Progressive Collapse Analysis of Steel Framed Structures with I-Beams and Truss Beams using Linear Static Procedure

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ABSTRACT

Progressive collapse starts with a local damage or loss of some members of the structure leading to failure at large parts of a structure. Due to the recent disastrous events like world Trade Center in USA, taking measures in reducing the potential of progressive collapse (PC) of structures during the analysis and design stages is becoming a necessity for the structures. A number of computational analysis programs, such as ETABS, SAP2000, ABAQUS can be used to simulate the structures and look into their potential of PC and also how to improve their design against PC.

This study investigates the potential of progressive collapse in steel framed structures using normal I-beams and truss beams in their floor systems. For this reason two steel framed buildings are considered having floors with normal I-beams and truss beams of 9 m,12 m and 15 m spans to investigate the effect of increasing the span of the beams onto the potential of progressive collapse of the buildings. General service Administration (GSA) guidelines with linear static procedure is used for the analysis of the above mentioned buildings and as a result of the analysis Demand Capacity Ratio (DCR), deflections and steel weights were also compared. Results indicate that due to column removal of all the frames with normal I-beams spanning 9 m,12 m and 15 m have higher potential of PC than the frame with truss beams since the additional loads are distributed onto the truss vertical and diagonal members. Furthermore, vertical displacement of the normal I-beams is also more than the truss beams. When 12m and 15m beam spans are considered buildings with truss beam floors have less steel weight than those with normal I-beam floors. However, when 9m beam spans are used then the case is opposite.

In the long side of the buildings generally the truss beam members manage to absorb the additional loads created by loosing a main column member. In the short side additional vertical bracings are used to reduce the DCR values below the acceptable limits of GSA.

Keywords: Progressive collapse, truss beam, normal I-beam, steel structure, linear static procedure.

ÖZ

Aşamalı çöküşün başlamasına neden genelde bölgesel hasarlar veya birkaç elemanın kırılışı sonucu yapının daha büyük bir kısmının çökmesidir. Yakın zamanlarda meydana gelen ABD'de Ticaret Merkezi binasının aşamalı çöküşü gibi felaketlerin olasılığını azaltma veya önleme için yapı analizi ve tasarımı yapılırken bir dizi önlemlerin alınması artık bir ihtiyaç olmuştur.

ETABS, SAP2000 ve ABAQUS gibi bir dizi analiz programlarında yapıların simulasyonu yapılarak aşamalı çöküş potansiyeli incelenebilir ve yapıların aşamalı çöküşe karşı dayanımını artırma yöntemleri araştırılabilir.

Bu araştırmada normal I-kirişi ve kafes kiriş döşeme sistemi olan çelik karkas yapılarda aşamalı çöküş potansiyeli araştırılmıştır. Bu nedenle bahsekonu iki tip çelik karkas yapıda kiriş açıklıklarının aşamalı çökme potansiyeline etkisi araştırılmıştır. Bu amaçla 9m, 12m ve 15m kiriş açıklıklı yapılar incelenmiştir. Genel Hızmet İdaresi (GSA) ilkeleri ve doğrusal statik analiz metodu kullanılarak yukarıda belirtilen yapılar analiz edilmiş, istek kapasite oranı (DCR), sehimler ve çelik yapı karkas ağırlıkları karşılaştırılmıştır. Sonuçlara göre, normal I-kirişli döşemesi olan yapıların tüm kiriş açıklıklarında aşamalı çöküş potansiyeli kafes kiriş döşemeli yapılara göre daha yüksektir. Ayni zamanda normal I-kirişlerin dikey sehimleri de kafes kirişlerden daha fazladır.

12m ve 15m kiriş açıklıkları olan çerçevelerde kafes kiriş döşeme sistemli yapıların çelik ağırlığı diğer yapılardan daha azdır. Bu durum 9m açıklıklı kirişlerde tamamen terstir. Yapının uzun kenarlarında bir kolonun hasara uğraması sonucu oluşan ilave yükler kafes kiriş döşemelerde daha iyi dağıtılmıştır. Kısa kenarlarda ise ilave yükler düşey destekler tarafından taşınarak DCR değerleri GSA tarafından kabul edilir sınırların altına düşürülmüştür.

Anahtar Kelimeler: Aşamalı çöküş, kafes kiriş, normal I-kiriş, çelik yapı, doğrusal statik analiz

This thesis is dedicated to my family

For their endless love,support and encouragement

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INTRODUCTION

1.1 General Introduction

Civil engineering structures can be subject to loads due to natural disasters like earthquakes, hurricanes, tornadoes, floods, fires and man-made and artificial disasters, such as explosion and impact, during their lifetime. The buildings are generally designed according to the design standards which usually considers dead, imposed, wind and earthquake loads. There are allowances for other loads, such as impact and explosion, if the structure is considered to have the risk of being subject to such loading during its lifetime. However, there are still circumstances that are unforeseeable at design stage. On the other hand, every project has a budget and engineers should meet the design requirements while producing an economical design within the allocated budget. Recent events, such as the 1994 Northridge earthquake, 1995 Kobe earthquake, bombing of Murrah Federal building in 1995 and the 2001 attack on the World Trade Center have led to the collapse of structures and consequential loss of life and finance.

A progressive collapse is identified as the initial local destruction of members that leads to the collapse of nearby members first and then leading to the collapse of a disproportionately large part of a building. Due to loss of lives and economic loss the progressive collapse of Ronan Point Apartment in Newham, London in 1968, caused a great concern to structural engineering society. Therefore, the Ronan Point collapse led to detailed investigation of the event and hence recommendations in some European design codes to improve the resistance of structures against progressive collapse (Kaewkulchai & Williamson, 2003).

Similar events within the last decade or so further urge the need of introducing new methods of assessing the potential of progressive collapse in buildings. There are several methods introduced to minimize the possibility of progressive collapse in new and existing structures. There are many building codes, standards and design guidelines for progressive collapse. Among them the General Services Administration (GSA, 2003) and Department of Defense (DoD, 2005) are the most widely used mehods for assessing the potential of progressive collapse and also reduce the occurance of progressive collapse. They present scientific and enforceable procedures for resistance against progressive collapse. These guidelines refer to indirect and direct approaches to concentrate on progressive collapse in structural design. The guidelines focus on the alternate load path method, a direct method, as the chosen approach for evaluating the progressive collapse potential of a structure (Kaewkulchai & Williamson, 2003).

In the alternate load path approach, due to removal of a column, the loads on the structure are studied to make sure that they are properly redistributed to the undamaged members. Designs based on the alternate load path analysis result in larger member sizes than those found when using all applicable load combinations (Ruth et al., 2006). Using bigger member sizes for the structural members in case of the need to redistribute loads is not realistic for existing buildings. Consequently, there is a need to find possible and viable approaches of retrofitting existing buildings to reduce the potential of progressive collapse (Ruth et al., 2006).

1.2 Research Objectives

The overall aim of this study was to evaluate the response of the steel structures due to a sudden loss of one or more columns by using computational modeling. The structures were designed to have three different beam spans in order to observe the effect of span length on the behavior of structure after the removal of the columns. There were two types of buildings, one was modeled with normal I-beam and the other was modeled with truss beams in the long direction. Three dimensional model of the structures were created in ETABS software [version 9.7.4] and the buildings were analyzed and designed according to General Service Administration (GSA, 2003). There are four different analysis procedures to evaluate the progressive collapse but in this study only linear static procedure was used to check the buildings against progressive collapse.

9 m, 12 m and 15 m beam spans were used for the buildings. A warren type truss was used for the beams. The connection between the truss beams and the I-section column is assumed to be a pinned joint. The test buildings were braced frame. The European steel section were used as structural members of the buildings. One of the objectives was to compare the progressive collapse potential of a building with normal I-beams and with truss beams:

1.3 Tasks

The major and specific tasks of this study are as follows:

1. Test the design building modeled in ETABS software [version 9.7.4] by removing exterior column in the first-story that may lead to progressive collapse.

2. Investigate the progressive collapse potential of these two kinds of steel building due to the removal of the column in the ETABS model.

3. Improve the structur of the 3-dimensional steel frame building to analyze and compare the result between using normal I-beam and truss beam structures.

4. Evaluate the response of two models of the building after removing a column and carrying out linear static analysis procedures.

5. Compare the Demand Capacity Ratio (DCR) values of each member for the building with normal I-beam and truss beam and compare the deformations due to the removal of the column.

1.4 Outline of the Thesis

This thesis is organized as five chapters which contain an introduction (Chapter 1), literature review (Chapter 2), research methods (Chapter 3), results and discussions (Chapter 4) and conclusion and recommendation for further investigations (Chapter 5).

Chapter 2 explains background researches concerning the progressive collapse of the structures. The description of well known examples of progressive collapse cases explained. Review current guideline for resistance against progressive collapse like GSA and DoD. The different design method also described in this chapter.

Chapter 3 is the methodology where the two different models of buildings (truss beam and normal I-beam) and their structural member arrangements were described.

Chapter 4 presents the detailed modeling assumption and analysis procedure. It provides the 3-dimensional computer models of each structure in ETABS computer program. The acceptance criteria suggested in GSA guideline and the loading conditions are also presented in this chapter. The result of the 3-dimensional linear static analysis procedure for both buildings designed for a number of beam spans are given and compared among themselves.

Chapter 5 provides the summary of the research and presents the results and conclusions. Finally some recommendation for future research is also given in this final chapter.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

This chapter provides background and study of literature concerning the progressive collapse of buildings. First, the description and well-known examples of progressive collapse are presented. Selected past studies on progressive collapse of structures are surveyed and summarized in this chapter. Also the design approaches and analysis procedures for progressive collapse of buildings are described. Finally, a review of the existing guidelines for the prevention of progressive collapse. In particular, the General Services Administration (GSA, 2003) is reviewed and the Department of Defense (DoD, 2005) guidelines are described.

2.2 Definitions of Progressive Collapse

A series of reaction to the failure initiated by the immediate failure of one or a few structural elements is called progressive collapse. Man-made hazards may cause progressive collapse, such as blast, explosion, vehicle collision and severe fire or by natural events including earthquakes.

When a structural element fails, the structure members should be arranged as such to form an alternative load transfer path to distribute the loads carried by the failed element to the adjacent elements. The loss of a structural member would cause release of inner energy and leads to raise in the dynamic internal forces of nearby elements. When the load is spread through a structure, each of the structural elements should be able to support its expected loads as well as the additional internal forces from the failed members. The bearing capacities of the nearby undamaged members may exceed the allowable values due to the redistribution of the loads and this can cause another local failure. Such serial failures can distribute from one element to another, finally causing the complete or a disproportionately large part of the structure to go through progressive collapse.

The definition of progressive collapse may incorporate the perception of disproportionate collapse which means that the extent of the final failure is not proportional to the size of the preliminary starting event. For instance, the American Society of Civil Engineer (ASCE) Standard 7-05 defines the progressive collapse as "the extend of a preliminary local failure from element to element resulting eventually in the collapse of an entire structure or a disproportionately large part of it" (ASCE 7-05, 2005). A similar definition of progressive collapse is given in GSA 2003 guidelines, "a situation where local failure of a primary structural component leads to the collapse of adjoining members, and hence, the total damage is disproportionate to the original cause" (GSA, 2003).

Progressive collapse, which is also designated as disproportionate collapse, refers to the total or partial collapse of a structure starting from a localized failure. The presently accepted definition of progressive collapse also involves the concept that the total area or volume of the structure that collapses is disproportionate to the area or volume of the structure destroyed by the initiating happening (Nair, 2006).

2.3 Examples of Progressive Collapse

Details about some of the most known examples of progressive collapse are given in the following sections. These are Ronan Point Apartment Tower in 1968, Alfred P. Murrah Federal Building in 1995 and World Trade Center in 2001. These three events had an important impact on the increase in research on progressive collapse which heavily contributed to the development of codes and standards with regards to measures to be taken to prevent progressive collapse in the design of buildings.

2.3.1 Ronan Point Apartment Tower Collapse

The first famous case of disproportionate progressive collapse is Ronan Point apartment tower collapse on May 16, 1968 (Griffiths et al., 1968). The building was located in Newham, England. It was 22-story building with precast concrete bearing wall system. The collapse was started by a gas leak in a corner kitchen on the 18th floor. The exterior walls of the apartment blew out due to the pressure of the small gas blast and also shift a load-bearing precast concrete panel near the corner of building. Figure 1 shows the partly collapsed structure. The Ronan Point collapse capture the attention of the structural engineering community and caused serious concerns relating to progressive collapse among the structural engineers throughout the world. This collapse led to a number of changes in building codes in England and Canada so that buildings would resist against progressive collapse (Griffiths et al., 1968).

Figure 1: A partial collapse of the Ronan Point Apartment tower in 1968 (Wikipedia, 2012)

2.3.2 The Oklahoma City Bombing

The bombing of the Alfred P. Murrah Federal building that was located in downtown Oklahoma City, OK on April 19, 1995 was a second important case of progressive collapse (FEMA-277, 1996). The falling down of this structure is a characteristic example of progressive collapse due to a bomb blast. At the base of the building three columns were damaged due to the bomb blast in a truck. When these columns loss their supports, a transfer girder failed. Failure of the transfer girder led to collapse of columns supported by the girder and floor areas supported by those columns. The outcome was the general collapse observed in Figure 2 (Nair, 2004). The north side of the building was where the main structural damage occurred and this was right in front of the explosion area. The blast damaged about half of the residential space in the nine-story Federal Building. As a result of the effect of this huge explosion followed by the collapse, 168 people were killed and over 800 people were wounded (Irving, 1995). The Murrah Building tragedy was obviously a progressive collapse by all the definitions of this term. Collapse of the large part of this building was caused by the damage to its few small members (a few column members). The collapse was also related to progression of actions: damage to columns; collapse of the transfer girder followed by failure of the structure above the transfer floor. After this event, there was increased concern of structural engineers on progressive collapse which encouraged more research into this matter. Further investigations were conducted on progressive collapse and findings were reflected in the design procedures in the design codes for structures.

Figure 2: External sight of Alfred P. Murrah Federal building collapse (FEMA-427, 2003)

2.3.3 World Trade Center Collapse

The twin towers of World Trade Center 1 and 2 progressively collapsed on 11 September 2001 due to terrorist attacks (NIST, 2005).

Boeing 767 jetliners crashed into two towers of WTC in New York City at high speed. Within a short time after the crash the towers were totally collapsed due to their huge self weight above the floors subject to crash. The structure collapse caused by a very large impact and fire; it is a progressive collapse but not a disproportionate collapse as shown in Figure 3 (Dusenberry et al., 2004).

Figure 3: The north and east faces of the World Trade Center towers, showing fire and crash destruction to both towers (FEMA-403, 2002)

The crash set off a strong fire inside the building and therefore, structural damage close to the position of impact; the structural members nearby the crash area lost its capability to carry the loads above that floor. As a result of damages caused in combination of crash and consequent fire the structure above the crashed floor collapsed, having lost its supports; the loads above the damaged floors collapsed on this floor followed by the progression of failures which continue all the way down to the ground. The death of more than 3000 people was the result of the collapse of the twin towers, as well as a wide range of damage to the neighboring buildings (Dusenberry et al., 2004).

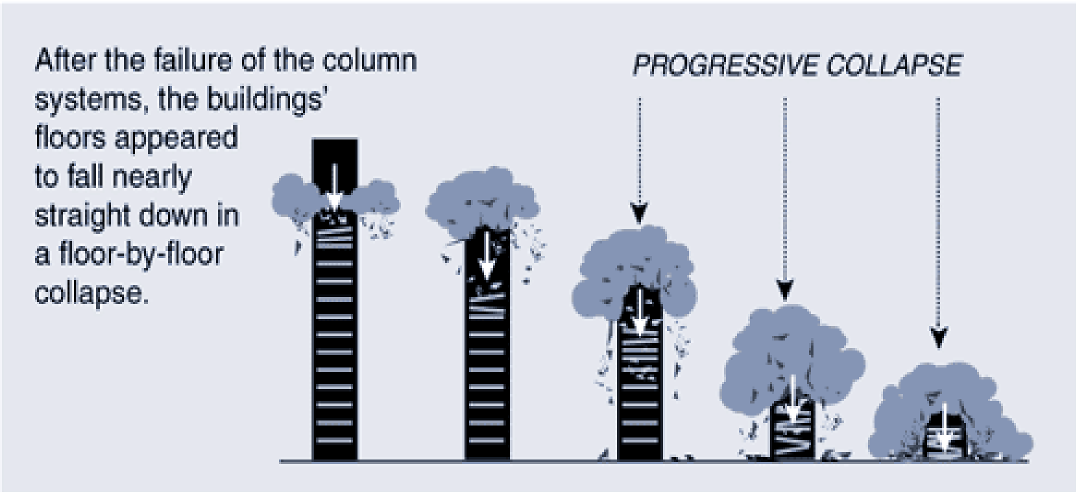

Figure 4: The Progressive collapse of World Trade Center towers
(New York Times, 2001)

2.4 Design Method for Progressive Collapse

Two common design methods to decrease progressive collapse potential is defined by ASCE 7-05 (ASCE 7-05, 2005), which are indirect design method and direct design method. In the following sections each of these approaches are explained.

2.4.1 Indirect Design Method

The provision of the lowest levels of potency, continuity and ductility approach is provided by indirect design method to prevent progressive collapse (ASCE 7-05, 2005). Improving joint connections by special detailing, improving redundancy, and providing more ductility to the structure, are examples of this method. Generally, most building codes and standards used the indirect design approach since it can make a redundant structure that will complete under any situation and improve overall structural response (ACI 318-08, 2008). This method is not suggested for the progressive collapse design owing to no special consideration of the removal of elements or exact loads.

2.4.2 Direct Design Method

During the design procedure the direct design method clearly considers resistance of a structure to progressive collapse (ASCE, 2005). There are two direct design approaches: the specific local resistance method and the alternate load path method. The specific local resistance method trying to improve and provide strength to be capable to resist progressive collapse. The alternate load path method seeks to provide alternative load paths to absorb constrained damage and resist progressive collapse (ASCE, 2005).

2.4.2.1 Specific Local Resistance Method

The critical structural element should be able to resist an abnormal loading by the exact local resistance method. Despite of the high loads, the structural element should not collapse because of its strength. For this method, an adequate amount of strength and ductility of the member must be determined during the design against progressive collapse. The essential member can be designed to have additional strength and stiffness to resist the loading, only by raising the design load factors (ASCE, 2005).

2.4.2.2 Alternative Path Method

In the alternate path (AP) approach, the design allows a region to collapse but seeks to prevent a major failure by providing alternate load paths to distribute the additional loads to members which are not direcly affected by the over loading. Collapse in a structural member severely changes the load path by carrying loads to the members next to the failed member. If the neighboring members have adequate capability and ductility, the structural system develops alternate load paths. Through this method, a building is designed for the potential of progressive collapse by immediately eliminating one or several of the load bearing members from the building and by assessing the capability of the remaining structure to prevent further damage. The benefit of this method is the fact that it is independent of the starting of the overload; therefore, the solution would likely be suitable to resist any type of danger which may cause loss of members (ASCE, 2005).

The alternate load path method is mainly suggested to be used in the existing building design codes and standards in the U.S., such as, General Services Administration (GSA, 2003) and the Department of Defense (DoD, 2005) guidelines. Therefore, investigations carried out as per the GSA and DoD guidelines mostly focus on the use of AP approach for progressive collapse analysis.

2.5 Analysis Procedures for Progressive Collapse

There are four different procedures to analyze the structural performance of a building; Linear Static (LS), Nonlinear Static (NLS), Linear Dynamic (LD), and Nonlinear Dynamic (NLD), in order of rising complexity. So far the advantage and disadvantage of each of the above mentioned procedure was investigated by many researchers. A complex analysis is preferred to achieve better and more rational results instead of the actual nonlinear and dynamic reaction of the structure during the progressive collapse. On the other hand, for the progressive collapse analysis, both GSA and DoD guidelines choose the simplest method, linear static, since this method is cost-effective and easy to perform. Consequently, one of the intentions in this study is to know the achievement of the simplest analysis procedure (i.e., Linear Static) for evaluation of the progressive collapse potential of two kinds of buildings.

2.5.1 Linear Static Process

The most important method of analysis offered in the GSA guidelines is the linear static (LS) method. Generally, the LS process is the most basic of the four procedures and therefore the analysis can be finished rapidly and it is simple to estimate the consequences. Though, it is not easy to forecast exact behavior in a structure, owing to the lack of the dynamic result and material nonlinearity by rapid failure of one or more members (Kaewkulchai & Williamson, 2003). The examination is run on the assumptions that the construction only undergoes small deformations and that the materials respond in a linear elastic mode. Hence, the LS method, is limited to simple and low to medium rise structures (i.e., less than ten stories) with expected behavior (GSA, 2003).

2.5.2 Nonlinear Static Process

In a nonlinear static (NLS) process, geometric and material nonlinear behaviors are examined during the investigation. The NLS method is generally accomplished for a lateral load called pushover analysis. A stepwise raise of vertical loads is applied till the greatest loads are attained or until the structure fall down, which is recognized as vertical pushover analysis. This process explains the linear static process because structural elements are permitted to undergo nonlinear performance during the NLS analysis. However, vertical push over analysis for the progressive collapse potential might lead to very conservative results (Marjanishvili, 2004). The NLS process still does not explain the dynamic effects and therefore it is unsuccessful to be used for progressive collapse analysis. NLS analysis is not used in this study.

2.5.3 Linear Dynamic Process

Dynamic analysis explains the factors which are calculated during analysis, such as, dynamic intensification factors, inertia, and damping forces. Dynamic analysis is more difficult and time-consuming than static analysis, whether it is linear or nonlinear. However, the linear dynamic (LD) process compared with static analysis, determines more accurate results. For the construction with large plastic deformations, one should be cautious to use this analysis process since it may wrongly calculate the dynamic parameters (Marjanishvili, 2004). In this research, linear dynamic analysis was not used.

2.5.4 Nonlinear Dynamic Process

The nonlinear dynamic (NLD) process is the most comprehensive method of progressive collapse analysis. This method contains both dynamic character and nonlinear behavior of the progressive collapse event. Extra accurate and realistic outcomes can be achieved from the NLD process while it is very time-consuming to calculate and confirm analysis results (Marjanishvili, 2004). This analysis method is not used for this research.

2.6 Design Guidelines to Defend Against Progressive Collapse

Progressive collapse is a substantial concern for the reason that local damage may cause huge destruction and collapse of a structural system. In recent years the progressive collapse by terrorist attacks has created an urgent necessity for the inclusion of design guidelines and criteria in design standards to avoid or decrease progressive collapse. Figure 5 shows the timeline of the most important events followed by changes in the building codes for reducing progressive collapse. The number of building disasters and related code changes has been considerably improved during the last decade.

