Model A.

6.6.5 Results of Scenario 4-7 (Removal of Top Floor Middle Column on Short Edge)

Figure 6.25 shows the vertical displacement of joint B2 on the roof (See “Original Design” in the figure). The analysis does not converge beyond the first 3.76 seconds when the maximum displacement of joint B1 reached 16.7 in. However, the level of damage occurred at the critical sections, which will be discussed below, along with the increasing slope of displacement-time history suggests that the structure is likely to collapse. The vertical displacement of joint B1 is used as the reference to define the
deformation stage of the structure while presenting the results below. Beams A1-B1, C1-B1 and B2-B1 are the three primary elements that transfer the load of removed column to the neighboring elements. The structure has a symmetric response about Axis B. Hence only the elements between Axes A and B are presented and discussed below.

By 16.7 in of vertical displacement, the top of column A1 experiences concrete crushing and compressive rebar buckling. The maximum tensile and compressive rebar strains are around 0.077 (close to rupture) and 0.073, respectively. The A1 end of beam A1-B1, however, remains elastic during analysis. Figure 6.26 shows the deformed shape of structure with a scale factor of 7. Note in the figure that all the plastic deformation is concentrated on the top of the column A1 at joint A1. This concentration can be explained by the difference between the demand/capacity ratios of the column A1 and beam A1-B1. As mentioned before, the same reinforcement pattern is used for top three floor beams and columns. The sixth floor beams governs the design of the top three floors, and therefore, the beams on the seventh floor and roof are overdesigned. The same concept is valid for the columns. In quantitative terms, the beam section at the A1 end of the A1-B1 beam on the roof is overdesigned by 92% while the 7th floor column A1 is overdesigned by 34%. To eliminate the consequence of overdesign of the beams and columns (on the top floor level) on the general response as well as on the significant difference between beam and column demands at Joint A1, reinforcement of top floor beams and columns are redesigned individually without changing the section dimensions and the structure is reanalyzed under scenario 4-7. Based on the new design, the A1 end of the A1-B1 beam on the roof is overdesigned by 7% while the seventh floor column A1 is overdesigned by 19%.

The displacement history of joint B1 for the redesigned structure is shown in Figure 6.25 (See “Redesign” in the figure). The analysis does not converge beyond the first 4.01 seconds. As one expects, the displacement response is larger than original design since the structure became less strong. The maximum tensile strain in the rebars of column A1 at its top end reaches the rupture strain of steel (0.09) at 22.8 in of vertical displacement. Compressive rebars at the section buckle at around 19 in of vertical displacement having a maximum strain of 0.04. Different than the results of original
design, the A1 end of transverse beam A1-A2 experiences tensile rebar yielding and maximum tensile rebar strain reached 0.059. Thus, the deformations of the elements connected to joint A1 are comparable as a result of new design mentioned above.

Based on the analysis results of the original design, the longitudinal interior beam B1-B2 deforms in double curvature before torsional cracking of exterior transverse beams. The rotational restraint at the B1 end of the beam is provided by the torsional stiffness of exterior transverse beams. After cracking, the torsional stiffness of exterior transverse beams drops to 5% of the uncracked stiffness (Valipour and Foster, 2010), and beam B1-B2 practically behaves as a cantilever. The change in the flexural behavior of the beams, which are neither continuous beyond the removed column nor connected to a column in upper floors, after losing rotational restraints at their ends, are described in detail before while presenting the results of Scenario 4-7.

The B2 end of beam B1-B2 experiences severe concrete crushing (starting at around 4.3 in) of vertical displacement. Moreover, the bottom rebars buckle at around 6.6 in of vertical displacement. The maximum tensile and compressive rebar strains are about 0.055 and 0.08, respectively. Figure 6.27 shows the axial force history of B2 end of beam B1-B2 versus vertical displacement of joint B1. The beam experiences compressive axial force until 5.8 in of vertical displacement as a result of beam growth and in-plane restraint provided by the floor. Beyond 5.8 in of vertical displacement, the beam experiences tensile axial force as a result of the reduced effect of beam growth due to concrete crushing and the increased contribution of geometric compatibility. Axial tensile force in the beam reaches 354 kips at 16.7 in of vertical displacement. Note that, at this level of displacement, around 12% of the transferred shear at the B2 end is due to the vertical component of the axial force in the beam.

