Progressive Collapse Analysis of Four Existing Reinforced Concrete Buildings Using Linear Procedure

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ABSTRACT

Progressive collapse occurs when local failure in a structural element spreads to the adjoining members, this might promote additional failure. The damage will propagate till the structure reaches to new static equilibrium configuration. Otherwise, entire collapse will happen. On the other hand, local damage in the structural system will be arrested if the facility has an adequate design to bridge over the damage and redistribute the loads.

In general, buildings are designed to resist the normal anticipated loads like gravity, occupancy, wind and seismic. However, some structures occasionally are being exposed to unforeseen loads due to natural, man-made, intentional or unintentional reasons. These unexpected loads induce progressive collapse event.

In the last few decades several catastrophic events were occurred. Those accidents have instigated vital debate amongst the structural engineering society in respect to the structural behavior under abnormal loading situations, such as collision, intense fire and explosion. Consequently, numerous studies have been conducted to improve the structural performance against extreme load hazards and progressive collapse phenomenon. These investigations proposed many suggestions to design and assess the new and the existing buildings in order to survive such unpredicted incidents and assure life safety of the occupants of those buildings. More concern was given to structures that are prone to progressive collapse event, such as military and federal offices besides skyscrapers. The recommendations have been specified by several researches and codes

related to the progressive collapse issue, such as British Standard, Eurocode, ACI and AISC were not sufficient to prevent the disproportionate collapse since it is not easy to predict the spread of progressive collapse and the structural behavior under various triggered events in their sort and magnitude.

Nevertheless, two governmental agencies in the United States have released detailed guidelines to mitigate the likelihood of progressive collapse. First document was published by the United States General Service Administration (GSA) in November 2000 and revised in June 2003. It was entitled “Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects”. The second guideline was the UFC 4-023-03 (DoD, 2010).

In this research DoD 2010 labeled as “Design of Buildings to Resist Progressive Collapse” that was issued by the United States Department of Defense has been followed as a guideline to analyze and re-design the case study buildings.

This document addresses the numbers and locations of the removed column according to the alternate load path method (sudden column loss). It stipulates that a column should notionally be removed from the middle of the short side, middle of the long side and from the corner of the building. Yet, it did not declare which one from the three cases is the most significant besides the influence of the building height. As a result, the prime objective of this study is to find out the impact of the building height and the most critical location of the removed vertical-support element.

In order to accomplish this, four existing apartments have been selected. They are located in Famagusta, Northen Cyprus. Two of them have four-stories and the other two have eight-stories.

Additionally, Linear static analysis of the three-dimensional (3-D) computer models of each selected building was carried out by using SAP2000 program. The failed members were identified and re-designed.

Ultimately, observations from this research demonstrate that the increase in the height of the structure and the removal of column from the middle or near the middle of the short side of the building is more significant to progressive collapse event.

Keywords: Progressive collapse resistance, Reinforced concrete frame structures, Alternate load path, Abnormal loading, Structural analysis.

ÖZ

Bir yapısal elemanda oluşan yerel bir göçmenin bu elemana bağlı diğer lemanlara yayılması ve daha çok elemanın göçmesine yol açması sonucu aşamalı çöküş oluşur. Bu hasar yapı yeni bir statik dengeye ulaşıncaya kadar yayılacaktır.Bunun olmaması durumunda tüm yapının göçmesi gerçekleşecektir. Diğer yandan, yapı tasarımının hasarlara karşı yeterli olması durumunda hasar sonucu oluşabilecek yükler o bölgede bulunan diğer elemanlar tarafından taşınabilecektir.

Genelde, yapılar normal olarak beklenen yükler düşey, kullanım, rüzgar ve deprem yükleridir. Bazen yapılar doğal, kasti veya kasti olmayan nedenlerle de beklenmedik yüklere maruz kalabiliyor. Bu beklenmedik yükler aşamalı göçme durumunu teşvik edebiliyor.

Son 20 yılda birkaç felaket olmuştur. Bu kazalar İnşaat Mühendisleri odalarında yapıların normal dışı yüklemeler durumunda, örneğin, çarpışma, yoğun yangın ve patlama, esnasındaki davranışları konusunda çok önemli tartışmaları körüklemiştir. Bu nedenle, yapıların olağanüstü yüklenme tehlikelerine ve aşamalı göçme olgusuna karşı yapısal davranışlarının gelştşrilmesi için çok sayıda çalışma yapılmıştır. Bu çalışmalar yeni ve mevcut binaların bu tür olağanüstü yüklemeler karşısında dayanıklılığını artırma ve bina sakinlerinin hayati riskini azaltma adına birçok tasarım ve değerlendirme yöntemleri geliştirmişlerdir. Aşamalı göçmeye yatkın olabilecek tür yapılara, örneğin,

gökdelenler, askeri ve devlet, bu araştırmalarda öncelik verilmiştir. Aşamalı göçme ile ilgili bazı araştırma ve standardlarda, örneğin, İngiliz, Avrupa, Amerikan Beton Enstitüsü ve Amerikan Çelik Yapı Enstitüsü standardlarında yer alan öneriler orantısız çökmeyi önleyebilecek yeterlilikte değidi. Bunun nedeni ise aşamalı göçmenin yayılma şeklini ve yapının bazı durumlarda oluşacak olayın türü ve büyüklüğü karşısında olası davranışının tahmininin zor olmasıdır.

Aşamalı çökme olasılığını azaltmak için, Amerika Birleşik Devletleri'nde iki devlet acentası detaylı standard yayınladılar. İlk dokuman Amerika Birleşik Devletleri Genel Hızmet İdaresi (GSA) tarafından Kasım 2000 yılında yayınlanıp Haziran 2003 yılında güncellenmiş olan "Yeni devlet dairesi binaları ve büyük çapta restorasyon projeleri için aşamalı göçme analiz ve tasarımı" başlıklı yayındır. İkinci rehber doküman ise Savunma Bakanlığı tarafından yayınlanan Birleştirilmiş Tesisler Kriteri UFC 4-023-03 (DoD, 2010) dokümanıdır.

Bu araştırmada örnek yapıların analiz ve tasarımı "Aşamalı göçmeye dayanıklı yapı tasarımı" başlıklı UFC 4-023-03 (DoD, 2010) doküman kullanılarak yapılmıştır. Bu dokuman alternative yol metoduna göre (ani kolon kaybı) çıkarılacak kolon yerleri ve sayılarını vermektedir. Bu dokuman bir kolonun yapının kısa ve uzun kenarının ortasından ve koşesinden çıkarılmış olduğunu varsaymayı şart koşmaktadır. Fakat bu üç ayrı yaklaşımdan hangisinin daha önemli olduğunu ve binanın yüksekliğinin önemine dikkat çekmemektedir. Sonuç olarak bu çalışmanın ana hedefi yapının yüksekliğinin ve de üç yerden çıkarılacak kolonun hangisinin aşamalı göçmeye en kritik etki yapacağının belirlenmesidir.

Bu heflere ulaşmak için dört mevcut apartman yapısı seçilmiştir. Bu apartmanlar Gazimagusa, Kuzey Kıbrıs'tadırlar. İki tanesi dört kat ve digger ikisi ise 8 kattırlar. İlaveten seçilmiş yapılar için SAP2000 bilgisayar programı kullanılarak doğrusal static üç-boyutlu analiz ve tasarım yapılmıştır.

Son olarak, bu araştırma sonucunda aşamalı göçme durumunun yüksek yapılarda ve de yapının kısa yönünün orta bölgesinden kolon çıkarılması durumunda daha kritik olduğu gözlemlenmiştir.

Anahtar Kelimeler: Aşamalı çöküşü direnci, Betonarme çerçeve yapılar, Alternatif yük yolu, Anormal yükleme, Aapısal analiz Takviyeli.

This thesis is dedicated to my beloved family to express appreciation for the principles and values they instilled in me at an early age. Without their desire to see me succeed, it would have been difficult for me to develop the motivation I have today. Also, I dedicate this work to my adored fiancée. Her assistance and support throughout this research was beyond what could have ever been asked of her.

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INTRODUCTION

1.1 General Background

Progressive collapse of structures refers to local damage due to occasional and abnormal events such as gas explosions, bomb attacks and vehicular collisions. The local damage causes a subsequent chain reaction mechanism spreading throughout the entire structure, which in turn leads to a catastrophic collapse [1].

In general, the size of resulting collapse is disproportionate with the triggering event. Progressive collapse might be concluded in two outcomes either partial collapse or global collapse. Moreover, the ratio of total destroyed volume or area to the volume or area damaged by the originated event could be defined as the degree of progressivity in a collapse.

Prior to the partial collapse of the multi-story Ronan Point apartment tower in Newham, London, UK in 1968 the nomenclature progressive collapse, disproportionate collapse, redundancy, robustness etc., were not part of structural engineering terms. In response to the collapse occurred in the 22 story block of pre-cast concrete apartment due to a moderate gas explosion on the 18^{th} floor, the provisions in the United Kingdom Building Regulations have been changed to necessitate that buildings having five or more stories should not sustain failure disproportionate to an initial local failure due to an abnormal load such as a gas explosion [2]. In addition, the requirement which was formulated in the aftermath of the Ronan Point tower to avoid disproportionate collapse and remained largely unchanged until the present day can be listed follows:

- prescriptive ‘tying force’ provisions which are deemed sufficient for the avoidance of disproportionate collapse,
- ‘notional member removal’ provisions which need only be considered if the tying force requirements could not be satisfied,
- ‘key element’ provisions applied to members whose notional removal causes damage exceeding prescribed limits [3].

The landmark event of the Ronan Point tower drew the interest of the engineers and research community for the first time towards the topic “Progressive Collapse”. The partial collapse of the 22 story pre-cast concrete triggered by a very modest gas explosion in the kitchen of the 18th floor knocked out the precast concrete panels and resulted in a chain reaction of collapse all the way to ground due to loss of their support. The impact of the upper floors on the lower floors caused more failure in the exterior wall. As a result, the entire corner of the building was collapsed.

It should be mentioned that the magnitude of the collapse was totally out of proportion with the triggering event.

Another famous example was the bombing of Oklahoma City in 1995. The Alfred P. Murrah Federal Building collapsed as a result of terrorists detonating a bomb in the north side of Murrah Federal Building which caused the damage to few of the ground columns and consequently the collapse of the building. This was the first progressive collapse in the recent history of the United States and it was triggered by the explosion of a bomb truck rather than manmade errors or natural disasters. After this tragedy, many authoritative papers were published on this phenomenon. Several investigations in the issue of progressive collapse and blast loading were performed. A lot of recommendations relating to future structural design were suggested.

The interest in progressive collapse has been at its highest level after this event. The issue of progressive collapse was brought to the forefront after the terrorist attack on the World Trade Centers and the Pentagon on September 11, 2001. The large number of the casualties put the progressive collapse at the climax of interest all over the world. Two hijacked aircrafts hit the Twin Towers in Manhattan, New York City, United States. Another airplane hit the Pentagon in Washington, DC, United States. The entire collapse of the World Trade Centers and the partial collapse of the Pentagon resulted in around three thousand civilians to lose their lives besides the accompanied vast economic loss.

The buildings and infrastructures have been designed to resist the normal loading such as those due to self-weight, occupancy, wind load, seismic effect, and other loading scenarios stipulated in building design codes. However, the accidental partial collapse of the multi-story large-panel apartment building at Ronan Point in the UK in 1968, which was caused by a gas explosion (human-error) drew the attention of structural engineering community to the issue of progressive collapse. Despite the fact that only four people died, a widespread attention has been given to progressive collapse as an important phenomenon that has to be considered in the design of structures. The progressive collapse-resistant performance of structures and extreme loading has become increasingly recognized as a significant issue in the development of several structural design codes. The landmark of Ronan Point apartment promoted the concern of academic researchers and practicing engineers which resulted in the performance many theoretical and experimental researches and production of numerous authoritative papers in the field of progressive collapse.

During the last four decades the prevention and/or mitigation of progressive collapse appears to be an essential issue for design and construction of the buildings as well as the government entities and civilian agencies. Since the partial collapse in 1968 many investigations have been done to provide sufficient integrity for the civil engineering structures and offer adequate ductility, redundancy, besides continuity in steel reinforcement to minimize the risk of progressive collapse.

The worldwide concern regarding the Ronan Point apartment collapse in UK in 1968 initiated extensive investigations related to this phenomenon and resulted in several changes in Canadian and United Kingdom Building Regulations. In addition, many structural engineers and designers have been engaged in research relating to evaluation and prevention of progressive collapse. The resistance of buildings against disproportionate collapse has been explored by many countries and civilian organizations. All this new approach has caused modifications in the design codes and inclusion of some recommendations and specifications relating to this topic. For instance, both European and American codes (ASCE 7-05, ACI 318) have suggested enhancement of structural robustness through the design to prevent or mitigate disproportionate collapse.

The second renowned event was the collapse of the Alfred P. Murrah building in Oklahoma City in 1995. The collapse was generated by a truck loaded with an ammonium nitrate and fuel oil bomb caused collapse of fully half of the total floor area of the nine-story conventional reinforced concrete building. This was the first intentional collapse in recent history of the United States.

The explosion of the truck bomb has destroyed three columns at the first floor in north side of the building. The destruction of those three columns has led to the failure of a transfer girder at the third floor of the mentioned building. The initial partial collapse evoked by the explosive charge propagated well to the adjacent elements causing the failure of complete half of the structure due to the lack of continuity, ductility and alternative load paths to absorb such an unanticipated load.

The Oklahoma City bombing in April 1995 drew the concern for the threat of the terrorism and the need for thorough research to develop the structural integrity and collapse resistance especially for critical buildings and high profile structures.

The total collapse of the World Trade Center towers in New York City on September, 11, 2001 is the most well-known example of this phenomenon where almost the whole world has seen it happening live. As a result of this attack and failure of the buildings around 3000 innocent residents lost their lives.

The terrorist attack was behind this tragedy when two Boeing 767 planes were flown into the north and south towers and caused the entire buildings collapse due to the massive weight of the structure above the impact zone.

The investigations in this planned attack have reported that the tow skyscrapers were completely collapsed because of the collisions by airplanes and the extensive fire from its fuel which was responsible for the ignition of the contents inside the towers.

It should be stated that the structures had sufficient redundancy to resist the collapse for about an hour in spite of the large impact and the intense fire. The part above the collision area was able to sustain the strong fire until it caused the columns to fail. Whilst the lower part was intact till the collapse of the upper part when it was not able to handle the extreme loads resulted from the debris.

It is obvious that during the last few decades the most disastrous collapse scenarios were due to the terrorist attacks. Therefore, the hazard of the terrorism has to be further highlighted particularly possible catastrophic consequences.

Prior to the partial collapse of Ronan Point apartment in London 1968 none of the international standards contained any information on disproportional collapse, but after the incident in UK 1968 the British Building Regulations set up a system of demolition of buildings and the temporary removal of gas in high rise constructions [4].

The consequences of the damage happened in multi-story pre-cast concrete tower created a great influence in the philosophy of structural engineering and led to large modifications in the international codes.

The spectacular nature of the progressive collapse phenomenon induced the interest and concern of the professional engineering to start thinking about new concepts such as progressive or disproportionate collapse, robustness, and integrity of the structures.

Since this landmark, the progressive collapse started to be considered in the design codes. The focus on this subject resulted in new revisions in guidelines and codes available in United States (ASCE7-05, and ACI318) and Europe (Eurocode). This notable concern in the matter of progressive collapse resulted in major improvements in the system of the newly constructed buildings as well as capabilities of the computer programs for analysis and design of the structures.

The next two decades has seen reduced interest in the issue of progressive collapse. Some academic researchers and bodies involved in standard development believed that the partial collapse of the Ronan Point building in 1968 was an anecdotic accident and the progressive collapse has a very low probability. Besides the cost of the buildings will be much higher if the disproportionate collapse needs to be considered.

The Oklahoma City bombing in 1995 raised the interest of the progressive collapse. The destruction of one column was extended to three other columns since the structure was not ductile and had lack of continuity in reinforcing. The damage in the transfer girder caused the full collapse of the nine-story reinforced concrete building.

The structural engineers have begun to refocus on the problem of progressive collapse. The design societies and researchers have shown a vast interest in the performance of the buildings under the situation of progressive collapse.

In order to reduce the risk of potential progressive collapse and to advance the behavior of the buildings against this phenomenon an enormous number of investigations have been conducted in this aspect to modify the design buildings codes.

The outcomes of those studies have recommended that prevention or minimization of the potential collapse in the structures mainly the susceptible buildings to collapse by terrorist attacks such as high-rise building, military offices, and federal buildings could be achieved by designing robust structures which could be reached by providing an adequate integrity, ductility, redundancy, continuity, and alternate load paths to redistribute the loads in case of one or more vertical carrying loads members are removed or destroyed due to abnormal loading.

Due to the savage terrorist attack on the World Trade Centers in September 11, 2001, unprecedented interest has been given to the buildings performance in case of progressive collapse. Because of the profound influence on the American society, an extraordinary concern has led to great number of studies have been carried out by the researchers and engineering communities in addition to extensive investigations both theoretical and experimental have been done in order to comprehensive understanding of this form of failure. Many approaches have been suggested to evaluate, mitigate, and prevent the progressive collapse in the new and existing buildings.

The increased interest in progressive collapse reflected into the significant enhancements in the computer modeling and analytical tools in the last few decades.

The disaster of the twin towers in New York City initiated many suggestions in the international code provisions. For example, in Europe and United States the civilian and governmental agencies stipulated a different philosophy in the design methodology seeking to form rational methods for the improvement of the structural robustness under abnormal events. They have published series of specifications, guidelines, and design codes to advance the structural redundancy to withstand the progressive collapse. Amongst these codes and guidelines are Eurocode, ASCE 7-05, ACI 318, GSA 2003 and DoD 2005.

It should also be mentioned that General Services Administration (GSA, 2003) and Department of Defense (DoD, 2005) highlighted the issues of progressive collapse explicitly and declare quantifiable and enforceable approaches for assessment, mitigation, and avoidance of progressive collapse potential for new, upgraded, and existing buildings.

Generally there are three methods to avoid progressive collapse in buildings, they are listed below:

- Event control
- Indirect design method
- Direct design method

The first method stipulates that the prevention of the collapse could be reached by following a system to control the probability of collapse occurrence, such as, safe design of gas pipes, using barriers around the buildings to prevent possible vehicles with ammunition and monitoring the parking areas.

This method is not very popular in the design of progressive collapse resistance and there are not much details about its usage since the events are not easily foreseeable.

The direct and indirect design methods are the general methods in the design of building against progressive collapse. They are proposed by several researchers and they have been cited in the (ASCE-7, 2002).

Indirect design method is used to decrease the possibility of progressive collapse occurrence by utilizing a good layout for the structures and improving the robustness of the building by increasing the redundancy, ductility, shear resistance and continuity of reinforcement. The mentioned method is a threat independent because it depends on developing the general intercity of the building to produce redundant structures which are able to perform under the extreme loads situations.

Moreover, many researchers and international code are not endorsing the using of the indirect design method in design of progressive collapse because there are no specific considerations for the loading scenarios or the removal of the load-bearing elements (bearing-walls or columns).

Two approaches are listed under the direct design method to resist the progressive collapse. One is the local resistance method which seeks to resist the failure by providing adequate ductility and strength (by increasing the load factors) for specific elements (key members) to resist the abnormal loading. These elements should remain intact regardless of the extreme load cases and no failure is allowed in the entire structure. The other one is the alternate load path method, which permits the local failure in the structure but provide alternative load paths to redistribute the loads to absorb that damage and avoid the major collapse.

The alternate load path method is the most renowned method in the design of progressive collapse resistance. Its philosophy stipulates that the structure should tolerate the local damage and it should be able to achieve an equilibrium state after theoretically removing of the load-carrying element (bearing-walls or columns) one at a time and then analyzing the structure.

1.2 Research Objectives and Scope

Most of the reported progressive collapse events have resulted in large number of casualties besides the enormous loss in the property. In order to mitigate or prevent the potential progressive collapse, the behavior of the damaged members due to abnormal loading and the neighboring elements during and after the occurrence of the initial damage should be clarified.

After several tragedies all over the world generated by progressive collapse due to design or construction errors, fire, impact, gas explosion, and terrorist attacks, there was a need to understand the response of the concrete frame structures when they are exposed to extreme loads.

Many nations have modified their design codes to include the progressive collapse phenomenon. In United States, the General Services Administration and the Department of Defense have published specific guidelines for progressive collapse analysis and designs for the structures GSA, 2003 and UFC 4-23-03 respectively.

The Department of Defense (DoD) explicitly identified the number of the removed load-bearing elements (bearing-walls and columns) in addition to their location.

The primary objective of this research was to assess the potential collapse in four regular existing buildings by employing the alternate load path method and to identify the worst case scenario of the removed load-carrying elements and the effect of building height (number of stories) in addition to retrofit of the damaged members. To achieve this aim, a thorough computational modeling and analysis has been conducted by employing the commercially available computer software SAP2000 V14.2, 2010 [5] to model and analyze four existing buildings in Famagusta, Northern Cyprus following the DoD (UFC 4-023-03) released in January, 2010 as an approach for this study. A linear static procedure for three dimensional (3-D) models for each building was developed in this research. The analysis outcomes for each building were tabulated and then a comparison amongst the four building has been done. The retrofitting procedure for the damaged elements was briefly illuminated.

1.3 Thesis Structure

This thesis consists of five chapters, namely an introduction (chapter one), review of literature (chapter two), the methodology of the research besides the modeling and computer analysis of the buildings (chapter three), discussion of the analysis results (chapter four), and the conclusion (chapter five).

The first chapter provides a historical background explaining the definition of progressive collapse besides the main objectives of this study. Chapter two presents a comprehensive literature review clarifying the definition of progressive collapse in various international codes and three examples of the most prominent progressive collapse events happened in the last century, the design approaches for progressive collapse and the methods of structural design (event control, direct and indirect design methods), the general analysis procedures (Linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses), and the current guidelines for analysis and design of progressive collapse (GSA, 2003 and DoD, 2010). The third one describes the methodology has been followed in this study according to UFC 4-023-03 (DoD, 2010). Additionally, the modeling of the four buildings by implementing the linear static analysis by using the SAP2000 software is presented in this chapter. Furthermore, the results of the analysis and the retrofitting of the damaged parts are detailed in chapter four of this research. The last chapter provides conclusion and recommendations. Ultimately, the references are presented.

Chapter 2

Literature Review

2.1 Introduction

Progressive collapse is a situation in which a local failure in a structure leads to load redistribution, resulting in an overall damage to an extent disproportionate to the initial triggering event [6]. While the disproportionate collapse is associated with local failure of a structural component leading to the total failure of the entire structure or a significant portion of the structure, that is, the extent of final failure is not proportional to the original local failure. An example for this sort of collapse, the failure of a single column in a frame system due to an abnormal event leads to a chain reaction of subsequent failures for the adjoining components resulting in the entire collapse of the building.

The term propagating action, used in the subsequent discussion, refers to the action that results from the failure of one element and leads to the failure of further similar elements [7].

In general, the buildings and the infrastructures are designed to resist the normal loads such as self-weight, wind, and occupancy. However, these structures may be subjected to unexpected loads including natural hazards such as earthquake, flood, and hurricanes or human hazards like design or construction errors, gas explosion, vehicle impact, fire, aircraft collisions, and blast due to civil or criminal actions.

Once the buildings are exposed to these unforeseen loads, one or more of the load-carrying elements (bearing-walls or columns) lose their capacity or been damaged. This destruction in the load supporting system leads to failure in small portion of the structure (local damage). Following this partial collapse, an alternate load paths start to transmit the gravity load and the other loads from the failed elements to the neighboring members throughout the beams and slabs if a ductile structure was designed and has an adequate catenary action in the beams until reaching an equilibrium status and the collapse does not occur. If the structure was not designed to have a suitable catenary action besides alternate load paths to redistribute the loads subjected to the damaged members, a global collapse for the structure could be happened which leads to a serious threat to public safety and properties. Therefore, in order to enhance the progressive collapse resistance and prevent the collapse or more specific the whole collapse in the civil engineering structures, the designer have to increase the structural redundancy.

It should be stated that the cause of the initiating damage to the primary load-bearing elements is unimportant; the resulting sudden changes to the building's geometry and load-path are what matters.

2.2 Progressive Collapse Definitions

The term "progressive collapse" has various definitions and this lexis still vague until the present day. However, the author reports several definitions of this phenomenon which have mentioned in standards and guidelines associated with the analysis and design of progressive collapse.