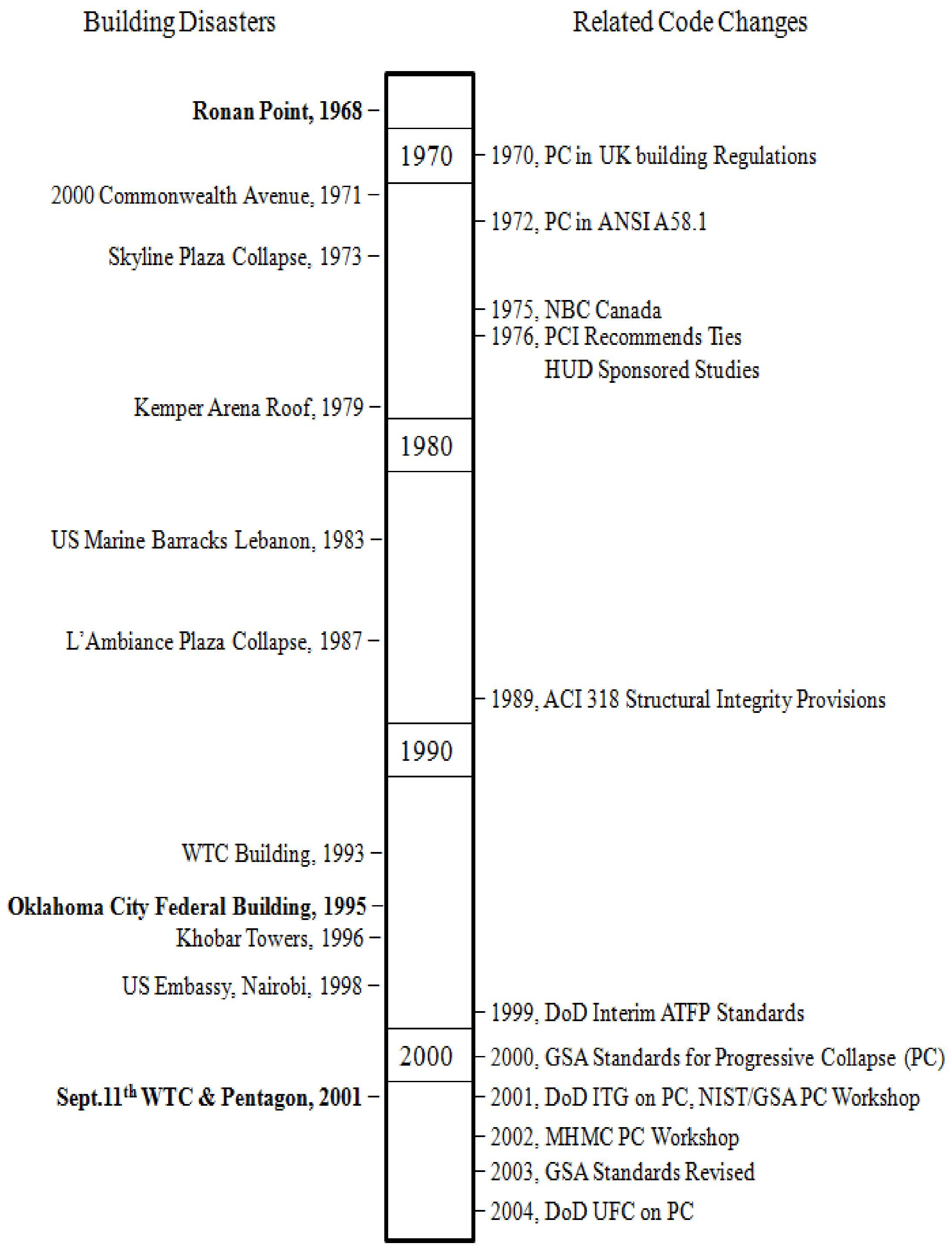

Figure 5: Timeline of main terrible events followed by major building code changes for progressive collapse lessening (Humay et al., 2006)

Some building codes, standards and design guidelines, such as the General Services Administration (GSA, 2003) and the Department of Defense (DoD, 2005), National Institute of Standards and Technology (NIST, 2005), American Society of Civil Engineering (ASCE 7-05, 2005) and American Concrete Institute (ACI 318-08,

2008) are used for the prevention of progressive collapse. These two US agencies (i.e., GSA and DoD) seriously considered preventative mesures against progressive collapse. ASCE 7-05 (2005) presents an explanation for progressive collapse, but do not offer detailed guidelines or requirements for the progressive collapse analysis. ACI 318-08 (2008) addresses provisions to develop the structural integrity of concrete structures, but does not particularly concentrate on provisions for progressive collapse. The design guidance issued by GSA and DoD addresses majority of the comprehensive information in the U.S. presently existing on the progressive collapse prevention, on the condition that these information is based on experimental and enforceable requirements (Humay et al., 2006).

2.6.1 DoD Guidelines

The U.S. Department of Defense issued a document, "*Design of buildings to resist progressive collapse*", in the casing work of the Unified Facilities Criteria (UFC) (DoD, 2005). This document was arranged for the new DoD structure such as military buildings and most important renovations. Particularly, all DoD buildings with three or more stories are necessary to consider progressive collapse. The DoD guideline can be assigned to reinforced concrete, steel masonry, wood and cold-formed steel structures and structural components.

The DoD guideline explains how to examine and design the building structures against progressive collapse. A combination of direct and indirect design methods were used, which relate to the necessary level of protection for the facility: indirect design used for very low and low levels of protection, and both indirect and direct design (Alternate Path) used for medium and high levels of protection. A suitable level of protection can be accommodated to decrease the risk of mass wounded for all DoD employees at a acceptable cost.

2.6.2 GSA Guidelines

The U.S. General Services Administration (GSA) guideline, characterized "*Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects*", was particularly arranged to make sure that the potential for progressive collapse is considered in the construction, planning, and design of new federal office buildings and most important modernization projects (GSA, 2003). The target of the guidelines is to avoid general collapse after a local failure has happened.

According to the GSA guidelines, progressive collapse analysis is accomplished by the performance of the alternate path method of design. The linear elastic and static method is the principal process of analysis in this design guideline. For low- to medium- rise structures, with ten or less stories and classic structural configurations linear methods are used. The buildings with more than ten stories, the GSA guideline suggests that the use of nonlinear procedures should be considered. The GSA guideline describes the whole procedures for the analysis of progressive collapse, the loads to be use for the analysis, and the acceptance criteria for progressive collapse. The issues associated with the avoidance of progressive collapse are considered for reinforced concrete and steel building structures (GSA, 2003).

The GSA guidelines are valuable guidance for reducing the potential for progressive collapse in the design of new and upgraded buildings, as well as for estimating the potential for progressive collapse in existing buildings. In this study, the progressive collapse potential of two steel buildings evaluated by GSA guidelines. The detailed GSA suggestions and loading combination for a computer model and the column removal procedure used in this study are explained in Chapters 3 and 4, respectively.

2.7 Progressive Collapse Studies

The analysis procedures explained in section 2.4 can be applied to new and existing buildings alike, but they are more amenable for the new buildings because structural particulars are readily accessible. Additionally, as oppose to the existing buildings, there is no anxiety relating to the strength properties of structural elements that may be overstressed for the reason of uncertainty about element properties. In the case that alternate load paths due to a column being missing, strategies used to reduce progressive collapse usually include upgrading the size of the critical members, upgrading of serious connection details, adaptations to the frame of the structures, or a combination thereof.

2.7.1 Member Size Upgrades

A case for the application of strategies to decrease the collapse potential of steel buildings can be attained in UFC 4-023-03 (DoD, 2005). The example of the structure was a five-story steel moment frame office building display on plan in Figure 6. The industry standard software and techniques used for the design of the structure. For accomplishing a nonlinear static progressive collapse analysis a 3-D mathematical model of the structure was done according to the guidelines on loading, hinge properties, hinge locations and material details contained in the UFC (DoD, 2005) and by using SAP 2000NL. The internal column shown in Figure 6 was eliminated.

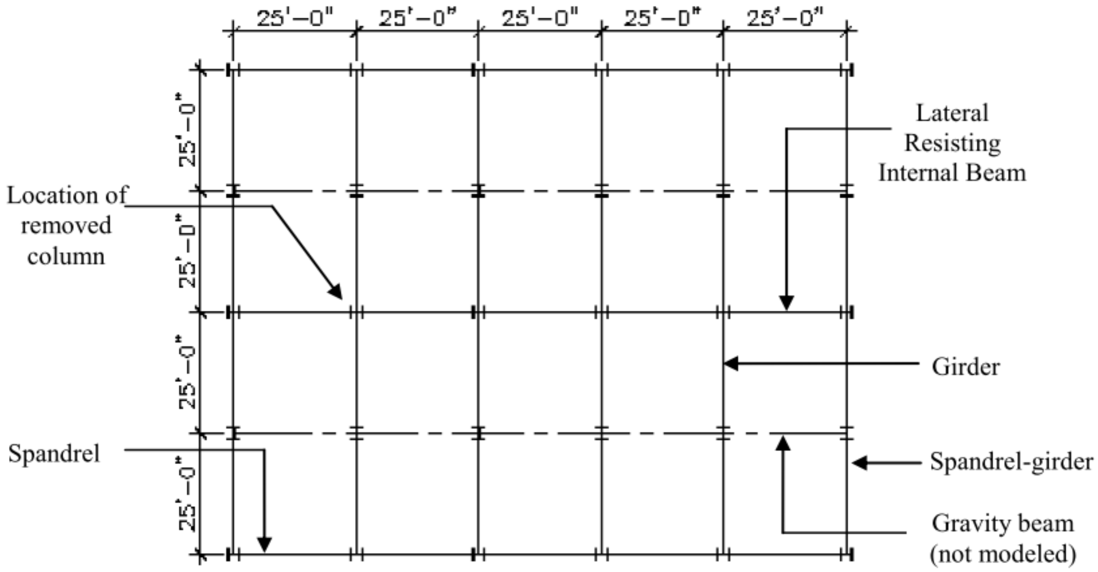

Figure 6: Plan of example construction given in UFC 4-023-03 (DoD, 2005)

The research was repeated with bigger member sizes after each failed analysis, until results showed that the building had a low potential for progressive collapse. The beginning and the final member sizes of the structure are given in Table 1 for comparison. Clearly, substantial increases in the section sizes of few member groups of this structure were necessary to improve its collapse resistance.

Table 1: Initial and final member sizes for UFC example (DoD, 2005)

Member Group	Prelim. Section	Final Section
Spandrels	W18x35	W18x35
Interior Beams	W18x35	**W18x65**
Girders	W18x55	**W21x83**
Spandrel-Girders	W18x40	W18x40
Bottom Columns (1st to 3rd Floor)	W14x145	W14x145
Top Columns (4th to 5th Floor)	W14x68	**W14x82**

2.7.2 Vertical Segmentation

A vertical transmission of collapse, like the Ronan Point failure, may happen in steel framed buildings if column removal leads to beam collapse that starts the collapse of the floors. To reduce losses from such kind of collapse, the theory of using considerably rigid horizontal systems that describe vertical segments within which failure is arrested has been presented by Crawford (2002) and Starossek (2008). By installing an alternate load path or by absorbing the energy related to local collapse the stiff horizontal systems arrest the progressive collapse.

In high-rise buildings, the use of trusses to decrease progressive collapse is briefly explained by Crawford (2002) and more about this approach is discussed in this section. When collapse is to be contained, two kinds of trusses may limit the vertical segment, Figure 7. The trusses are situated between designated floor levels and incorporate columns and floor beams as web and chord members respectively, as well as diagonal web members. The loads conveyed by a column with the truss at the bottom of the segment and the truss on the above of a section are considered to sustain loads as a strong panel. Consequently, if a column is missing within a segment or a truss, a protected alternate load path develops in which columns on top of the removed column become tension elements that transfer floor loads to trusses above. The trusses in chain distribute the loads to the foundation through the undamaged or unaffected columns.

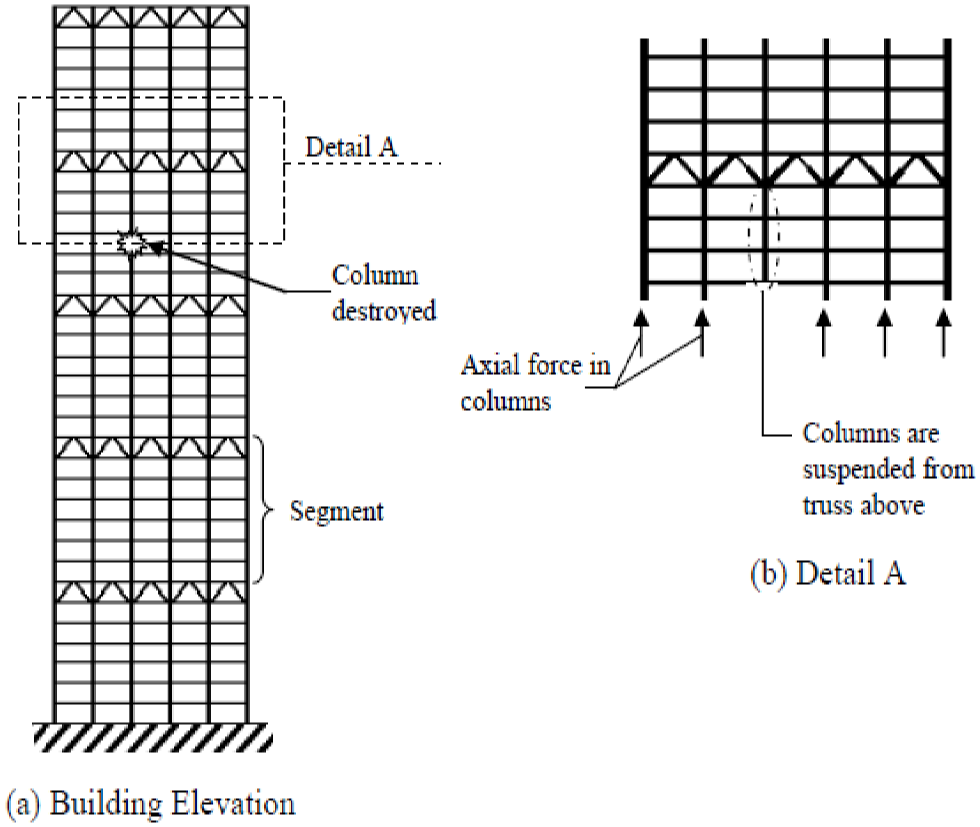

Figure 7: Vertical segmentation with trusses
(Crawford, 2002)

2.7.3 Improve Connection

The beam-to-column connections are one of the hypotheses of the alternate load path analysis that provide adequate strength between beams transverse to a removed column. Therefore, they do as a single beam with two span lengths. Sometimes, to have adequate capacity to supply the necessary continuity between beams it may require upgrade of the existing connections.

If a column is removed, Crawford (2002) argues the use of a SidePlate™ system, shown in Figure 8, to provide a solid connection across beams. The SidePlate™ system was improved in reaction to the connections collapse detected during the 1994 Northridge earthquake for the new and retrofitted structure (Houghton, 2000) . One of the key properties of the link system is that when using full-depth of plates beside beams that cause to deform first before column panel zones. The side plates supply improved rotational and energy dissipation ability that is beneficial for alternate load path and explosion loading scenarios (Houghton, 2000).

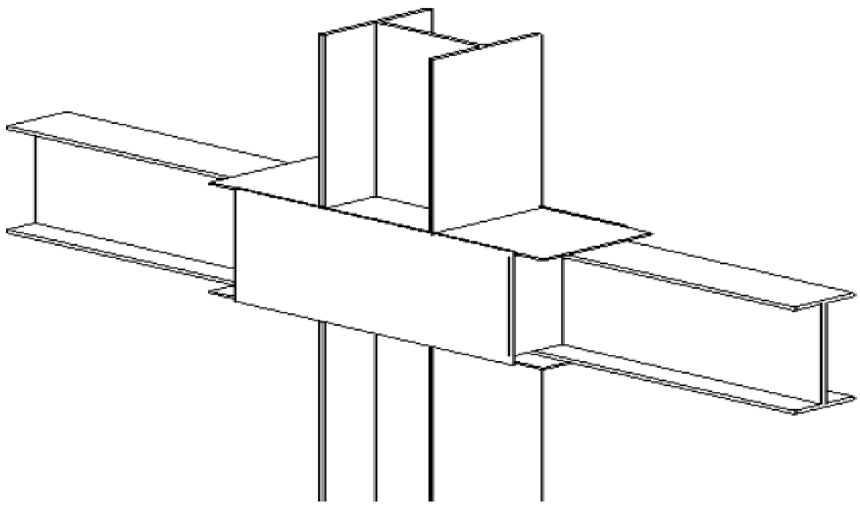

Figure 8: SidePlate™ connection aspects
(Houghton, 2000)

2.7.4 Vierendeel Action

Improvement of the critical element sizes are not realistic for existing buildings. As a substitute, a structural design configuration to support the formation of alternate load paths when a column is missing is a more useable explanation. Herrle and McKay explained a theory that uses structural alterations in a 5-story federal structure to decrease progressive collapse. The Vierendeel truss is the improvement idea included the establishment of new vertical structural elements between the second and third floors. All shear links along the outside of the structure on all floors were improved to fully-restrained moment links (Figure 9). A linear-static study of the structure was accomplished by GSA guidelines for progressive collapse analysis (GSA, 2003). Therefore, demand-capacity ratios (DCR) were analyzed to evaluate the achievement of the structure. The analysis carried out after careful elimination of a corner column and a worst-case moment DCR of 38.2 was discovered after analysis of the original construction. With improved model applied to invoke Vierendeel action when a column is removed, the maximum moment DCR was decreased to 0.78 (Herrle & McKay, 2008).

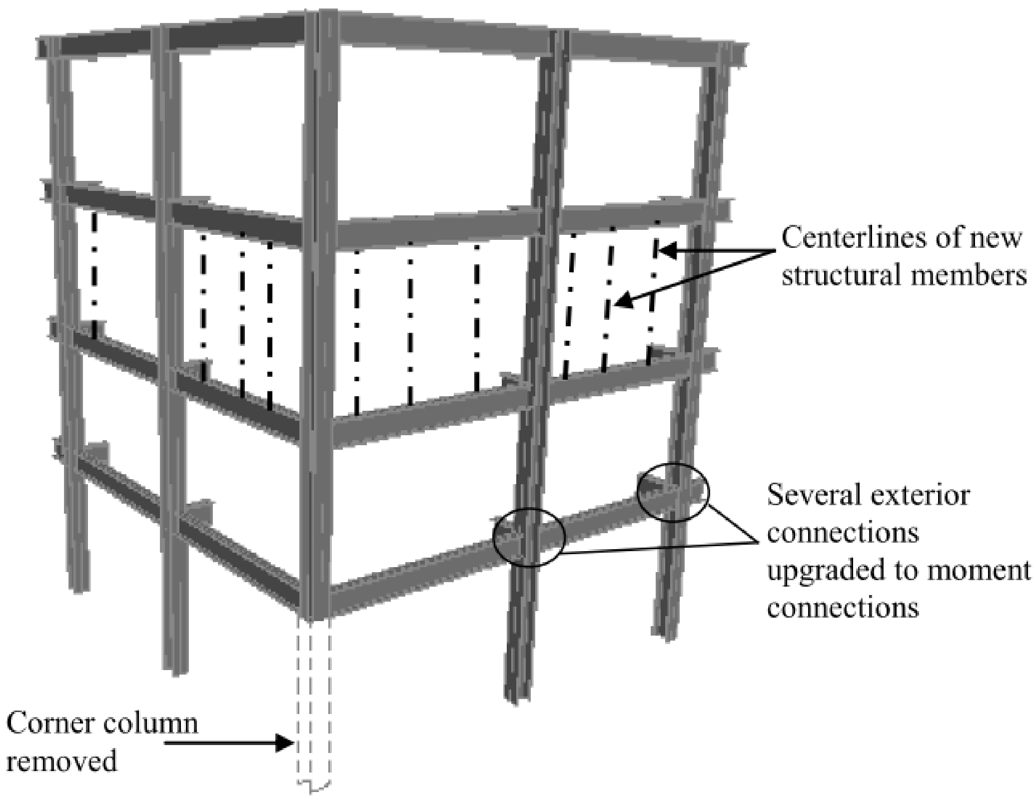

Figure 9: Improved Concepts for Existing 5-Story Federal Building
(Herrle & McKay, 2008)

2.7.5 Use of Cables for Existing and New Buildings

Following the same strategy of modifying an existing structural configuration of a building to resist againt progressive collapse, the concept of using steel cables in buildings for which exterior column loss (not corner) is considered has been investigated at the University of California Berkley (Astaneh, 2003). For existing structures the cables are located along the side of the beam, but for new building cables are located in the slab on top of the spandrels (Figure 10). In both models the cables are joined to all outside columns. Except for removing a corner column, the loss of an external column actuates the cables to convey loads to the other side of the structure to stop the floor at that stage from collapsing. The study tested the performance of an exterior frame without cables, a frame with cables in the slab, and a frame with cables joined to the side of the spandrel when subjected to progressively rising downward load functional at the place of a loss column (Astaneh, 2003).

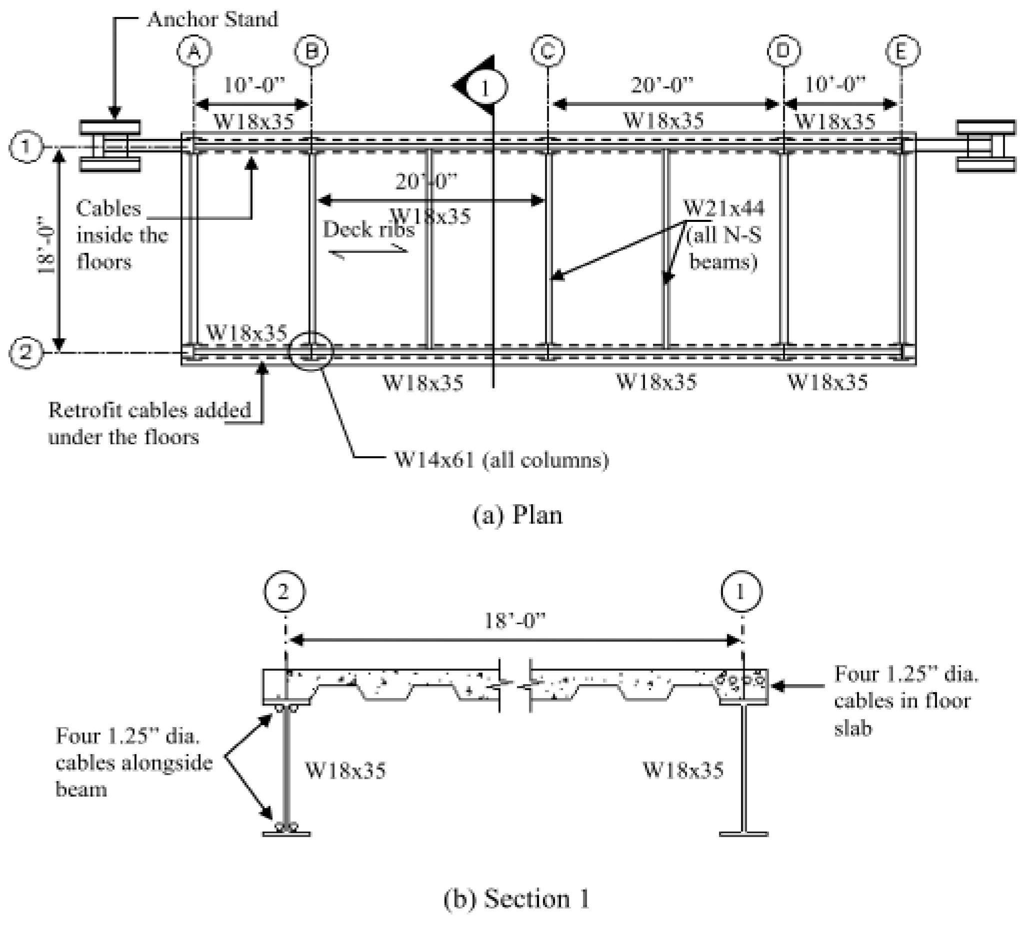

Figure 10: Schematic of test set-up
(Astaneh, 2003)

The consequences of the research showed that the design of new structure in which cables are located inside the slab could sustain 3.1 times the design load without collapse. The retrofitted sample withstood 1.5 times the design load as the sample not including cables failed at the design load. An impact factor of 1.5 of the design load is included for each cases in the design (Astaneh, 2003).

Chapter 3

RESEARCH METHOD

3.1 Introduction

The progressive collapse performance of two steel buildings was examined through computational analysis. The first building under consideration had normal I-beam as primary beams and the second building hadt russ beams in three sizes of spans as primary beams. The details for these two steel buildings are presented in this chapter. The details of the structural elements for each building are also presented.

There were two most important objectives of this study; the first objective was to investigate progressive collapse in two kinds of steel building and compare their behavior and potential of progressive collapse. The second objective was to compare the rate of deflection for the steel buildings especially for the long spans.

3.2 Truss beam and Normal Beam

A truss beam is a structural assembly by small interconnected elements. A network of elements in a truss, which assumed to sustain tension or compression. A beam is a structural element considered mainly to resist bending. The connection between the members of a truss is assumed to be pin jointed. Therefore, assuming that the loads are acting at these joints then each bar can only be in tension or compression and do not subject to bending. If connected into triangles, then members appear as a rigid truss structure that acts as a stiff portion. A truss is lightweight and simple to handle.

One of its most important advantage is strength-to-weight ratio. Usually, most of the space inside a truss is unfilled; it is the skeleton of elements that shapes the structure (Wisegeek, 2012).