Based on the analysis results of a new design in which the top floor beams and columns are designed individually, the B2 end of beam B1-B2 experiences larger deformations. Maximum tensile rebar strain reaches 0.077 while the maximum compressive rebar strain reached 0.093. Note that having weaker sections for beam B1-B2 changes the axial forces developed in the beam. The peak compressive force
developed due to beam growth is increased to 120 kips (redesign) from 75 kips (original design). Since the section is weaker in the new design, larger tensile strains developed at the center of section that amplified the beam growth, and consequently, the axial compressive forces increase. After the effect of geometric compatibility overcame the effect of beam growth on the axial behavior of beam, the axial force changes from compression to tension. The level of axial tensile forces for the new design is less than that for the original design simply because the weaker sections can provide less resistance to geometric compatibility.

The exterior transverse beams axial forces after column removal remain in compression for both the original and new design. The axial compressive forces are due to the growth of transverse beams and restrain provided by the in-plane action of the floor system. It can be concluded that these beams could not develop Catenary Action as a load transferring mechanism.

6.7 Summary

Analytical modeling techniques for progressive collapse evaluation of RC structures used in this paper are primarily based on the experimental and analytical studies conducted by the authors. Progressive collapse resistance of a seven-story RC structure under 15 scenarios of sudden column removal is evaluated. The numerical simulations demonstrate the following:

For the 13 scenarios that the structure resists progressive collapse, the dominant load resisting mechanism is a combination of Vierendeel frame action and the axial compression-flexural response of beams. The maximum vertical displacement for these scenarios is limited to 4.1 in. Due to small vertical displacements, Catenary action is not developed.

On average, 65-75% of the load is redistributed by the beams that are connected to the joints above the removed column. About 20-25% of the load is redistributed by the joists in the vicinity of the damaged region. The remaining load is transferred by the floor slabs.
In Scenario 1-7 as well as 2-7, the roof transverse beam connected to the joint above the removed column (i.e. removal of an interior column of the exterior longitudinal frame) is neither continuous over the removed column nor does it interact with a column at that location. However, its flexural response is affected by the torsional response of the exterior longitudinal beams in the perpendicular direction and as a result develops Vierendeel frame action and deforms in double curvature. Note that the exterior longitudinal beams in these scenarios do not crack under torsion due to small displacements and deformations. Similarly, the neighboring joists also deform in double curvature due to the torsional stiffness of the exterior longitudinal beams they are connected to.

In Scenarios 3-7 and 4-7, the primary reason for which the structure cannot redistribute the loads and shows local collapse is that the roof beams which are not continuous over the removed column could not develop Vierendeel frame action (i.e. could not deform in double curvature). This is in part due to the fact that unlike Scenarios 1-7 or 2-7, because of large torsional demands on exterior perpendicular beams in Scenarios 3-7 and 4-7, they form cracks and therefore their torsional stiffness drops significantly. As a result, the discontinued beams deform like cantilever elements in single curvature resulting in larger moment demands and eventually section flexural failure. It is important to note if the torsional cracking of the elements were mistakenly not considered in the analyses, the structure would have been able to redistribute the load of the removed column and stabilize and the progressive collapse resistance of the structure would have been overestimated.

Structures are less susceptible to collapse if there are at least two floors (and connecting columns) above the removed column such that Vierendeel frame action can effectively develop.

In order to account for RC beam growth due to flexural cracking and yielding, axial and flexural deformation of floor systems (i.e. beams, joists, and slabs) needs to be accounted for. Therefore, modeling structures using plastic hinges that do not include
axial deformation as a section degree of freedom, can significantly underestimate the progressive collapse resistance of RC structures.