2.2.1 ASCE/SEI 7-02 Minimum Design Loads for Buildings and Other Structures

The American Society of Civil Engineers Standard 7-05 defines progressive collapse as "the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it." [8].

2.2.2 NIST Best Practices for Reducing the Potential for Progressive Collapse in Buildings

The United States National Institute of Standards and Technology (NIST) proposes that the professional community adopt the following definition: "Progressive collapse is the spread of local damage, from an initiating event, from element to element, resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it; also known as disproportionate collapse." [9].

2.2.3 Eurocode 1 – Actions on Structures – Part 1-7: General Actions – Accidental Actions

There is no specific definition for the progressive collapse in the Eurocode, yet the phenomenon is mentioned in the general accidental actions in term of "robustness". Therein, a definition of robustness is described as "the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause". [10].

2.2.4 United States General Services Administration

The Progressive collapse Analysis and Design Guidelines (GSA, 2003) which was issued by the United Stated General Services Administration labeled the progressive collapse as “a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause.” [9].

2.3 Progressive Collapse Case Studies

In the structural engineering field, the designers and practicing engineers apply their experiences for an adequate design and construction for the civil engineering buildings to achieve the safety and luxury requirements. In general, those structures are being designed to resist the normal loads such as self-weight, wind, snow, live, and seismic loads. Nevertheless, the mentioned buildings are being subjected to extreme loads like design, construction or operation error, thermal, impact, and explosion. Since the structures are not designed to resist such unanticipated loads, a local damage happens for a small portion of the structure. If the building was not designed properly to redistribute the loads carried by the destroyed components to the neighboring elements and provide the catenary action in the beams, the local failure might be propagated to a major part of the building leading to entire collapse of the structure.

Due to unforeseen loads, several structures have suffered from either partial or whole collapse within the last century. For instance, the partial collapse of the Ronan Point apartments in London, United Kingdom in 1968 (gas explosion), the 2000 Commonwealth Ave. tower in Boston, United States in 1971 triggered by punching of insufficiently hardened slab (Fig. 1), the Jewish Community Centre, Buenos Aires, Argentina 1994 (terrorist attack), Alfred P. Murrah Federal Building, Oklahoma, United States in 1995 (terrorist attack), Khobar Towers bombing (Fig. 2), Dhahran, Saudi

Arabia in 1996 (terrorist attack), the World Trade Centers, New York ,United States in 2001 (terrorist attack), and the Windsor Tower, Madrid, Spain in 2005 due to intensive fire (Fig. 3).

For further understanding of the mechanism of progressive collapse, the most three popular cases are provided by the author as examples of evolution of local damage.

Figure 1. The 2000 Commonwealth Ave. Tower in Boston, United States [11]

Figure 2. Khobar Towers Bombing, Dhahran, Saudi Arabia [12]

Figure 3. The Windsor Tower, Madrid, Spain [13]

2.3.1 Ronan Point Apartment

The collapse of the Ronan Point apartment could be considered as the first well-known and the most publicized example of progressive collapse. The Ronan Point tower was a multi-story residential building consisted of 22 stories located in Newham, East London, United Kingdom constructed between July, 1966 and March, 1968. The overall dimensions of the plan were 24.4m by 18.3m and the total height of the apartment was 64m. It was easy to be built since the structural flat plate floor system contained precast concrete for the walls, floors and staircases. The walls and floors were bolted together and the connections were filled with dry packed mortar. This means that the floors did not have a high potential to withstand bending, especially if overhanged, so that each floor was supported directly by the walls in the lower story [9].

On May 16, 1968, Mrs. Ivy Hodge, a tenant on the 18th floor of the 22-story Ronan Point apartment caused the partial collapse of the tower due to gas-stove leak in her kitchen (Fig. 4). This unintentional incident has led to a small gas explosion in the corner kitchen which blew out the exterior precast load-bearing wall in the corner of the structure. The loss of support in turn caused the collapse of the upper floors (19 to 22) because the building was not design to have alternate load paths to redistribute the loads carried by the failed members. The weight and impact of the falling debris resulted from the upper floors failure set off a chain reaction which caused the collapse of floors seventeen down to the ground level.

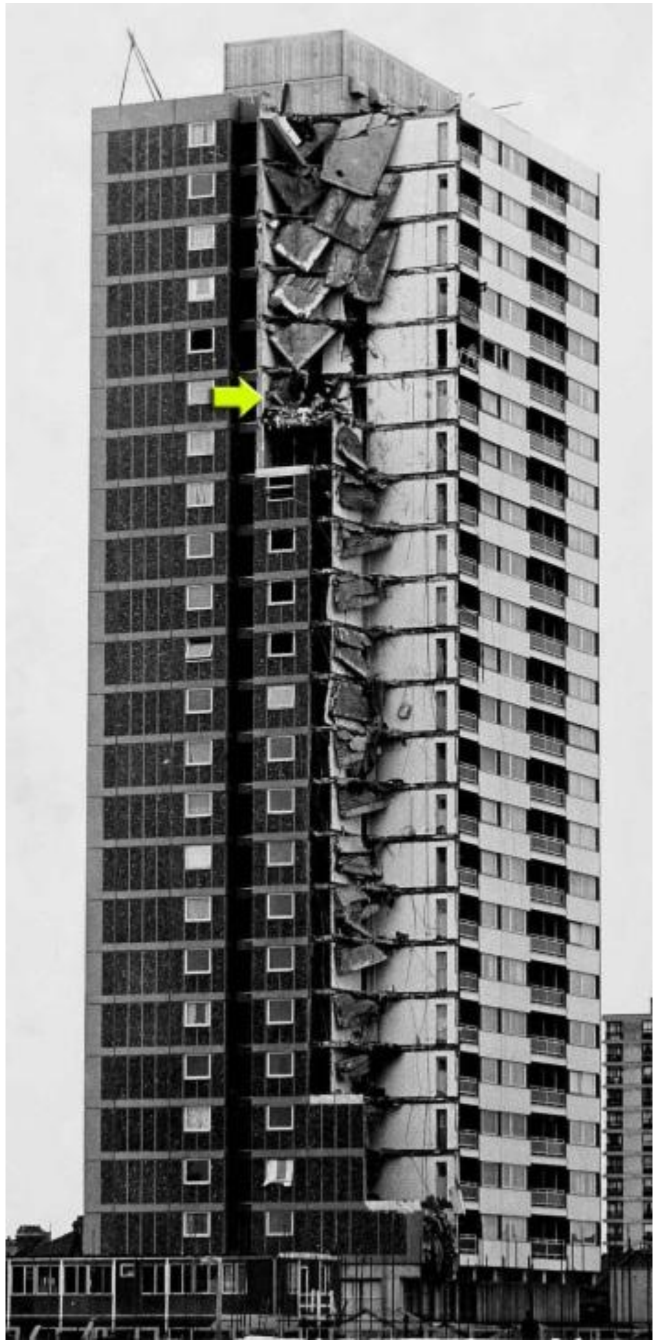

Figure 4. The Ronan Point Apartment, London, UK [14]

It is obvious in (Fig. 5) that the eventual result of the very moderate gas explosion was the collapse of the corner bay for the full height of the tower (entire collapse of the south-west corner). The consequences of the partial collapse of the 22-storey precast Ronan

Point apartment were a building bereft of one of its corners besides four dead residents and seventeen injured but the tenant of the flat number eighteen Mrs. Hodge who triggered the incident survived.

Despite the truth that the partial collapse of the Ronan Point tower in London, England in 1968 was not categorized as one of the biggest buildings disasters of recent years, it was such a shocking accident because the extent of the failure was absolutely out of proportion to the evoked event. The degree of "progressivity" or the ratio of it in this case was of the order of 20. [15].

It should be stated that the wall system was designed only to withstand the extreme wind pressure; hence the continuity in the vertical load path was lost for the upper floors [16]. The collapse was attributed to the lack of structural integrity, mainly in terms of redundancy and local resistance. In other words, the structural system was not designed to provide alternate load path to redistribute the stresses. Another reason of this disproportionate collapse was the building had been constructed with very poor workmanship, and thus its overall structural robustness was considerably compromised. [17].

Further investigations in this collapse reported that stronger interconnection amongst the structural elements is the key for such kind of facilities where this improvement in the connections between the wall panels and floors is likely to have great reduction in the damage scale of the Ronan Point apartment. [18].

Ultimately, the building was demolished in 1986 in the last century due to safety concern.

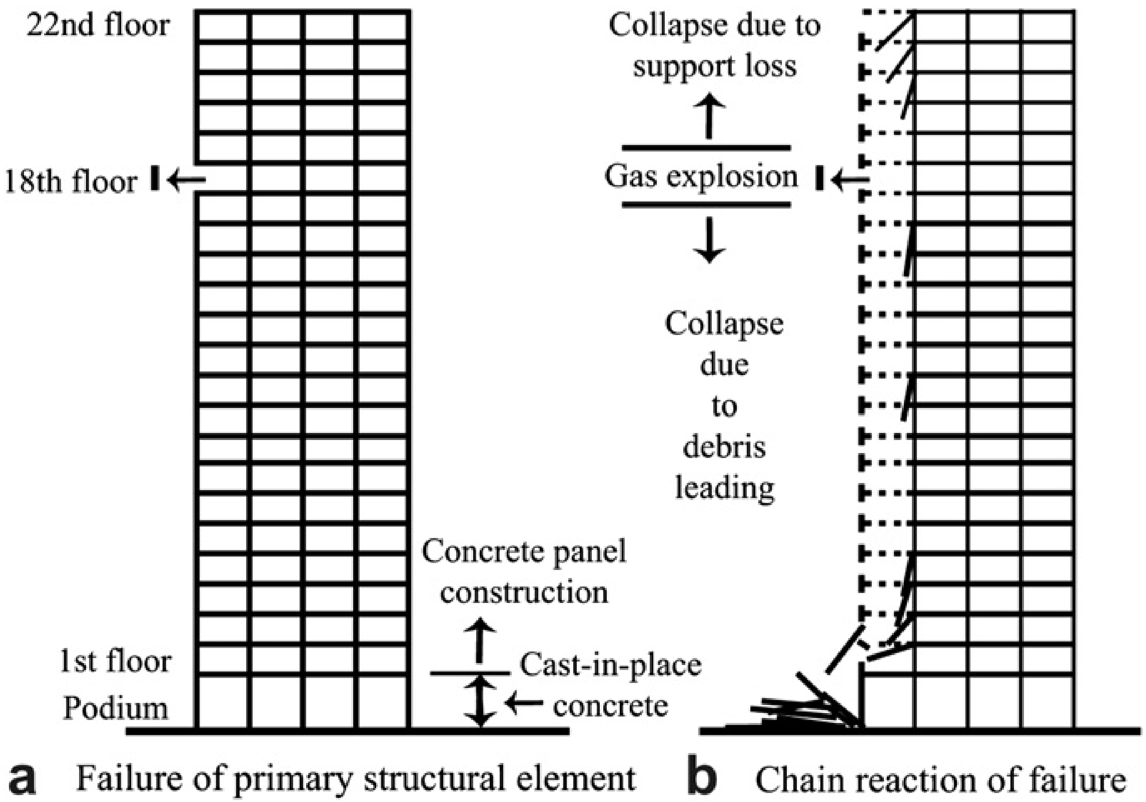

Figure 5. Progressive Collapse of Ronan Apartment [19]

2.3.2 Alfred P. Murrah Federal Building

The Alfred P. Murrah Federal Building was a federal office facility belonged to the United States government located at the downtown of Oklahoma City, Oklahoma, United States. The building was contained regional offices for the Social Security Administration (SSA), the Drug Enforcement Administration (DEA), and the Bureau of Alcohol, Tobacco, and Firearms (ATF) [20]. It was constructed between 1970 and 1976. The structural system was a conventional (non-ductile) reinforced concrete frame with one way floor slab systems besides an interior shear walls system to resist the lateral loads of the building. The federal building was consisted of nine-story with a rectangular floor plan approximately 61m in length and 21.4m in width. It encompassed of two 10.7m-long bays in the north-south direction and ten 6.1m-long bays in the east-west direction. The specific feature of the mentioned governmental facility was the presence of a transfer girder situated in the third floor of the frame building in the north side of the exterior front face of the structure. The unique transfer girder had 12.2 m in span and it was supporting the upper floor columns which in turn spaced at 6.1m [21].

On April 19, 1995, the Alfred P. Murrah Federal Building was a target of a bombing terrorist attack. A truck loaded with 1800 Kg TNT equivalent denoted the north side of the building (Fig.6).

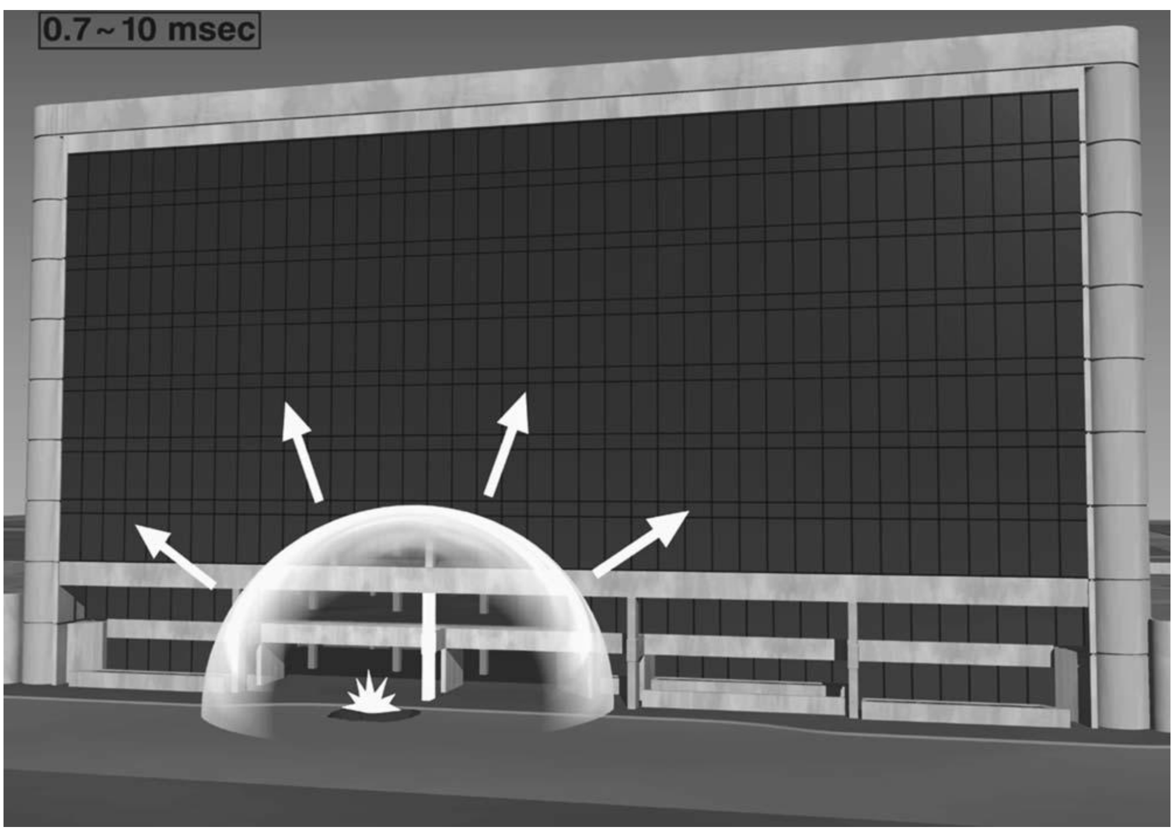

Figure 6. Isometric View Showing Location of Blast [22]

The bomb was delivered by a truck loaded with an ammonium nitrate and fuel oil bomb centered at a distance of 4.9m in front of the north side of the building. The explosion of the truck bomb caused instant destruction or extreme damage for three of the exterior columns in the first floor. Following the destruction of the ground floor columns the transfer girder has suffered from loss of supporting system which in turn has led to the failure of the transfer girder in the third floor. The consequence of this failure was the collapse of the above columns supported by the transfer girder in addition to the total collapse of the beams and floor areas in the upper levels which was supported by the mentioned columns. Eventually, a global collapse for half of the entire structure has occurred (Fig. 7). As a result of this eminent terrorist attack 168 victims were killed and over 800 people were injured accompanied by huge economic loss [23].

Figure 7. Alfred P. Murrah Federal Building [24]

It should be cited that according to the all definitions of the progressive collapse phenomenon described in the international standards, codes, and guidelines the disaster of the Alfred P. Murrah Federal Building was progressive collapse because the blast which was the triggered event has led to local damage represented in the destruction of three ground floor columns and then the local failure has extended to transfer girder located in the third floor caused more failure to the columns, beams, and slab floors above it and finally a general collapse for the half of the full building height occurred.

Several studies have explored the catastrophic collapse of Murrah building and reported that the prime cause of this collapse was the impropriate design for the transfer girder due to the lake of sufficient ductility and the lack of continuity in the steel reinforcement. This poor design prevents the transfer girder to withstand the destruction of the first level columns and extends the local collapse to involve more components and evoked the total collapse.

On the other hand, some investigations concluded that the failure of the ground columns would have also lead to a major part of collapse for the structure [9].

On the contrary of the Ronan Point apartment collapse which was quite simple to be characterized as a complete disproportionate collapse, the progressive collapse of the Alfred P. Murrah Federal building was not easy to be identified whether it was disproportionate collapse or not. There was a consensus that the Murrah collapse was large, but the triggered event was very huge as well. The explosive charge was large enough to cause severe damage over an area of several city blocks.

Finally, the judgment of the researchers and professional engineers was “possibly disproportional”. This conclusion was given because some fair modifications in the design approach would improve the structural behavior and the influence of the bombing attack could be reduced significantly [15].

Further investigations in respect to the Murrah building collapse reported that the prevention of the first floor columns destruction by enhancing the resistance within “plausible limits” would not change the scenario of the columns failure. However, improving the ductility throughout the building and the interconnection and continuity might have helped in the mitigation of the collapse [18]. Furthermore, the employment of the seismic design approach for such type of moment resisting frame would decrease the collapsed area by 50 to 80 percent [17].

Ultimately, Corley (2002) stated that the structural damage and the number of casualties would be reduced by 80 percent if fully continuous reinforcement has been applied [25].

2.3.3 World Trade Centers

The World Trade Centers located in Lower Manhattan, New York City, United States were designed and constructed in the 1960s. The towers also known as WTC 1 and WTC 2 and they have been the world tallest building in 1972. The construction of the WTC 1 was completed in December 1972 while the WTC 2 was finished in July 1973. Both of the World Trade Centers were consisted of 110 stories. They are similar but not exactly identical. The height of the north one (WTC 1) was 417m whereas the south tower (WTC 2) had a total height of 415m. Additionally, the north tower was supporting a radio and television transmission tower with a full height of 110m. Another difference between the two towers is the orientation of the service core situated in the center of the structures. In the north WTC the service core was oriented east to west, but it was oriented north to south in the south WTC.

Each skyscraper was built as a box tube moment-resisting steel frame comprised of closely spaced exterior columns and widely spaced interior columns. The towers have square floor plane of 63m in side length. Each tower was provided by a rectangular service core located in the center of the structure with approximate dimensions of 27m by 42m. The mentioned core contained 99 elevators, 16 escalators, and 3 exit stairways [26].

The World Trade Centers were a target of a terrorist attack in September 11, 2001. Two hijacked commercial airliners hit the two towers. The north tower (WTC 1) was hit first by a Boeing 767-200ER aircraft with estimated velocity of 750 km/hr between 94th and 98th floor in the center of the north face of the building (Fig. 8). Nevertheless, the south skyscraper (WTC 2) was struck seventeen minutes later with higher velocity airplane with approximate velocity of 910 km/hr between 78th and 84th floor nearly the east side of the south face of the tower.

Figure 8. World Trade Center Towers [27]

The destruction of the World Trade twin towers intentional incident was the worst building disaster in the recent history which could be considered the largest mass murder in the United States resulted in the largest loss of lives from any building collapse all over the world. This attack killed 2749 people as well as immense economic loss.

The disaster of the World Trade Center collapse was trigged by deliberate flown of two commercial jetliners into the two skyscrapers. The crash of the jets accompanied by the massive fireball that evoked by the instant ignition caused severe damage of the structure and led to partial failure for several core and exterior columns in the impact zone. However, the ignition of the building contents besides the flow of the fuel across the building floors in the collision zone and down the elevators led to extent of the fire over wider areas on many levels of the structures simultaneous with the ventilation resulting from the air feeding throughout the broken windows and breached walls. Therefore, there is a consensus that the intense fire resulted from the immediate impact and grew later caused the structure burn and had a big part to play in the catastrophic collapse [28].

It should be stated that the World Trade Centers had sufficient robustness and redundancy to withstand such unforeseen loads. The north tower was capable to arrest the local failure in the impact zone for 1 hour and 42 minutes until the structure near the crashed zone was not able to support the load from the upper part of the tower due to the impact and the influence of the overheating resulted from the massive fire caused the failure of the upper part which extended downward all the way to the ground (Fig. 9). On the other hand, the WTC 2 remained standing for 56 minutes after the first trigged event because it sustained more initial damage including primary and incessant fires subjected to the east side of the tower where the insulation for the steel structure was widely stripped due to the plane crash.

Nevertheless, the fall of the World Trade Centers was a big surprise for the structural engineering community because no practicing civil engineer who was watching the accident expected this dominion collapse. Likewise, there was no skyscraper collapse in the history due to fire. Furthermore, several researchers reported that the fire would not have led to entire collapse unless significant portion of the steel insulation has been dislodged because of the impact [29].

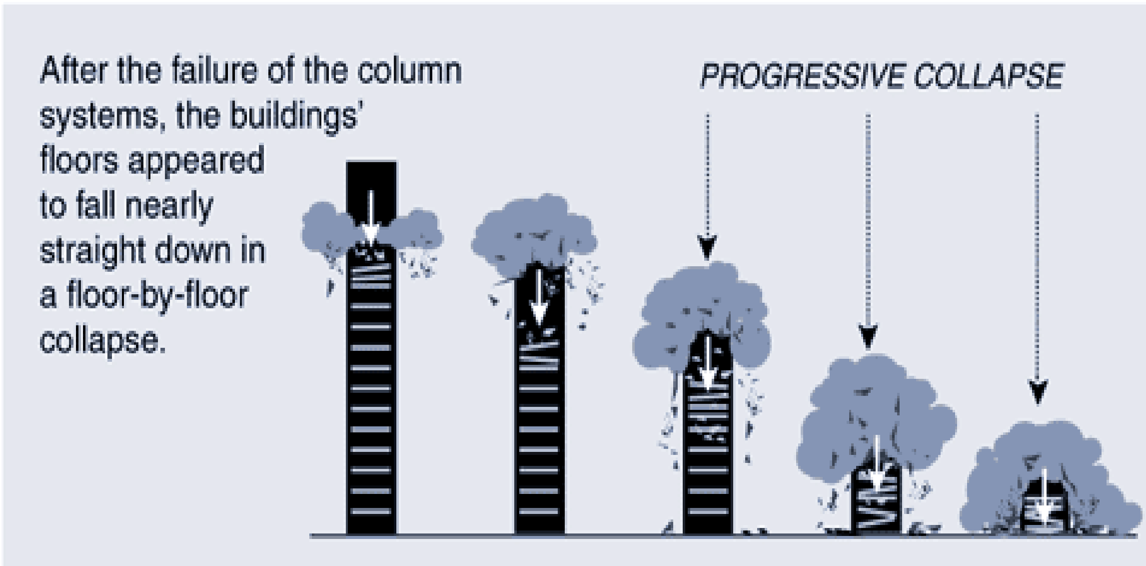

Figure 9. Progressive Collapse of the Twin Towers [30]

The destruction of the World Trade Center towers demonstrated that even the robust and redundant facilities could be vulnerable to progressive collapse phenomenon [23].

In respect to the prevention or mitigation of the entire collapse of the Twin Towers, Nair (2004) concluded that the greater local resistance was not a practical proposition in the case of the WTC towers. In addition, the improvement of the interconnection would not have been valuable. The conclusion of his report and the findings of other researchers were none of the methods for progressive collapse mitigation or prevention would have made any changes in the collapse scenario of the World Trade Center towers [18].

Lastly, the entire collapse of the WTC towers cannot be labeled "disproportionate collapse" like the case of the Ronan Point apartment London, UK in 1968 or the case of Alfred P. Murrah Federal Building, Oklahoma City, USA in 1995 because in this terrorist attack the towers were subjected to two abnormal loads (impact and fire) and none of the means for enhancing the structure robustness would have prevented the entire collapse [15].