When an I-shaped beam is subject to simple bending the bulk of the resistance to bending moment is obtained by a couple created by the forces acting at the two flanges of the beam multiplied by the distance between the flanges. It is assumed that all resistance to bending is offered in this way. The most efficient system will be the one in which the flange forces are reduced to a minimum to save material, and the distance between them are increased accordingly (Wisegeek, 2012).

The truss beam composite with steel deck and concrete slab is considered particularly for composite floor structure where column free long spans are necessary such as factories, workshops and railway stations. The open web configuration of the steel truss beam allows for easier passage of services. This includes the ability to cross over services within the depth of the lattice beam, which can be more difficult to achieve with normal I-beams. The trusses generally recognized as suitable for structure with spans from 10 meters to 100 meters. Generally a span to depth ratio of parallel boom trusses are approximately 15:1 for light loading to approximately 10:1 for heavier loading (Wisegeek, 2012).

3.3 The Buildings with Truss Beams

3.3.1 Description of the Building

For this building three types of plan layouts were used with three different spans of truss beams for the design. The building has four stories. The finishing floor heights for the basement and the other stories are 2.75 metres and the span to depth ratio of the truss is 1:10. There are four bays in the longitudinal direction (x-direction) and six bays in the transverse direction (y-direction). For case 1, the four bays in the longitudinal direction has 9 m column spacing and the six bays in the transverse direction has 3 m column spacing. Figure 11 shows the ETABS [version 9.7.4] model of case 1 building with truss beams and Figure 12 shows the plan layout of the same building with the positions of the columns removed.

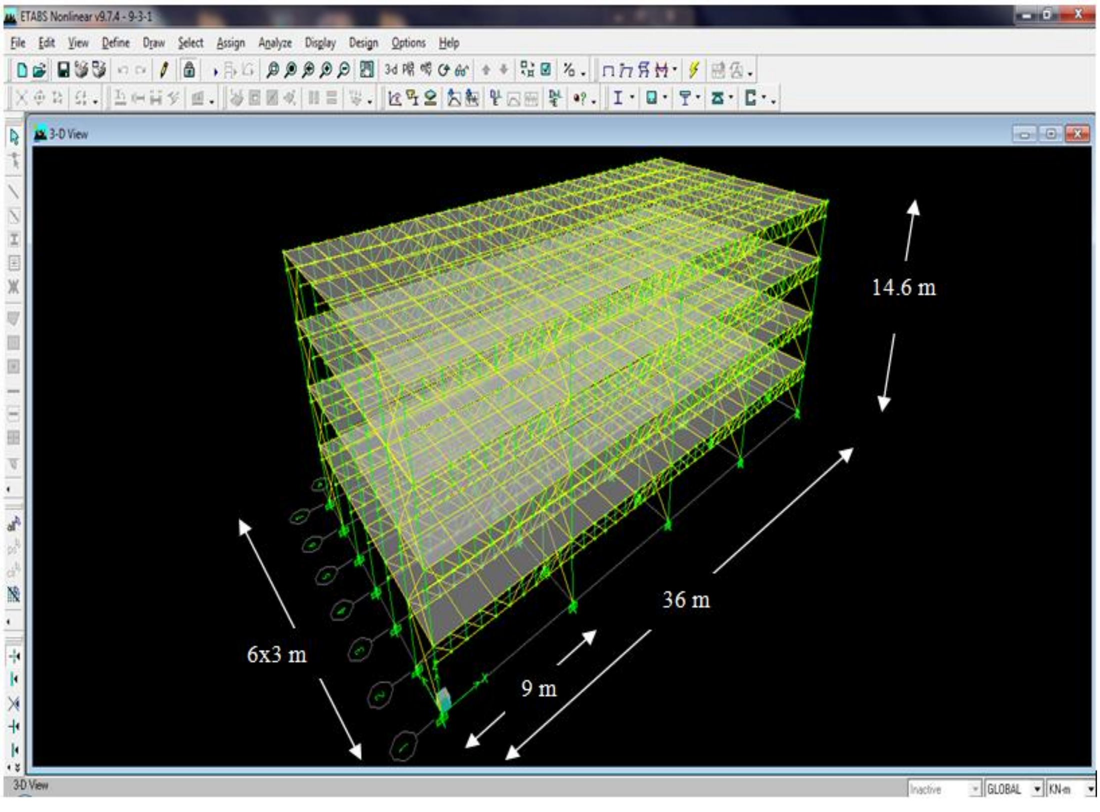

Figure 11: A 3-D model of case 1 structural building with truss beams

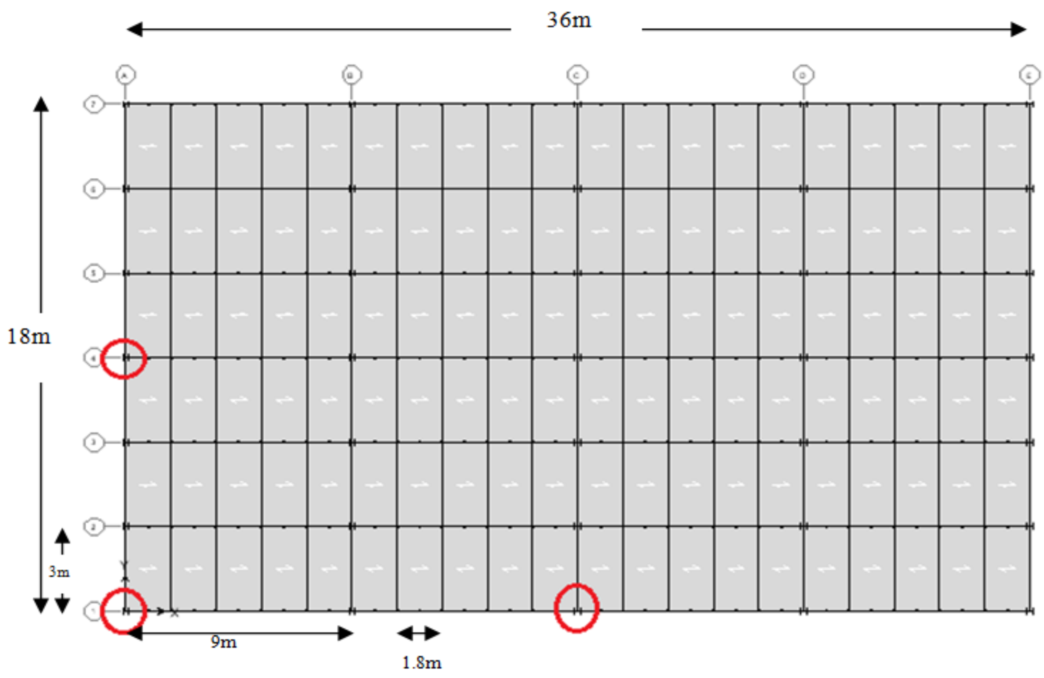

Figure 12: Typical plan layout for the case 1 building with truss beams and the columns removed are highlighted

Figure 13 shows the elevation of the four-story high building with composite truss beams in the longitudinal direction, case1.

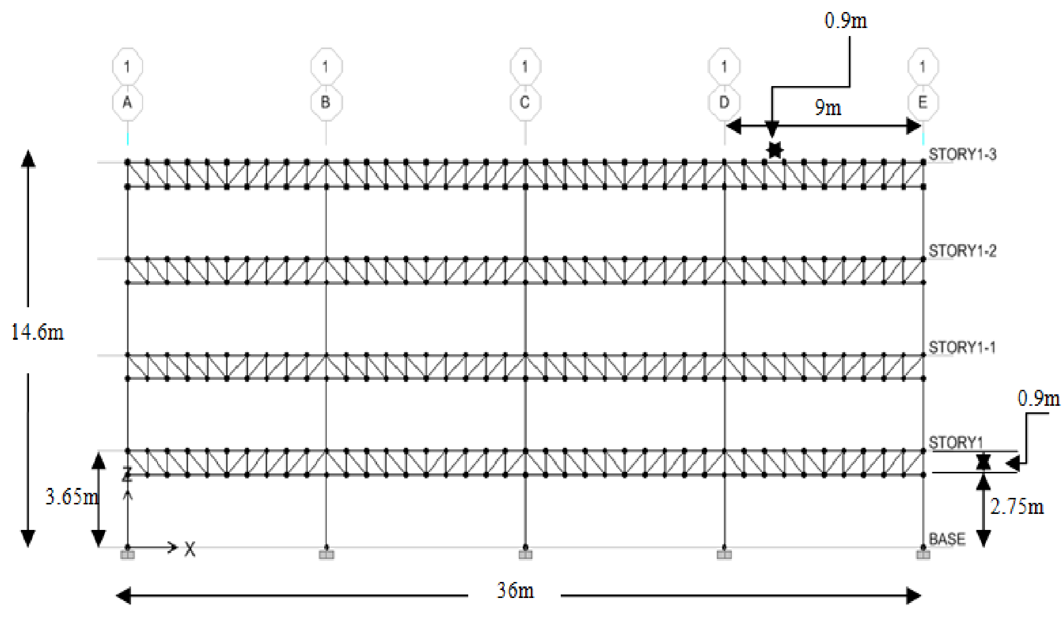

Figure 13: The elevation of the four-story high building with composite truss beams in the longitudinal direction, Case1

Case 2 can be seen in Figure 14 where the building in the longitudinal direction has a column spacing of 12 m (four bays) and 4 m in the transverse direction (six bays).

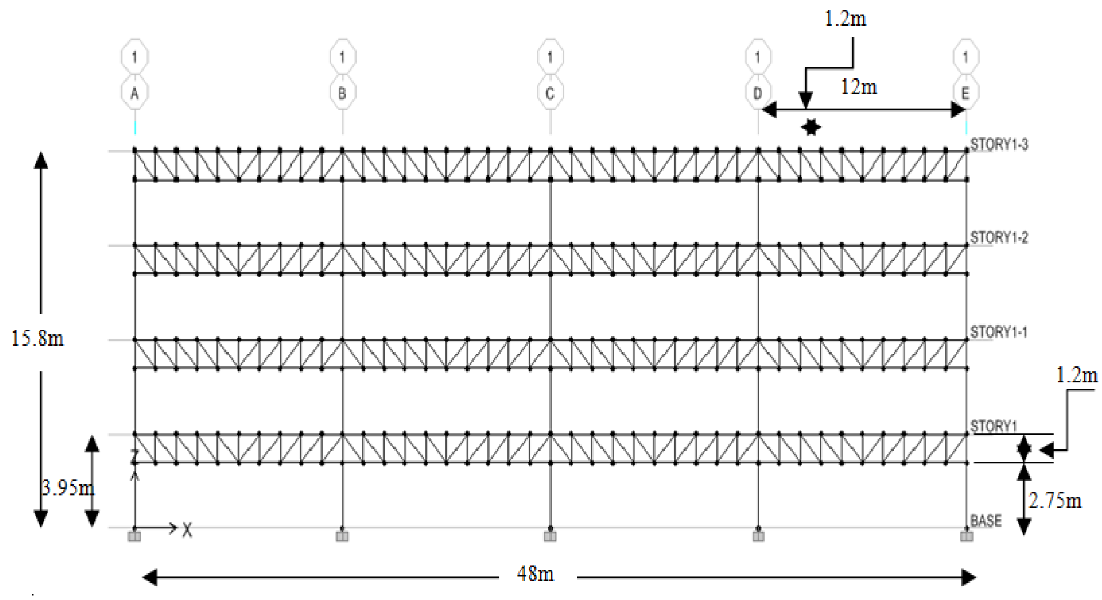

Figure 14: The elevation of the four-story building with truss beam in the longitudinal direction, Case 2

Case 3 is shown in Figure 15 where the column spacing of 15 m (four bays) in the longitudinal direction and 6 m in the transverse direction (six bays) are used.

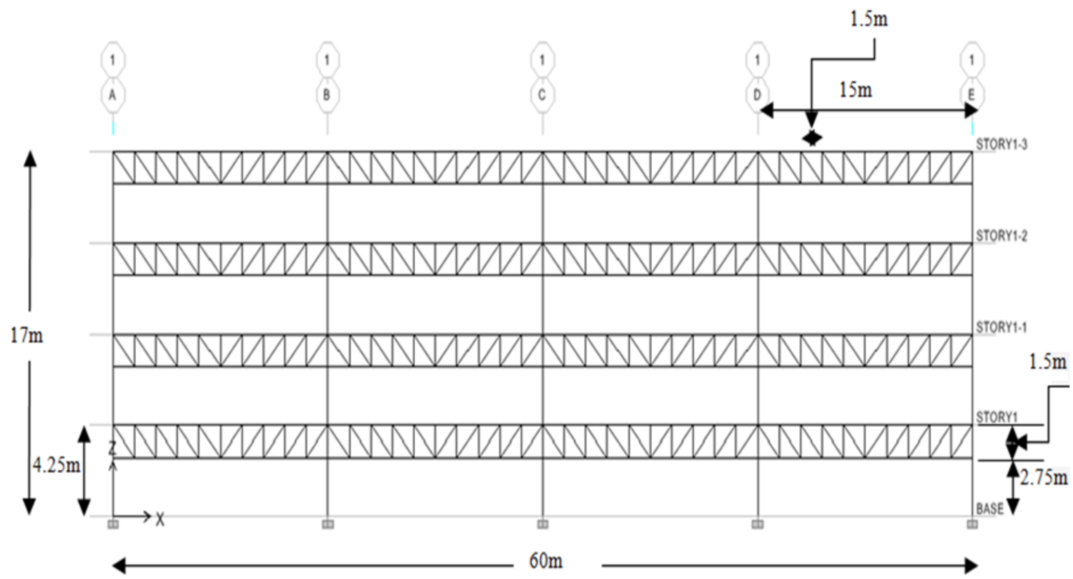

Figure 15: The elevation of the longitudinal direction of four-story building with truss beams, Case 3

3.3.2 Properties of Structural Members

The building with truss beams is a truss beam frame in x-direction and concentrically braced frame in the y-direction. The properties of truss beams and columns are shown in Tables 2 to 4. These are the European steel sections with the relevant European designation.

Table 2: Steel sections for Case 1

Case 1 European Steel Sections	
Storey Designation	**Column Section**
Storey 1 Storey 1-1 Storey 1-2 Storey 1-3	HD320x97.6 HD320x97.6 HD320x74.2 HE180B
Truss Member Sections	
Section	**Type**
Top chord Bottom chord Diagonal elements Vertical elements	TUB60x60x5 TUB140x140x16 TUB60x60x4 TUB60x60x4
Beam Sections	
All storey	IPE180

Table 3: Steel sections for Case 2

Case 2 European Steel Sections	
Storey Designation	**Column Sections**
Storey 1 Storey 1-1 Storey 1-2 Storey 1-3	HD320x158 HD260x114 HD260x93 HD260x68
Truss Member Sections	
Section	**Type**
Top chord Bottom chord Diagonal elements Vertical elements	TUB60x60x5 TUB180x180x20 TUB70x70x10 TUB70x70x10
Beam Sections	
All storey	IPE220

Table 4: Steel sections for Case 3

Case 3 European Steel Sections	
Storey Designations	**Column Sections**
Storey 1 Storey 1-1 Storey 1-2 Storey 1-3	HD400x237 HD360x196 HD360x179 HD260x93
Truss Member Sections	
Section	**Type**
Top chord Bottom chord Diagonal elements Vertical elements	TUB60x60x5 TUB240x240x20 TUB100x100x10 TUB100x100x10
Beam Sections	
All storey	IPE330

3.4 The Buildings with Normal I-Beams

3.4.1 Description of the Building

The second buildings has standard I-beams. This building is also four-story high steel framed structure. The finishing heights at the basement and other stories are 2.75 meters (red numbers shows the height of beams that are used in the structures). Figure 16 shows the 3-D view of the building and the columns removed are highlighted. Three different plan layouts were used for the design. There are four bays in the longitudinal direction and six bays in the transverse direction.

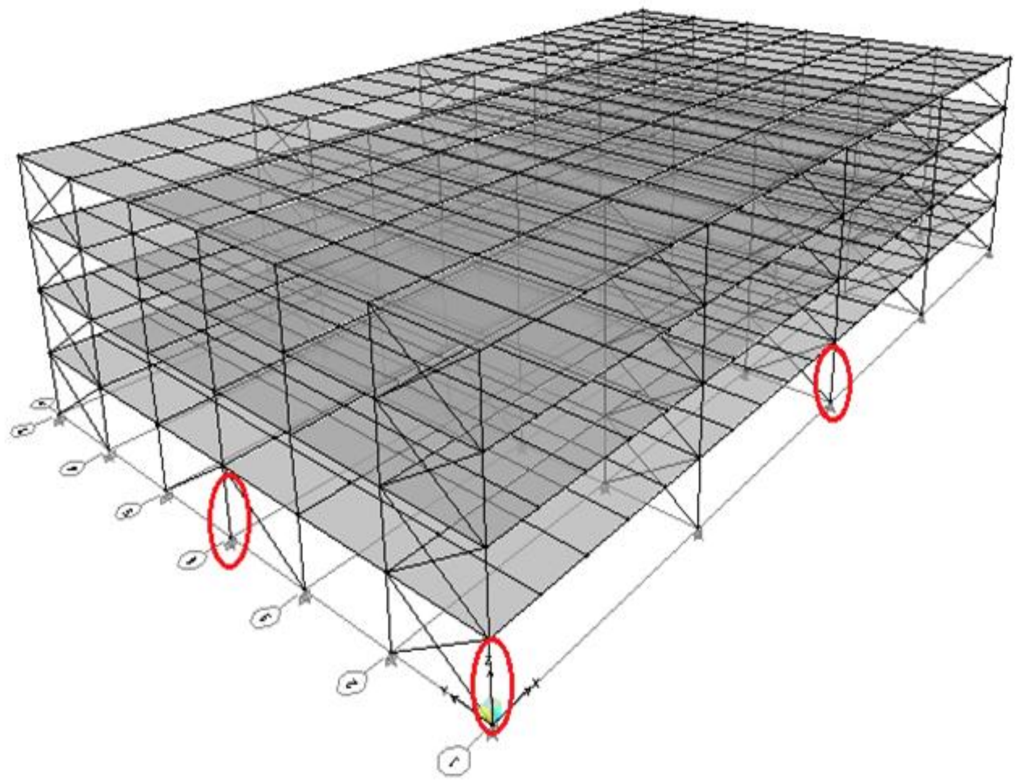

Figure 16: Columns removed are highlighted

In Case 1, column spacing of 9 m (four bays) in the longitudinal direction and 3 m in the transverse direction (six bays), are used as shown in the Figure 17.

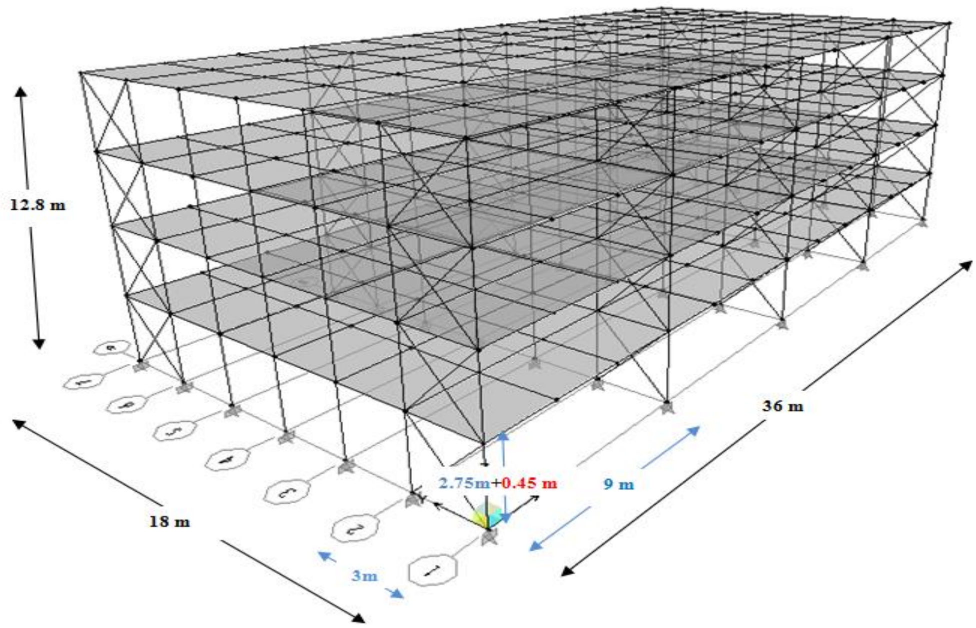

Figure 17: Three-dimensional ETABS model of normal I-beam, Case1

In case 2, shows in Figure 18 the longitudinal direction with column spacing of 12 m (four bays) and 4 m in the transverse direction (six bays).

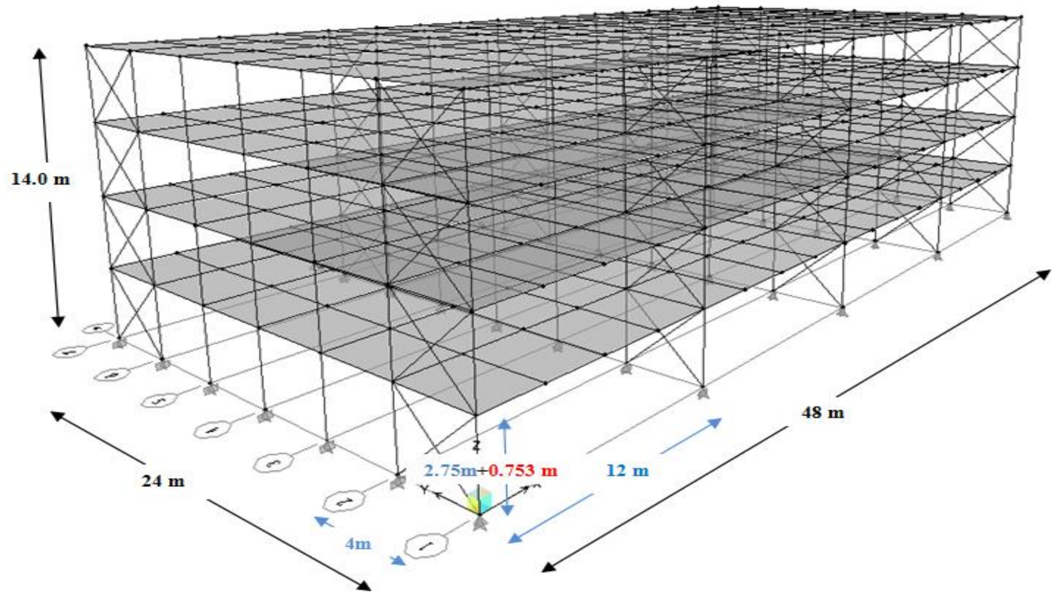

Figure 18: Three-dimensional ETABS model of normal I-beam Case2

Case 3 is shown in Figure 19 the longitudinal direction with column spacing of 15 m (four bays) and 6 m in the transverse direction (six bays).

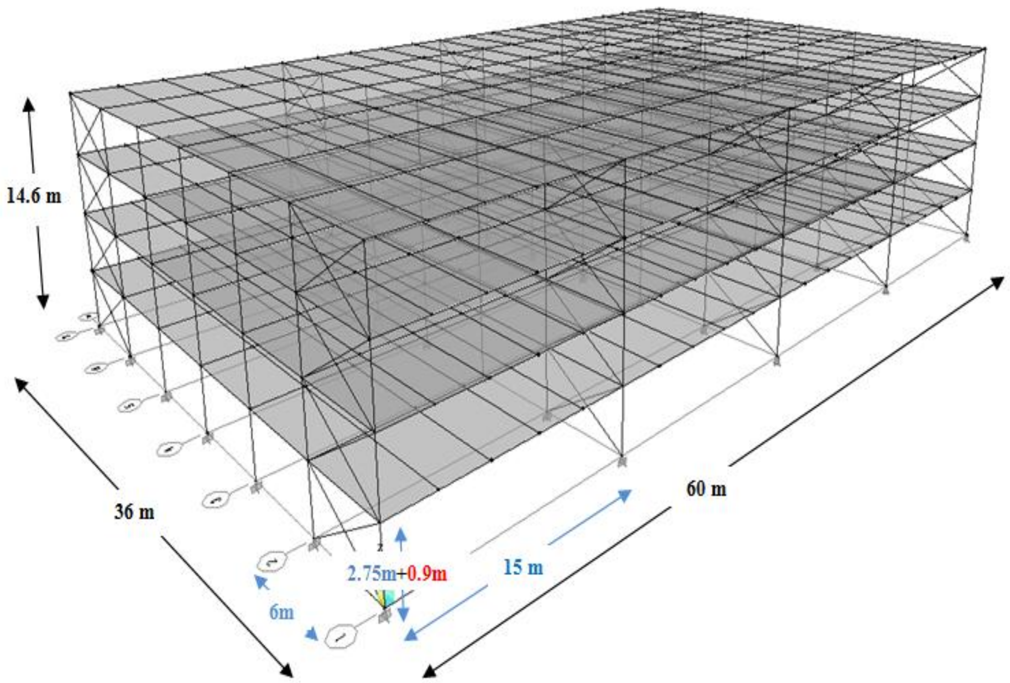

Figure 19: Three-dimensional ETABS model of normal beam Case3

3.4.2 Properties of Structural Members

In this study the test building was a brace frame structure. The properties of beams and columns show in Tables 5 to 7. In these tables, for HD sections the first and last numbers are the width of the section (in millimeter units) and mass per unit length (kg per linear m) of the column, respectively. For HE sections the number is the width of the section and for IPE section the number is the height of the section.