The axial compressive forces developed in beams due to beam growth improves their flexural capacities (up to about 100% more), and in turn enhances the beam flexural and shear carrying capacities. Therefore disregarding the beam growth in analytical modeling will result in an underestimation of the structure’s resistance to progressive collapse.

The axial forces in the columns above the removed column drop rapidly following the removal of the column indicating that each floor redistributes its own load and there is not notable vertical load transfer between floors through the columns above the removed column.

Including the elevation difference between the center lines of beams and floor slabs in the analytical modeling increases the restraint provided by the slab elements to beam growth. In turn, the axial forces in the beams and slabs are increased. Larger axial compressive force due to beam growth makes flexural response of beams stronger but less ductile.

In Scenario 4-7, Catenary action of the exterior transverse beam bridging over the lost column develops in neither the original design nor the redesigned structure. This is in spite of a rather large vertical displacement. In the original design, the plastic deformation is concentrated at the top of the corner columns and the beam remained almost elastic. In the redesigned structure, although the beam yielded significantly, its axial force remains compressive even at a vertical displacement of about 23 in, where the column top bars start to rupture.
Table 6.1 Peak and permanent displacements

<table>
<thead>
<tr>
<th>Column removed from:</th>
<th>1st Floor</th>
<th>4th Floor</th>
<th>7th Floor</th>
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<tr>
<td></td>
<td>Peak</td>
<td>Permanent</td>
<td>Peak</td>
</tr>
<tr>
<td>Scenario 1</td>
<td>1.66&quot;</td>
<td>1.39&quot;</td>
<td>1.65&quot;</td>
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<tr>
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<td>1.57&quot;</td>
<td>1.95&quot;</td>
</tr>
<tr>
<td>Scenario 3</td>
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<td>2.04&quot;</td>
<td>2.67&quot;</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>3.13&quot;</td>
<td>2.92&quot;</td>
<td>4.10&quot;</td>
</tr>
<tr>
<td>Scenario 5</td>
<td>2.26&quot;</td>
<td>1.88&quot;</td>
<td>2.34&quot;</td>
</tr>
</tbody>
</table>

Table 6.2 Beams bridging over removed column for each scenario

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Beams bridging over removed column</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 1-1, 1-4, 1-7</td>
<td>Longitudinal Exterior Beams C3-C4 and C4-C5</td>
</tr>
<tr>
<td>Scenario 2-1, 2-4, 2-7</td>
<td>Longitudinal Exterior Beams C1-C2 and C2-C3</td>
</tr>
<tr>
<td>Scenario 4-1, 4-4</td>
<td>Transverse Exterior Beams A1-B1 and B1-C1</td>
</tr>
<tr>
<td>Scenario 5-1, 5-4, 5-7</td>
<td>Longitudinal Interior Beams B3-B4 and B4-B5</td>
</tr>
<tr>
<td>Scenario 5-1, 5-4, 5-7</td>
<td>Transverse Interior Beams A4-B4 and B4-C4</td>
</tr>
</tbody>
</table>
Figure 6.1 Typical floor plan and typical size of structural elements (beams, columns and joists)

Figure 6.2 Elevation views of Axis A and Axis 1
Figure 6.3 Joist floor details
Figure 6.4 Reinforcement details of exterior longitudinal beams
Figure 6.5 Reinforcement details of exterior transverse beams
Figure 6.6 Reinforcement details of interior longitudinal beams
Figure 6.7 Reinforcement details of interior transverse beams
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Figure 6.8 Reinforcement details of exterior and interior columns

Figure 6.9 Locations of columns to be removed on floor plan
Figure 6.10 Locations of columns to be removed for each scenario
Figure 6.11 Linear-nonlinear regions
Figure 6.12 Stress-strain relationship of concrete

Figure 6.13 Stress-strain relationship of steel
Figure 6.14 Displacement histories
Figure 6.15 Moment diagram of elements of Axis C for Scenario 3-1 (after structure stabilized)