2.4 Analysis Procedures for Progressive Collapse

In order to analyze the structures and investigate their response to the progressive collapse phenomenon, there are several analytical methods. These methods vary extensively in respect to time consumption and the structural knowledge required to perform the analysis.

The most common analysis methods have been used to explore the general structural behavior in order of increasing complexity are Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP), and Nonlinear Dynamic Procedure (NDP).

Several researchers in the realm of progressive collapse examined the advantages and drawbacks of the different analysis methods in terms of time and accuracy.

Generally, the response of the building to the local damage in the load-bearing components besides the loads redistribution leads to nonlinear deformation in the structural elements before the failure. As a result of this, the progressive collapse phenomenon is a nonlinear dynamic event.

In spite of the fact that the more complicated analysis procedure would yield to more accurate results which is desirable in the case of progressive collapse analysis, employment of the nonlinear dynamic procedure in progressive collapse analysis is less frequent than linear static procedure because of the complexities and costs required for the fidelity structural models and the time consumption to execute the analysis software.

Additionally, the difficulties relating to the evaluation of the analysis results for the nonlinear dynamic procedure due to the general lack in the behavior of the structural elements and specifically the beam to column connections for concrete and steel structures leads to more implementation of the linear static procedure [31].

It should be mentioned that both DoD and GSA guidelines advocated the usage of a simple analysis approach for design and analysis of low and mid-rise, new and existing buildings. Consequently, the linear static procedure is more widely used to assess the progressive collapse potential in low and mid-rise structures.

The analysis approach of buildings subjected to earthquake specified by FEMA 356 is almost similar to the one used in the progressive collapse analysis. As mentioned before, four methods have been suggested by many researchers and international standards for structural analysis. On the contrary, the two events differ in the load application. In which the seismic analysis is considering gravity loads besides lateral forces for iterative design of the building until the acceptance criteria are being fulfilled. However, the analysis of the building to evaluate progressive collapse potential, a critical column is notionally removed. Two load sequences could be followed, the first sequence is suggested by the General Service Administration (GSA) which stipulates to remove the column and then apply the gravity load to the structure. The other methodology is proposed by the Department of Defense (DoD) which specifies the removal of the critical column after applying the gravity load, and finally analyze the structure [32].

2.4.1 Linear Static Procedure

The linear static method is a basic approach and the simplest method for structural analysis. In this method the structural analysis incorporates only linear elastic materials and it is not considering the geometric and materials nonlinearity. Moreover, it is difficult to correctly predict the structural behavior of the buildings particularly under blast or progressive collapse scenarios. For this reason, the implementation of this analysis method has some errors when compare with more sophisticated approaches. For example, this method is not permitted to be used for structures more than ten stories in height and it is limited only for low-to-medium rise structures according to GSA design guideline. In spite of these disadvantages, the linear static procedure is a popular method for analysis and design of the structures since it is quick, simple, and economic analysis approach.

In order to employ this method for progressive collapse simulation, critical columns are notionally removed from the structure and the gravity load is applied. The analysis was completed quickly and gives fundamental results to aid the analytic to conceptually grasp the behavior of the structure. Subsequently, the response of the structure to the column removal is assessed via demand to capacity ratios (DCRs) for each element.

Finally, some recommendations have been proposed by DoD and GSA analysis and design guidelines for utilizing the linear static procedure for the evaluating progressive collapse potential. For instance, it should be used for regular (typical) structures, for routine analysis of low and medium rise building (not exceeding ten floors), and to account the dynamic influence by applying an amplified factor of "2" to the load combination.

2.4.2 Linear Dynamic Procedure

In general, the linear dynamic analysis method is more precise approach than linear static procedure. The accounting of the damping forces and inertia besides the considerable increase in the accuracy level of the analysis outcomes is the major advantages of this analysis approach. In addition to these benefits there is no necessity to estimate the dynamic amplification effects since they are being considered. Instead, this method has several drawbacks such as more complicity, time consumption, and not including the materials and geometric nonlinearity which is the main feature of the progressive collapse phenomenon.

Furthermore, only those buildings expected to remain elastic during the progressive collapse event could be analyzed by this analysis approach. Lastly, in the implementation of the linear dynamic procedure the researcher has to be more aware for the buildings that have large plastic deformations due to the inaccurately computed dynamic parameters [23].

2.4.3 Nonlinear Static Procedure

Nonlinear static approach is a more intricate and accurate analysis method than the linear static procedure to identifying the progressive collapse in structures. This analysis method is known as pushover analysis and it is extensively used for the earthquake analysis (lateral load). The nonlinear static procedure is only one step above the linear static one since it allows capturing of both geometric and materials nonlinearity in which the most widespread model is an elastic-perfectly plastic curve. Although the materials and the geometric nonlinear behavior are accounted for, but the dynamic effects still be neglected and the analytic should imply the amplification factor of “2” in the loads combination. Thus, this procedure provides limited improvement towards understanding the structural response. In the case of progressive collapse analysis, the structural behavior is evaluated by applying a stepwise increase of vertical loads (incremental or iterative approach) until structure collapse or maximum loads are attained. These step-by-step increases are complicated to be performed and time consuming. Additionally, the nonlinear static procedure generally leads to overly conservative findings during the structural analysis to assess the progressive collapse potential. Finally, Marjanishvili and Agnew, (2006) stated that “Nonlinear static analysis procedure is limited to structures where dynamic behavior patterns can be easily and intuitively identified” [33].

2.4.4 Nonlinear Dynamic Procedure

Nonlinear dynamic method is the most sophisticated and detailed structural analysis method. It performs if the structures are expected to experience nonlinear behavior. This method is known in the structural engineering community as Time History analysis where the structural response is determined as a function of time. In time history analysis procedure inertial effects, nonlinearities for both geometry and materials including second order effects such as P-delta, and the dynamic nature are accounted. In this approach, an assumption has to be considered for the plastic hinges locations and their behavior since the members are permitted to enter the inelastic range of deformation. Furthermore, the moment-rotation relationship is used to define the behavior of the plastic hinges.

The nonlinear dynamic approach (NLD) is the most integrated and vital method for progressive collapse potential assessment provides the most realistic and accurate results. In this evaluation, a critical load-carrying element is instantaneously removed, then the loads are applied without the amplification factor and structural materials are allowed to undergo nonlinear behavior. Following this procedure, the loaded structure is analyzed.

It should be mentioned that time history analysis is generally avoided because of its complexity and the enormous time to generate the model and the long amount of time to execute the model. Also, the evaluation and validation of the outcomes can become an economical concern.

2.5 Methods for Collapse Mitigation

Several approaches have been developed and implemented in the design standards to make the new and existing facilities more resistant to the risk of the progressive collapse due to the abnormal loads. These sorts of loads are extremely difficult to be defined. Therefore, the analysis procedures to evaluate the progressive collapse are established in such a way that it may not always consider loading from these triggered events.

There are, generally, three methods to mitigate the hazard of progressive collapse, being (1) event control, (2) indirect design method, and (3) direct design method as can be seen in Figure 10.

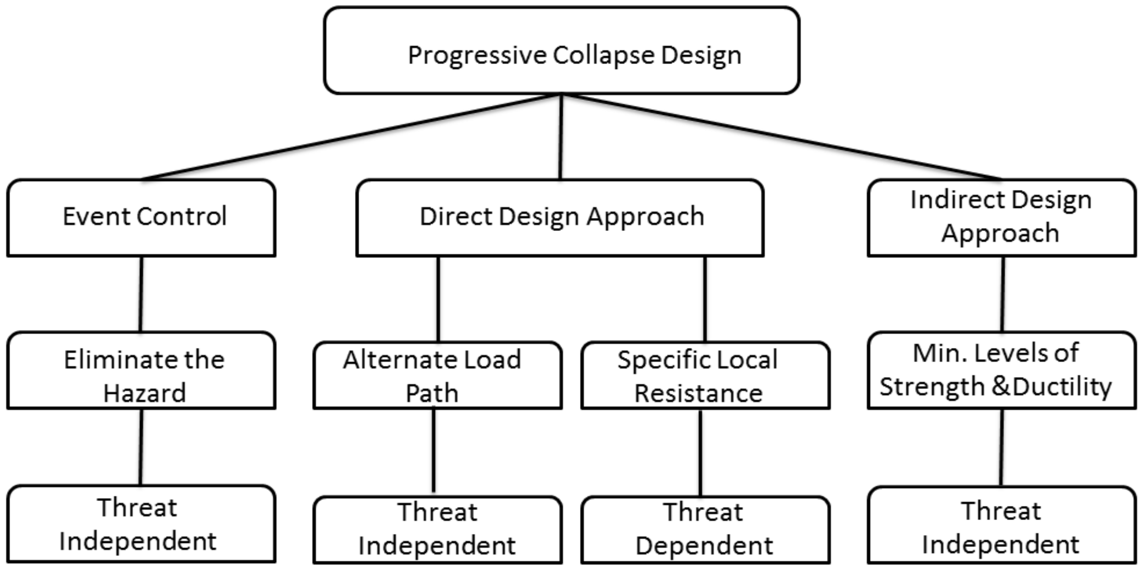

Figure 10. Progressive Collapse Design Methods

The event control design method is not very popular in the design of structures against the threat of progressive collapse event. On the other hand, the rest of the design approaches are distinct processes in the design and assessment of new and existing facilities to the progressive collapse potential. Additionally, the design provisions of the direct and indirect design procedures are addressed in many design codes, standards, and guidelines such as American Society of Civil Engineers (ASCE) “Minimum Design Loads for Buildings and Other Structures” (ASCE 2002), General Service Administration (GSA) “Progressive Collapse Analysis and Design Guidelines” (GSA, 2003), and Department of Defense (DoD) “Design of Buildings to Resist Progressive

Collapse” (DoD, 2010). In the following subsections, the details of each design procedure and its implementation will be explored.

2.5.1 Event Control

The event control design method is used to mitigate or prevent the risk of progressive collapse by considering indirect actions to protect the buildings by eliminating the exposure to abnormal loads.

To mitigate the threat of extreme loads, many suggestions have been proposed by structural engineers and researchers. For examples, isolation of parking zones, elimination of gas installation in multi-story structures as employed in France, and providing a stand-off distance for the facilities at risk of bombings which will avoid large explosion from being close to cause serious damage to the structures [34].

Although this design method is very economical approach to lessen the possibility of progressive collapse occurrence, however it cannot be deemed as an applicable design method in the realm of the disproportionate collapse prevention since it is impractical and difficult to ensure the entire avoidance of the progressive collapse.

2.5.2 Indirect Design Methods

Indirect design methods are threat independent design approaches aim at providing sufficient general integrity for the buildings without any considerations for extreme loads.

In general, to improve the stability and integrity of structures the designer has to develop the redundancy and ductility of the structural system by using continuous interconnections across joints. Additionally, some researchers and design codes recommended that the structure shall be appropriately held together. This can be easily achieved by employing tie forces (Fig. 11) that will give the structural components proper tensile capacity to mature catenary action after local failure occurrence.

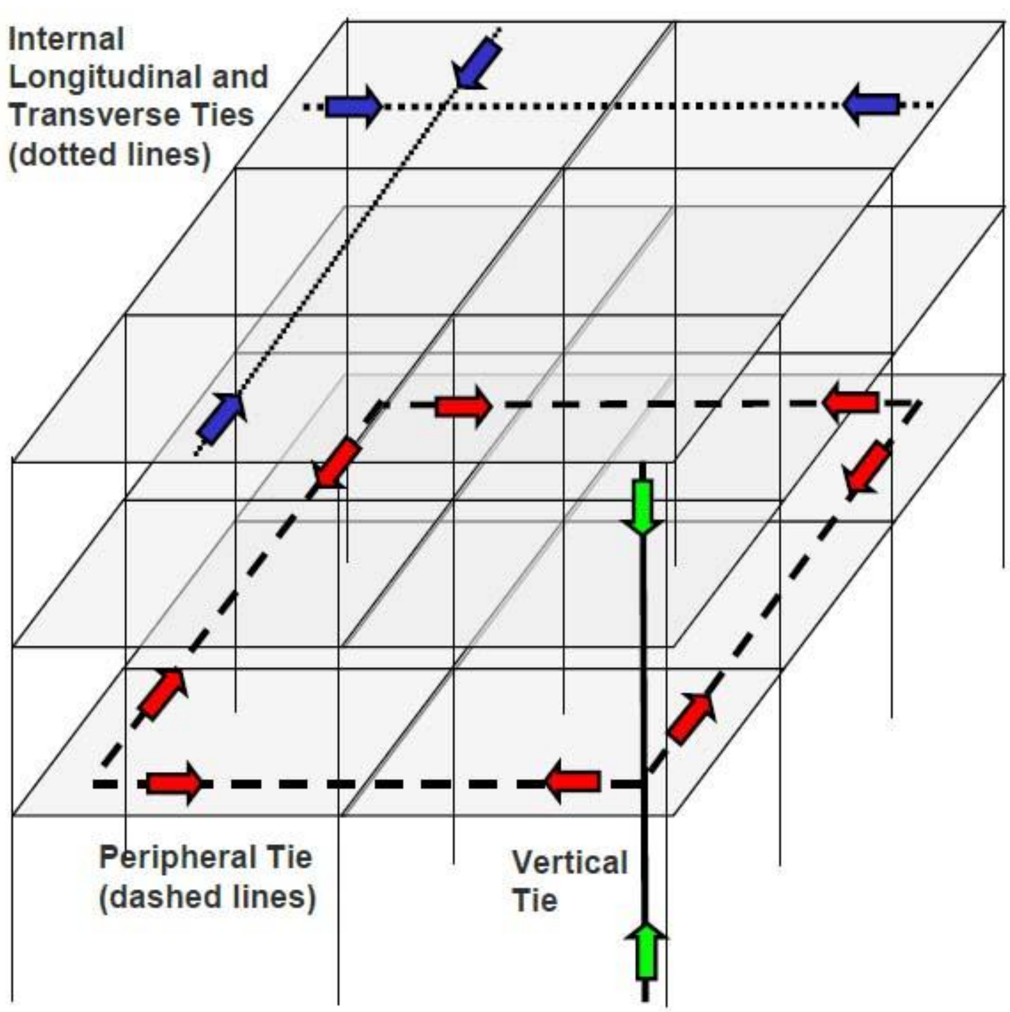

Figure 11. Tie Forces in a Frame Structure [35]

Several design codes and guidelines identified this design approach. For instance, ASCE 7-05 (2005), ACI-318-05 (2005), (GSA, 2003), and UFC 4-023-03 (DoD, 2010) suggest many recommendations regarding the structural ductility, redundancy and continuity in order to increase the ability of the buildings for redistribution of forces. In case of (GSA, 2003), beam-to-beam continuity through a column, more redundant and resilient connections are needed. These suggestions can be accomplished via symmetric reinforcement and increased torsional and minor axis bending strength. ASCE 7-05 (2005) stipulates that adequate redundancy, continuity, and ductility have to be provided. Alternatively, more details have been given in ACI-318-05 (2005) with respect to the building integrity and redundancy, such as, continuous positive reinforcement and mechanical splices have to be exploited, besides the use of moment-resisting frames. Whilst UFC 4-023-03 (DoD, 2010) proposes to employ tie forces to create catenary response of the facility.

2.5.3 Direct Design methods

The abnormal loads are considered in this design approach to resist the progressive collapse in structures. This is done by adopting specific provisions for the design of major elements such as load-carrying members, connections, and beams to provide a good structural performance in the progressive collapse situation. The alternate load path method and the specific local resistance method are the two design approaches that fall under the direct design procedure.

2.5.3.1 Alternate Load Path Method

This design approach is primarily endorsed in the current analysis and design standards associated with progressive collapse phenomenon. Since this method is mainly recommended by the Department of Defense (DoD, 2010) and General Services Administration (GSA, 2003) which are the most two eminent guidelines in this realm, this study has been carried out by implementing the alternate load path method.

The philosophy of this method is to permit the occurrence of the local damage; however the collapse of large portion of the structure is avoided by providing alternate load paths in the neighboring elements to redistribute the loads that were applied on the damaged components if they have designed sufficiently (Fig. 12). Finally, prevent any major failure happening in the facility.

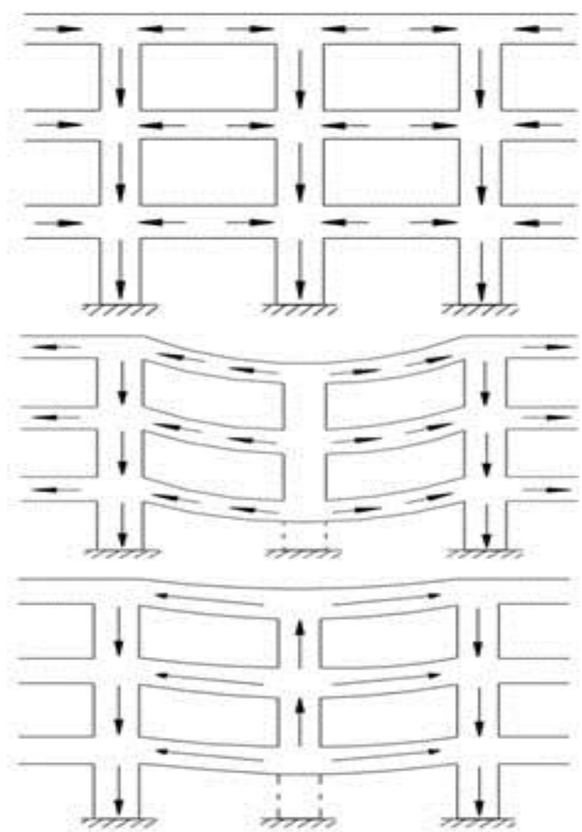

Figure 12. Alternate Load Path Method and Catenary Action

From analytical point of view, exploitation of this method is done by applying the design loads and then different scenarios of instantly removing for one load-carrying member are performed. Last step of this design approach is done by evaluating the capacity of the remaining structure to resist the subsequent failure.

Lastly, the threat independent approach is the prime advantage in this design method, so that it can be effective against any sort of the risk which may cause element loss [23].

2.5.3.2 Specific Local Resistance Method

The specific local resistance is a threat dependent design method. The resistance of structures against extreme load hazard is achieved by strengthening critical elements (key members) in the configuration of the building. This design procedure specifies that the key elements shall remain intact irrespective of the magnitude of the abnormal loads applied on the facility. The strengthening can be reached by providing an adequate ductility and strength to the critical components to resist the progressive collapse. For example, increasing the load factors leads to an additional strength for the key elements.

The changes in the United Kingdom Building Regulations, after the collapse of Ronan Point tower in 1968, specify that the structural members shall be designed to sustain a static pressure of 34 KN/m^2 for gas explosion are an example of this method implementation [32].

2.6 Progressive Collapse Provisions in Codes and Guidelines

2.6.1 British Standard

The United Kingdom was the first country to address the progressive collapse issue in the building standards after the partial collapse of the Ronan Point apartment tower in 1968. The indirect design method is used in these standards by providing horizontal tying forces to enable the beam-column connections to withstand the local damage. These horizontal forces are intended to ensure that the building can bridge over the damaged elements through catenary actions.

In case of providing effective tying forces cannot be met, an approach of alternative element removal has to be adopted. The suggested approach stipulated that a critical load-carrying element is removed once at a time for each floor and then analyze the remaining structure to check if the building can bridge over the removed column albeit in a considerably deformed condition. The behavior of the structure is evaluated throughout the damaged area. The building is deemed to be satisfactory if the collapsed area is limited to 15% of the total floor area or 70 m^2, whichever is the less, and the collapse shall not be propagated further than the immediate adjacent floors [17]. However, no dynamic amplification factor is identified and no computational procedure to assess the propagation of the collapse.

Finally, those members whose notional removal leads to a damage extent exceeds the acceptance criterion, several key members have to be designed to resist a static pressure of 34 KN/m^2.

2.6.2 Eurocode

The Eurocode classify the structures into four categories depend on the consequences of the collapse once it occurs. It adopts three approaches for mitigation of progressive collapse. For the first group, no considerations are required for progressive collapse. For the next class, requirements for horizontal tie force are mentioned. For the last two classes, the requirements for a sufficient tie force need to be met, as well as the building must be designed to withstand the column removal and the damage has to be limited for a particular region. If the vertical load-carrying components resulted in an extensive damage, these elements are deemed as key members and they must be designed to tolerate an additional static pressure of 34 KN/m^2. Finally, the Eurocode is similar to the British Standard in the matter of not specifying quantifiable procedure for the alternate load paths analysis.

2.6.3 U.S. National Institute of Standards and Technology (NIST)

The document entitled "Best Practices for Reducing the Potential for Progressive Collapse in Buildings" was published by NIST in 2007. Even though this document does not offer comprehensive computational procedures to simulate the progressive collapse phenomenon, nevertheless it provides several recommendations for general structural integrity, a review of the methods employed to evaluate and mitigate the potential of progressive collapse, besides an overview of the current codes for buildings design to resist progressive collapse, such as, DoD and GSA guidelines [9].

2.6.4 ASCE 2002 "Minimum Design Loads for Buildings and Other Structures"

Section 1.4 of ASCE 2002 provides some recommendations to improve the general integrity of the structures. These requirements in respect to the progressive collapse design mitigation stipulated that all buildings must be designed to sustain the local failure and the whole structural system has to remain stable and the damage should not be disproportionately extent due to the local failure. These provisions can be achieved by a good arrangement for the structural components to offer stability for the whole building. Additionally, all the structural elements should have satisfactory redundancy, steel reinforcement continuity, ductility, or a combination thereof [36].

In spite of the fact that the ASCE 2002 does not consider an extreme loading scenario or specific threat, yet, it addresses two design methods (direct and indirect design method) to design the building against progressive collapse.

Direct design method deems the progressive collapse resistance by utilizing either the Specific Local Resistance Method or the Alternate Path Method. In the first method, the dimensions and steel reinforcement details are designed to resist particular loads or threats. Whilst, the second allows the occurrence of local failure and through providing alternate load paths for the gravity load and preventing the major collapse.

The indirect design method dictates that the mitigation of the progressive collapse can be reached by providing minimum level of continuity, strength, and ductility.

The commentary section of this code offers some recommendations associated with the specific local resistance method related to some specific charge sizes (weight of TNT), but it does not define the stand-off distances, therefore the actual blast loading is not quantified.

2.6.5 ACI 318-05 "Building Code Requirements for Reinforced Concrete"

The ACI code employs the indirect design method that suggested by the ASCE to address the progressive collapse event.

In order to improve the structural behavior, specific requirements for the steel reinforcement details have to be utilized to enhance the overall stability and integrity of the buildings [37]. The general integrity can be done by including continuous rebar in the parametric members, a specified amount of splicing and the interconnections do not lean on the gravity. Ultimately, there is no assurance that these recommendations might effectively prevent the progressive collapse because the basic concepts are not apparent [38].

2.6.6 GSA "Progressive Collapse Analysis and Design Guidelines"

The United States Public Service Authority (GSA) released a document entitled "Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects" in November 2000 and revised in June 2003. The (GSA, 2003) guideline follows a threat independent methodology for analysis and design of buildings to mitigate the risk of progressive collapse. This guideline was the first document providing an explicit step-by-step process to aid the structural engineering to assess the potential of progressive collapse of federal facilities [39].

The prime feature of this code is the implementation of the alternate load path approach to model the structure under various load-bearing removal scenarios.

Herein, the major characteristics of the (GSA, 2003) will be summarized by the author to offer a brief clarification of this document and for a simple elucidation.

First of all, the main purpose of this guideline is to contribute to the evaluation of the potential progressive collapse in new and existing buildings besides the improvement of potential upgrades to facilities.

This document divides the buildings into two groups according to their configurations typical or atypical structures. All the buildings are typical unless they have one or more of these configurations:

- Plan Irregularities
- Variations in Bay Size/Extreme Bay Sizes
- Vertical Discontinuities/Transfer Girders
- Closely Spaced Columns
- Combination Structures

The GSA guideline can be applied for the majority of the steel and reinforced concrete structures low-to-medium-rise unless these structures are exempted from the progressive collapse considerations. The exemption of the facilities relies on certain concepts. For instance, building classification whether reinforced concrete or steel frame buildings, building occupancy, seismic zone and number of floors. The four analysis approaches; Linear Static procedure, Linear Dynamic procedure, Nonlinear Static procedure, Nonlinear Dynamic procedure can be employed [40].