Table 5: Sections of case 1 (Normal I-beam)

Case 1 European Steel Sections	
Storey Designation	**Column Section**
Storey 1 Storey 1-1 Storey 1-2 Storey 1-3	HE360B HE280B HE220B HE160B
Beam Section (Long side)	
All storey	IPE450
Beam Section (Short side)	
All storey	IPE200

Table 6: Sections of case 2 (Normal I-beam)

Case 2 European Steel Sections	
Storey Designation	**Column Section**
Storey 1 Storey 1-1 Storey 1-2 Storey 1-3	HE450B HE360B HE280B HE200B
Beam Section (Long side)	
All storey	IPE750x147
Beam Section (Short side)	
All storey	IPE240

Table 7: Sections of case 3 (Normal I-beam)

Case 3 European Steel Sections	
Storey Designation	**Column Section**
Storey 1 Storey 1-1 Storey 1-2 Storey 1-3	HD400x347 HE600B HE400B HE300B
Beam Section (Long side)	
All storey	HE900B
Beam Section (Short side)	
All storey	IPE330

3.5 Modeling Procedures

Computational progressive collapse analysis of the two buildings was performed by the commercially available computer program, ETABS [Version 9.7.4] through the use of General Services Administration (GSA) guidelines (GSA, 2003). The buildings under consideration have three different span of beams which one has trusses as floor beams and the other one has used normal I-beams. This chapter presents three-dimensional (3-D) computer models of each of these buildings using ETABS program. The assumptions and complete procedures for the modeling of these buildings are described. Also, the calculations for loading and the criteria regulated in the GSA guidelines are provided.

3.6 Modeling Assumptions

While a building was modeled in this study, a number of assumptions were made to make things easier and to show the steps of progressive collapse analysis. These assumptions are described below:

(1) The buildings were modeled as braced frames.

(2) The base plate to foundation connections were assumed to be fixed connections at the x-direction and pinned at y-direction.

(3) Secondary members (e.g., transverse joist beams and braces) were ignored and did not contribute to the progressive collapse resistance.

(4) The occupancy of these buildings was assumed as offices.

(5) The depth of trusses assumed as span /10.

3.7 Arrangement and Modeling of the Buildings

Progressive collapse of two buildings was investigated using the ETABS computer program (ETABS version 9.7.4). ETABS is a widely known structural analysis and design software, generally used in traditional building design. For modeling steel deck of the slab design, it could be used and analysis in ETABS software. This program was used to develop the 3-D frame of each building and then analayse them.

3.7.1 Model of Buildings with Truss Beams and Normal I-Beams

ETABS program was used to investigate the progressive collapse potential. Figure 11 and Figure 16 shows 3-D model and plan layout of the buildings with truss beam and normal I-beam frames. The circles indicate the order in which the column would be removed. 3-D models can sufficiently account for 3-D property and keep away from very conservative consequences. DoD and GSA, both of these guidelines suggested the use of 3-D models in the progressive collapse analysis (DoD, 2005; GSA, 2003) .

3.8 Material Properties

The building which is used truss beam and normal beam was a regular steel frame structure, including steel columns, beams, and connections. The connection between truss elements is pinned and also the steel girders, beams, and columns were connected with simple connections.

S275 steel grade with a minimum yield strength of 250 N/mm^2 is used for all members of the steel frame for the two buildings. The modulus of elasticity of steel was set equal to $2.0E^{+8}$ kN/m^2 .

3.9 Loading Conditions for Analysis

For evaluating the progressive collapse for every structural member in the building, GSA (2003) recommended a common loading factor to be used. According to GSA, for the linear static analysis of a structure, the following gravity loading conditions are recommended to be used:

$$\text{Load}=2(DL+0.25*LL) \qquad (1)$$

Where DL is the self-weight of the structure (i.e., Dead Load), which can be automatically generated from steel and slab weight by ETABS based on element volume and material density. For the finishes of the slab and the roof the total dead load was assumed as 2.5 kN/m^2. LL is the live load of the structure and for these analysis it is assumed to be 3.0 kN/m^2 since the buildings are assumed to be used as offices.

3.10 Deck Design

Figures 20 and 21 illustrates details of the galvanized steel deck (RLSD, 2012) used for the slab design. In this study, the slab was considered as one-way spanning slab. The self weight (dead load) of the deck was distributed to the transitional beams supporting the edges of the slab with its tributary region and then transferred to the main beams as a consistent load then acting like a point load to each column. The stiffness of the building due to the slab was not considered in the model since the joint connections of the intermediate beams were acting as a member trusses (pinned connections).

Ribdeck E60 Section Properties (per metre width)

Gauge mm	Self Weight kg/m²	Self Weight kN/m²	Area mm²	Inertia cm⁴	Y_{NA} mm
0.9	9.3	0.091	1,140	80.4	37.1
1.0	10.3	0.101	1,273	89.8	37.2
1.2	12.3	0.121	1,538	108.7	37.2

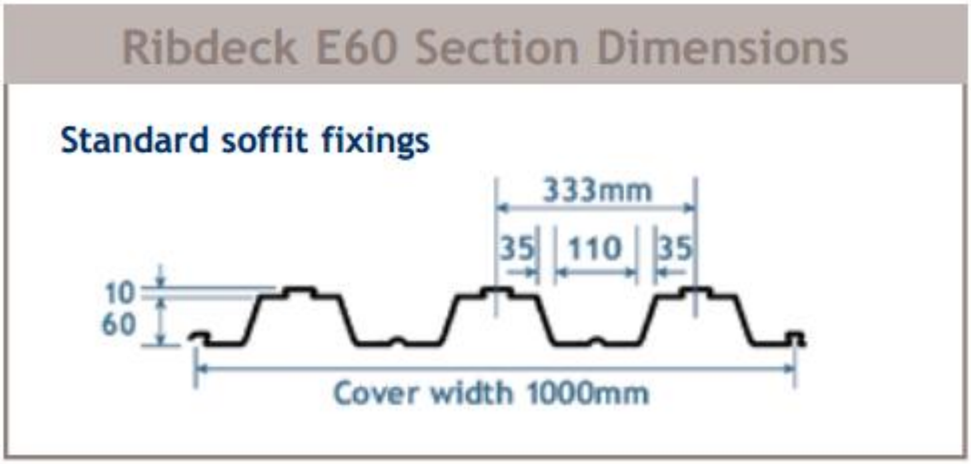

Figure 20: Ribdeck E60 section dimensions
(RLSD, 2012)

Span/load table															Normal weight concrete
Support Condition	Slab Depth (mm)	Concrete Volume (m^3/m^2)	0.9 Gauge Imposed Load				1.0 Gauge Imposed Load				1.2 Gauge Imposed Load				
			FW	5.0	6.7	10.0	FW	5.0	6.7	10.0	FW	5.0	6.7	10.0	
Single - Unpropped	130	0.094	2.74	2.74	2.74	2.59	3.10	3.10	3.10	2.76	3.44	3.44	3.44	3.06	
	140	0.104	2.68	2.68	2.68	2.68	3.03	3.03	3.03	2.91	3.36	3.36	3.36	3.25	
	150	0.114	2.62	2.62	2.62	2.62	2.96	2.96	2.96	2.96	3.29	3.29	3.29	3.29	
	160	0.124	2.57	2.57	2.57	2.57	2.90	2.90	2.90	2.90	3.23	3.23	3.23	3.23	
	175	0.139	2.49	2.49	2.49	2.49	2.82	2.82	2.82	2.82	3.14	3.14	3.14	3.14	
	200	0.164	2.39	2.39	2.39	2.39	2.71	2.71	2.71	2.71	3.01	3.01	3.01	3.01	
	250	0.214	2.22	2.22	2.22	2.22	2.52	2.52	2.52	2.52	2.81	2.81	2.81	2.81	
Multiple - Unpropped	130	0.094	3.31	3.31	3.22	2.59	3.67	3.67	3.47	2.76	4.00	4.00	3.93	3.06	
	140	0.104	3.22	3.22	3.22	2.72	3.58	3.58	3.58	2.91	3.90	3.90	3.90	3.25	
	150	0.114	3.14	3.14	3.14	2.87	3.49	3.49	3.49	3.07	3.81	3.81	3.81	3.45	
	160	0.124	3.07	3.07	3.07	3.00	3.41	3.41	3.41	3.23	3.72	3.72	3.72	3.64	
	175	0.139	2.96	2.96	2.96	2.96	3.30	3.30	3.30	3.30	3.61	3.61	3.61	3.61	
	200	0.164	2.79	2.79	2.79	2.79	3.14	3.14	3.14	3.14	3.45	3.45	3.45	3.45	
	250	0.214	2.53	2.53	2.53	2.53	2.87	2.87	2.87	2.87	3.19	3.19	3.19	3.19	
Multiple - Propped	130	0.094	4.55	3.35	2.93	2.43	4.55	3.61	3.14	2.58	4.55	4.10	3.53	2.85	
	140	0.104	4.90	3.49	3.07	2.54	4.90	3.78	3.30	2.71	4.90	4.32	3.72	3.00	
	150	0.114	5.25	3.64	3.20	2.66	5.25	3.95	3.45	2.84	5.25	4.52	3.90	3.16	
	160	0.124	5.60	3.78	3.32	2.77	5.60	4.10	3.59	2.96	5.60	4.72	4.08	3.31	
	175	0.139	5.88	3.96	3.50	2.92	6.13	4.32	3.79	3.13	6.13	5.00	4.33	3.52	
	200	0.164	5.59	4.24	3.76	3.15	6.23	4.64	4.09	3.40	6.50	5.40	4.71	3.84	
	250	0.214	5.11	4.70	4.21	3.56	5.71	5.17	4.60	3.86	6.16	6.07	5.35	4.41	

Figure 21: Normal weight concrete-Ribdeck E60(RLSD,2012)

3.11 Acceptance Criteria of Demand Capacity Ratio (DCR)

To estimate the results of a linear static analysis, according to GSA guidelines Demand Capacity Ratio should be considered based on the following equation (GSA, 2003):

$$DCR= \frac{QUD}{QCE} \qquad (2)$$

Where: QUD= Acting force (Demand) determined or computed in element or connection/joint

QCE= Probable ultimate capacity (Capacity) of the component and/or connection/joint

Table 8 provides the GSA particular DCR limits for each steel frame section. If structural members with DCR values exceed those given in Table 8, the members are considered to be failed, resulting in severe damage or potential collapse of the structure (GSA, 2003).

DCR < 2.0: for typical structural configuration

DCR < 1.5: for atypical structural configuration (GSA, 2003)

Cases which were chosen for this study have typical structural configuration, therefore, if a DCR value is more than 2.0, in theory the member has exceeded its ultimate capacity at that location.

Table 8: GSA specified DCR acceptance criteria for the steel building (GSA, 2003)

Component/Action	Values for Linear Procedures DCR
Beams – flexure	
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	3
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	2
Columns – flexure	
For 0 < P/P_{CL} < 0.5	
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{300}{\sqrt{F_{ye}}}$	2
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{460}{\sqrt{F_{ye}}}$	1.25
For P/P_{CL} > 0.5	
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{260}{\sqrt{F_{ye}}}$	1
b. $\frac{b_f}{t_w} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{400}{\sqrt{F_{ye}}}$	1

b_f = Width of the compression flange
t_f = Flange thickness
F_{ye} = Expected yield strength
h = Distance from inside of compression flange to inside of tension flange
t_w = Web thickness
P_{CL} = Lower bound compression strength of the column
P = Axial force in member taken as Q_{uf}

3.12 ETABS Analysis Procedures

In this investigation, ETABS computer program was used to create the model of the two buildings and to examine the redistribution of loads after the first story columns were removed. The steps of the complete analysis for the linear static method are described below.

The most straight forward method of progressive collapse analysis is linear static method. This method is used only for very simple construction with predictable behavior. The analysis procedure involves the following steps and determines DCR value and displacement:

1. Build a 3-D model in the ETABS computer program.
2. Select the exterior frames with high potential of progressive collapse.
3. Select GSA guideline based on linear static
4. Apply the static load combination as defined in Equation 1.
5. Removing the column based on GSA guideline.
6. Analyze the structure after removing the column
7. Compute DCR for each element (beams and columns)
8. Evaluate the results according to DCR value.

Chapter 4

RESULTS AND DISCUSSIONS

4.1 Introduction

In this chapter, progressive collapse performance of two buildings was investigated. The result of the analysis and values of DCR for beams and columns are presented in this chapter. These buildings were modeled in ETABS to carry out three-dimensional (3-D) analysis and compare the progressive collapse potential. The result of each structure with truss floor beam and normal I-beam are compared. Three different beam spans for each of these buildings were evaluated to examine the potential of progressive collapse scenarios.

The assumptions made and the procedures followed for the modeling are explained in Chapter 3. For the linear procedure the factor of dead load is 2.0 and the factor of live load is 0.5.

4.2 Progressive Collapse Analysis

4.2.1 Linear Static Analysis of Case 1 with Modeling of the Building with 9 m Span Beams

The linear static analysis is the simplest method usually used to study the progressive collapse potential of a structures (ASCE 7-05, 2005). The calculation of DCR due to GSA was defined in Equation 2. According to GSA and Table 8 a DCR value of 2.0 is the limiting value for each steel element in this study.

4.2.2 Column Removal Procedure

The GSA guidelines demand the removal of first-story columns. As it can be seen from Figure 22, GSA (2003) suggests that a structure should be analyzed by immediately removing a column from the near middle of the short side of the building, near middle of the long side of the building and at the corner of the building. It was implied that immediate removal of an exterior column leads to critical damage to the structural bays exactly linked to the removed column or to an area of 1,800 ft^2 at the floor level exactly above the removed column (Figure 23).

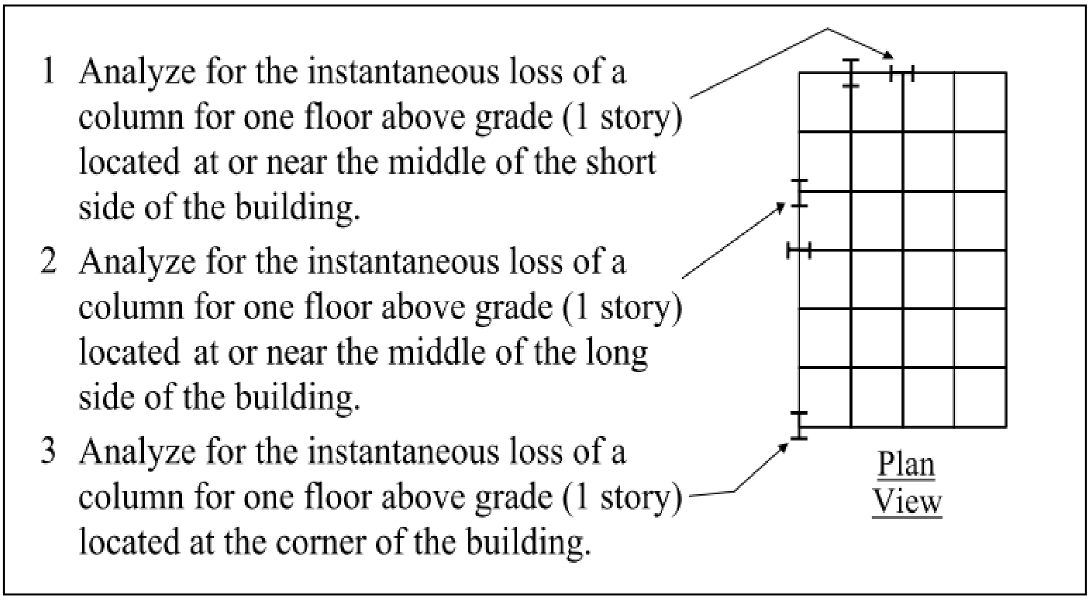

Figure 22: Progressive Collapse Analysis required for the framed structure (GSA, 2003)

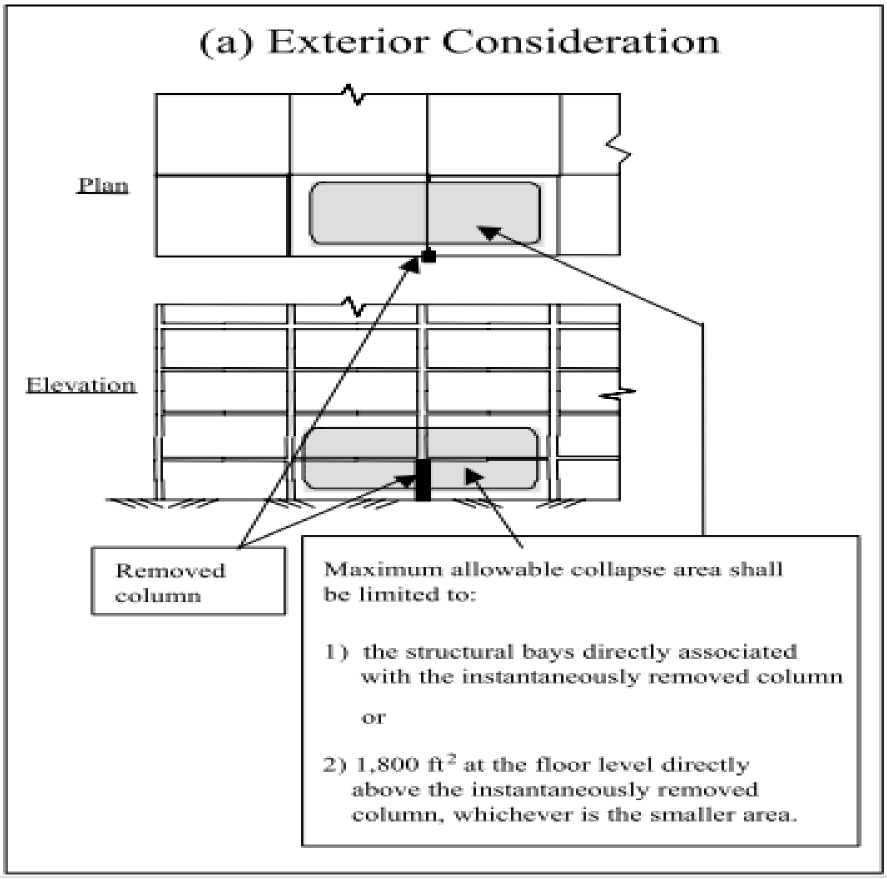

Figure 23: Maximum allowable collapse areas of structure that uses columns for vertical support system (GSA, 2003)

4.3 Modeling of the Building with 9 m Span Beams

4.3.1 Modeling of Buildings with Normal I-Beams And Truss Beams

These steel buildings have braced frames in both x- and y-directions and the design is based on the European Standard Code. After the instantaneous removal of columns analysis of the building was carried out based on GSA guidelines to evaluate the potential of progressive collapse.

Removal of columns in each sequence has been modeled in ETABS [version 9.7.4] software. Figure 24 shows the cases established for the removal of each column. Case 1 is the removal of column from the middle of the long side of building, case 2 removal of column in the middle short side of the building and case 3 indicates removal of column at the corner of the structure.

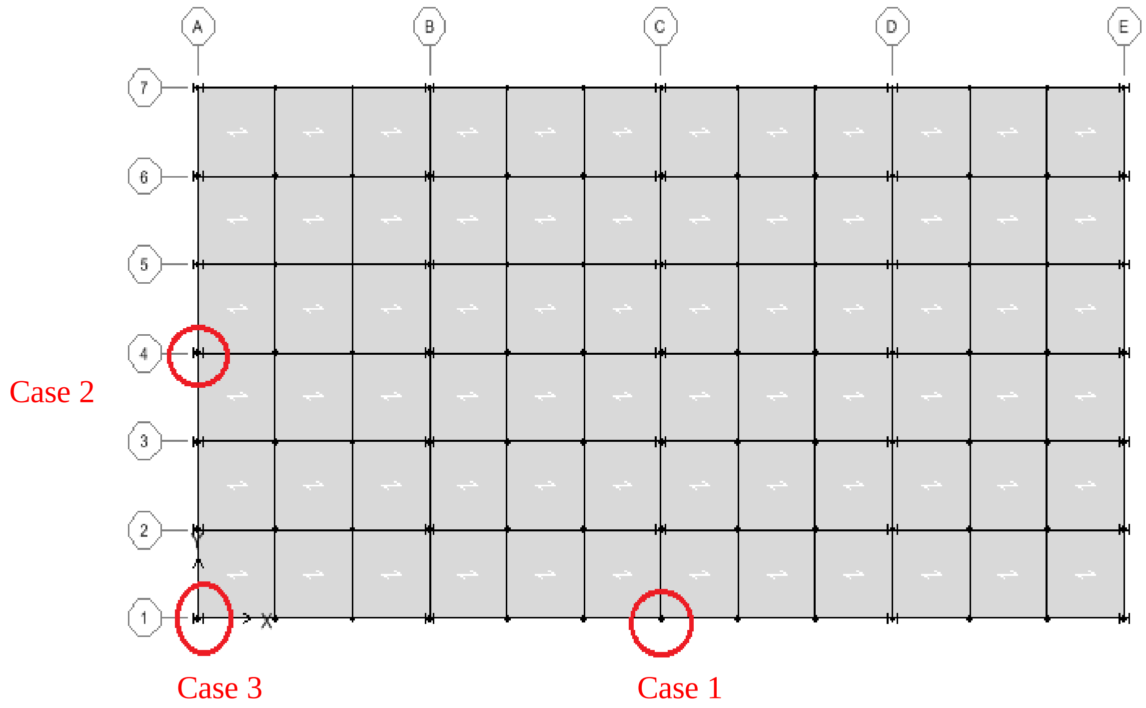

Figure 24: The locations of columns to be removed based on GSA guideline

4.3.2 Demand Capacity Ratio for The Buildings with 9 m Span Beams

Figure 25 shows the middle column removed due to GSA guideline and DCR's calculated and compared for each element of the building with normal I-beam and truss beam in this bay.

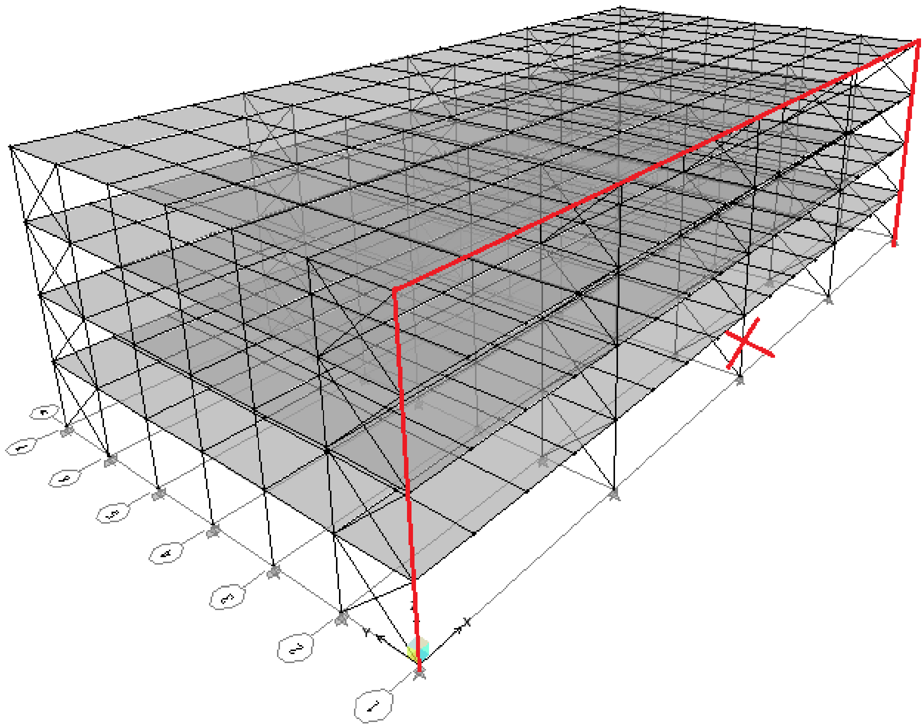

Figure 25: Remove column from the middle of long side of the building (9 m span)

Figures 26 and 27 indicate that none of the DCR's elements are more than 2.0 and shows that the potential of progressive collapse due to the removal of a column in the long side of the building is low. As the results show DCR values for normal I-beam have higher value than truss beam.