Figure 6.16 Edges of vicinity regions for each scenario
Figure 6.17 Axial force diagrams of beams and floor elements (a) on second floor for Scenario 1-1 (b) on fifth floor for Scenario 3-4

Figure 6.18 Displacement history of joint C1 at roof for Model A (without raised slab) and Model B (with raised slab) for Scenario 3-7
Figure 6.19 Transition of bending moment pattern of Beam C1-C2 on roof for Scenario 3-7
Figure 6.20 Elements in corner panel B1-B2-C1-C2
Figure 6.21 (a) Axial Force and (b) elongation histories of longitudinal beam C1-C2 segments for Scenario 3-7
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Figure 6.22 Elongation of beam C1-C2 and parallel slab elements at peak axial compressive force in beam (Scenario 3-7)

Figure 6.23 Moment histories of critical beam sections for Model A (without raised slab) and Model B (with raised slab) (Scenario 3-7)
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Figure 6.24 Axial force histories of critical beam sections for Model A (without raised slab) and Model B (with raised slab) (Scenario 3-7)

Figure 6.25 Displacement history of joint B1 on roof for Scenario 4-7
Figure 6.26 Deformed shapes of 7th floor columns and roof beams (Original Design) under Scenario 4-7 (Scale factor = 7)

Figure 6.27 Axial force history of B2 end of beam B1-B2 for Scenario 4-7 (Original Design)
Chapter 7

Conclusions

In experimental and analytical progressive collapse evaluation of the structures studied in this dissertation, three main resisting mechanisms are observed that help resist progressive collapse; Vierendeel frame action, Catenary action, and the axial compression-flexural response of beams. The conclusions related to these three mechanisms are given in the following sections. The conclusions related to behavior of beams, columns, and infill walls are also presented in the following sections along with other findings.

7.1 Vierendeel Frame Action

- Experimental results obtained in this study show that Vierendeel frame action was the dominant progressive collapse resisting mechanism following element removal in all evaluated structures (three full scale existing buildings, one two-dimensional scaled frame structure, and one analytically studied building). Vierendeel frame action develops in the beams or slabs in the vicinity of the removed column(s), which can be characterized by relative vertical displacement between beam ends (or slab end regions) and double curvature deformations of beams and slabs, which interact with columns. Such a deformed shape provides shear forces in beams (or slabs) to redistribute gravity loads following column removal.

- The capability of the structure to develop Vierendeel Frame Action is crucial in resisting progressive collapse. Structures are less susceptible to collapse if there are at least two floors (and connecting columns) above the removed column such that Vierendeel frame action can effectively develop. However, in cases where only one floor above the removed column (top floor column loss) exists, beams connected to each other in perpendicular directions can deform in double curvature through their torsion-flexure interaction and Vierendeel action can still develop. Therefore,
accounting for torsional cracking in beams is very important in reliably estimating the progressive collapse resistance of the structure.

- In Vierendeel frame action, the contribution of the elements in a floor that are directly connected to the columns above the removed one depends on their relative stiffness, strength and ductility capacities. Assuming the same vertical displacement at their ends on the removed column side and negligible deflection at their other ends on the remaining part of the structure, element end moments and in turn shear forces are functions of the element stiffness and strengths.

- Different than Catenary action, Vierendeel frame action can form in beams, joists or slabs interacting with the columns in the vicinity of the removed column even though the elements (beams or slabs) are not continuous beyond the line of the removed column.