The analysis of the typical facilities with a simple layout is performed according to the following scenario:

- Framed or Flat Plate Structures
  i. Exterior Considerations

Analyze the structure after the notional removal for a load-carrying element for the first floor situated at or near the middle of short side, middle of long side, or at the corner of the building as shown in Figure 13 [40].

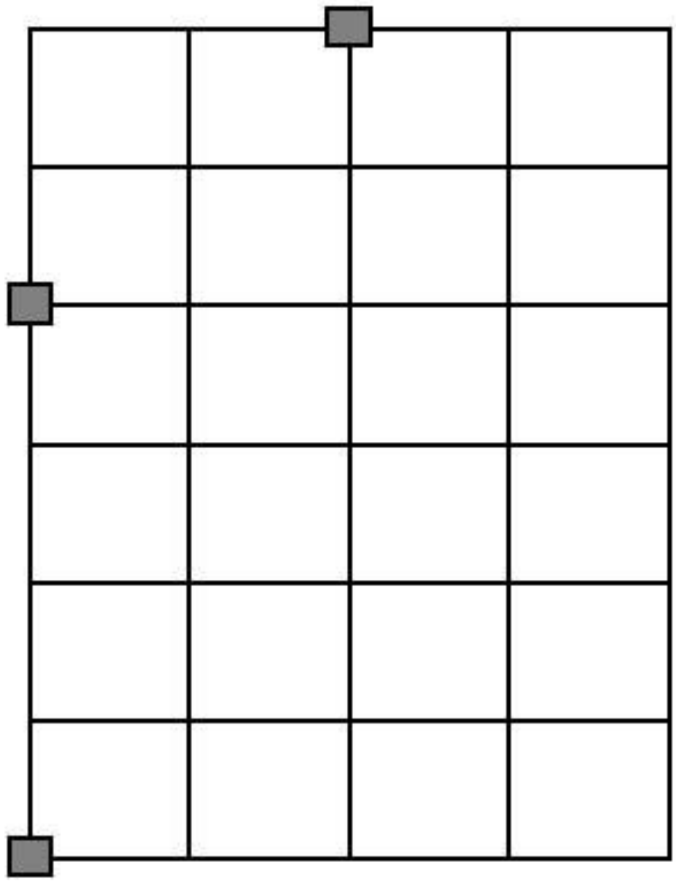

Figure 13. Locations of Removed Column for Exterior Considerations [40]

ii. Interior Considerations

This analysis procedure is carried out for the buildings having underground parking and/or uncontrolled public ground area.

The analysis is carried on by an instantaneous loss of one load-bearing component that extends from the floor of the underground parking area or uncontrolled public ground floor area to the first story as displayed in Figure 14 [40].

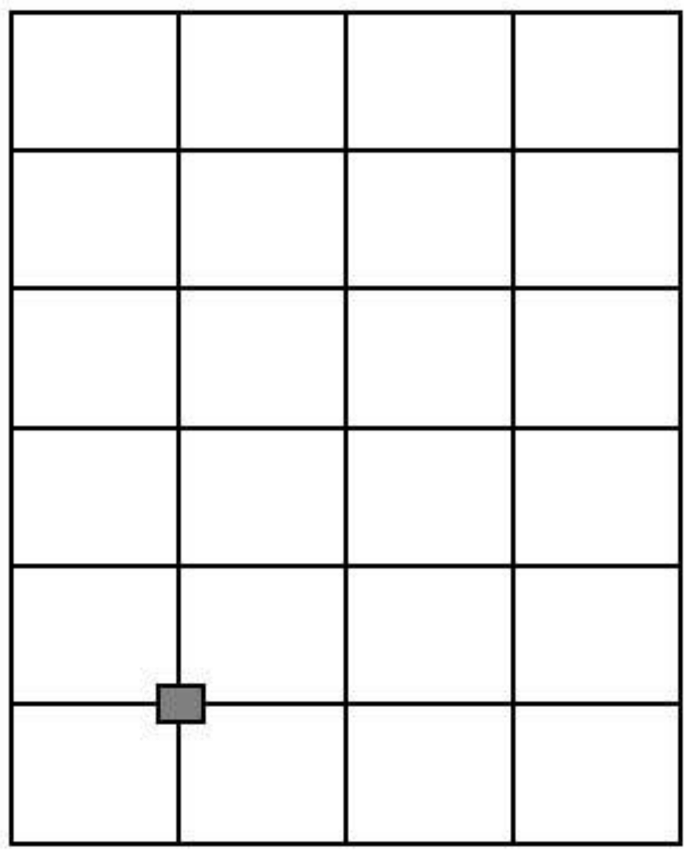

Figure 14. Locations of Removed Column for Interior Considerations [40]

- Shear/Load Bearing Wall Structures

  i. Exterior Considerations

Analyze the structure after the notional removal for one structural bay or 30 linear feet whichever is less from the exterior wall section for the first floor located at or near the middle of short side or middle of long side. Whilst, the loss of wall section for the corner consideration is accomplished by removing of the whole bearing-wall along the perimeter at the corner bay or 30 linear feet (15 feet in each main direction) whichever is less as shown in Figure 15 [40].

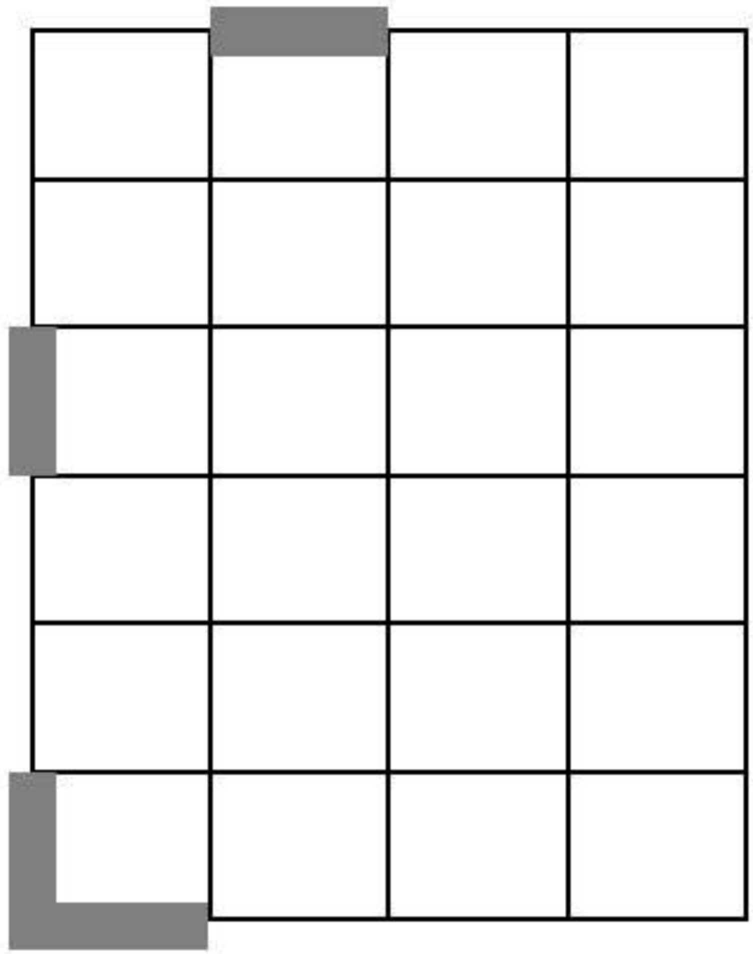

Figure 15. Locations of Removed Load-Bearing Wall for Exterior Considerations [40]

ii. Interior Considerations

The analysis is performed by an instantaneous loss of one structural bay or 30 linear feet whichever is less at the floor level of the uncontrolled ground and/or underground parking area as presented in Figure 16 [40].

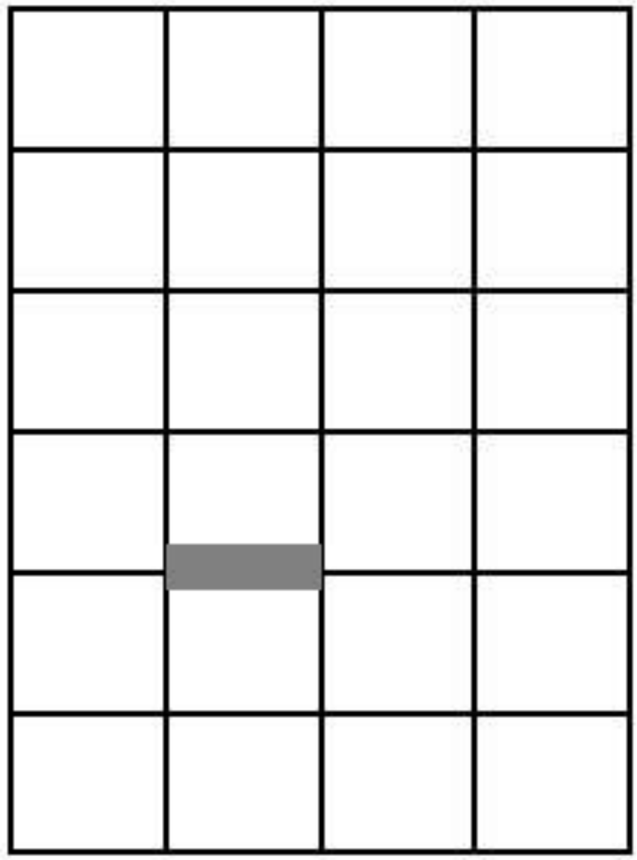

Figure 16. Locations of the Load-Bearing Wall Removed for Interior Considerations [40]

For the evaluation of the atypical facilities the engineering knowledge should be used to define the critical analysis situations in addition to those mentioned above.

For the assessment of the building under consideration the following load combination has to be used for the static analysis procedure whether linear or nonlinear.

$$\mathbf{Load = 2(DL + 0.25LL)} \tag{1}$$

where,

DL = dead load

LL = live load

The (GSA, 2003) guideline specifies that only 25 percent of the live load has to be applied in vertical load combination because of the possibility of presence of the full live load during the collapse being very low. A magnification factor of 2 is used in the static analysis approach to account for dynamic effects [31].

For the linear or nonlinear dynamic analysis procedure, the equation of the load combination will be:

Load = DL + 0.25LL (2)

For the exterior considerations the maximum acceptable extend of collapse resulting from the removal of an exterior critical load-bearing element must be restricted to the structural bay directly related to the removed load-carrying components in the floor level directly above the removed element, or 1,800 square feet at the floor level directly above the removed member whichever is less [40].

However, for the interior considerations, these criteria are limited to the structural bay directly linked to the instantaneously removed element, or 3,600 square feet at the floor level of the primary vertical support system [40].

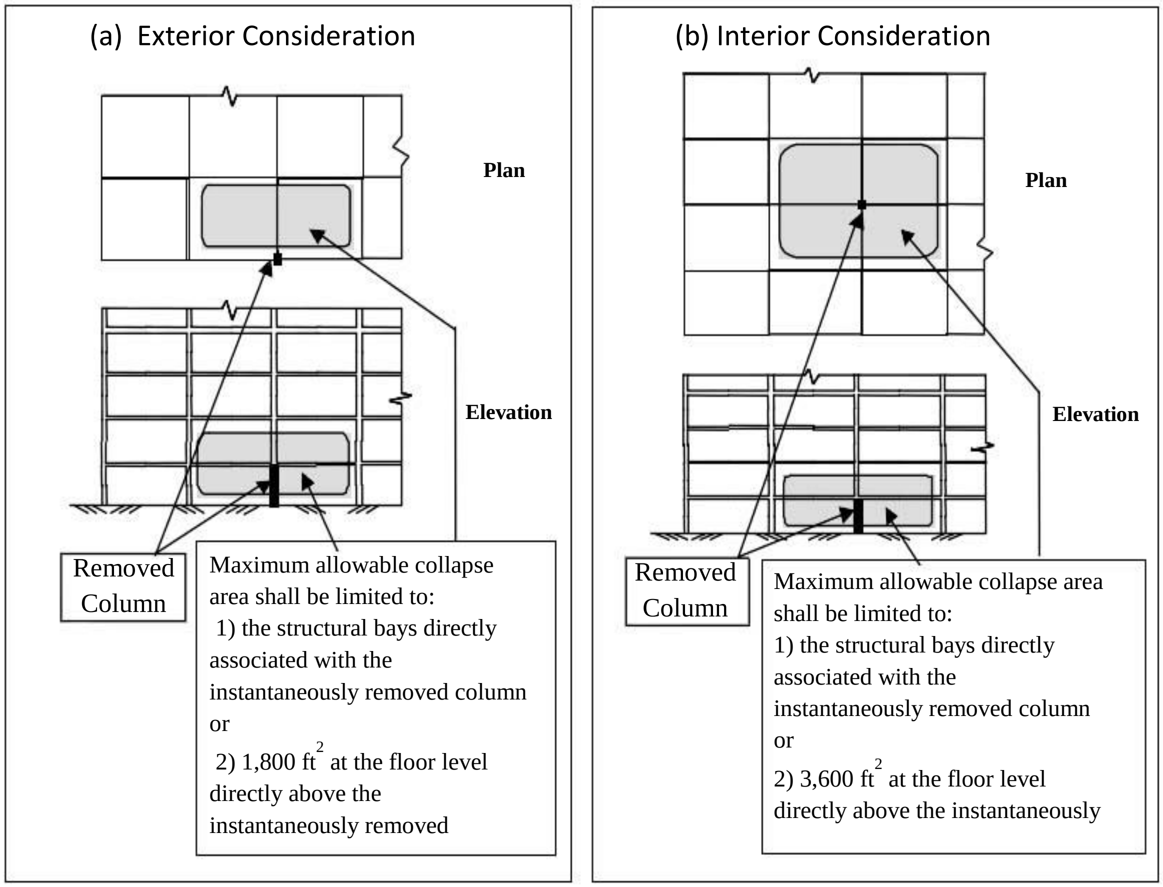

Figure 17. Maximum Allowable Collapse Areas for A Structure that Uses Columns for the Primary Vertical Support System [40]

Finally, after finishing the analysis, the demand-capacity ratios (DCRs) are computed for each single element of the structure.

$$DCR=\frac{QUD}{QCE} \tag{3}$$

where,

$\mathbf{Q_{UD}}$ = Acting force (demand) determined in component or connection/joint (moment, axial force, shear, and possible combined forces).

$\mathbf{Q_{CE}}$ = Expected ultimate, un-factored capacity of the component and/or connection/joint (moment, axial force, shear and possible combined forces).

Generally, the acceptable DCR values for primary and secondary elements are:

- DCR $\leq$ 2.0 for typical structural configurations
- DCR $\leq$ 1.5 for atypical structural configurations

2.6.7 DoD: "Design of Buildings to Resist Progressive Collapse"

The "Design of buildings to resist progressive collapse" was issued by the United States Department of Defense in June 2005. This document was prepared for the design of the military facilities with three or more stories that necessitate progressive collapse considerations. The DoD guideline provides a thorough explanation for progressive collapse design and assessment for new and existing buildings respectively constructed by reinforced concrete, steel structures, masonry, wood and cold-formed steel structures. The analysis and design procedure is based on the level of protection (LOP) and the occupancy category (OC).

A comprehensive description of this code will be presented by the author in the next chapter since this document is utilized as the main guideline for this research.

Figure 18 shows the timeline of major catastrophic events followed by major building codes changes for progressive collapse mitigation.

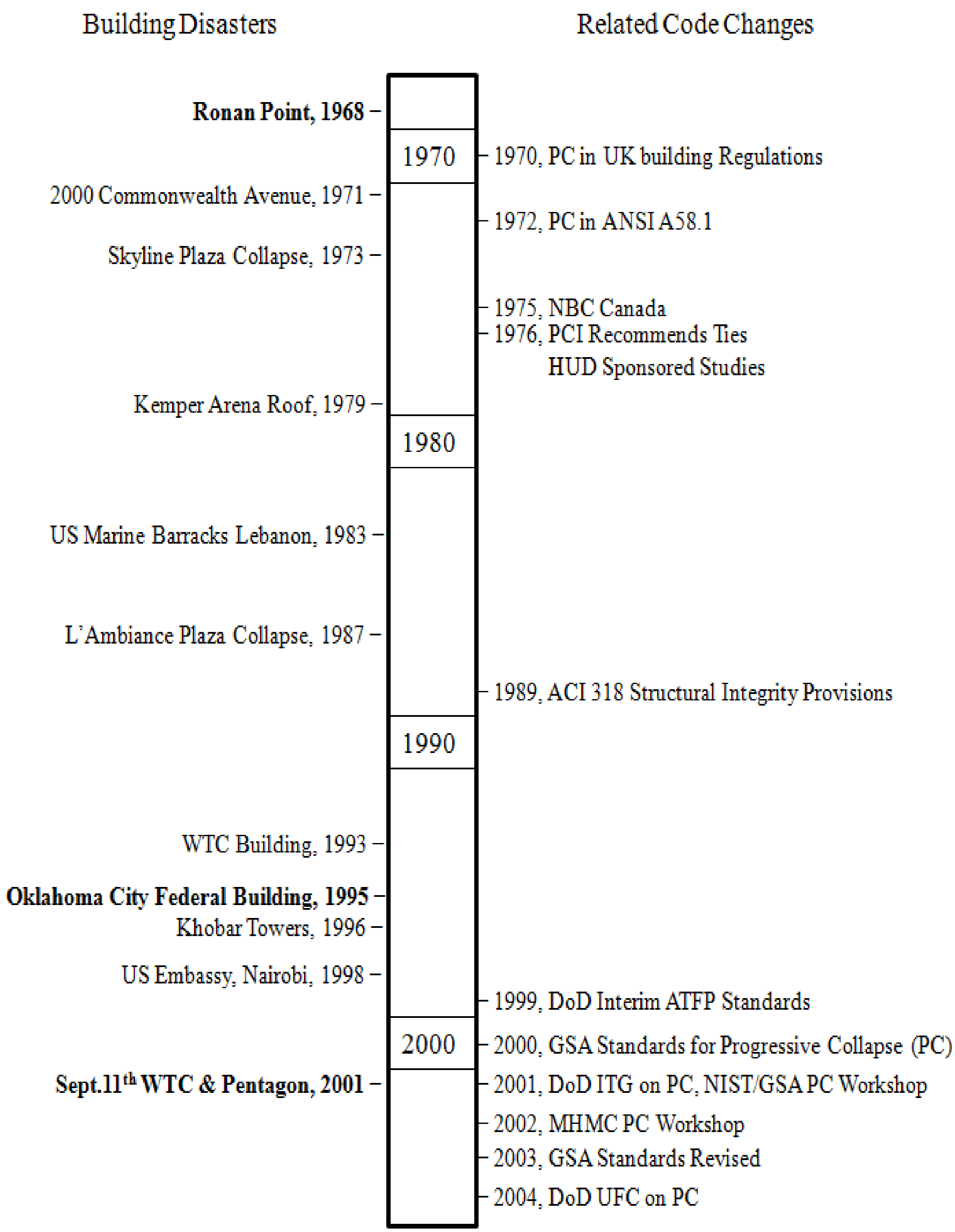

Figure 18. Timeline of Major Disastrous Events Followed By Major Changes in Structural Design Standards [23]

2.7 Typology of Progressive Collapse

There is a possibility of six different types of progressive collapse in a structure. They will only be named without any more details since they have no influence on this study [7].

- Pancake-type Collapse
- Zipper-type Collapse
- Domino-type Collapse
- Section-type Collapse
- Instability-type Collapse
- Mixed-type Collapse

Chapter 3

METHODOLOGY AND COMPUTATIONAL ANALYSIS

3.1 Introduction

Initially, the methodology of this research will be described by following UFC 4-023-03 "Design of buildings to resist progressive collapse" (DoD, 2010) guideline. Moreover, brief description for the case study buildings will be presented. Finally, the details about the structural analysis to assess the progressive collapse performance of four existing buildings will also be presented in this chapter.

3.2 Analytical Approach

3.2.1 Background

The Unified Facilities Criteria (UFC) of the Department of Defense released "Design of buildings to resist progressive collapse" guideline (DoD, 2005) and revised it in 2010. This analysis and design document is extensively accepted and referenced in disproportionate collapse scenarios. It has been established and developed to provide the essential requirements in order to decrease and/or eliminate the hazard of progressive collapse event in new and existing federal and military facilities under the situation of sudden unexpected extreme loads. The next revision (DoD, 2010) is formed with the intention that it will be applicable for all types of new facilities or retrofitting of existing buildings that are constructed from reinforced concrete, steel structures, wood, masonry and cold-formed steel. Additionally, according to this guideline, all buildings with three stories or higher have to be designed to prevent progressive collapse situation. With respect to the number of stories of the structure, if 25 percent or more of the interior operational area is occupied, the progressive collapse considerations have to be applied for the whole structure and not only for the used portion. On the other hand, if any floor is utilized as storage or it is not occupied by residents, that story must be neglected from calculation of the building height [35].

3.2.2 Risk Considerations

The records for progressive collapse incidents due to natural disasters, bomb attacks and vehicle collisions are very limited. Therefore, it is not easy to reasonably evaluate the progressive collapse occurrence as a result of individual threat or group of threats. In general, the assessment of progressive collapse risk and its consequences are standardized by the number of casualties. In other words, the most stringent issue in disproportionate collapse assessment is the building occupancy. Despite the fact that the function of the building is a critical issue in the design considerations of this phenomenon, the approaches of this document are essentially emphasize the occupancy of the intended designed facility. For that reason, the criticality of the building or its function and the occupancy level are the two major factors in progressive collapse measurements and its consequences.

The structure occupancy is defined as the Occupancy Category (OC) of the building according to this DoD guideline. The Occupancy Category (OC) is the prime function in the evaluation and design of existing and new facilities against the progressive collapse event. Moreover, this term will either be identified by the building holder or will be determined according to the requirements of this document as stipulated in Table 1[41].

Table 1. Occupancy category of buildings and other structures [41]

Occupancy Category	Nature of Occupancy
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities
II	Buildings and other structures except those listed in Categories I, III, IV and V
III	Buildings and other structures that represent a substantial hazard to human life or represent significant economic loss in the event of failure, including, but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300 people • Buildings and other structures containing elementary school, secondary school, or daycare facilities with an occupant load greater than 250 • Buildings and other structures with an occupant load greater than 500 • Group I-2 occupancies with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities • Group I-3 occupancies • Power-generating stations; water treatment facilities for potable water, waste water treatment facilities, and other public utility facilities that are not included in Categories IV and V • Buildings and other structures not included in Categories IV and V containing sufficient quantities of toxic, flammable, or explosive substances to be dangerous to the public if released • Facilities having high-value equipment, as designated by the authority having jurisdiction

IV	Buildings and other structures designed as essential facilities, including, but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities • Fire, rescue, and police stations, and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers, and other facilities required for emergency response • Emergency backup power-generating facilities required for primary power for Category IV • Power-generating stations and other utility facilities required for primary power for Category IV, if emergency backup power generating facilities are not available • Structures containing highly toxic materials as defined by Section 307, where the quantity of material exceeds the maximum allowable quantities of Table 307.7(2) • Aviation control towers and air traffic control centers required for post-earthquake operations where lack of system redundancy does not allow for immediate control of airspace and the use of alternate temporary control facilities is not feasible. Contact the authority having jurisdiction for additional guidance. • Emergency aircraft hangars that house aircraft required for post-earthquake emergency response; if no suitable back up facilities exist • Buildings and other structures not included in Category V, having DoD mission-essential command, control, primary communications, data handling, and intelligence functions that are not duplicated at geographically separate locations, as designated by the using agency • Water storage facilities and pump stations required to maintain water pressure for fire suppression
Vb	Facilities designed as national strategic military assets, including, but not limited to: • Key national defense assets (e.g. National Missile Defense facilities), as designated by the authority having jurisdiction. • Facilities involved in operational missile control, launch, tracking, or other critical defense capabilities • Emergency backup power-generating facilities required for primary power for Category V occupancy • Power-generating stations and other utility facilities required for primary power for Category V occupancy, if emergency backup power generating facilities are not available • Facilities involved in storage, handling, or processing of nuclear, chemical, biological, or radiological materials, where structural failure could have widespread catastrophic consequences, as designated by the authority having jurisdiction.

The UFC stated that the occupancy category has an influence on the selection of the design approaches whether Tie Forces method, Alternate Load Path method or Enhanced Local Resistance method. Furthermore, Table 2 illustrates the design requirements for each level of occupancy. In the following subsections more details will be provided for the design method of this research.

Table 2. Occupancy categories and design requirements [35]

Occupancy Category	**Design Requirement**
I	No specific requirements
II	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.
III	Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first story columns or walls.
IV	Tie Forces; Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first and second story columns or walls.