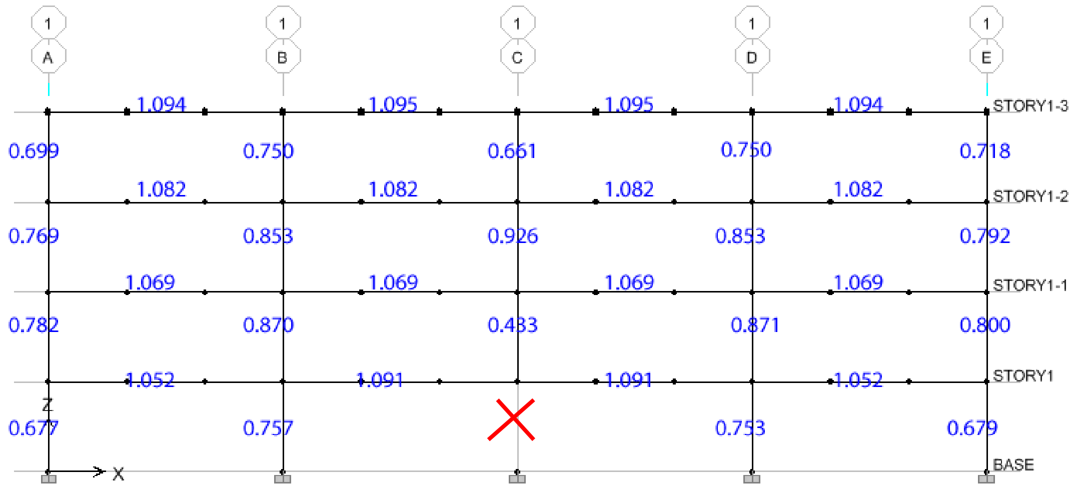

Figure 26: Demand Capacity Ratio (DCR) when the column on the longitudinal side of the structure is removed (9m span Normal I-beam)

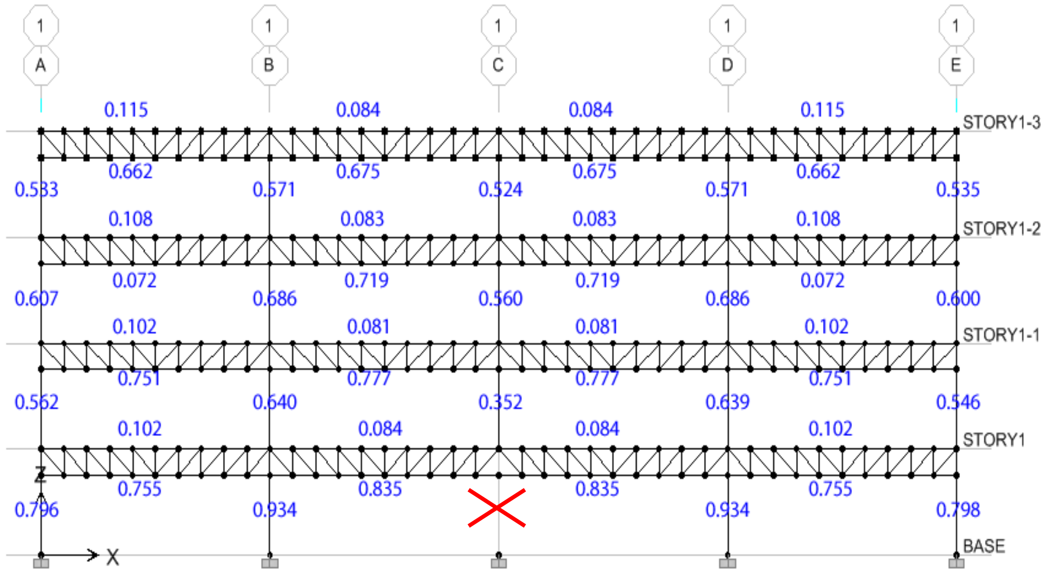

Figure 27: Demand Capacity Ratio (DCR) when the column on the longitudinal side of the structure is removed (9m span Truss beam)

Figure 28 indicates by removing the middle column of long side, the more load is distributed at behind bay. Figure 29 and 30 comparing the results of these two buildings.

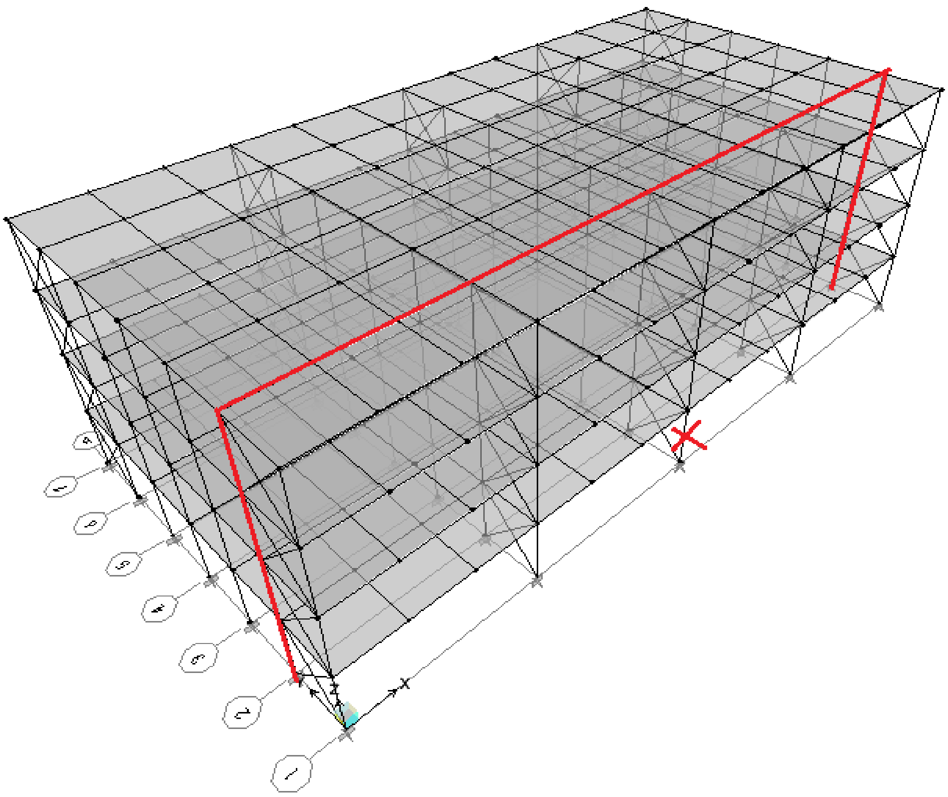

Figure 28: Remove column at long side and distribute load to the behind bay
(9 m span)

Figures 29 and 30 show the DCRs value of the building, after removing column on the long side of the structure. The results show that all of the DCR values for truss beam are less than DCR value of normal I-beam.

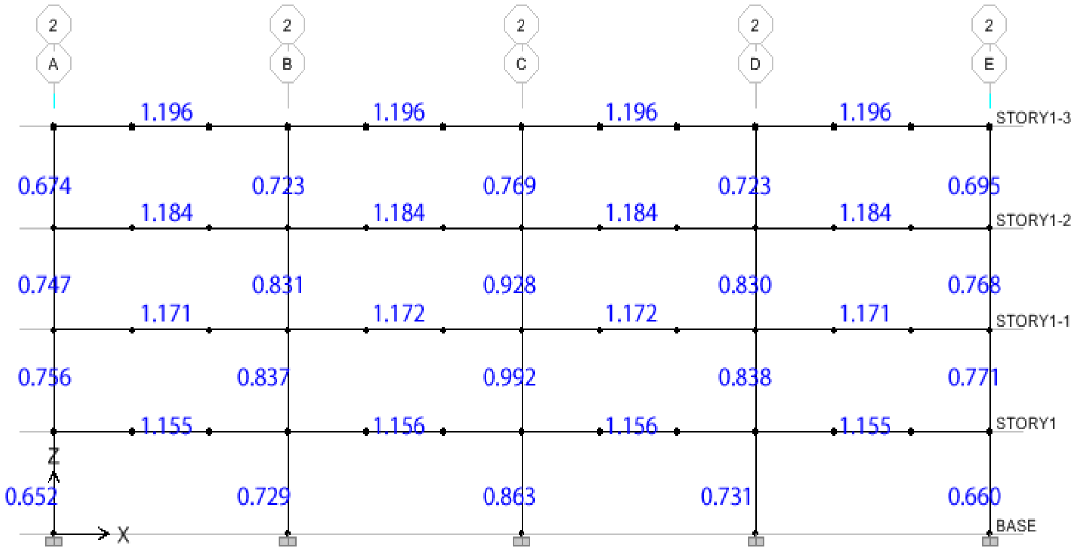

Figure 29: Demand Capacity Ratio (DCR) when the the external column on the longitudinal side of the structure is removed (9m span Normal I-beam)

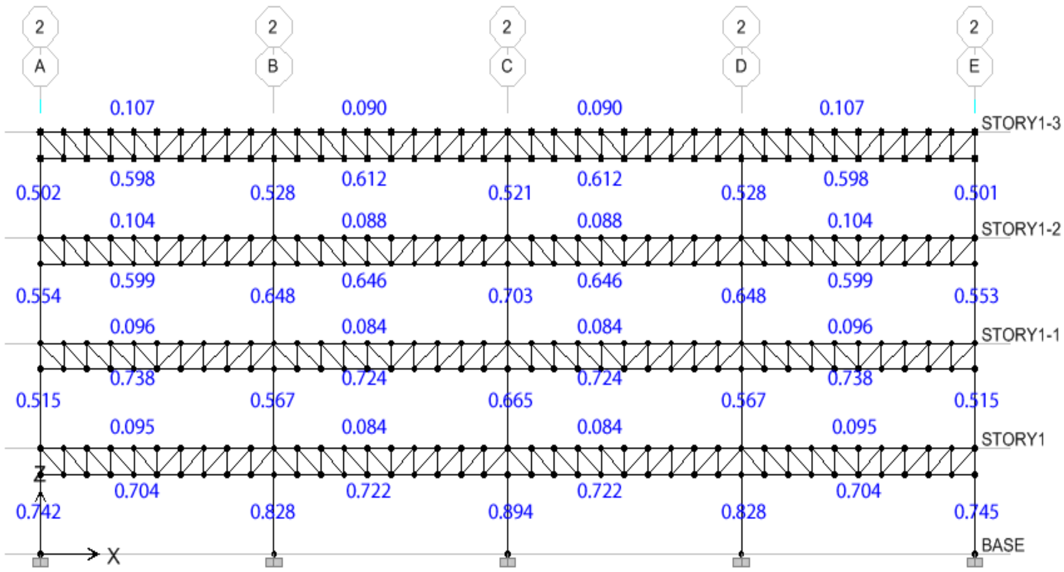

Figure 30: Demand Capacity Ratio (DCR) when an external column on the longitudinal side of the structure is eliminated (9m span Truss beam)

When the DCR values of the building with truss beams and normal I-beams are compared, the truss beam appears to be distributing more loads and the DCR values for the top and bottom chords are less than the DCR values of the normal I-beams in the other structure. Therefore, according to GSA guideline the building with a lower DCR value is safer when a column is suddenly removed due to an accidental impact or explosion.

In Figure 31, after removing the column at the middle of the short side, DCR value of this bay for both structures are compared.

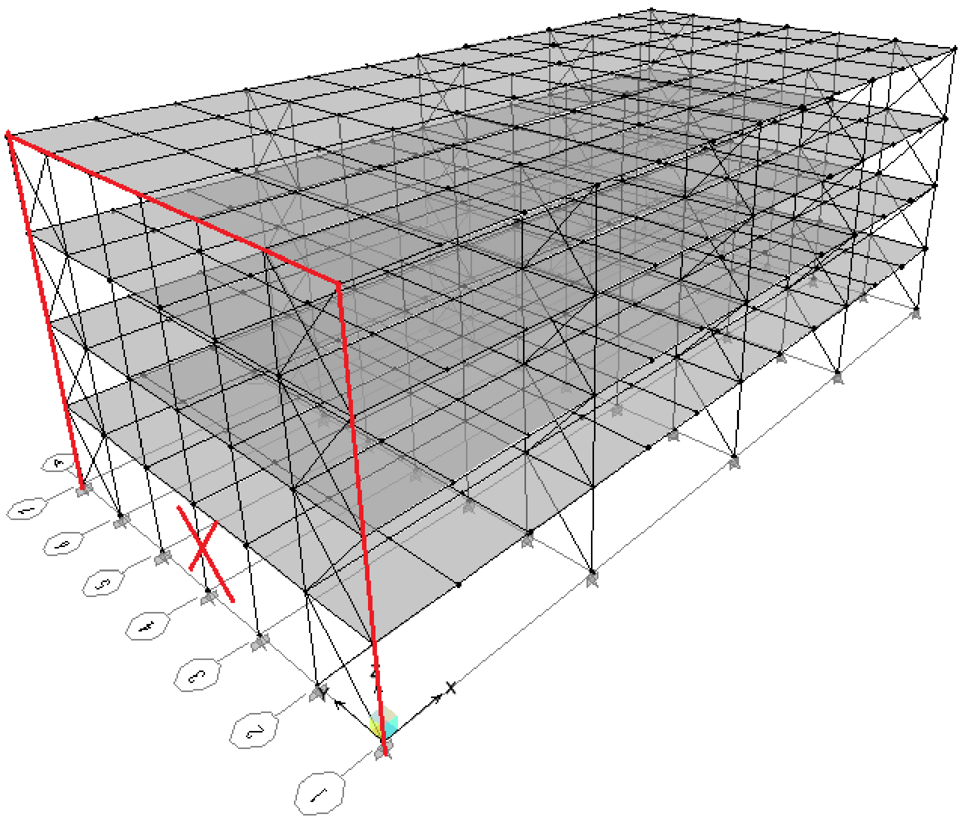

Figure 31: Removing a column on the short side of the structure (9 m span)

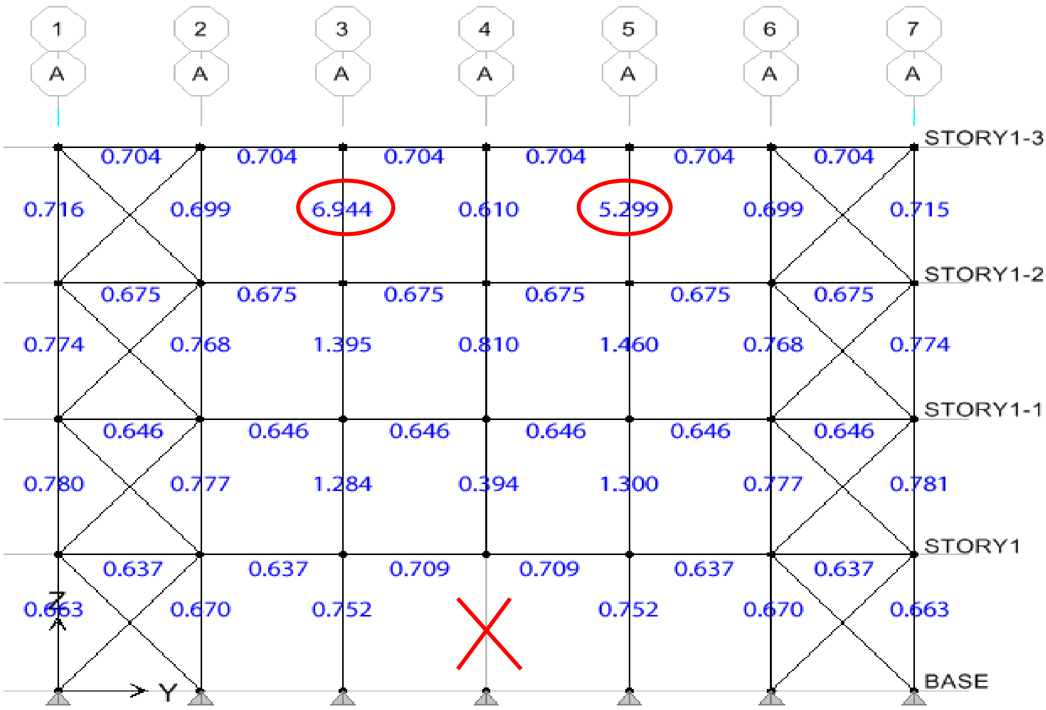

Figure 32: Demand Capacity Ratio (DCR) when a column is removed on the short side of the structure (9m span Normal I-beam)

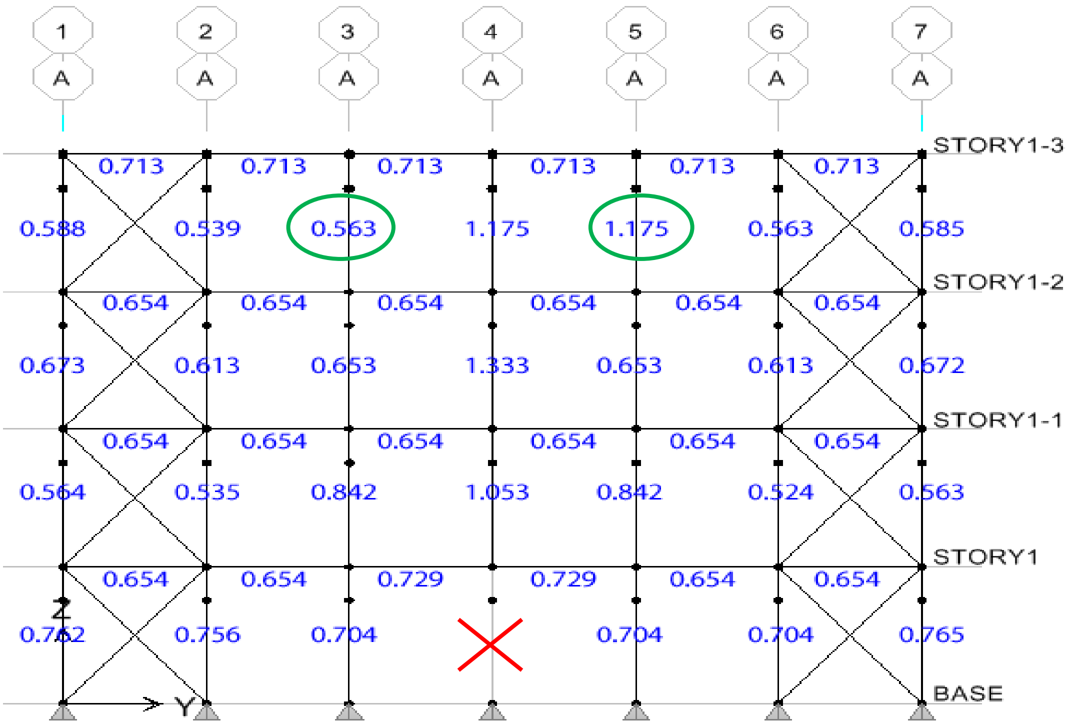

Figure 33: Demand Capacity Ratio (DCR) due to removal of the column on the middle short side of the structure (9m span Truss beam)

As can be seen in Figures 32 to 33 by removing the column in the short side of the building with normal I-beams and truss beams, some of the members achieved DCR more than the accepted limits. In Figure 32 the maximum DCR is 6.944 this high values of DCR indicates that the structures are more susceptible to progressive collapse.

The Figure 34 shows that if the column at the corner of these structures is removed due to an explosion or accident then loads will be distributed in direction 1 and A.

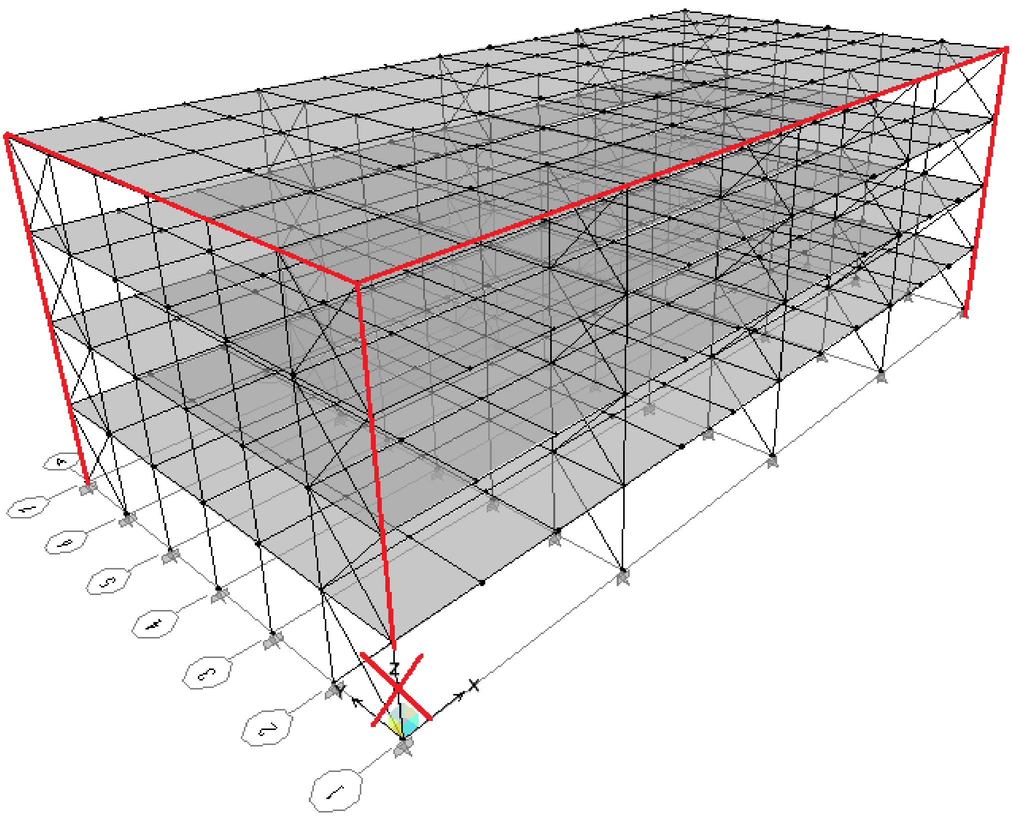

Figure 34: Removing a column at the corner of the structure (9 m span)

The maximum DCR value of the building with normal I-beams is 1.095 (Figure 35). Figure 36 shows the maximum DCR value of truss beam structure at the bottom chord of the truss which is closer to the removed column is 0.776.