- In the evaluated structures, three types of brittle failure risks are observed that are associated with the development of Vierendeel frame action:
  
  o Before removal of one column, the bending moment in the beams and the slabs in the vicinity of the removed column were negative under gravity loads, as expected. After removal of the column, however, the bending moment in the elements around the removed column changes to positive as a result of relative end displacement and, in turn, double curvature deformation of the elements occurs. If the beams (or slabs) do not have the minimum bottom reinforcement at those sections, which is not needed when designing under gravity loads, brittle flexural failure may be observed if the moment demand at such a section reaches the cracking moment capacity. Of course, the amount of bottom reinforcement at the end regions of the beams depends on the designed lateral load resistance of the structure.

  o In the case of having bottom reinforcement at such sections mentioned above, development of the full flexural strength of the section depends on the existence of proper anchorage of the bottom reinforcement into the joint. Not
having proper anchorage of the bottom reinforcement into the column is not an issue when designing under gravity loads. However, after column removal and the development of positive bending moment in the beams (or slabs) at the face of the removed column, the bottom bars become susceptible to bar pull out due to potential lack of anchorage.

- While development of Vierendeel frame action imposes positive bending moment to the beam (or slab) end on the removed column side, it imposes additional negative bending moment to the other end of the beam (or slab). Another type of brittle failure in beams (or slabs) associated with the development of Vierendeel frame action could happen when the increased negative bending moment demand in the beams (or slabs) extends beyond the top bar cut off locations. If the minimum reinforcement was not continued after such locations, brittle flexural failure could happen when the bending moment reaches to the cracking bending moment capacity. The modulus of rupture of the concrete plays an important role in the behavior of such sections.

- The potential brittle failure mechanisms mentioned above can negatively affect the contribution of Vierendeel frame action in the progressive collapse resistance of the structure. Therefore, they need to be taken into account in the design of new structures. In the evaluated structures in this dissertation, however, some of the structures were susceptible to such potential brittle failures, but they resisted progressive collapse through their three dimensional response and redundancy without experiencing any failures mentioned above.

- The contribution of columns above the removed column in Vierendeel frame action depends on the geometry of the structure. If the beams over the removed column are continuous and symmetric with respect to the axis of the removed column (as in Memphis Memorial Hospital and two-dimensional frame), then the bending demands of the beams in both sides of the column simply cancel out and the columns do not experience any bending in the corresponding direction. However, if the beams are not
continuous or not symmetric (as in the Hotel San Diego and University of Arkansas Dormitory), then the columns deform in double curvature along with the beams to satisfy the joint equilibrium.

7.2 Catenary Action

- Catenary action is clearly observed in the two-dimensional scaled frame when it is subjected to a monotonically increased displacement at the top of the removed column. Before the displacement-controlled loading, the frame is tested as well as analyzed following removal of the first floor center column. Both experimental and analytical results showed that the dominant resisting mechanism was Vierendeel frame action of the beams that bridged over the removed column and permanent displacement is limited to around 0.4” (3.2” in full scale). As the frame was loaded beyond that displacement in the second phase of the test, Vierendeel frame action diminished as bar ruptures occurred in the beams but the contribution of Catenary action become more significant as the slope of the bridging beams increased.

- Development of Catenary action requires the structure to experience large displacements to transfer the loads to the neighboring columns through the tensile forces in the beams (or slabs) bridging over the removed column. For instance, in the case of the two-dimensional frame, the effect of Catenary action on the horizontal displacements of the ends of the bridging beams, which tends to pull these joints inwards, became equal with the effect of beam growth, which pushes the beam ends outwards, at 1.5” of vertical displacement (12” in full scale). In other words, the effect of the Catenary action become comparable with the effect of Vierendeel frame action after the maximum displacement in the structure reached 12 in. Maximum vertical displacements observed in the evaluated full scale buildings were in the range of 0.24”-0.38” suggesting that the contribution of Catenary action in the resisting mechanism of those buildings was negligible.

- Even though the contribution of Catenary action in the resistance of the two-dimensional frame to applied vertical displacement (2\textsuperscript{nd} phase of the test after column removal) is clearly observed experimentally and analytically, neither in the evaluated
buildings nor in the two-dimensional frame did the Catenary action develop after column removal. The maximum deflections after column removal in all structures were too small for Catenary action to be effective in load redistribution.