3.2.3 Primary and Secondary Components

The structural components are generally designated as either primary or secondary elements. The structural members and components which provide the capacity against progressive collapse event as a result of theoretical removal of a vertical load-carrying member are classified as primary elements. All other members and components are categorized as secondary elements [35].

3.2.4 Force-and Deformation-Controlled Actions

All actions are either categorized as force-controlled or deformation-controlled using the element force and deformation curve presented in Figure 19. Moreover, the member perhaps would be force-controlled accommodated by deformation-controlled. The provisions of this document have to be followed to define force- or deformation-controlled action and it is not up to the designer.

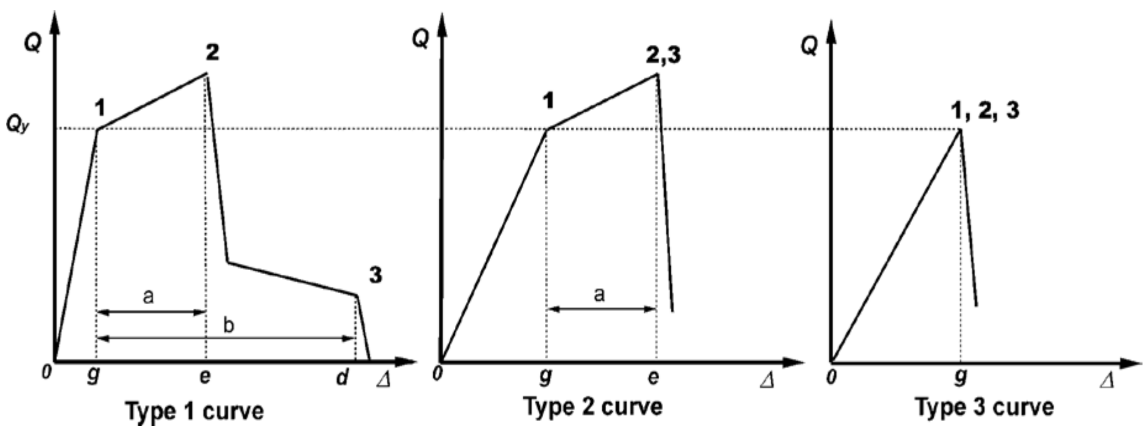

Figure 19. Force-Controlled and Deformation-Controlled Actions [35]

If the primary component has a Type 1 curve with e ≥ 2g, or, it has a Type 2 curve with e ≥ 2g, it is classified as deformation-controlled action. Whereas, the secondary element is classified as deformation-controlled action if it has a Type 2 curve with e < 2g, or, it has a Type 3 curve.

Nevertheless, the primary element is defined as force-controlled action if it has a Type 1 or Type 2 curve with e < 2g, or, if it has a Type 3 curve. If the secondary component has a Type 2 curve with e < 2g, or, it has a Type 3 curve, then it is categorized as force-controlled action [35]. Some examples of force- and deformation- controlled actions are listed in Table 3.

Table 3. Examples of deformation-controlled and force-controlled actions [35]

Component	Deformation- Controlled Action	Force- Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V[1]
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	P, V, M[2]	P, V, M
1. Shear may be a deformation-controlled action in steel moment frame construction. 2. Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.		

3.2.5 Expected and Lower Bound Strength

The expected bound strength is used to evaluate the behavior of deformation-controlled actions. The expected strength, $\mathbf{Q_{CE}}$ represents the statistical average value of the strength, such as, compressive, tensile, yield, etc. as suitable for tested material properties with the inclusion of the inconsistency in strength of materials and strain hardening. However, the lower bound strength is defined as the statistical average value minus one standard deviation of the strength (tensile, compressive, yield, etc. as appropriate) for a population of similar elements. The lower strength $\mathbf{Q_{CL}}$ is used to assess the performance of force-controlled actions.

It should be mentioned that the appropriate strength reduction factors have to be employed in the calculation of $\mathbf{Q_{CE}}$ and $\mathbf{Q_{CL}}$.

For instance, the load reduction factor $\mathbf{\Phi}$ in the ACI 318 code must be applied for reinforced concrete structures.

3.2.6 Alternate Load Path Design Method

This design approach focuses on the behavior of the structural system following the occurrence of the extreme event. According to this approach, the structure is required to redistribute the loads after the loss of a primary load bearing member and to remain stable. The method provides a check of the capability of the structural system to resist the removal of specific elements, such as, columns and/or load bearing walls, through careful assessment of the capacity of the remaining structure.

The ALP method consists in considering stresses redistribution through-out the structure following the loss of a vertical support element. The structure is bound to find alternative paths for the forces initially carried by the failing elements. It is thus a threat-independent approach to progressive collapse: the main purpose is to analyze the progressive spread of damage after localized failure has occurred.

The GSA and DoD requirements for the ALP application include the analysis of the structural response to a key structural element removal, in order to simulate a local damage comparable to the one produced in a blast or impact load scenario.

3.2.7 Removal of Load-Carrying Members for the Alternate Path Method

In the alternate load path method, the beam-to-beam continuity has to be sustained through the removal of the load-bearing element. Figure 20 shows the correct and incorrect approach to column removal.

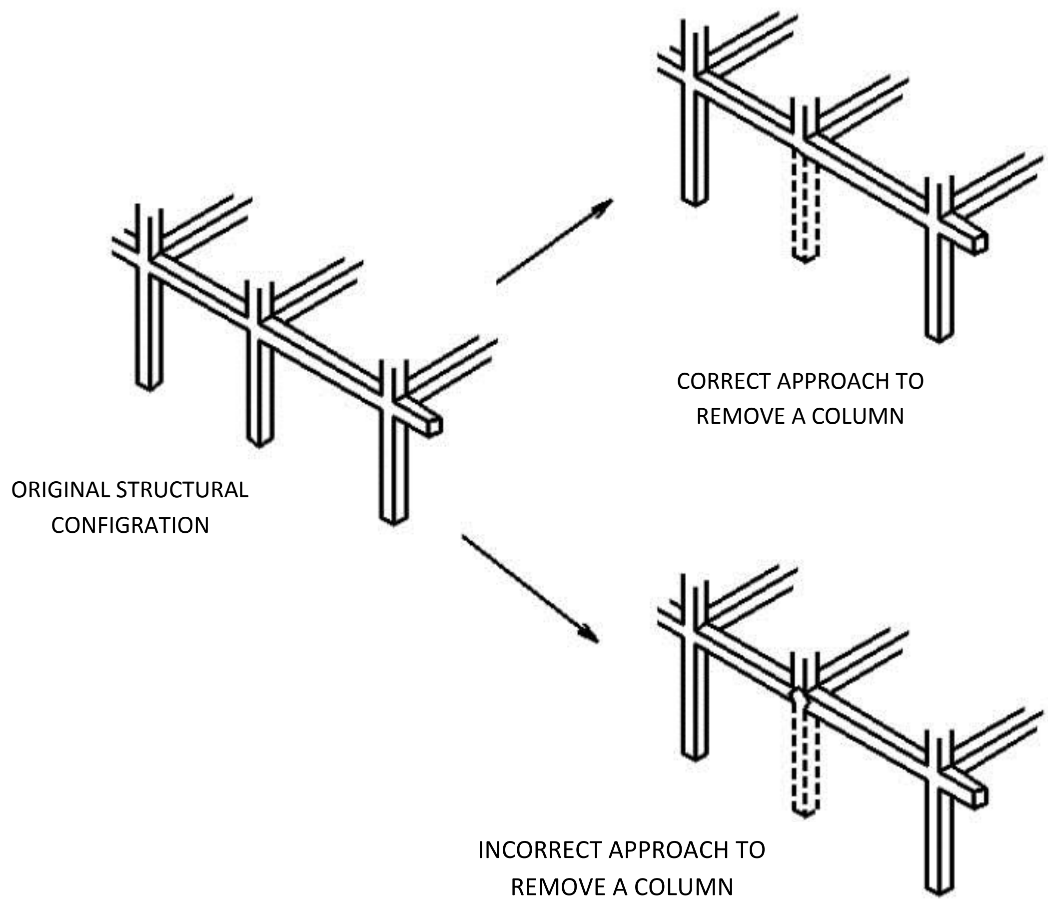

Figure 20. Notional Column Removal in Alternate Load Path Method [35]

The Unified Facilities Criteria (UFC 4-023-03) classify the load-carrying element removal into four clusters. These groups are: interior column and exterior column removal, interior load-bearing wall and exterior load-bearing wall removal. The internal load-carrying member removal is considered if the facility has underground parking or other uncontrolled areas, while the external load-bearing element removal is performed for the remaining structural configurations. In this study, the external column removal scenario was conducted. Therefore, only the design requirements for this case will be thoroughly described.

The DoD guideline stipulates that the minimal external column removal scenarios are as follows (Figure 21):

- at or near the middle of the short side
- at or near the middle of the long side
- at the corner of the facility

Additionally, the engineering judgment must be implemented to identify other critical column locations that have to be removed if there are significant changes in the structural geometry. For example, unforeseen decrease in the size of bay, frame elements located at variant elevations or orientations, locations where ad-joining load-bearing components have lighter loads.

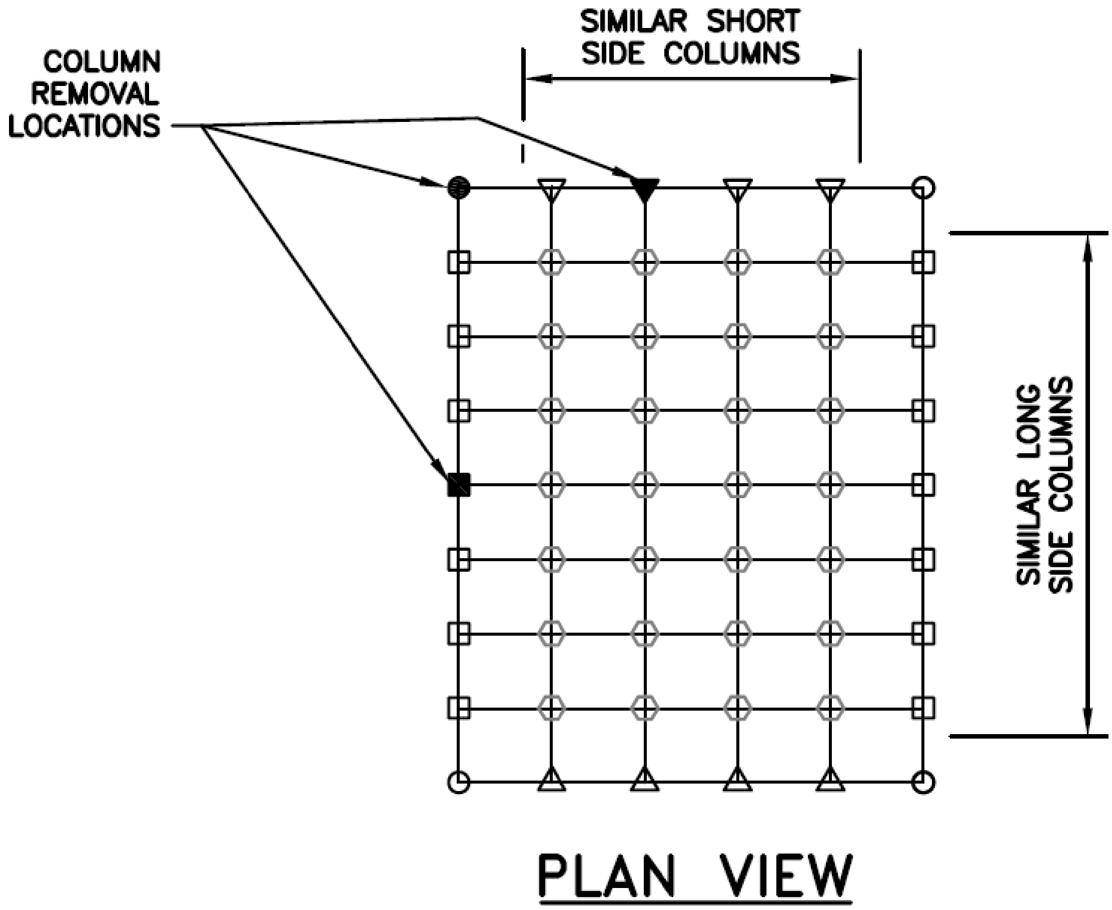

Figure 21. Location of External Column Removal [35]

The locations of the notionally removed column are:

- First story above grade (first floor)
- Story directly below roof (last floor)
- Story at mid-height
- Story above the location of a column splice or change in column size

For instance, if a column situated at the middle of the short side is selected to be removed in a nine-story structure with a change in column side in the third story, the first alternate load path analysis is done for removal of the ground floor (first floor); the next analysis is conducted for the column located at the middle of the short side in the ninth floor (last floor); the third alternate load path analysis is conducted for the column in the fourth story (floor above the changed column size) and the last analysis is performed for the fifth story (mid-height floor).

3.2.8 Limitations on the Use of Linear Static Procedure

If the structure meets the conditions for Demand-Capacity Ratios (DCRs) and structural irregularities, the linear static procedure can be used for building analysis.

The linear static analysis method can be conducted and there is no necessity to calculate the demand capacity ratios if there are no structural irregularities. It is possible to use the linear static procedure if the building is irregular and the computed DCRs for all the structural members are less than or equal to 2.0. Nevertheless, it is not permitted to perform this analysis method if the DCR for any component is more than 2.0.

The Unified Facilities Criteria guideline considers a structure as irregular if one or more of the following parameters are available [35]:

- Significant discontinuities exist in the gravity-load carrying and lateral force-resisting systems of a building, including out-of-plane offsets of primary vertical elements, roof “belt-girders”, and transfer girders (i.e., non-stacking primary columns or load-bearing elements). Stepped back stories are not considered as irregularity.

- At any exterior column except at the corners, at each story in a framed structure, the ratios of bay stiffness and/or strength from one side of the column to the other are less than 50%. Three examples are;

  a) the lengths of adjacent bays vary significantly

  b) the beams on either side of the column vary significantly in depth and/or strength

  c) connection strength and/or stiffness vary significantly on either side of the column (e.g., for a steel frame building, a shear tab connection on one side of a column and a fully rigid connection on the other side shall be considered irregular).

- For all the external load-bearing walls, except at the corners, and for each story in a load-bearing wall structure, the ratios of wall stiffness and/or strength from one side of an intersecting wall to the other are less than 50 percent.

- The vertical lateral-load resisting elements are not parallel to the major orthogonal axes of the lateral force-resisting system, such as the case of skewed or curved moment frames and load-bearing walls.

In order to compute the demand capacity ratios for either load-bearing or framed buildings, the designer simulates the linear static model for the structure. All the primary members shall be included except the removed column or load-carrying elements. Applying the deformation-controlled load condition with dead and live loads multiplied by the load increase factor ($\mathbf{\Omega_{LD}}$), the outcome of the analysis (internal forces and flexures) are defined as $\mathbf{Q_{UDLim}}$.

$$\mathbf{DCR = Q_{UDLim}/Q_{CE}} \quad (4)$$

where,

$\mathbf{Q_{CE}}$ = Expected strength of the component.

3.2.9 Loading

In the design of structures to resist progressive collapse event, two load conditions have to be performed. The first load case is used for the deformation-controlled actions, and the second one is utilized for the force-controlled actions. These two various load cases are applied as a result of the difference in the calculation of force-controlled and deformation-controlled actions.

3.2.9.1 Load Case for Deformation-Controlled Actions $\mathbf{Q_{UD}}$

In order to compute the deformation-controlled actions, the combination of lateral and gravity loads are concurrently applied as follows:

3.2.9.1.1 Increased Gravity Loads for Floor Areas above Removed Column or Wall

The increased gravity load combination has to be applied for floor area above the removed load-bearing elements. They are applied to those bays directly adjoining to the removed member besides above the removed member at all levels, as shown in Figure 22.

$$\mathbf{G_{LD} = \Omega_{LD}\ [(0.9\ or\ 1.2)\ D + (0.5\ L\ or\ 0.2\ S)]} \quad (5)$$

where,

$\mathbf{G_{LD}}$ = Increased gravity loads for deformation-controlled actions for Linear Static Analysis

D = Dead load (lb/ft2 or kN/m2)

L = Live load (lb/ft2 or kN/m2)

S = Snow load (lb/ft2 or kN/m2)

$\mathbf{\Omega_{LD}}$ = Load increase factor for calculating deformation-controlled actions for Linear Static analysis

3.2.9.1.2 Gravity Loads for Floor Areas Away From Removed Column or Wall

For the remaining floor areas (those apart from removed elements) only gravity load shall be applied. For those bays as displayed in Figure 22, the following gravity load combination is used:

$$\mathbf{G = (0.9 \text{ or } 1.2)\, D + (0.5\, L \text{ or } 0.2\, S)} \tag{6}$$

where,

G = Gravity loads

3.2.9.1.3 Lateral Loads Applied to Structure

The lateral load is applied to the building accompanied by the combination of the gravity loads $\mathbf{G_{LD}}$ and **G**. The lateral load is applied to each side of the structure, one side at a time. Therefore, four separate analyses have to be conducted, one for each major direction of the facility.

$$\mathbf{L_{LAT} = 0.002\Sigma P} \tag{7}$$

where,

$\mathbf{L_{LAT}}$ = Lateral load

$\mathbf{0.002\Sigma P}$ = 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

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

3.2.9.2 Load Case for Force-Controlled Actions Q_{UF}

In order to calculate the force-controlled actions, the combination of lateral and gravity loads are concurrently applied as follows:

3.2.9.2.1 Increased Gravity Loads for Floor Areas above the Removed Column or Wall

This increased gravity load combination is applied to the immediate neighboring bays to the removed vertical load-bearing component and all floors above them as presented in Figure 22.

$$\mathbf{G_{LF} = \Omega_{LF}\ [(0.9\ or\ 1.2)\ D + (0.5\ L\ or\ 0.2\ S)]} \quad (8)$$

where,

$\mathbf{G_{LF}}$ = Increased gravity loads for force-controlled actions for Linear Static analysis

D = Dead load (lb/ft2 or kN/m2)

L = Live load (lb/ft2 or kN/m2)

S = Snow load (lb/ft2 or kN/m2)

$\mathbf{\Omega_{LF}}$ = Load increase factor for calculating force-controlled actions for Linear Static analysis

3.2.9.2.2 Gravity Loads for Floor Areas Away From the Removed Column or Wall

The gravity load is similar to that one used for the deformation-controlled actions.

3.2.9.2.3 Lateral Loads Applied to Structure

The lateral load is similar to that one used for the deformation-controlled actions.

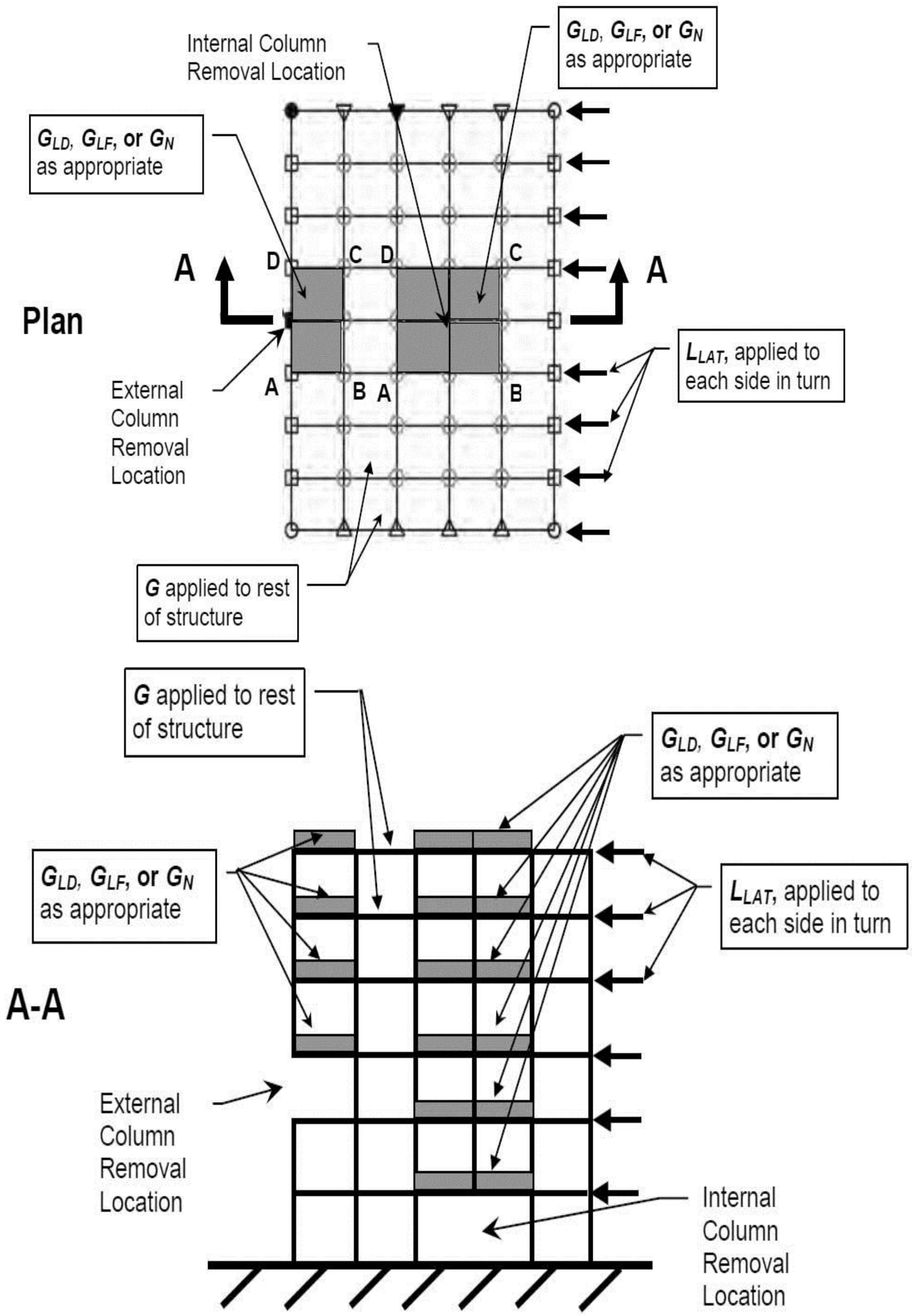

Figure 22. Loads and Load Locations for the Removal of the External Column for Linear Static Analyses [35]

3.2.9.3 Load Increase Factor

The load increase factors are used for both force and deformation- controlled actions for the removal of the load-carrying members; their values are displayed in Table 4. The values of $\mathbf{m_{LIF}}$ that is mentioned in this table should be the lowest m-factor of the primary members (girder, beam and wall member) which are immediately connected to load-bearing element directly above the removed load-carrying members. On the contrary, there is no consideration for the $\mathbf{m_{LIF}}$ values for the columns. The values of m-factor for linear approach of reinforced concrete beams as well as two-way slab floors and column-slab connections which are modified and replaced in Table 6-11 and Table 6-15 in ASCE 41 respectively, are shown in Tables 5 and 6.

Table 4. Load increase factors for linear static analysis [35]

Material	Structure Type	ΩLD, Deformation-controlled	ΩLF, Force-controlled
Steel	Framed	0.9 mLIF + 1.1	2.0
Reinforced Concrete	Framed	1.2 mLIF + 0.80	2.0
	Load-bearing Wall	2.0 mLIF	2.0
Masonry	Load-bearing Wall	2.0 mLIF	2.0
Wood	Load-bearing Wall	2.0 mLIF	2.0

Table 5. Acceptance criteria for linear models of reinforced concrete beams [35]

Conditions			m-factors[1] Component Type Primary	Secondary
i. Beams controlled by flexure[2]				
$\frac{\rho - \rho'}{\rho bal}$	Trans. Reinf.[3]	$\frac{V}{bwd\sqrt{fc'}}$ [4]		
≤ 0.0	C	≤ 3	16	19
≤ 0.0	C	≥ 6	9	9
≥ 0.5	C	≤ 3	9	9
≥ 0.5	C	≥ 6	6	7
≤ 0.0	NC	≤ 3	9	9
≤ 0.0	NC	≥ 6	6	7
≥ 0.5	NC	≤ 3	6	7
≥ 0.5	NC	≥ 6	4	5
ii. Beams controlled by shear[2]				
Stirrup spacing ≤ d /2			1.5	3
Stirrup spacing > d /2			1.5	2
iii. Beams controlled by inadequate development or splicing along the span[2]				
Stirrup spacing ≤ d /2			1.5	3
Stirrup spacing > d /2			1.5	2
iv. Beams controlled by inadequate embedment into beam-column joint[2]				
			2	3

1. Linear interpolation between values listed in the table shall be permitted. See Section 3-2.4 for definition of primary and secondary components.
2. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
3. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ *d*/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (*Vs*) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
4. 4. V is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.4.2.4.1 of ASCE 41.