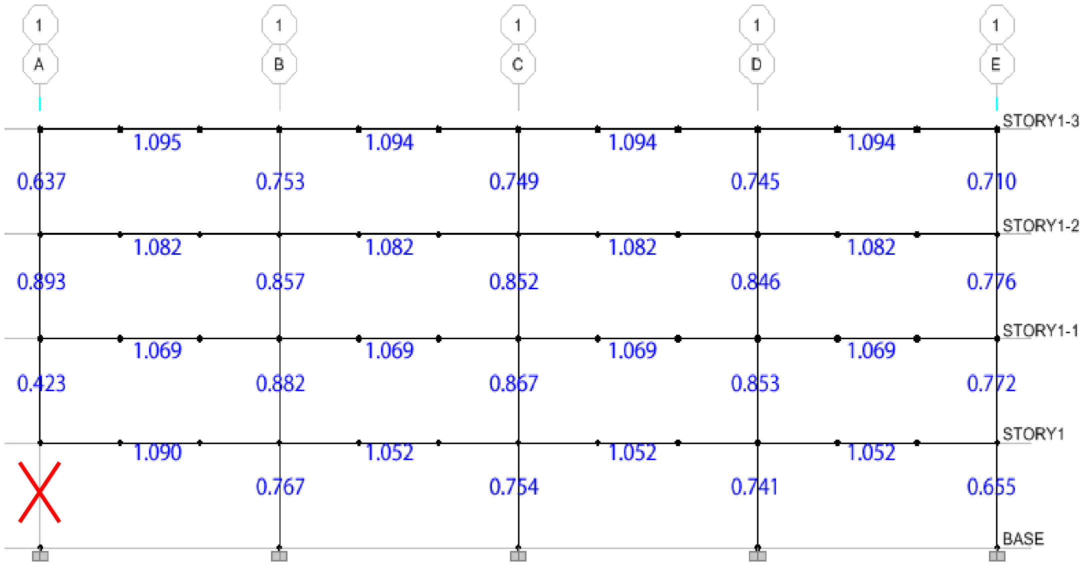

Figure 35: Demand Capacity Ratio (DCR) in the longitudinal side due to the elimination of the column at the corner of the structure (9 m span Normal I-beam)

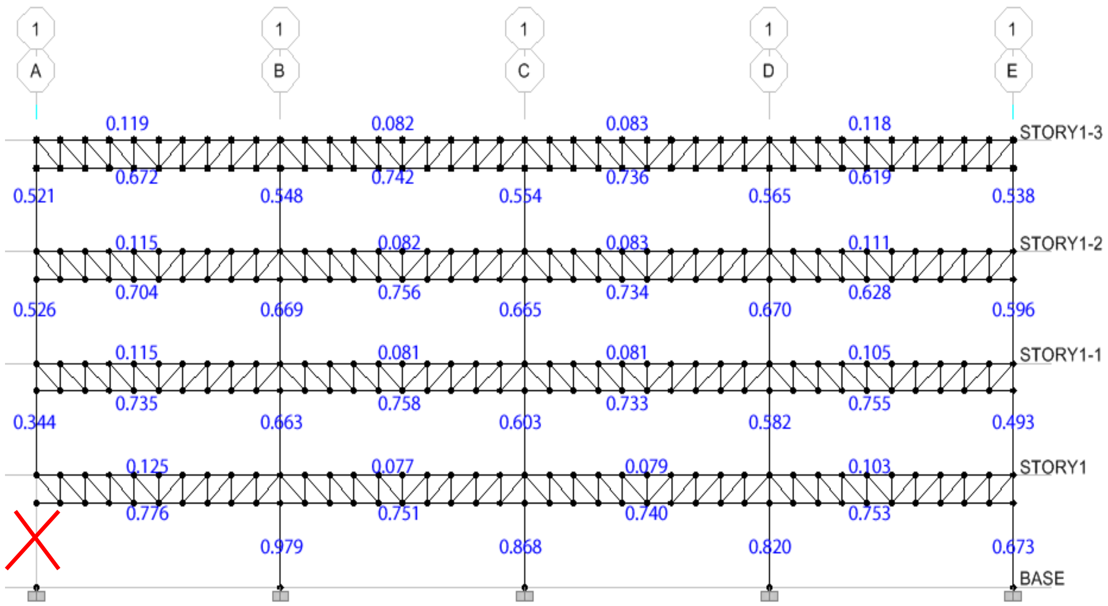

Figure 36: Demand Capacity Ratio (DCR) in the longitudinal side due to elimination of the column at the corner of the structure (9 m span Truss beam)

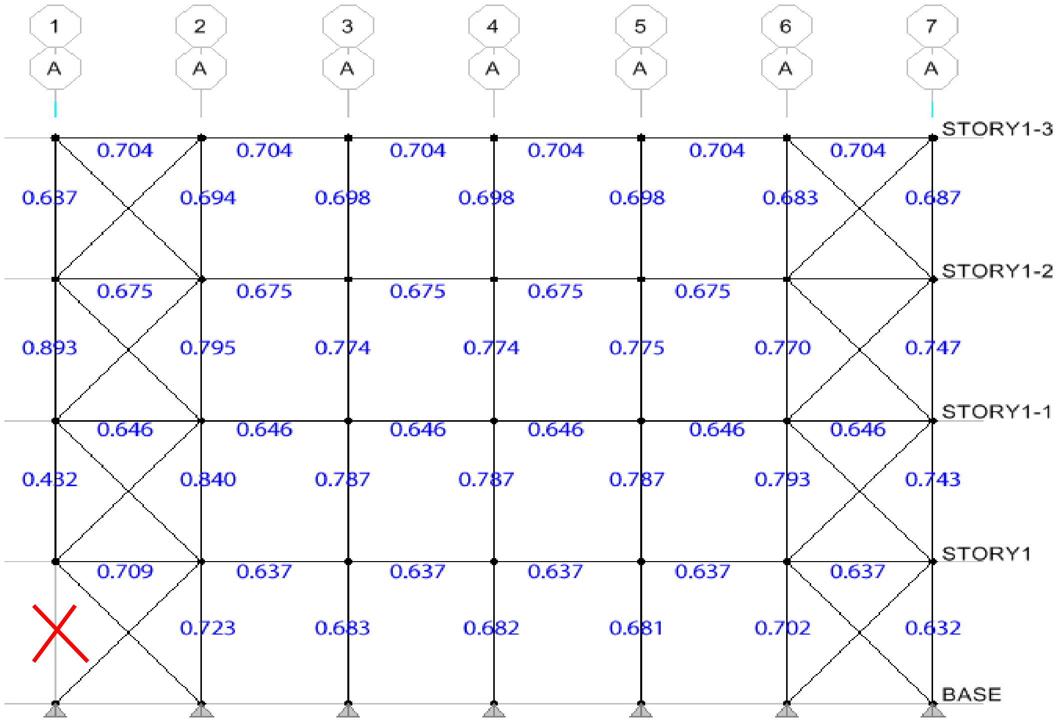

Figure 37: Demand Capacity Ratio (DCR) in the short side due to the elimination of the column at the corner of the structure (9 m Normal I-beam)

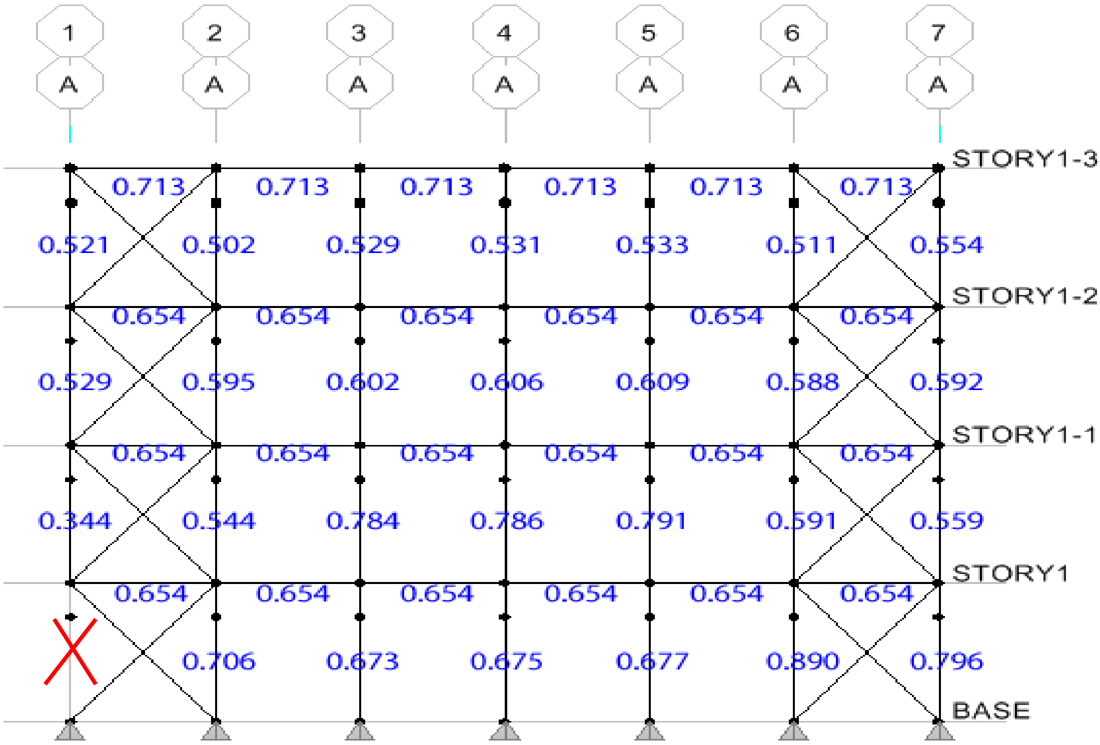

Figure 38: Demand Capacity Ratio (DCR) in short side due to the elimination of the column at the corner of the structure (9 m span truss beam)

The results show, in the short side both structures have roughly same values (Figures 37 and 38), although in the long side maximum DCR values belong to the structure with normal I-beam (Figure 35).

4.4 Modeling of the Building with 12 m Span Beams

4.4.1 Modeling of Buildings with Normal I-Beams And Truss Beams

This steel building has modeled in ETABS software [version 9.7.4] and constructed by brace frame in both direction based on the European Standard Code. The immediate removal of columns are analyzed based on GSA guideline and the potential of progressive collapse was evaluated. Figure 24 shows the removal of column in each sequence.

4.4.2 Demand Capacity Ratio for The Buildings with 12 m Span Beams

Due to GSA guideline by removing the column on each side of the structure, the DCR values will be obtained. The DCR is calculated for each element of the buildings which are using normal I-beams and truss beams as part of their floor structure.

The middle column removed due to GSA guideline and DCR's calculated and compared for each element of the building with normal I-beam and truss beam for 12 m span (Figure 39).

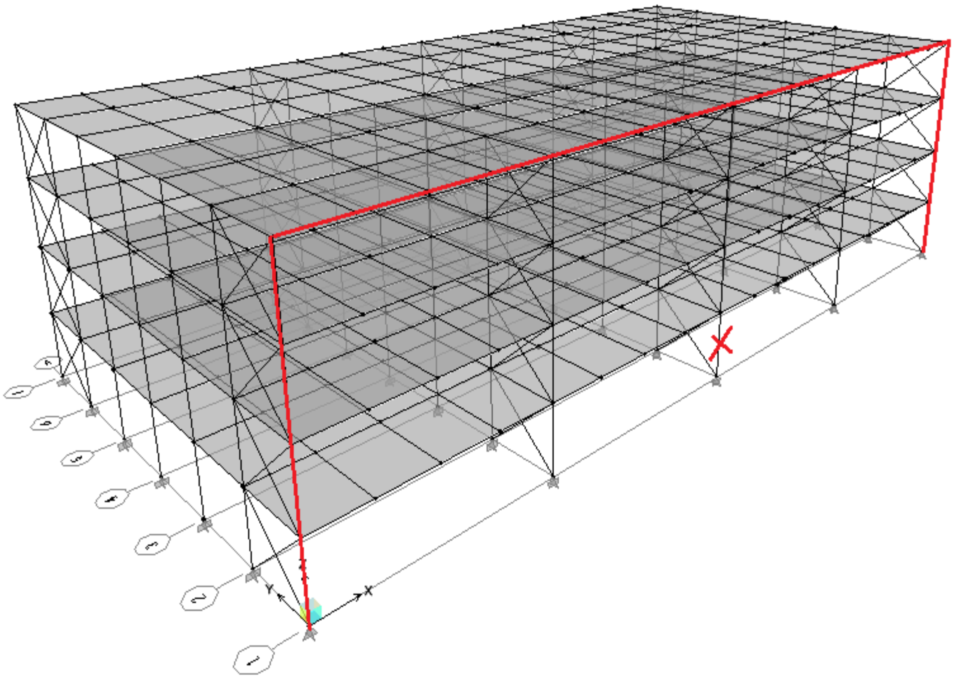

Figure 39: Remove column from the middle of long side of the building (12 m span)

Figure 40 and 41 show the DCR value of each structures. In normal I-beam all the beam members have same value, although in truss beam by becoming far from the removed column DCR value decrease.

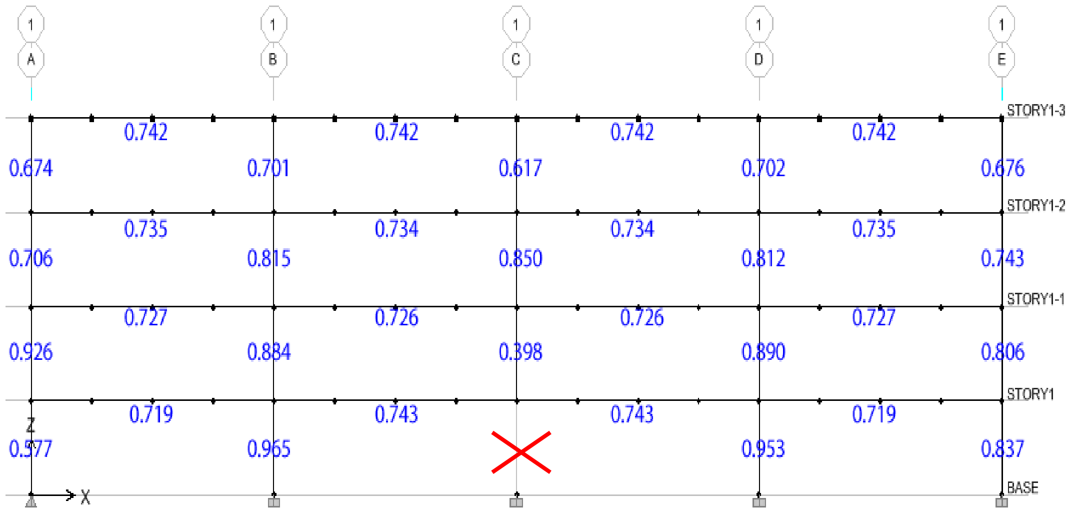

Figure 40: Demand Capacity Ratio (DCR) due to the elimination of the column on the longside of the structure (12m span Normal I-beam)

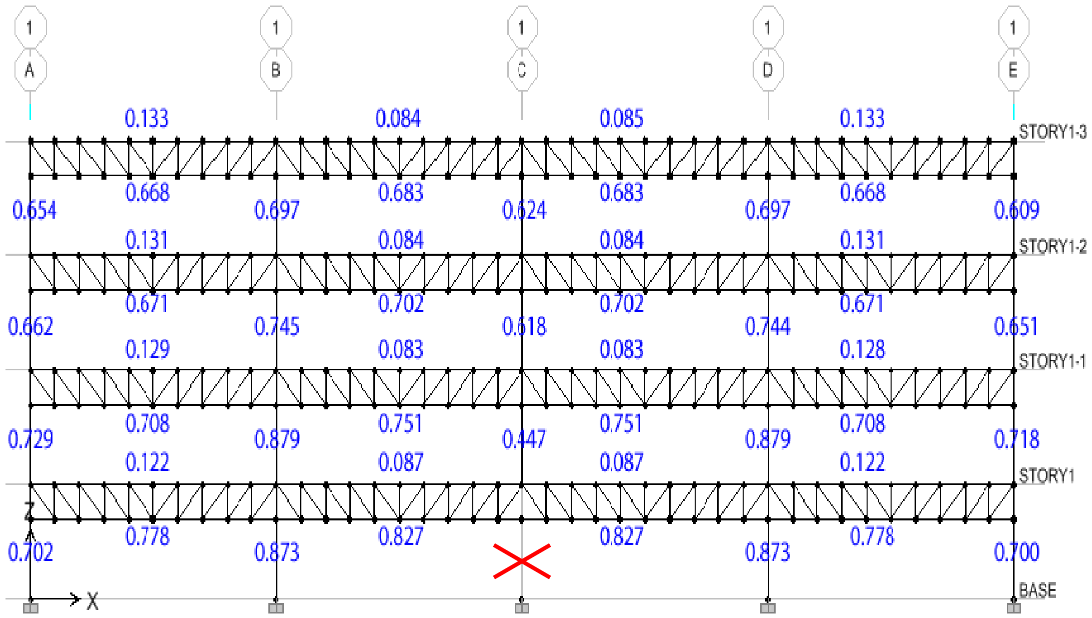

Figure 41: Demand Capacity Ratio (DCR) when the column on the longitudinal side of the structure (12m span truss beam) is eliminated.

By sudden removal of the column at the long side, as in Figure 42 shows, more load distributed to the bay under consideration.

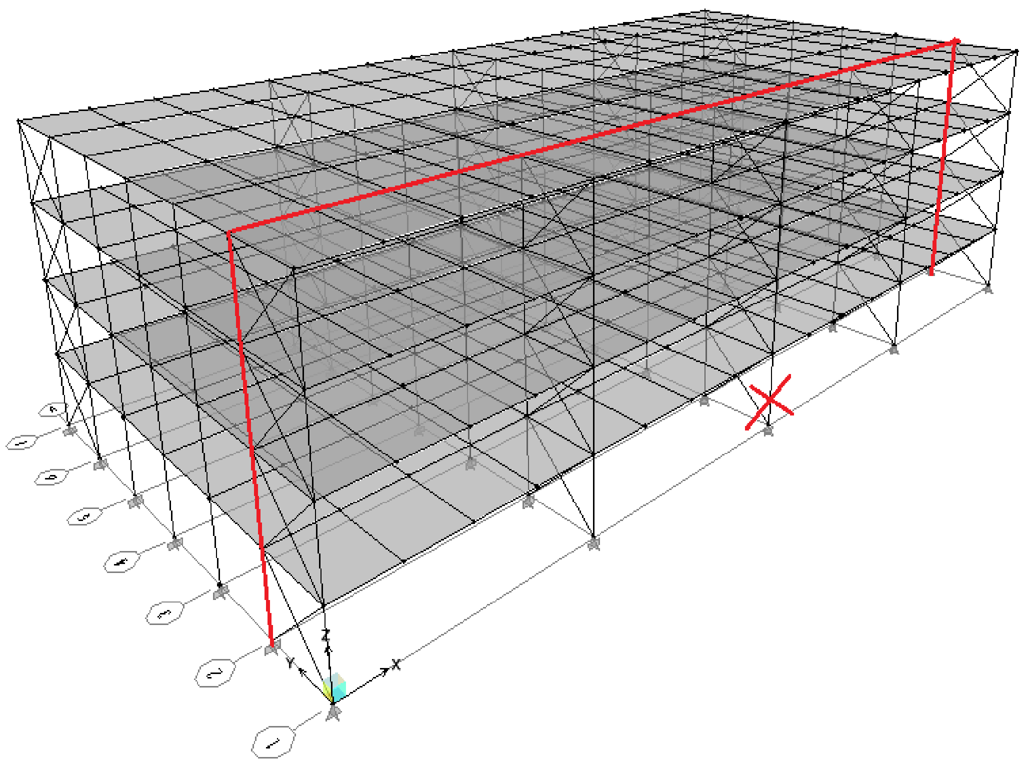

Figure 42: Remove column at long side and distribute load to the behind bay (12 m span)

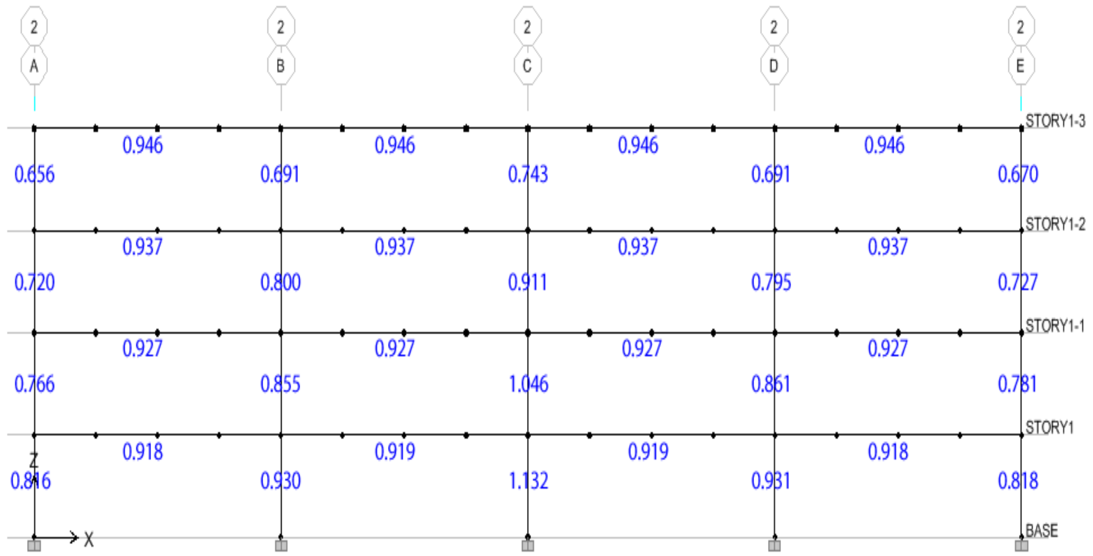

Figure 43: Demand Capacity Ratio (DCR) due to elimination of the external column on the longitudinal side of the structure (12 m span Normal I-beam).

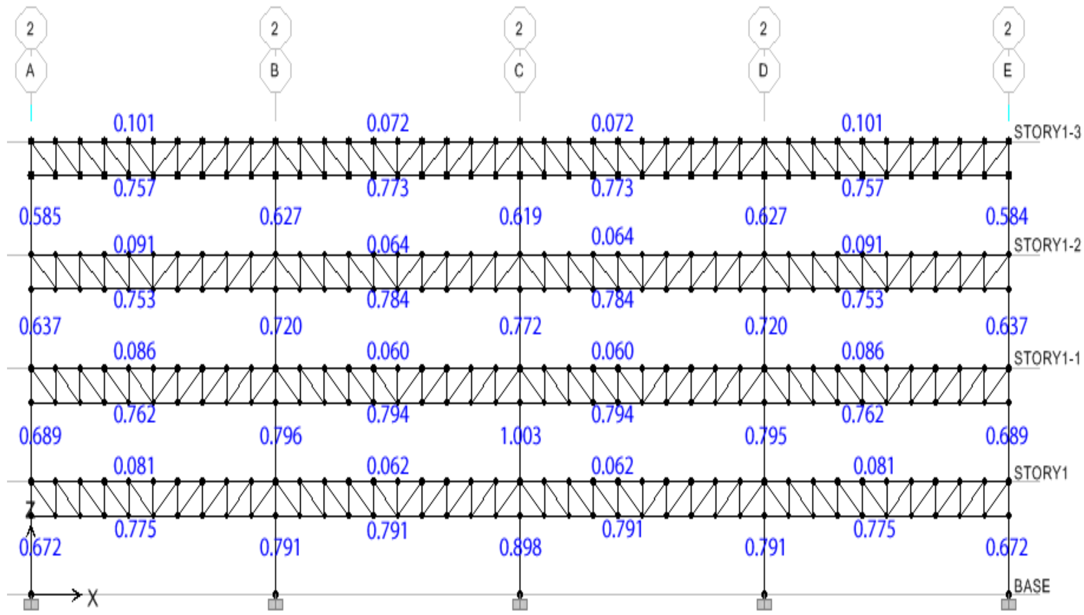

Figure 44: Demand Capacity Ratio (DCR) when an external column on the longitudinal side of the structure (12 m span truss beam) is eliminated.

Figures 43 and 44 show maximum DCR values due to removal of the columns on the long side of the structure for normal I-beam and bottom chord of truss beam are 0.946 and 0.721 respectively. The column's DCR calculated is 1.132 for building with normal I-beam and 1.003 for building with truss beam. Therefore, it indicates that the truss beam have a better behavior than norml I-beam.

In Figure 45, the results of the sudden removal of the column at the middle of the short side are given as, DCR values for this bay and these values are calculated and compared for both of the buildings.