7.3 Axial Compression-Flexural Response of Beams

- Beam growth in the beams bridging over the removed column is described. It is shown that the center of a beam section after cracking and yielding experiences tensile strain which results in an elongation of the beam. When the tendency of the beam to grow is restrained by the rest of the structure, axial compressive forces develop in the beams. The axial compressive force due to beam growth enhances the flexural strength of the beam sections through axial force-moment (P-M) interaction. Moreover, the compressive force also reduces the extent of flexural cracking in the beams and in turn enhances their stiffness. Therefore, including beam growth in the modeling improves the progressive collapse resistance of the structure and limits the deflections. Ignoring the effects of beam growth on flexural behavior of the beams causes underestimation of strength and stiffness of the structure.

- Another effect of the compressive force developed in the beam due to the beam growth is that it imposes additional bending moment on the beam. Note that for the axial load to have an augmented effect on the beam moments, the geometric nonlinearity (P-Delta) needs to be included in the analysis.

- Beam growth can be included in the analytical model if strain variation over the section of the elements can be tracked during the analysis due to both flexural as well as axial deformations. For example, modeling of the beams with 2-node Bernoulli beam elements with plastic flexure-only-plastic hinges cannot capture the beam growth and its effects on the behavior of the beams. Using fiber sections, however, allows tracking the strain variation over the sections due to flexural as well as axial deformations and including the beam growth in the analysis.

- While for small displacements the axial compressive force in the beams (due to beam growth and the constraint provided by the rest of the structure) governs the resulting
axial demand in the beams over the removed column, for large deformations axial tensile force due to change of geometry (which is the indication of Catenary action) overcomes the compressive force and the resultant axial force in the beam changes to tension. The beam sections may crack if the tensile stresses in the section due to axial tensile force exceed the tensile strength of concrete. Moreover, from the flexural behavior point of view, tensile axial force developed in the beam would reduce the moment capacity of the sections due to axial force - moment (P-M) interaction. These facts are accounted for when the beam is modeled with fiber sections (or with springs). In a model where beams are modeled with 2-node linear Bernoulli beam elements and yielding under flexure is modeled with localized plastic hinges, flexural plastic hinges that can account for the P-M (Axial Force-Moment) interaction can be used to include the effect of axial tensile force under large displacements on the flexural behavior of beams. Note that the geometric nonlinearity that accounts for large displacements (i.e. equilibrium equations written in the deformed configuration) needs to be included in the analysis to observe axial forces in the beam elements.

- Considering the Vierendeel frame action, the geometric nonlinearity does not need to be included in the analysis to capture the double curvature deformation of the beam and corresponding load redistribution through shear forces in the beam. However, the effect of the axial compressive force that develops due to beam growth on the moment in the beam would be ignored if the P-Delta effect is not accounted for in the analysis. As for Catenary action, the analysis should include geometric nonlinearity that accounts for large displacements to capture the axial tensile force in the beam due to change of geometry.

- It is demonstrated that the amount of axial force that can develop in a beam due to beam growth is also a function of the constraints of the beam. In-plane action of floor system provides restrain to elongation of the beam significantly. Adjacent frames also provide restrain against beam growth. As is observed, an exterior beam close to the middle of an axis can develop more axial force than a beam that is at the edge of the same axis. Axial compressive force enhances the flexural behavior of the beam better and in turn improves the performance of Vierendeel frame action.
7.4 Behavior of Columns in Progressive Collapse Analysis

- The columns above the removed column lose their axial forces a few milliseconds after column removal, while it takes much more time (an order of magnitude) for the neighboring columns to carry the additional compressive forces due to redistribution of loads. This difference in the response is explained by the fact that the reduction in the axial force of columns above the removed one is due to axial wave propagation, while the increase in the compressive force of neighboring columns is due to flexural wave propagation that requires the structure to deform and redistribute the gravity loads.

- The difference between the axial and flexural wave propagation can also be seen in the comparison of deformation of the structure at the time the columns above the removed column had lost their compressive strain (force). That is, the displacement of the structure is just a fraction of the peak displacement when the columns above the removed one lose their axial force.