Table 6. Acceptance criteria for linear models of two-way slabs and slab-column connections

Conditions			m-factors[1]	
			Component Type	
			Primary	Secondary
i. Slabs controlled by flexure, and slab-column connections[2]				
$\frac{Vg}{V0}$ [3]	Continuity Reinforcement[4]			
≤ 0.2	C		16	19
≥ 0.4	C		9	9
≤ 0.2	C		9	9
≥ 0.4	C		6	7
ii. Slabs controlled by inadequate development or splicing along the span[2]				
			-	4
iii. Slabs controlled by inadequate embedment into slab-column joint[2]				
			3	4

1. Linear interpolation between values listed in the table shall be permitted. See Section 3-2.4 for definition of primary and secondary components.
2. Where more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
3. *Vg* = the gravity shear acting on the slab critical section as defined by ACI 318*;* *Vo* = the direct punching shear strength as defined by ACI 318.
4. Under the heading "Continuity Reinforcement," use "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, use "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, use "No."

3.2.10 Elements Acceptance Criteria

All the members have to be categorized as primary or secondary elements, and then they will be classified in respect to the type of actions into force-or deformation-controlled actions prior to checking the acceptance criteria.

3.2.10.1 Deformation-Controlled Actions

For both primary and secondary elements under the deformation-controlled actions situation, the acceptance criteria must be checked as follows:

$$\boldsymbol{\Phi\ m\ Q_{CE} \geq Q_{UD}} \tag{9}$$

where,

Q_{CE} = Deformation-controlled action, from Linear Static model

m = Component demand modifier (m-factor)

Φ = Strength reduction factor from the ACI 318 code

Q_{UD} = Expected strength of the component for deformation-controlled actions

3.2.10.2 Force-Controlled Actions

In this case, there is slight difference in the checking for primary and secondary components as follows:

$$\boldsymbol{\Phi\ Q_{CL} \geq Q_{UF}} \tag{10}$$

where,

Q_{UF} = Force-controlled action, from Linear Static model

Q_{CL} = Lower-bound strength of a component for force-controlled actions

Φ = Strength reduction factor from the ACI 318 code.

3.3 Buildings Description

In this research, four buildings were selected as a case study to evaluate the potential of progressive collapse event. These apartments are located in Famagusta, Northern Cyprus and they were constructed between 1992 and 2004 according to Turkish Standard. The structural system is a conventional (non-ductile) reinforced concrete frame. Two of these apartments are comprised of four-story with a rectangular plan dimensions of 15.15m in width and 17.2m in length for the first building whereas the second one has a plan dimension of 15.3m by 16.0m in width and length respectively. But, the third and fourth buildings are consisted of eight-story with rectangular floor plan dimensions of 15.0m in width and 17.6m in length and 14.8m in width and 15.1m in length respectively.

It should be mentioned that due to the copy right, the author could not include the drawings of the case study buildings.

3.4 Analytical Modeling Procedure

Computational progressive collapse analysis of the four case study apartments was conducted using the commercially available software, SAP2000 (V14.2, 2010) [5], and following the Unified Facilities Criteria (UFC) guideline (DoD, 2010). Linear static three-dimensional (3-D) computer models of each tested building using SAP2000 program were performed. The tested buildings were Occupancy Category II according to the classification of the UFC 4-023-03 document. Alternate load path for specified columns (Option 2) was selected as design requirements of this research. For each building, force-and deformation-controlled actions have been checked for each load case. Different notional removal scenarios were carried out according to the guidelines mentioned. The models were analyzed and the outcomes were summarized. The model did not consider the effect of large deflections since the purpose of these analyses and re-designs are to evaluate the potential of progressive collapse and only prevent the failure of the structure. The failed members were identified and finally they were retrofitted.

The methodology of selecting, modeling and executing the models of the buildings by SAP2000 [5] software was conducted following a step-by-step approach of the DoD guideline.

First of all, all the case study apartments were selected carefully to fulfill the structural regularity stated in the UFC guideline in order to permit the usage of the linear static procedure and avoiding the calculation of the demand capacity ratio for each element of the four selected buildings. Secondly, the dead load of each floor in the structure was computed by hand and added to the live load that specified in the Turkish Standard to calculate the notional lateral load for each floor which will be included in the load combination. Additionally, the researcher divided the analysis and the re-design of each building into two major groups contained of deformation-controlled and force-controlled actions as explained in subsections 3.2.8.1 and 3.2.8.2 respectively. Next step was the division of each prime group into four classes, one class for each column removal location as described in subsection 3.2.6. For the apartments consisted of four-story, three columns were removed from first story (ground floor), fourth story (last floor), second and third stories (mid-height floor) located at the corner, at or near the middle of the short side and at or near the middle of the long side of the facility. The buildings consisted of eight-story, three columns were removed from first story (ground floor), eighth story (last floor), fourth and fifth stories (mid-height floor) situated at the corner, at or near the middle of the short side and at or near the middle of the long side of the facility. In the case of the column removal from the mid-height of the structure, two cases were performed since the total number of building stories are even and it is not possible to define which floor represents the mid-height of the apartment. Also, each class of the four classes was divided into four clusters. This step was done to apply the lateral load on each principle side of the structure, one time for each side; north, south, east and west sides as elucidated in 3.2.8.1.3 and 3.2.8.2.3 subsections. As a result of this organization, each apartment was analyzed and re-designed for 96 times to cover the required scenarios of this document.

Eventually, after executing the computer program and completing the analysis of each structure, the failed elements (beams and columns) were identified without consideration of the large deflection of the structural components. Consequently, the failed members were re-designed according to the ACI 318-05 design code by implementing the factors stipulated in the elements acceptance criteria for the retrofitting requirements. More details for the analysis and re-design outcomes will be provided in the next chapter.

Chapter 4

RESULTS AND DISCUSSION

4.1 Introduction

The findings from the analysis of the case study buildings will be presented and examined in this chapter.

The analysis and design of structure against progressive collapse are different from the conventional analysis and design procedure. In the normal analysis and design several criteria are taken in the considerations. For instance, there is a limitation for the deflection for each structural component and if the deflection in any member exceed the maximum allowable value, that element is deemed as a failed member and it should be re-design. In addition, there was no damage has to be occurred for any component of the building under the service design loads. Alternatively, when the structure is able to provide sufficient load paths to redistribute the gravity and other loads subjected to the destroyed members as well as an adequate catenary action, the local failure in the structure is permitted to happen according to the conditions of the alternate load path method which is the procedure followed in this investigation. Since the prime purpose of the design of buildings is to resist the potential of progressive collapse and to minimize the damage, then, any structural member which could withstand the extreme load hazard regardless of the large deflection is classified as successful member against progressive collapse situation.

In this research, the author has implemented the alternate load path procedure for the analysis of the four selected buildings. After completion of the modeling of each building and executing the analysis software, only the failed members were specified and categorized in respect to their actions, floor number, location of the column removal and direction of the lateral loads applied to each face of the structure as displayed in Figure 23. Finally, one building was chosen as an example to be retrofitted. This classification will be detailed in the next subsections.

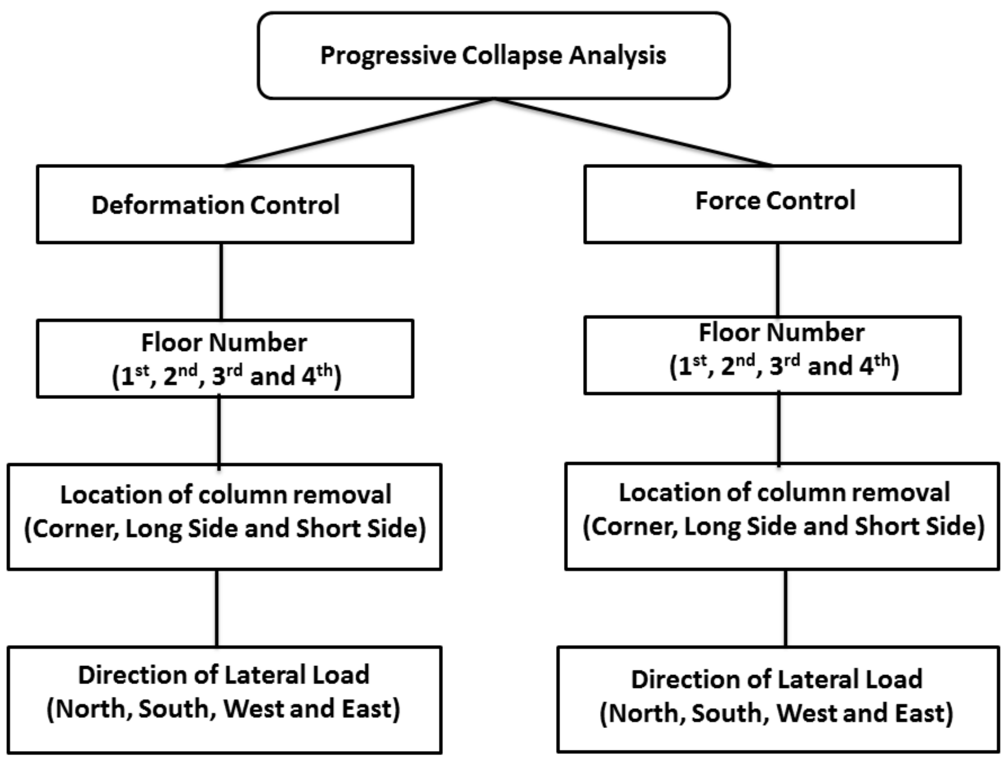

Figure 23. Analytical Approach of the Study

4.2 Analytical Results

4.2.1 First building

This apartment is consisted of four-story with a floor plan of 17.2m by 15.15m as shown in Figure 24. The dimensions and reinforcement for the cross section is presented in Table 7. The plan for the four buildings is presented in the appendix.

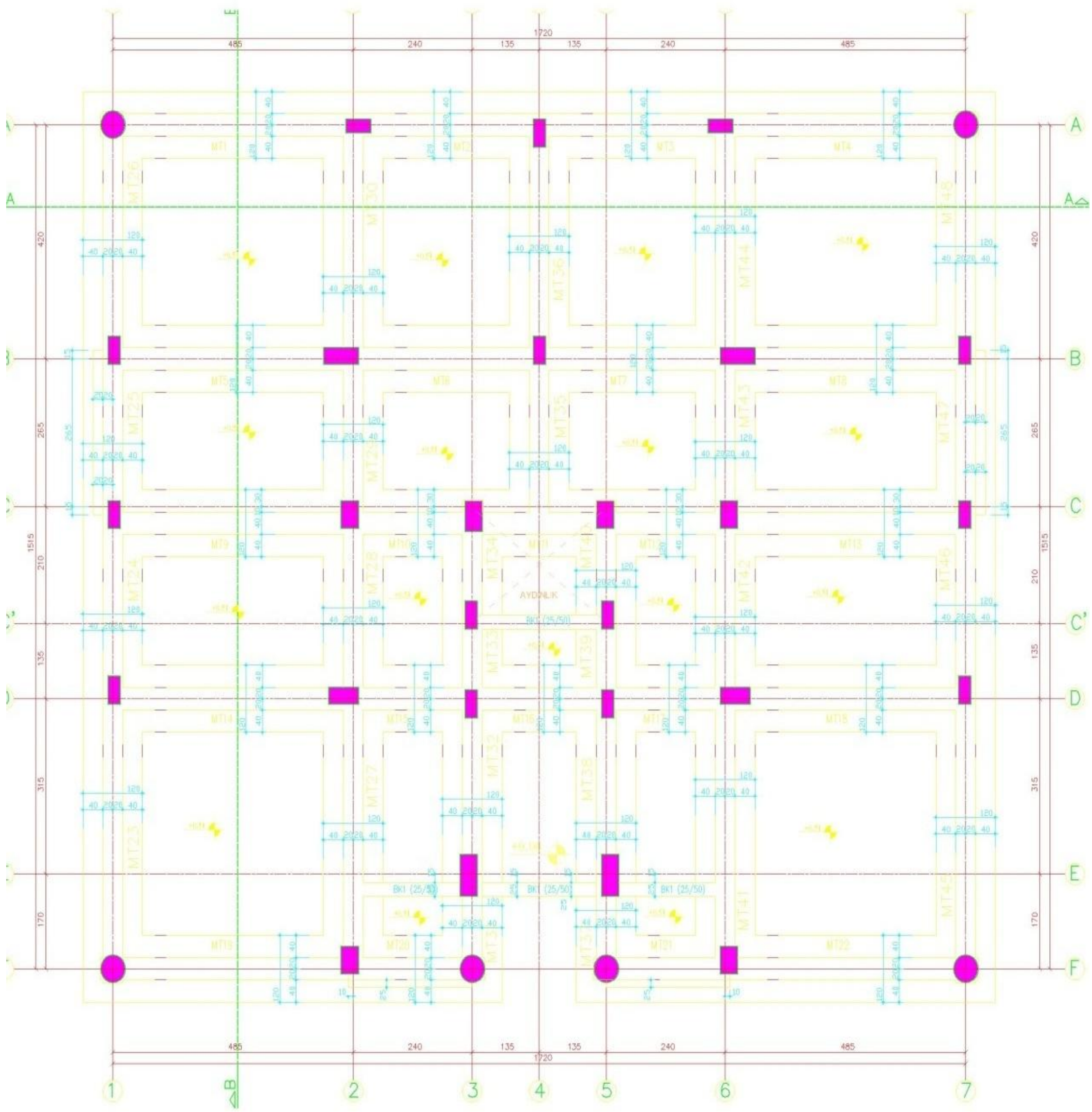

Figure 24. Plan of the First Building

Table 7. Dimensions and reinforcement of the cross sections for the first building

Section	Dimensions (cm)	Reinforcement	
		Longitudinal	Stirrups
Column	25X50	10ϕ14	10@15cm
Column	35X50	12ϕ14	10@15cm
Column	35X75	16ϕ14	10@15cm
Column	50X25	10ϕ14	10@15cm
Column	60X30	12ϕ14	10@15cm
Column	70X30	14ϕ14	10@15cm
Column	D50	10ϕ16	10@12cm
Beam	25X50	10ϕ14	10@12cm

The analysis of the building was divided into two major classes; force-and deformation-controlled according to the actions. The load increase factor in the case of deformation-controlled (**Ω_{LD}**) was 2.6. This value is bigger than the load increase factor for force-controlled (**Ω_{LF}**) which has a constant value of 2.0 for all the construction materials. As a result, the number of the failed elements was much higher accompanied by larger deflection values. Figure 25 displays the deflection values for this building at the combination of gravity load and the notional loads with a maximum deflection value of 0.0745m in the Z-direction at the corner of the last floor.

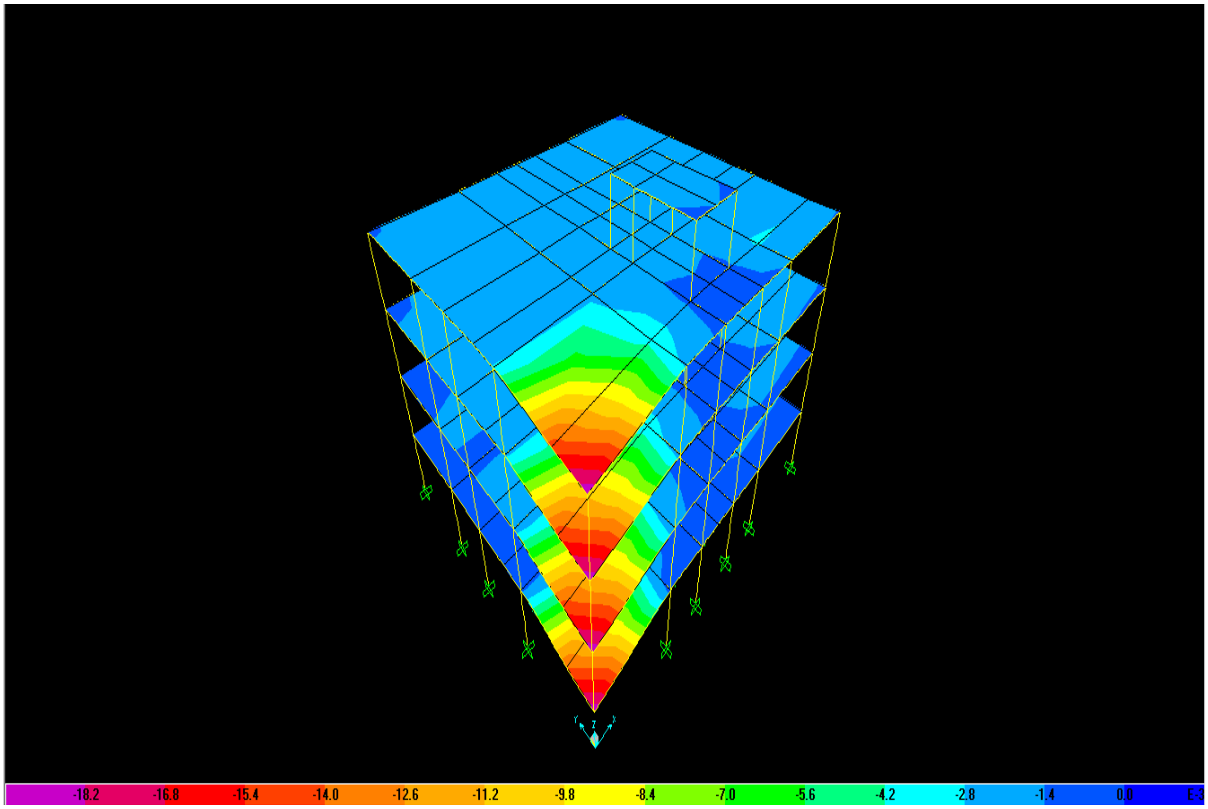

Figure 25. Deformed Shape and Deflection Values for the First Building [5]

However, the column removed from the middle of the short side of the building provided the worst results for both force-and deformation-controlled cases. The maximum number of the failed elements was 13 for the deformation-controlled actions, and 9 for the force-controlled actions. Moreover, the column removal from the first floor resulted in more failed members when compared with other stories. The direction of the lateral loads applied to each major face of the structure has a minor influence on the number of the failed members. Tables 7 and 8 show the summary of the failed elements according to Figure 23.

Location of column removal is given as:

C: the removed column located at the corner of the building.

L: the removed column located at or near the middle of the long side of the building.

S: the removed column located at or near the middle of the short side of the building.

Table 8. Number of the failed elements for first building in the case of deformation-controlled actions

Floor Number	First Story			Second Story			Third Story			Last Story		
Location / Direction	C	L	S	C	L	S	C	L	S	C	L	S
North	5	0	11	4	0	8	2	0	4	0	0	2
South	5	0	11	4	0	8	2	0	5	0	0	2
West	5	0	13	4	0	8	2	0	4	0	0	2
East	5	0	11	4	0	8	2	0	4	0	0	2

Table 9. Number of the failed elements for first building in the case of force-controlled actions

Floor Number	First Story			Second Story			Third Story			Last Story		
Location / Direction	C	L	S	C	L	S	C	L	S	C	L	S
North	4	0	9	2	0	6	1	0	4	0	0	2
South	4	0	8	2	0	6	1	0	4	0	0	2
West	4	0	9	2	0	6	1	0	4	0	0	2
East	4	0	9	2	0	6	1	0	4	0	0	2

4.2.2 Second Building

The second apartment also has four-story in height and the dimensions of the plan are 16.0m by 15.3m for width and length respectively. Figure 26 presents the floor plan of this building.

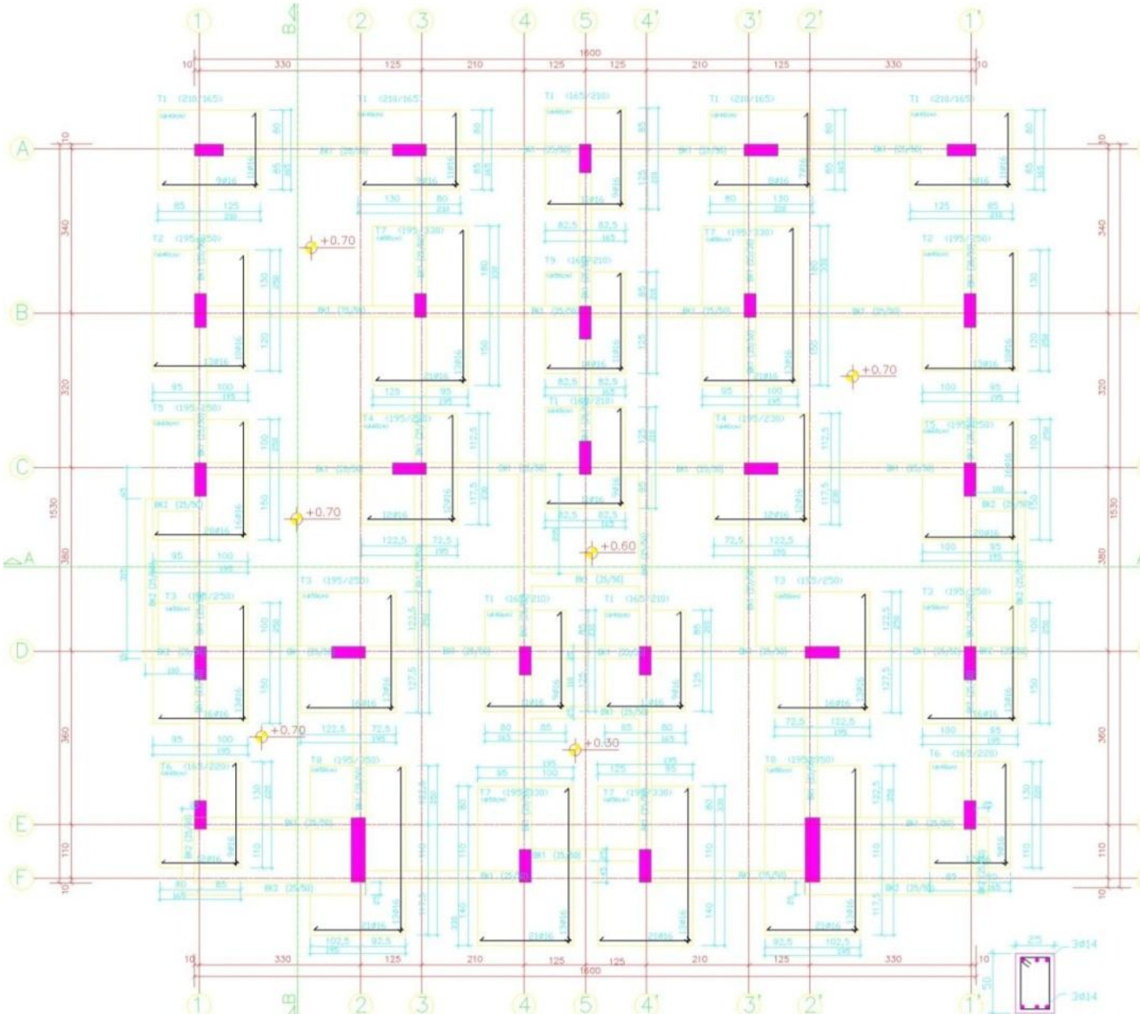

Figure 26. Plan of the Second Building

Table 10. Dimensions and reinforcement of the cross sections for the second building

Section	Dimensions (cm)	Reinforcement	
		Longitudinal	**Stirrups**
Column	25X50	10ϕ14	8@12cm
Column	25X60	10ϕ14	8@12cm
Column	25X70	12ϕ14	8@12cm
Column	30X135	24ϕ14	8@12cm
Column	60X25	10ϕ14	8@12cm
Column	70X25	12ϕ14	8@12cm
Beam	25X50	10ϕ14	8@10cm

Figure 27 shows the deflection values for the largest loads combination. The maximum deflection was 0.0123m in the Z-direction at the corner of the last floor.