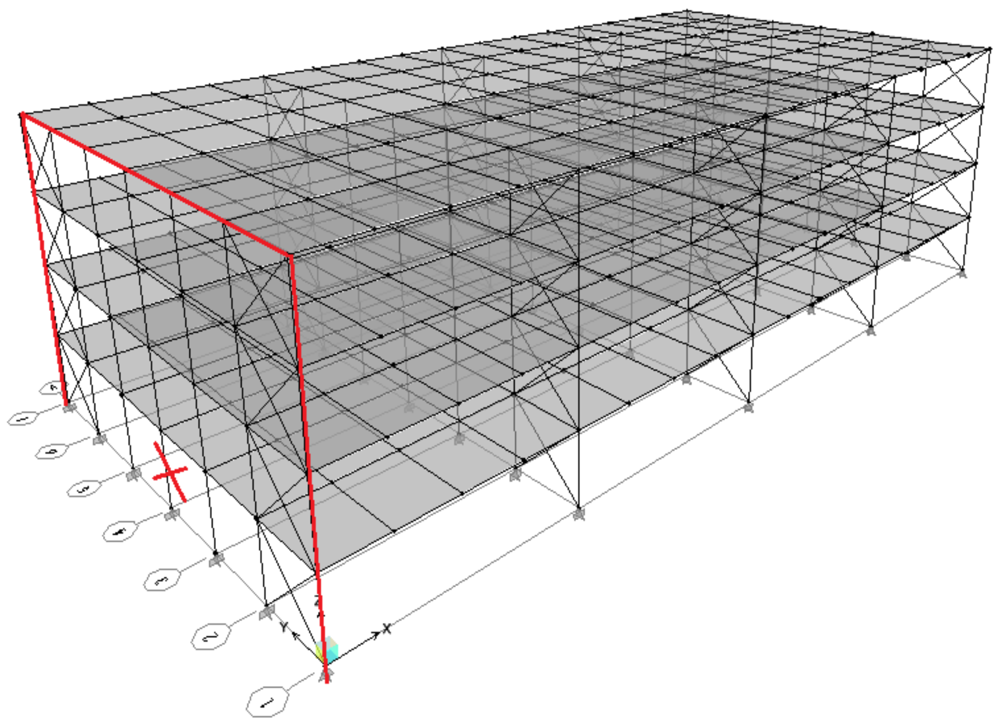

Figure 45: Removing a column on the short side of the structure (12 m span)

As the Figure 46 and 47 show the beam in both structure have roughly same DCR value. The columns in the structure with normal I-beam have higher DCR value than the other one.

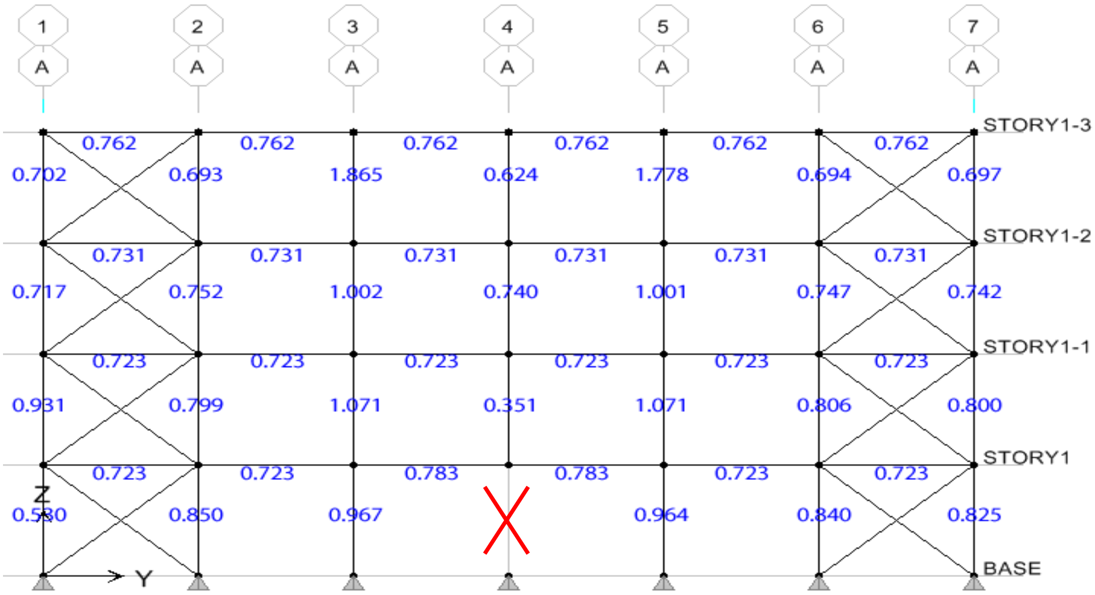

Figure 46: Demand Capacity Ratio (DCR) when a column is removed on the short side of the structure (12m span Normal I-beam)

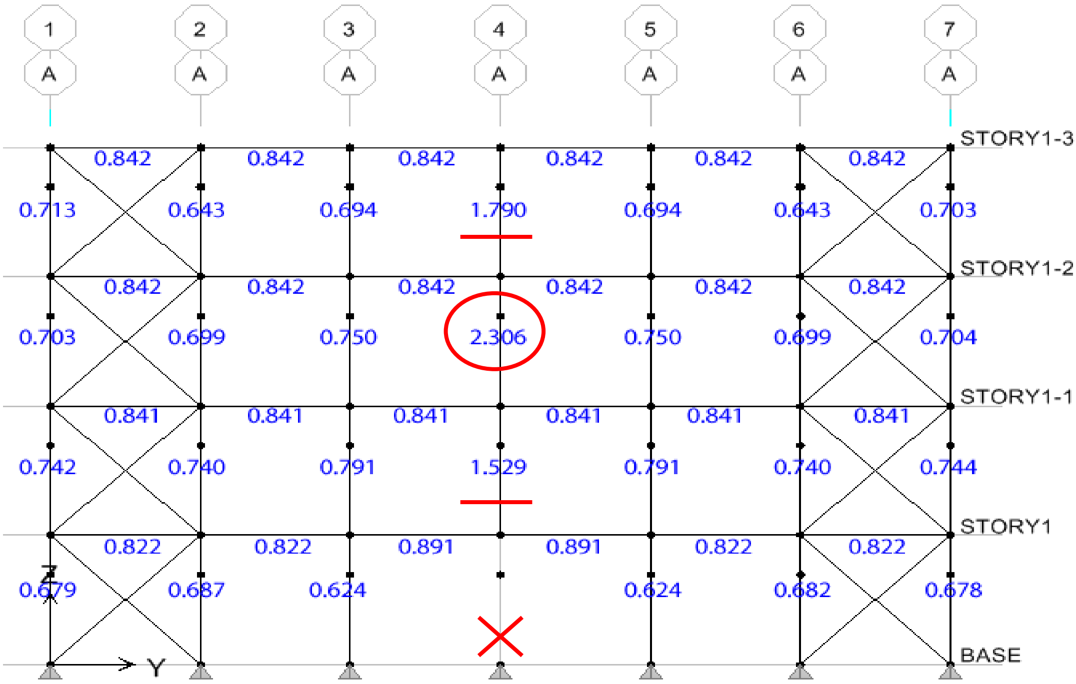

Figure 47: Demand Capacity Ratio (DCR) when a column is removed on the short side of the structure (12m span truss beam)

Figure 47 shows DCR values (2.306) higher than the stated limit. Therefore, this column could not resist against progressive collapse. These values were observed in columns rather than beams. Therefore, by using braces in the middle, the DCR value increases slightly for all the columns. Max DCR : 2.306 reduces to 0.686 (Figure 48).

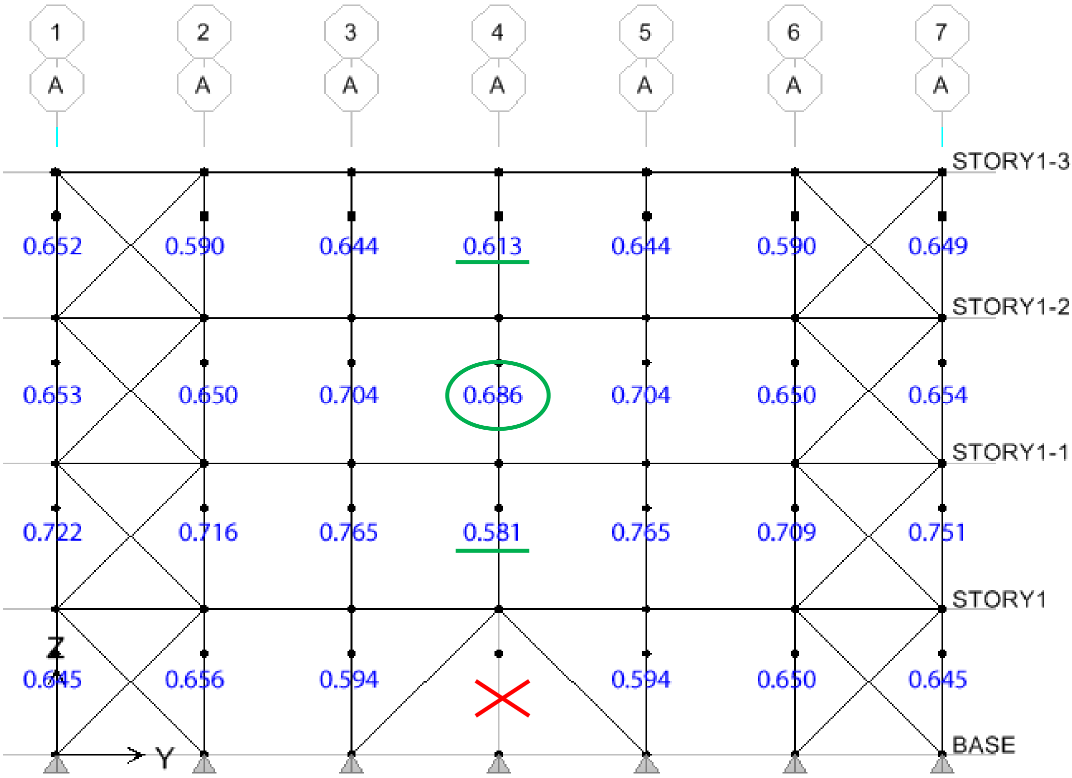

Figure 48: Demand Capacity Ratio (DCR) when a column is eliminated at the short side. A bracing system is introduced at the ground floor level of the structure as part of rehabilitating the building (12m span truss beam)

After removing a column at the corner of the building (Figure 49), the DCR values for all the beams and columns in normal I-beam and truss beam structures are calculated.

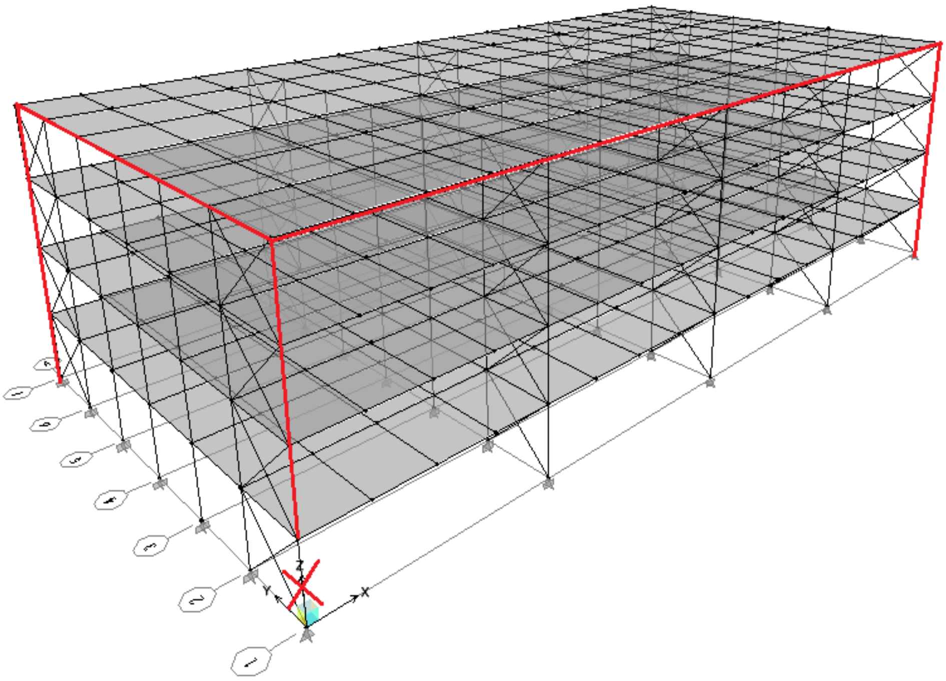

Figure 49: Removing a column at the corner of the structure (12 m span)

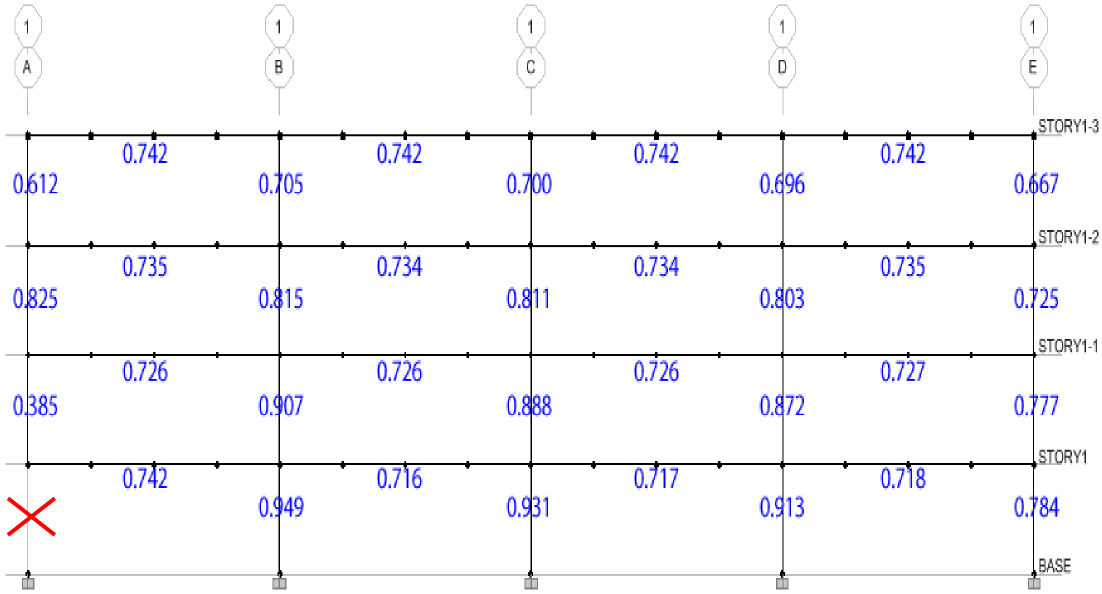

Figure 50: Demand Capacity Ratio (DCR) in the longitudinal side due to the eliminate the column at the corner of the structure (12m span Normal I-beam)

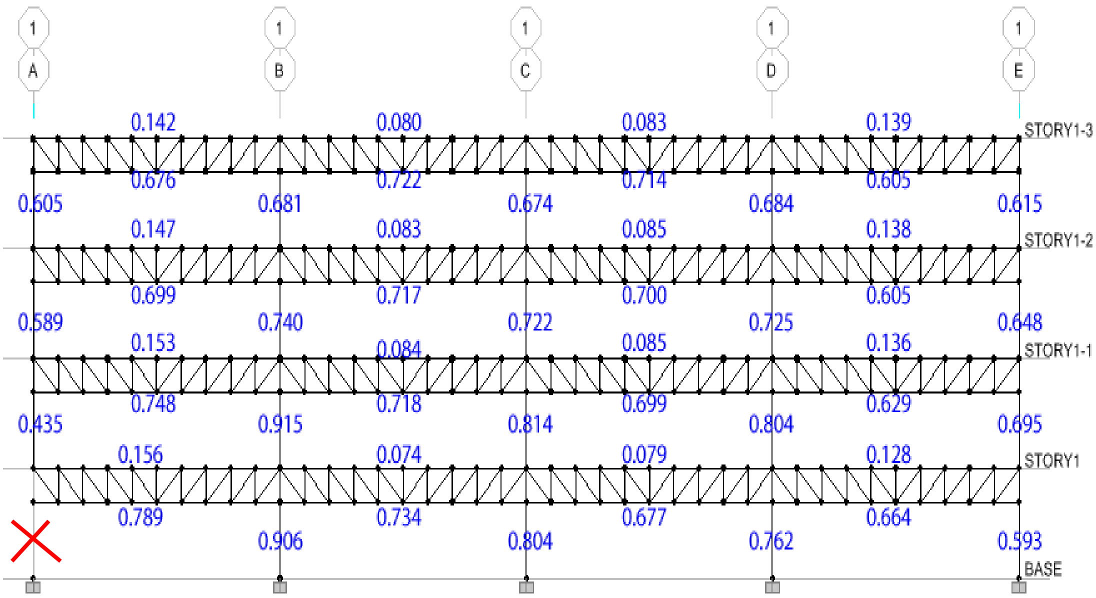

Figure 51: Demand Capacity Ratio (DCR) on the longitudinal side, when a column is removed at the corner of the structure (12m span truss beam)

According to Figures 50 and 51 when the column in the corner of the building is removed, for the structure with normal I-beams all DCRs are less than 0.949 and in the structure with truss beam all the DCR's are less than 0.915. In direction x, DCR values show that truss beam have a better behavior than normal I-beam. In the short side around removed column, the columns in truss beam building have lower value than the normal I-beam (Figure 52 to 53).

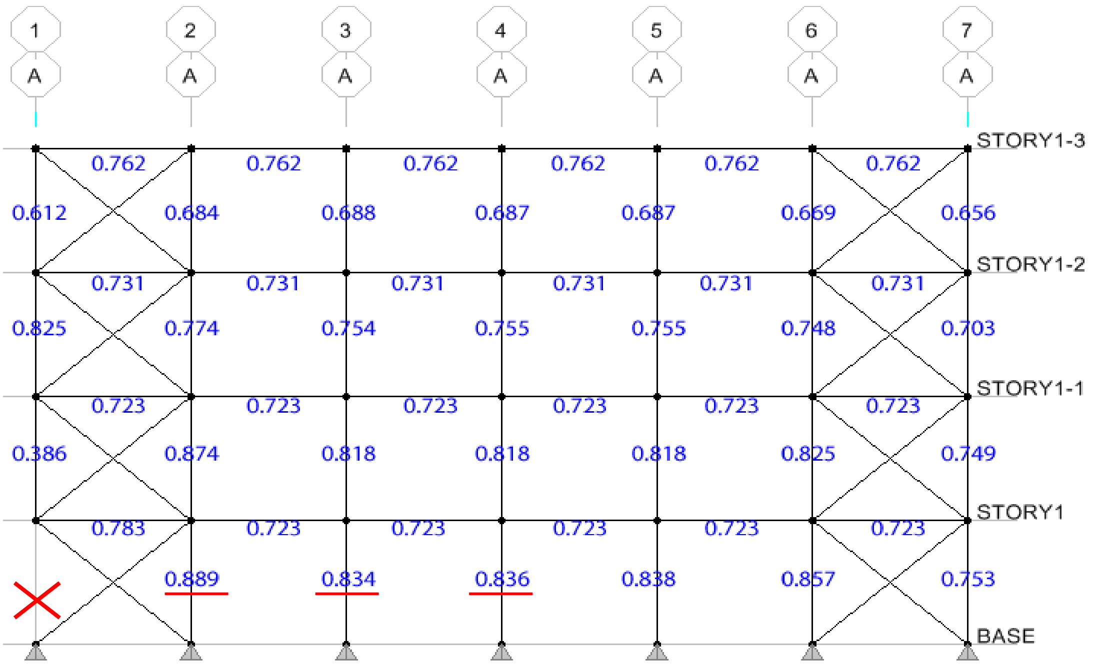

Figure 52: Demand Capacity Ratio (DCR) in the transverse side due to eliminate the column at the corner of the structure (12m span Normal I-beam)

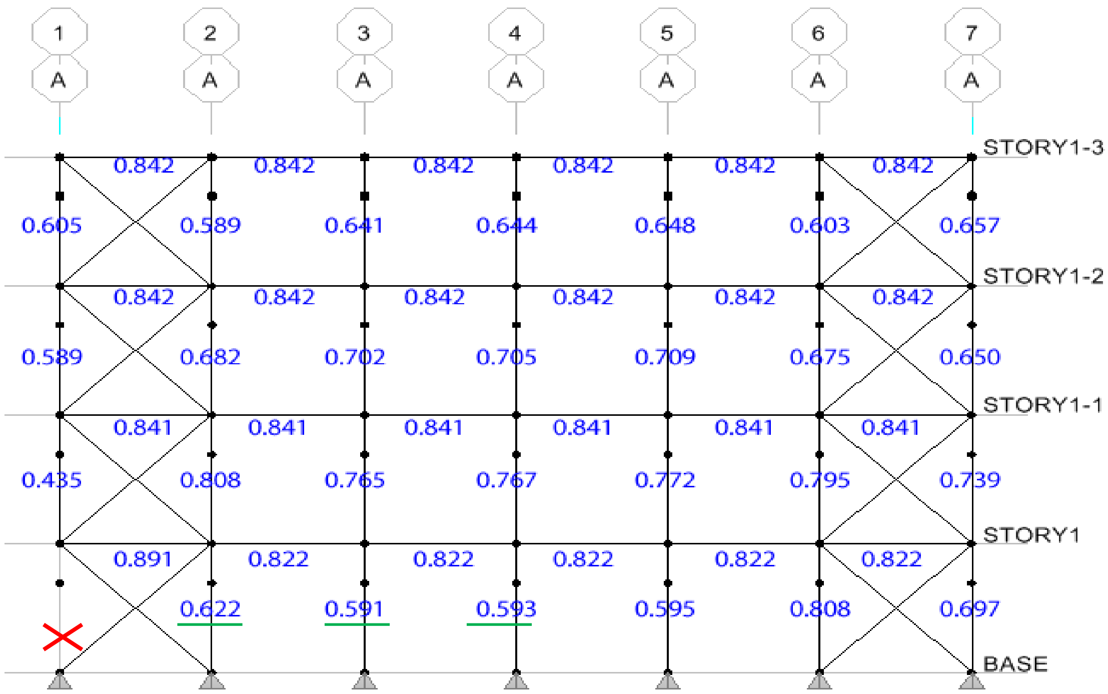

Figure 53: Demand Capacity Ratio (DCR) on the transverse side, when a column is removed at the corner of the structure (12m span truss beam)

4.5 Modeling of the Building with 15 m Span Beams

4.5.1 Modeling of Buildings with Normal I-Beams And Truss Beams

Figure 24 shows the removal of a column in each series. This steel building has modeled in ETABS software with brace frame system based on the European Standard Code. DCR value will be evaluated by immediate removing columns based on GSA guideline.

4.5.2 Demand Capacity Ratio for The Buildings with 15 m Span Beams

By removing the column of each side of the structure, the DCR value will be evaluated for each element of buildings. Normal I-beams and truss beams were used for the floors.

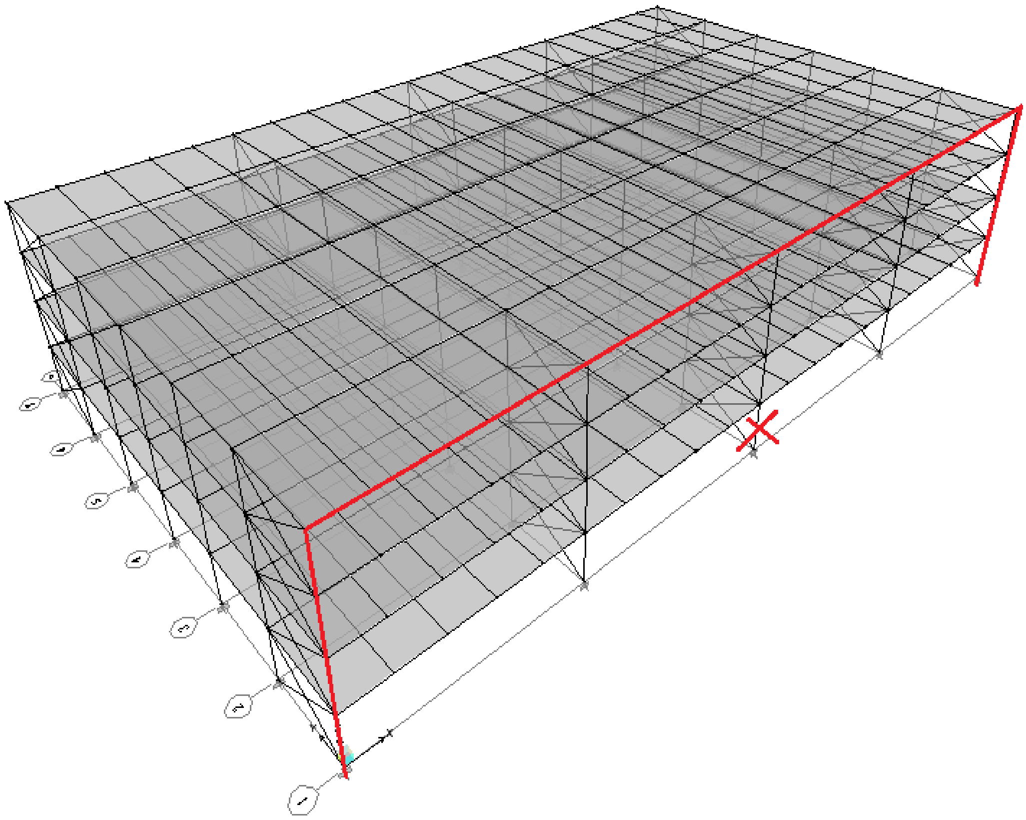

Figure 54: Remove column from the middle of long side of the building (15 m span)

By removing the column at the long side of the structure (Figure 54) the DCR value are shown in Figure 55 and 56.