- After column removal, the columns above the removed one lose their axial compressive forces and elongate. As a result of the elongation of these columns, the joints above the removed column in upper floors experience slightly less vertical displacement than the ones in the lower floors.

- While the columns above the removed one lose their axial compressive forces and elongate, the neighboring columns experience compressive strains due to the additional compressive forces as a result of the load redistribution and they shorten. As a result of the combination of these two facts, the beams and the floor systems in the lower floors experience more relative end displacement in Vierendeel frame action and in turn develop more shear forces (assuming they have the same stiffness) in different floors. Therefore, the beams and the floor systems in the lower floors participate in load redistribution more than those in the upper floors.

- While the axial compressive force in the columns that are in the lines adjacent to the removed one show increases, the columns in the next line experience a reduction in
their axial compressive forces. This phenomenon is observed in both evaluated buildings and the 2-dimensional frame and can be explained with the new deformed shape of the structure using influence line concept.

### 7.5 Behavior of Infill Walls in Progressive Collapse Analysis

- The contribution of the infill walls to the stiffness and load redistribution of the structure were found to be considerable especially when the peak displacements in the structure were small and infill walls did not have significant damage. In the progressive collapse evaluation of Hotel San Diego, ignoring the infill walls in the analytical modeling of the structure resulted in a 240% increase in the maximum displacement. Note that the model that included infill walls predicted the peak displacement of the structure closely with only 4 percent error.

- Modeling the infill walls with the compressive strut method based on FEMA 356 resulted in underestimation of the stiffness of the walls. In the case of small deflections, 2-dimensional shell elements (including the cracking) were found to be more appropriate to model the infill walls. If the deflections were large such that the extent of cracking in the infill walls would be severe and in turn the effective zone of the infill walls would reduce to only compression, then the results of the model with the compressive struts might have predicted the response more closely.

### 7.6 Additional Conclusions

- The diaphragm effect of the floor system on the constraint provided to the beams in both beam growth and the Catenary action is considerable. Therefore, the slab (floor system) shall be modeled in a way such that it provides beams with distributed constraint along their lengths. Potential in-plane failure of the slabs due to beam growth as well as due to large deformations also needs to be considered in the modeling.

- The effect of the initial damage location in a structure on progressive collapse resistance of a building is studied. Based on the results, it is concluded that in cases where the column removal is performed on the top floor rather than on floors below,
buildings are more susceptible to progressive collapse. One reason that lies behind this fact is that the performance of Vierendeel frame action is limited since the top floor beams and floor elements do not have an interaction with columns.

- The effect of the element length in a model that uses distributed plasticity to include the material nonlinearity is discussed for the case of yielding in the elements. It is emphasized that the length of the elements in such analytical models, especially in the regions where yielding is expected, should be chosen equal to the assumed plastic hinge length.

- The effect of the element length used in the case of bar rupture is also discussed and is demonstrated utilizing the analytical results of the two-dimensional frame. Note that element length has an effect on the nonlinear response of the structure when distributed plasticity is used. It is shown that in the analytical evaluation of the frame when it was subjected to a monotonically increased displacement at the top of the removed column, using smaller element size caused the first bar rupture in the frame to happen at a smaller vertical displacement at the top of the removed column compared to the model where element sizes were larger. The effect of the element size in the post bar rupture behavior of the frame is also discussed.

- The column removal during the experimental program is achieved by explosion of the columns. After explosion, the longitudinal rebars of the exploded columns were bent out but not broken up while the concrete fell apart. The effect of the remaining bars on the damping of the structure as well as their load carrying capacity is discussed. In the evaluation of the Baptist Memorial Hospital, following column removal the axial load carried by the remaining rebars is found out to be about 13% of the axial load of the removed column before explosion. It is demonstrated that the energy dissipated by the rebars corresponded to an additional effective damping ratio of about 15% in the first mode (vertical vibration).
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