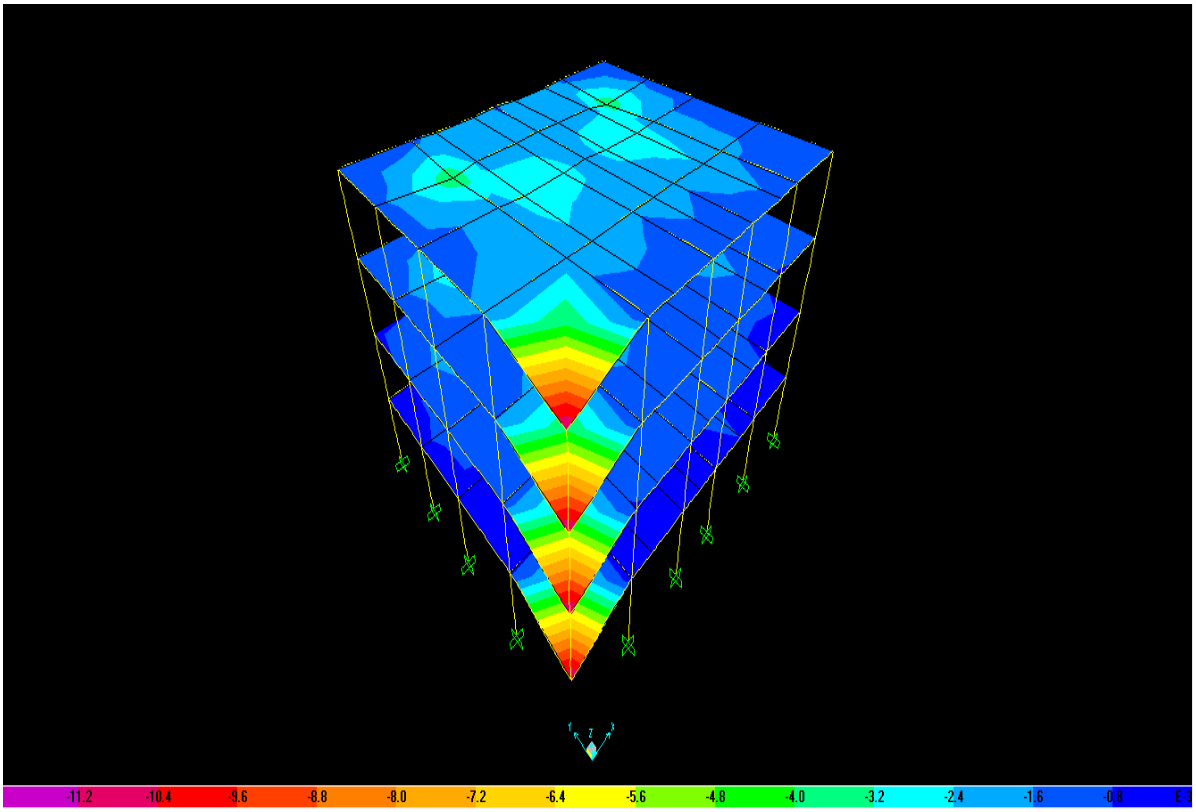

Figure 27. Deformed Shape and Deflection Values for the Second Building [5]

In addition, similar categorization has been followed for this case study structure. The column removed from the middle of the short side located at the ground floor also gives the larger number of the failed elements 12 and 10 for deformation-and force-controlled actions respectively. When the column in the corner and the middle of the long side of the building on the ground floor was removed the number of failed members indicated that the two cases are very similar. This was not the case for the first apartment.

It should be stated that no remarkable difference in the findings was noticed when the direction of the notional loads was changed. The important findings extracted from the outcomes of the second building are presented in Tables 9 and 10.

Table 11. Number of the failed elements for second building in the case of deformation-controlled actions

Floor Number	First Story			Second Story			Third Story			Last Story		
Location / Direction	C	L	S	C	L	S	C	L	S	C	L	S
North	8	7	12	6	2	9	5	1	6	3	0	3
South	8	8	12	6	2	9	4	0	6	2	0	3
West	8	7	12	6	2	9	5	1	6	2	0	4
East	8	7	12	6	2	9	5	0	6	2	0	3

Table 12. Number of the failed elements for second building in the case of force-controlled actions

Floor Number	First Story			Second Story			Third Story			Last Story		
Location / Direction	C	L	S	C	L	S	C	L	S	C	L	S
North	6	6	7	3	0	7	4	0	5	0	0	3
South	6	5	10	3	0	3	2	0	3	1	0	1
West	6	5	10	3	0	6	3	0	5	1	0	1
East	6	6	8	3	0	6	3	0	4	1	0	1

4.2.3 Third Building

The third selected building for this research comprised of eight-story. The floor plan of this apartment has dimensions of 15.0m by 17.6m as presented in Figure 28.

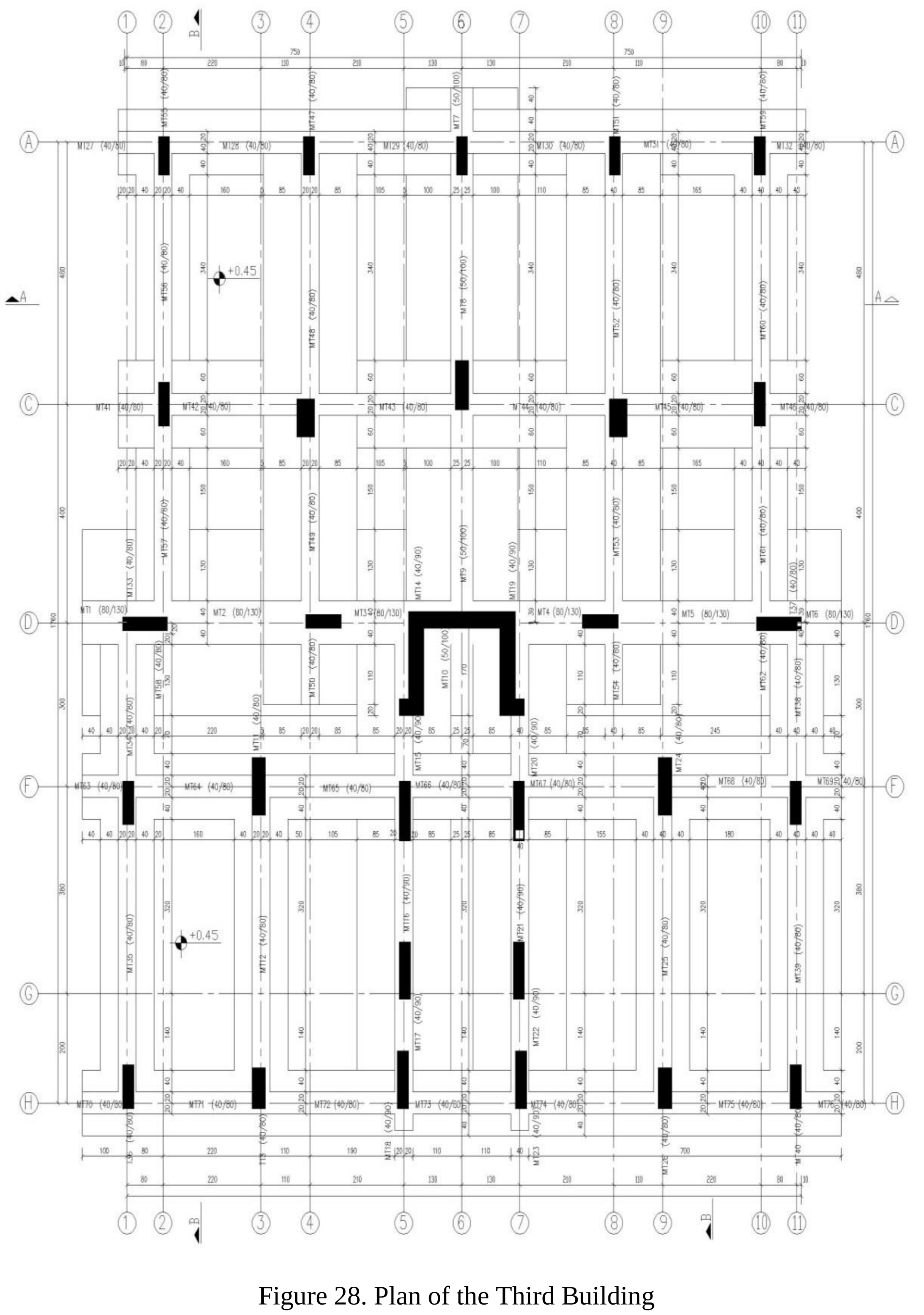

Figure 28. Plan of the Third Building

Table 13. Dimensions and reinforcement of the cross sections for the third building

Section	Dimensions (cm)	Reinforcement	
		Longitudinal	Stirrups
Column	25X70	12ϕ14	10@15cm
Column	25X80	14ϕ14	10@15cm
Column	25X105	16ϕ14	10@15cm
Column	25X110	16ϕ14	10@15cm
Column	30X75	14ϕ14	10@15cm
Column	30X90	16ϕ14	10@15cm
Column	30X105	16ϕ14	10@15cm
Column	40X70	14ϕ14	10@15cm
Column	80X25	14ϕ14	10@15cm
Column	100X25	16ϕ14	10@15cm
Beam	25X60	10ϕ14	10@12cm

Figure 29 shows the deflection values for the entire structure under the biggest loads combination. The highest deflection value was 0.0101 at the corner of the last floor.

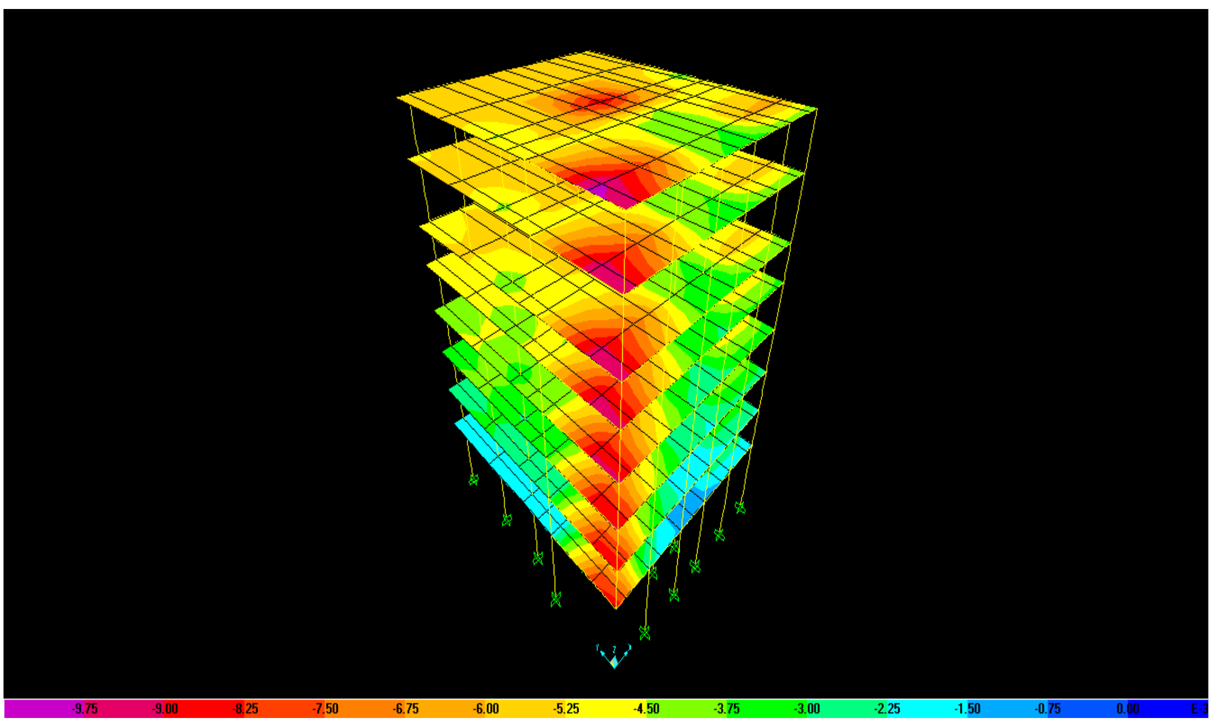

Figure 29. Deformed Shape and Deflection Values for the Third Building [5]

Similar to the four-story apartments, the outcomes of the linear static procedure demonstrate that the worst case scenario resulted from the notional removal of the column located at or near the middle of the short side of the structure at the first floor. Obviously, the more the number of floors the more failed elements due to the abnormal load condition. Therefore, the maximum number of the failed members was 13 and 6 for deformation-and force-controlled actions respectively.

Also, no substantial difference in the total number of the failed elements for the various locations of the removed columns was perceived particularly for the deformation-controlled actions.

The numbers of failed elements are given in Tables 11 and 12. As can be observed from these tables there is slight divergence in the number of failed elements due to the change in the direction of the lateral loads subjected to the side face of the building.

Table 14. Number of the failed elements for third building in the case of deformation-controlled actions

Floor Number	First Story			Fourth Story			Fifth Story			Last Story		
Location / Direction	C	L	S	C	L	S	C	L	S	C	L	S
North	12	11	13	11	11	13	10	10	13	9	9	12
South	12	11	13	11	10	13	10	9	13	9	8	12
West	12	12	13	11	11	13	10	9	13	9	9	12
East	12	11	13	11	11	13	10	9	13	9	9	12

Table 15. Number of the failed elements for third building in the case of force-controlled actions

Floor Number	First Story			Fourth Story			Fifth Story			Last Story		
Location / Direction	C	L	S	C	L	S	C	L	S	C	L	S
North	1	1	6	0	0	3	0	1	2	0	0	0
South	1	1	6	0	0	3	0	0	2	0	0	0
West	1	1	6	0	1	3	0	0	2	0	0	0
East	1	1	6	0	0	3	0	0	2	0	0	0

4.2.4 Fourth building

The last apartment that was chosen in this case study contained eight-story with floor plan of 14.8m in width and 15.1m in length. Figure 30 illustrate the floor plan of the building.

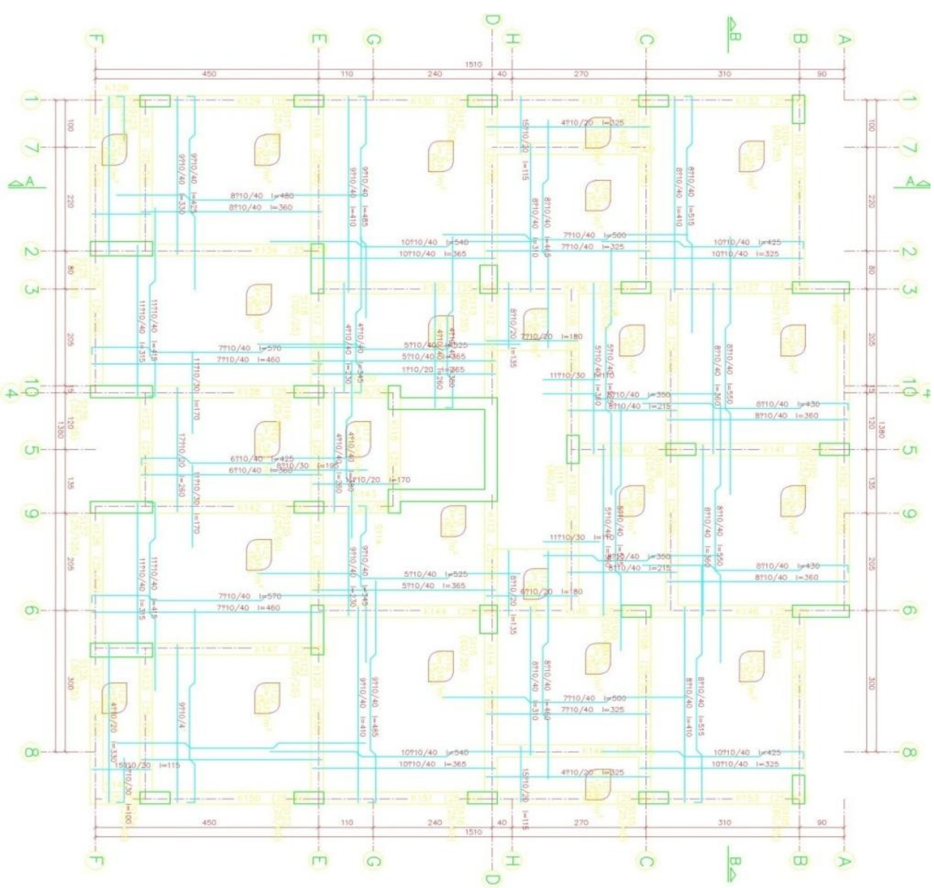

Figure 30. Plan of the Fourth Building

Table 16. Dimensions and reinforcement of the cross sections for the fourth building

Section	Dimensions (cm)	Reinforcement	
		Longitudinal	Stirrups
Column	25X60	10ϕ14	8@15cm
Column	25X105	14ϕ14	8@15cm
Column	50X25	8ϕ14	8@15cm
Column	60X25	10ϕ14	8@15cm
Column	60X25	10ϕ14	8@15cm
Column	120X25	16ϕ14	8@15cm
Beam	25X50	10ϕ14	8@12cm

Similar to the previous building, the maximum deflection value of 0.0163m was observed in the corner of the last story of the structure. The overall values of the deflection under the gravity and notional load combinations are presented in Figure 31.

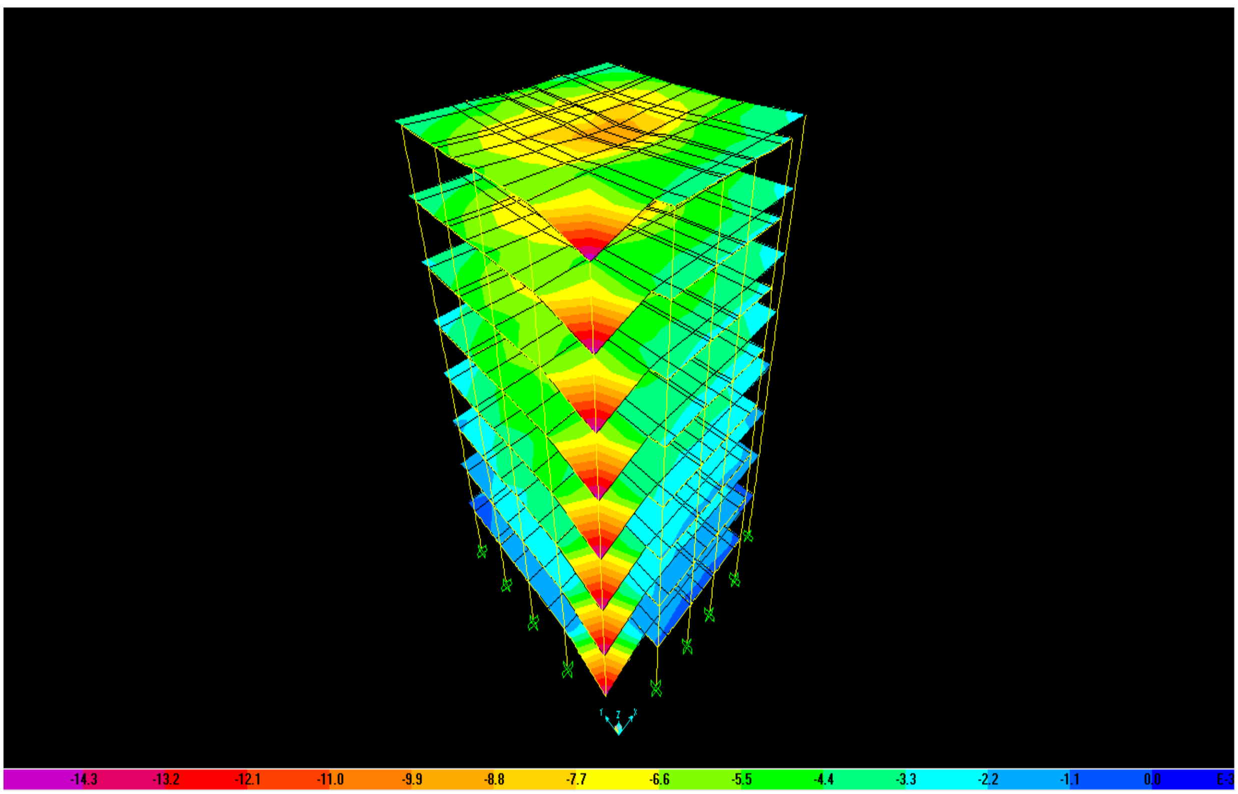

Figure 31. Deformed Shape and Deflection Values for the Third Building [5]

The bigger spans of the vertical-support elements lead to more components to be failed. The analysis results indicate that the removal of the middle of the short side of the building from the first story resulted in more failure in the structural members. For the deformation-controlled actions, the largest number of failure is in 23 members. However, the greatest number of the failed elements was only 8 for the force-controlled actions.

In spite the fact that the column removal from the middle of the short side leads to the higher number of the failed components, for this building the analysis findings elucidate that the total failed elements resulted from the column removal from the middle of the long side of the apartment was very close to the worst case situation.

Finally, the direction of the notional loads applied to each principle side of the structure has no remarkable diversity in the number of the failed components as stated in Tables 13 and 14.

Table 17. Number of the failed elements for fourth building in the case of deformation-controlled actions

Floor Number	First Story			Fourth Story			Fifth Story			Last Story		
Location / Direction	C	L	S	C	L	S	C	L	S	C	L	S
North	11	20	22	11	14	18	10	14	17	10	10	10
South	12	20	23	10	14	17	10	13	15	10	10	10
West	11	19	22	10	14	18	10	14	15	11	9	11
East	13	22	23	11	17	18	11	15	16	11	10	11

Table 18. Number of the failed elements for fourth building in the case of force-controlled actions

Floor Number	First Story			Fourth Story			Fifth Story			Last Story		
Location / Direction	C	L	S	C	L	S	C	L	S	C	L	S
North	1	6	8	0	2	3	0	1	3	0	0	1
South	0	6	8	1	1	4	0	1	3	0	1	1
West	1	5	7	2	2	5	1	2	4	1	1	2
East	1	8	8	1	3	3	1	2	4	1	1	1

4.2.5 Retrofitting Procedure

In order to re-design the failed elements, the building that has the least number of failed components was selected as an example to describe the retrofitting procedure. This was done to simplify this process.

The retrofitting procedure that proposed by the UFC 4-023-03 (DoD, 2010) was implemented in this study to re-design the failed elements. This process can be easily performed by increasing the dimensions of the cross-section and area of the steel reinforcement of the structural members.

To do this, all the failed components were identified, and then their cross-sections and reinforcement were modified for each one of them. Iterative analysis and re-design were conducted for each trail until overall the structural members were succeeded.

4.3 Summary

To conclude all these results, the worst case of the four-story buildings was selected and the failed elements were plotted as shown in Figures 32 and 33 for deformation-and force-controlled respectively.

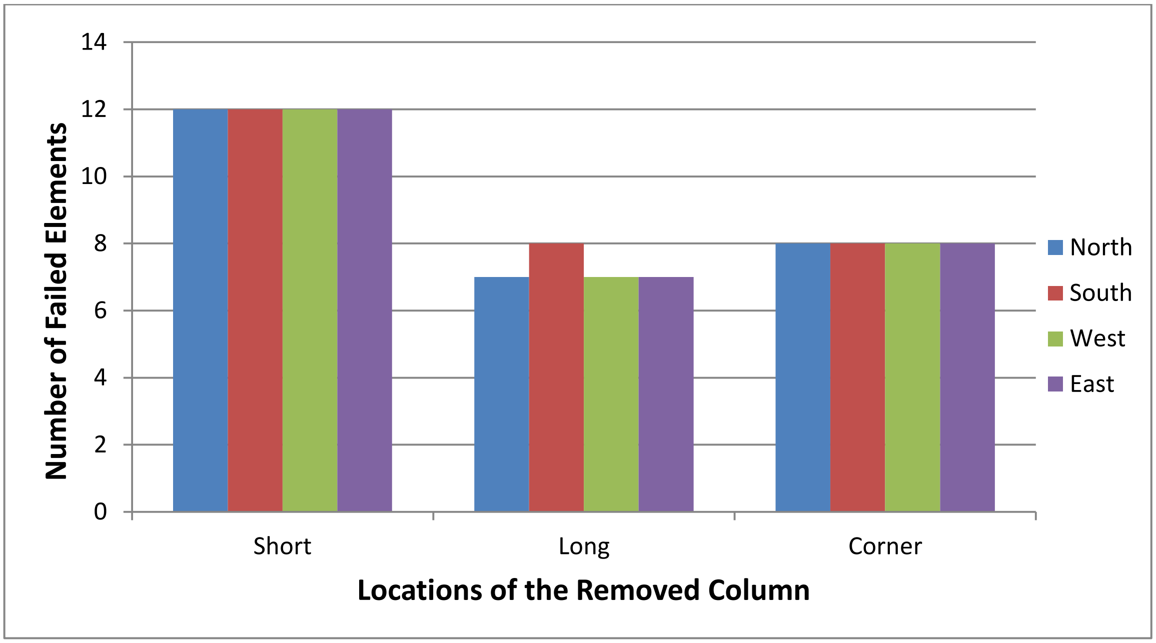

Figure 32. Total Failed Elements for the Worst Case of the Four-Story Buildings in Deformation-Controlled Actions Condition

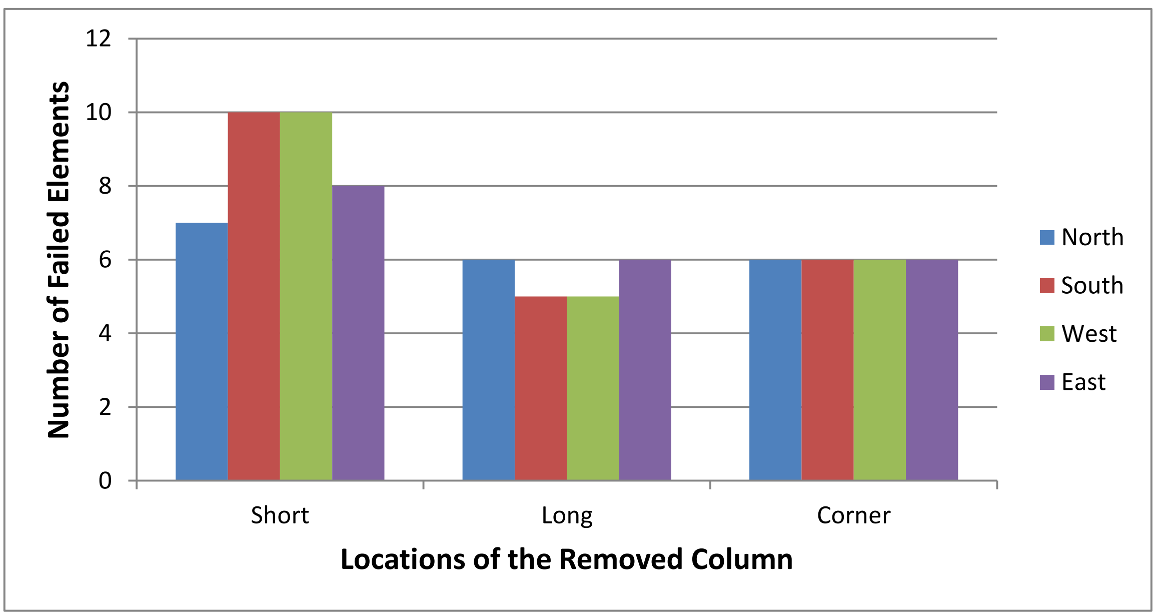

Figure 33. Total Failed Elements for the Worst Case of the Four-Story Buildings in Force-Controlled Actions Condition

Nevertheless, Figures 34 and 35 illustrate the worst case for the eight-story apartments. The first figure is for the deformation-controlled actions, whereas the second one is for the force-controlled actions.

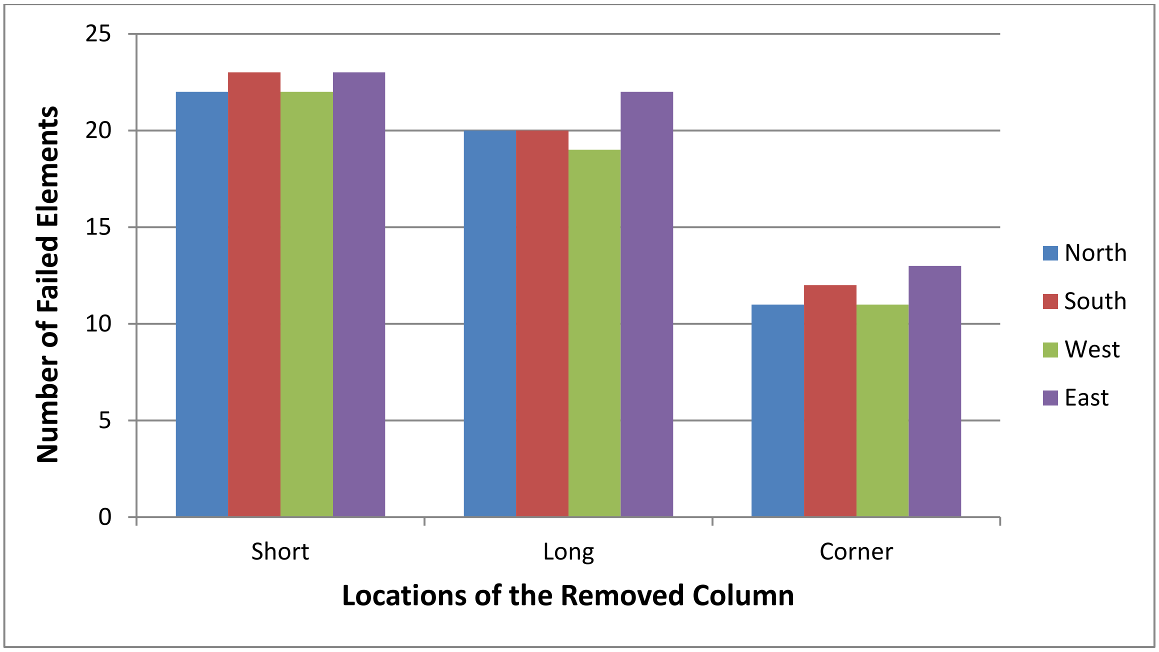

Figure 34. Total Failed Elements for the Worst Case of the Eight-Story Buildings in Deformation-Controlled Actions Condition

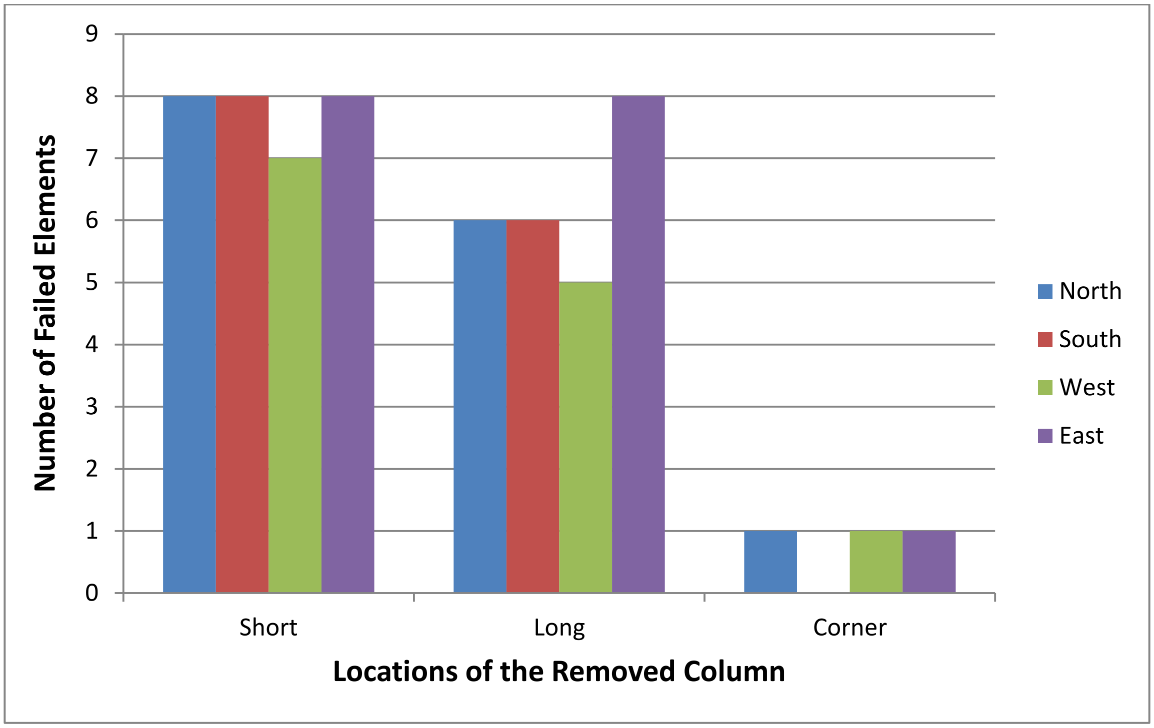

Figure 35. Total Failed Elements for the Worst Case of the Eight -Story Buildings in Force-Controlled Actions Condition

Ultimately, for all the buildings regardless of the type of actions, the number of stories and the direction of the notional loads, the removal of column from the first floor located at or near the middle of the short side of the facility leads to more damage and greater number of failed members. As a result, more concentration has to be given for this critical load-carrying element in the analysis and design procedures. However, Figure 36 demonstrates the overall approach of this study.

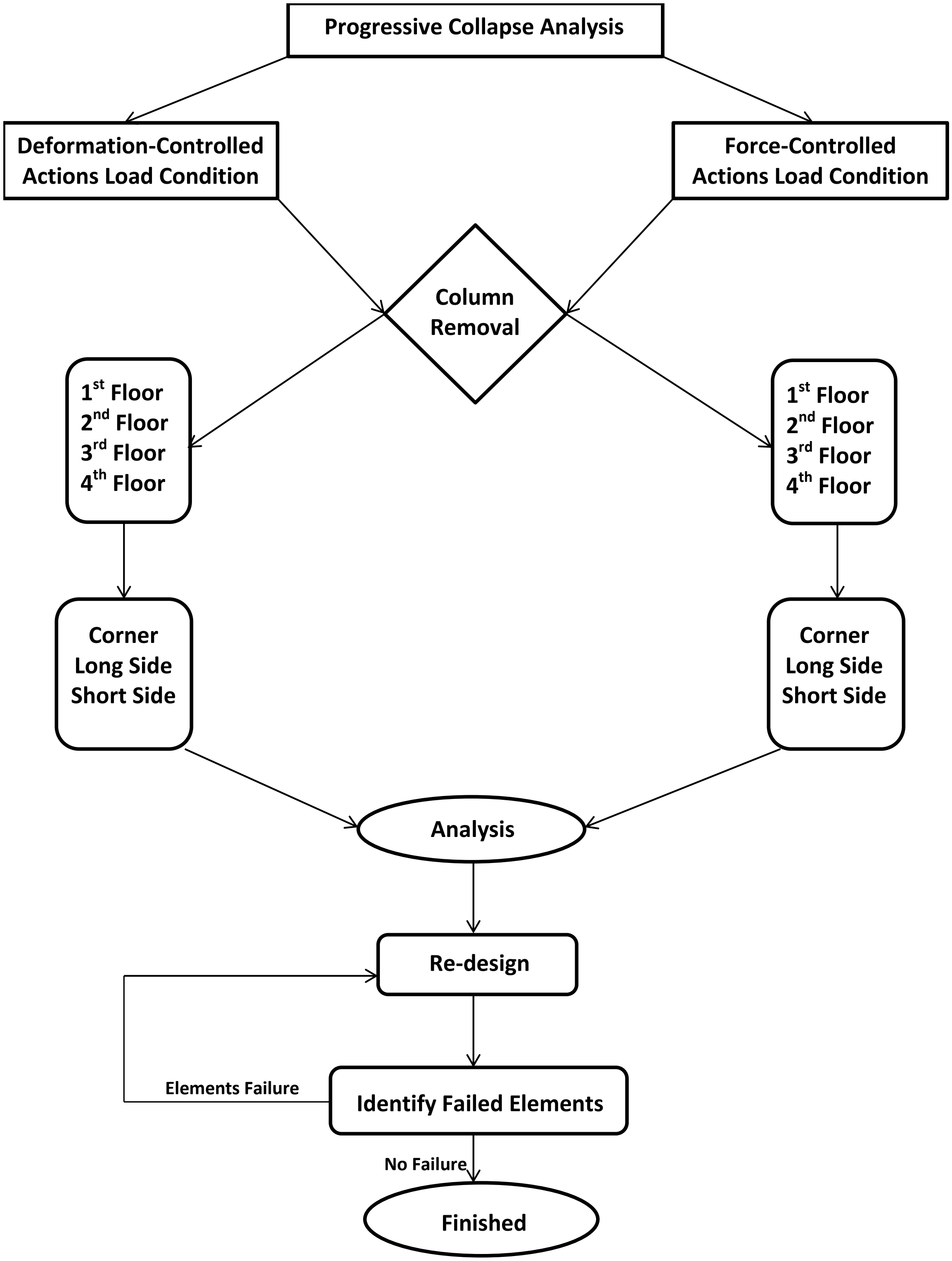

Figure 36. The Overall Approach

Chapter 5

CONCLUSION

5.1 Summary

Progressive collapse is a disastrous failure with low-probability of occurrence. It is classified into two; disproportionate progressive collapse and proportionate progressive collapse. First type occurs when the resulted collapse is disproportionate with the triggered event, such as the partial collapse of the Ronan Point apartment in London, UK in 1968, whilst the second one happens if the provoked event is very large and the structure is not able to resist it even if it has been designed properly like the destruction of the World Trade Center towers in New York, USA in 2001.

Normally, the vast majority of structural engineers do not consider the progressive collapse phenomenon in the design process because of the low possibility for it to happen. Nonetheless, for the vulnerable facilities those defined as being at risk, it is essential to consider the progressive collapse criteria in the structural design to minimize the potential of collapse at a reasonably minor level. The implementation of progressive collapse event in the design procedure is accomplished by considering the effect of the abnormal loads such as design and construction errors, intensive fire, vehicular collision, aircraft impact, gas explosions and bomb detonations.

Moreover, to investigate the progressive collapse situation, several analytical approaches have been proposed. These procedures diverge widely in the accuracy of their outcomes and time consumption to simulate and execute the model. The linear Static procedure (LSP), Nonlinear Static procedure (NSP), Linear Dynamic procedure (LDP) and Nonlinear Dynamic procedure (NDP) are the general methods in the structural analysis.

The linear static procedure (LSP) and nonlinear dynamic procedure (NDP) are the most common procedures in the analysis of building subjected to potential of progressive collapse.

Each of the mentioned methods has its advantages and drawbacks. The linear static approach is a popular method in this field since it is quick, easy, and economic and do not need vast structural knowledge to model and analyze the building. However, the findings of structural analysis have some errors and they are overestimated. This procedure is limited to be utilized only for low-to-medium rise (not exceeding ten stories) according to GSA design guideline. On the contrary, the nonlinear dynamic method is the most precise structural analysis procedure. Yet, it is generally avoided due to its complexity and the enormous time to generate the model and the long amount of time to execute it.

In respect to the design methods to mitigate the progressive collapse event, three methods have been specified to mitigate the hazard of progressive collapse, being (1) event control, (2) indirect design method, and (3) direct design method. The event control design method considers indirect actions to protect the buildings by eliminating the abnormal loads exposure. Despite the fact that it is very economic but it cannot be used to prevent progressive collapse since it is impractical and difficult to ensure the entire avoidance of the progressive collapse event. The indirect design methods are threat independent design approaches aim to provide sufficient general integrity for the buildings without any considerations for extreme loads. Furthermore, the alternate load path method and the specific local resistance method are the two design approaches which are part of the direct design procedure. The first method permits the occurrence of the local damage; however the collapse of large portion of the structure is avoided by providing alternate load paths in the neighboring elements to redistribute the loads that were applied on the damaged components. Whilst, the second approach stipulates that sufficient resistance of structures against extreme load hazard has to be provided. This is achieved by strengthening critical elements (key members) in the configuration of the building. This design procedure specifies that the key elements shall remain intact irrespective of the magnitude of the abnormal loads subjected to the facility.

5.2 Overall Conclusion

In this study the (DoD, 2010) is followed as a guideline. This document addresses the load cases according to the structural actions whether deformation-or force-controlled actions. Also, defines the numbers and locations of the vertical-support element removal. It states that a column should be removed from the first, last, mid-height and the story above the location of a column splice or change in column size. For each case one column situated at or near the middle of the long side, at or near the middle of the long side and at the corner of the building should be notionally removed. This step-by-step process has been followed in this investigation to identify the most critical location of the removed column. Besides, the influence of the building height to the results of the analysis has been demonstrated. This is not defined in the DoD, 2010.

In order to perform this research, four apartments have been chosen as a case study. They are existing structures in Famagusta, Northern Cyprus. All of them have a regular rectangular plan. Two of them contained of four-stories, while the others consisted of eight-stories.

The retrofitting of the failed members (beams and columns) has been done according to the DoD, 2010 document which requires to increase the cross-sectional dimensions of failed elements and the total area of the reinforcement.

The results of the analysis illustrate that the higher the building the more failure in the structural members. Additionally, based on the analysis outcomes of this investigation, the removal of column from the middle or near the middle of the short side at the first floor of the building is the worst case scenario. Therefore, only the column located at or near the middle of the short side sited in the first story in the building is removed to simplify this type of comprehensive analysis procedure. In this research 96 different analysis cases have been conducted for each building to satisfy the requirements of this guideline.

Eventually, it should be emphasized that these findings and recommendations are only to be employed for the regular structures since this study was carried out for regular buildings.

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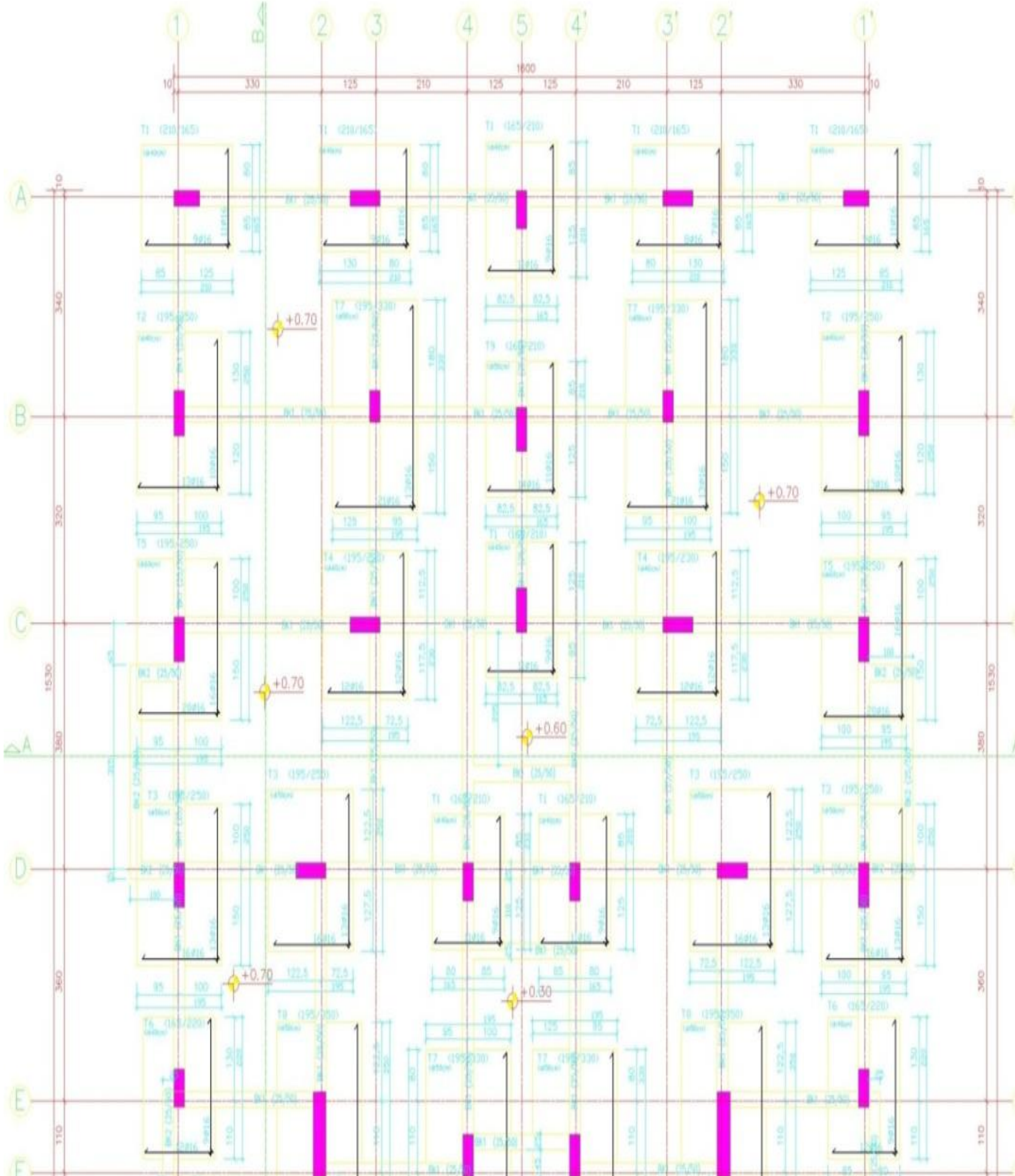
APPENDIX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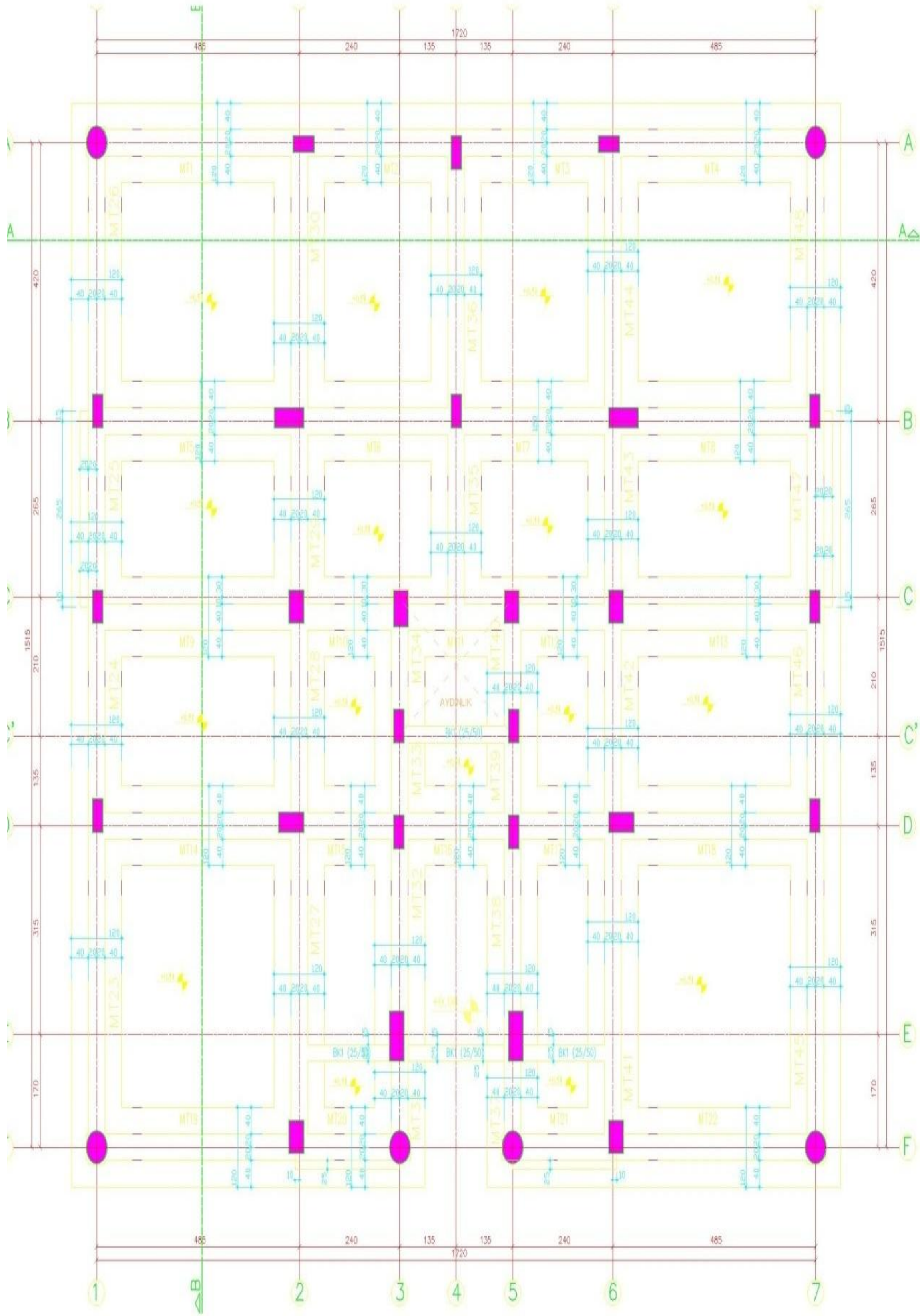

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