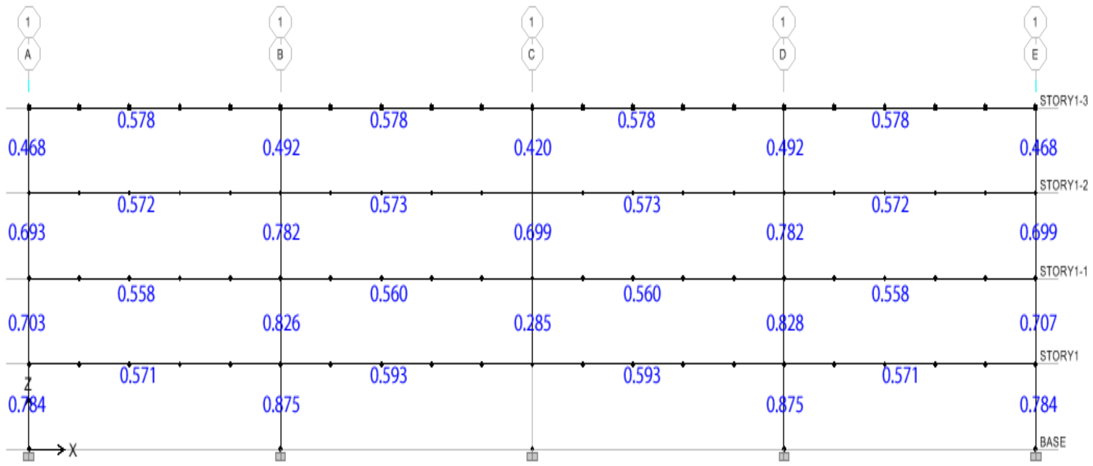

Figure 55: Demand Capacity Ratio (DCR) due to the elimination of the column on the long side of the structure (15 m span Normal I-beam)

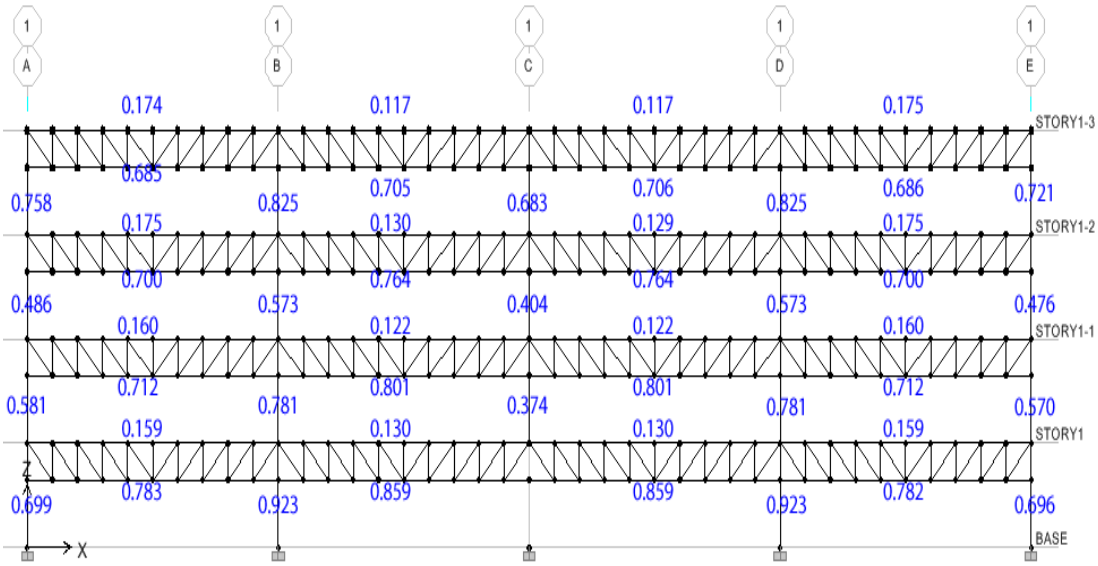

Figure 56: Demand Capacity Ratio (DCR) due to elimination of the column on the longitudinal side of the structure (15 m span Truss beam)

By comparing the DCR values of these two structures (Figure 55 and 56) , indicates in the long side they have roughly the same value in long direction.

Figure 57 shows by removing column in long side more loads are distributed in behind span and in Figure 58 to 59 the results are compared.

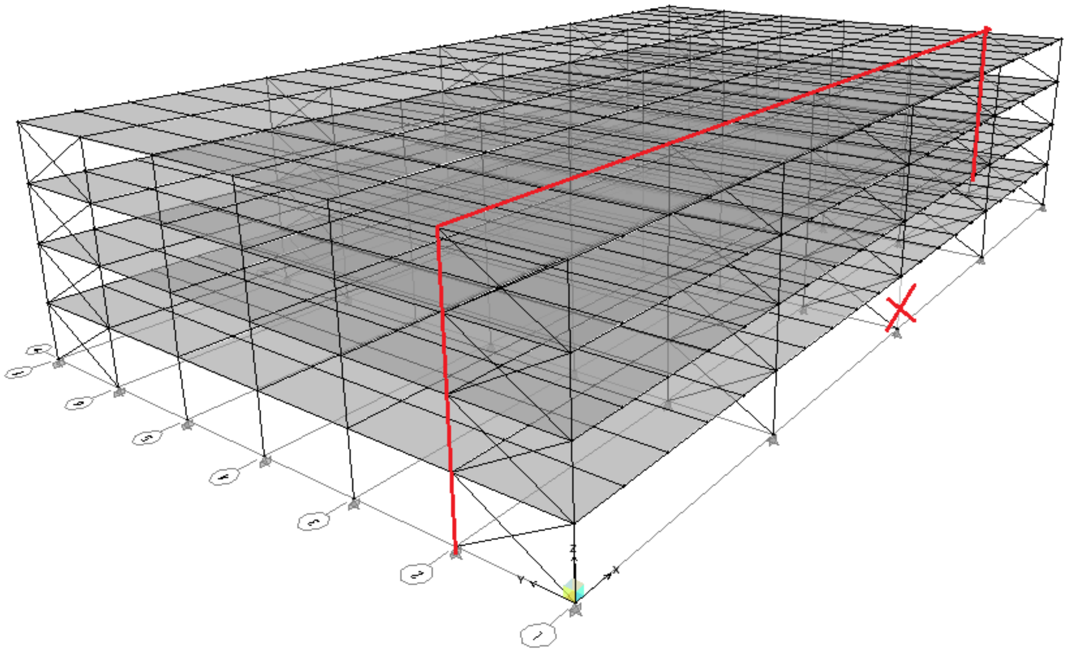

Figure 57: Remove column at long side and distribute load to the behind bay (15 m span)

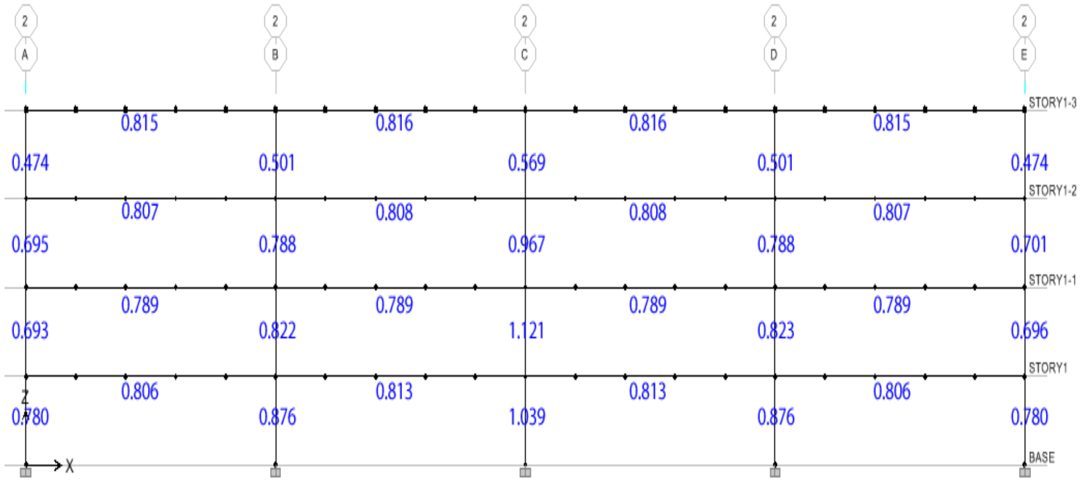

Figure 58: Demand Capacity Ratio (DCR) due to the elimination of the external column on the longitudinal side of the structure (15 m span Normal I-beam).

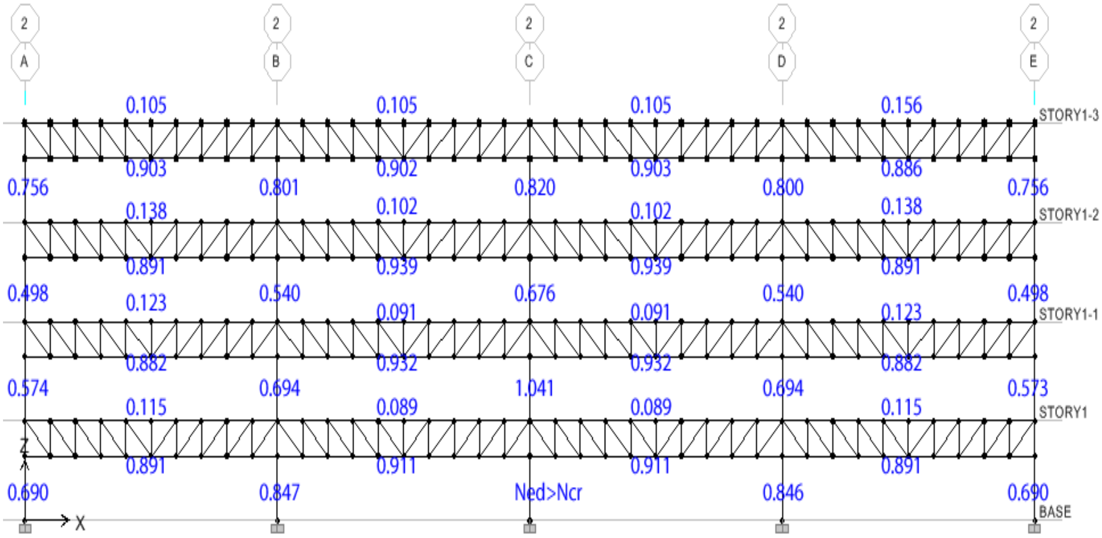

Figure 59: Demand Capacity Ratio (DCR) when an external column on the longitudinal side of the structure (15 m span Truss beam) is eliminated.

By removing the column at the long side of the structure, the DCR values were be evaluated for each element of the buildings. The DCR values in the long direction for both building are found to be around the same value. For the columns in such a condition the progressive collapse is inevitable (Figure 58 and 59).

N^{ed}: Design normal force

N^{cr}: Elastic flexural buckling force (Eurocode 1)

When $N^{ed}>N^{cr}$, columns could not resist any more axial force. These overloads are created by the accidental removal of a column.

By removing the column in the short side of the structure due to impact or explosion the DCR value of each buildings calculate (Figure 60).

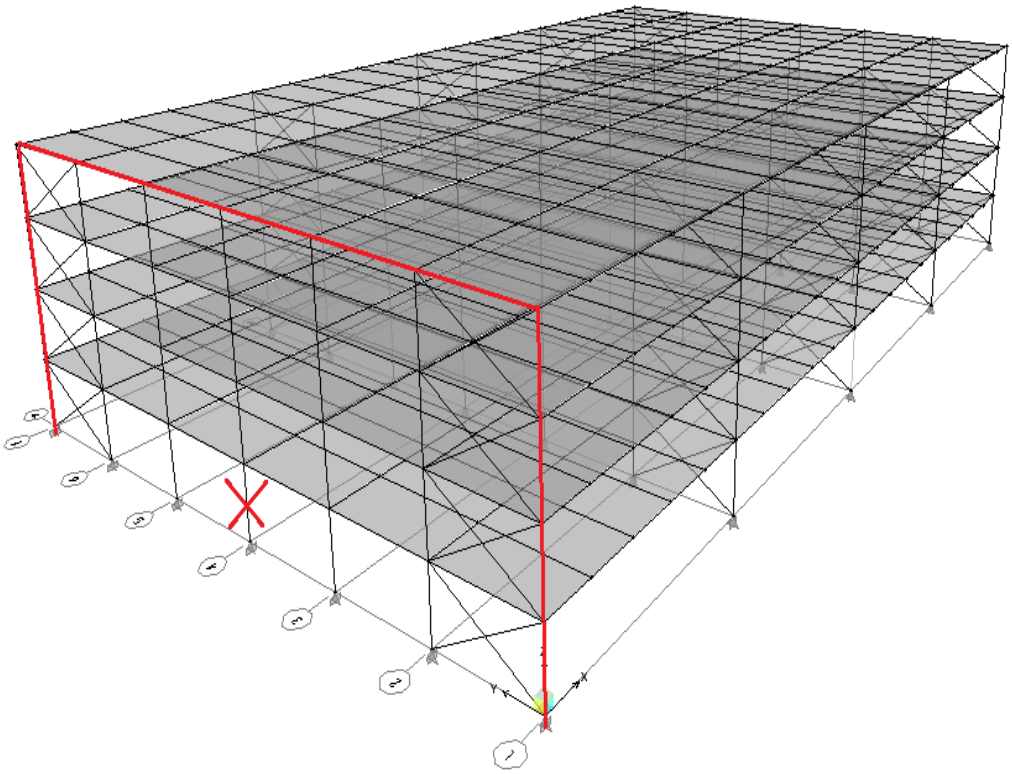

Figure 60: Removing a column on the short side of the structure (15 m span)

If the middle column in the short side of the structure is removed, the DCRs values for the building with normal I-beam is 5.814 and with truss beam is 45.003 (Figure 61 and 62). These DCRs show that the potential of the progressive collapse of both structures is high. Therefore, by using vertical brace in the middle of the short span, the DCR values were reduced to an acceptable level and the structure could resist against progressive collapse. In Figure 62 maximum DCR was 45.003 and by using vertical braces this value changed to 0.529 (Figure 63).

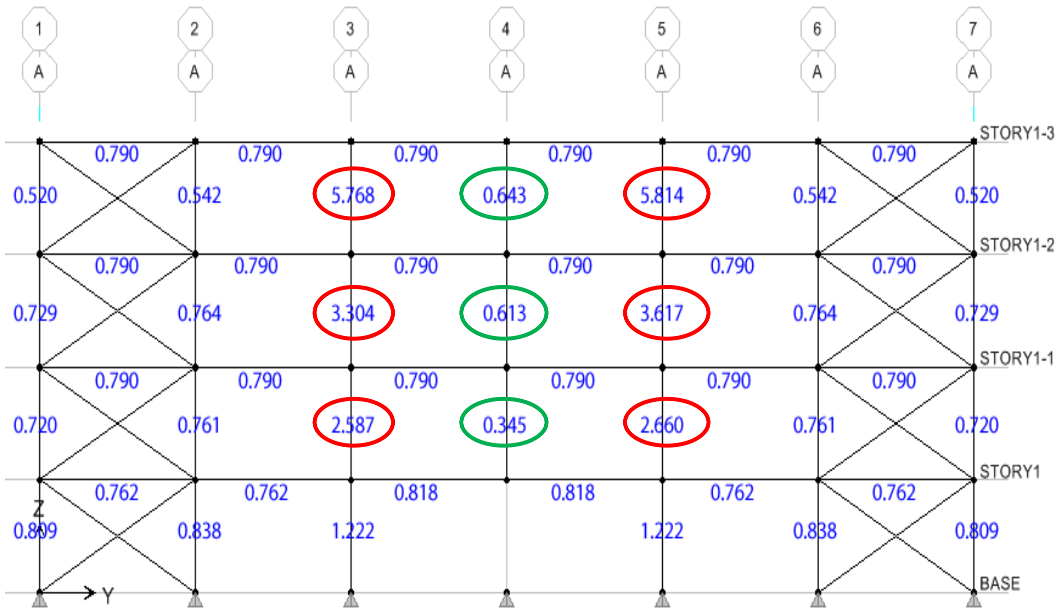

Figure 61: Demand Capacity Ratio (DCR) when a column is removed at the short side of the structure (15 m span Normal I-beam)

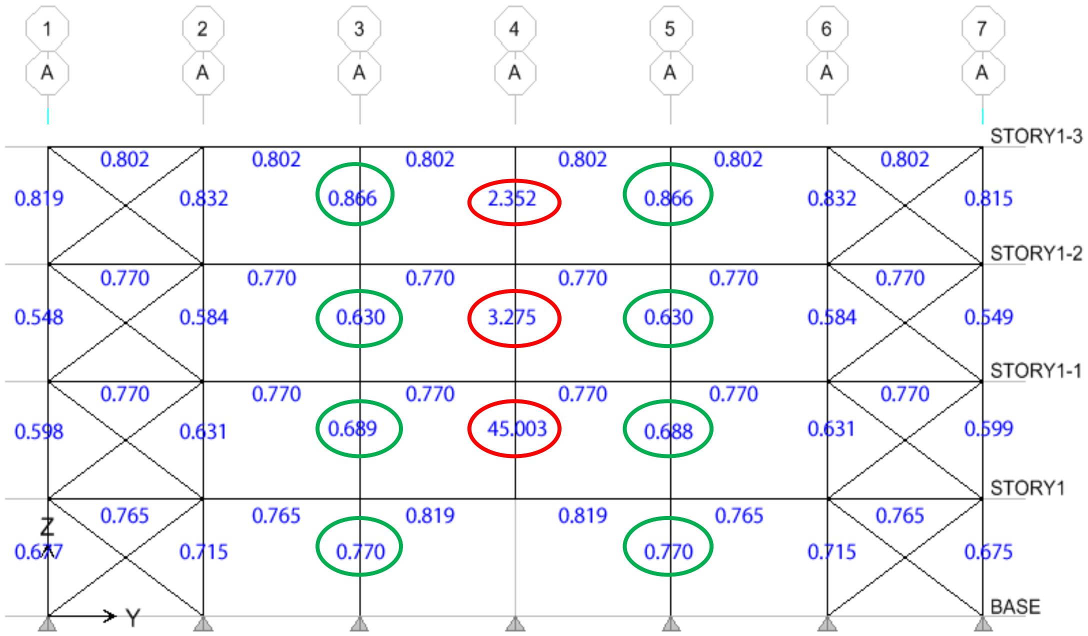

Figure 62: Demand Capacity Ratio (DCR) due to removal of the columnon the middle short side of the structure (15 m span truss beam)

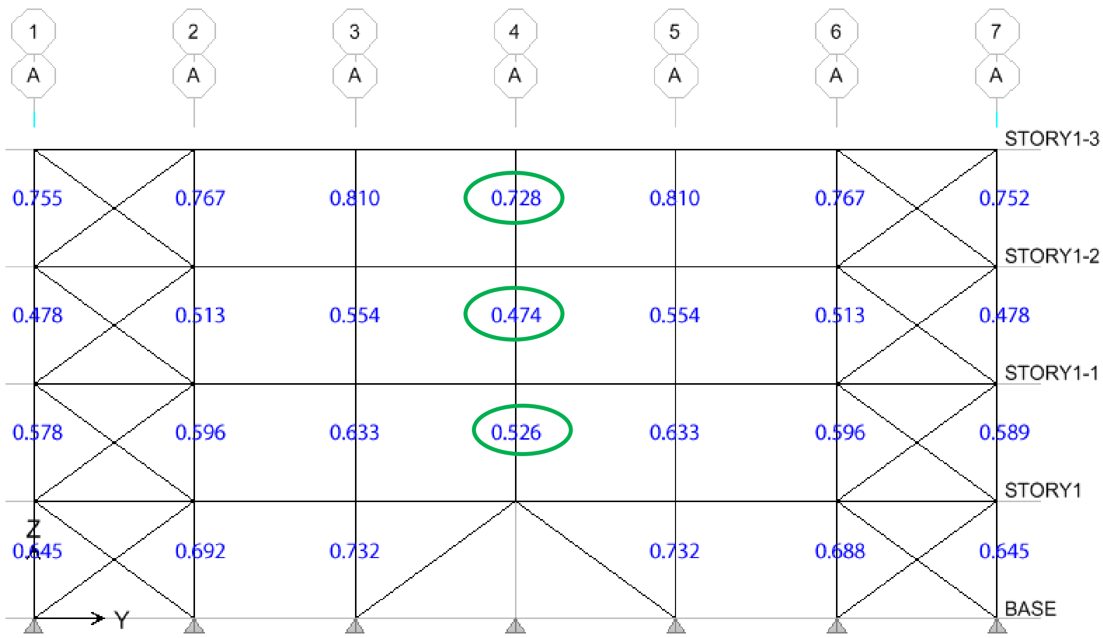

Figure 63: Rehabilitating the building and reducing Demand Capacity Ratio (DCR) by using brace system at the ground floor level of the structure (15 m span truss beam)

Figure 64 shows, sudden removal of the corner column of the structure and then the results were compared for the two types of structures.

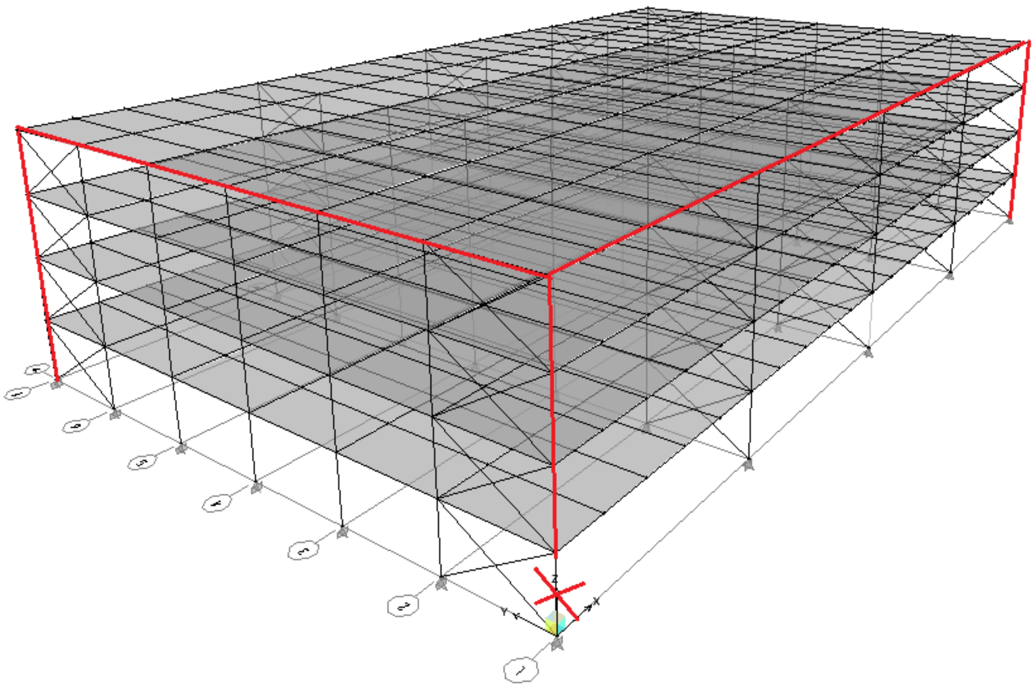

Figure 64: Removing a column at the corner side of the structure (15 m span)

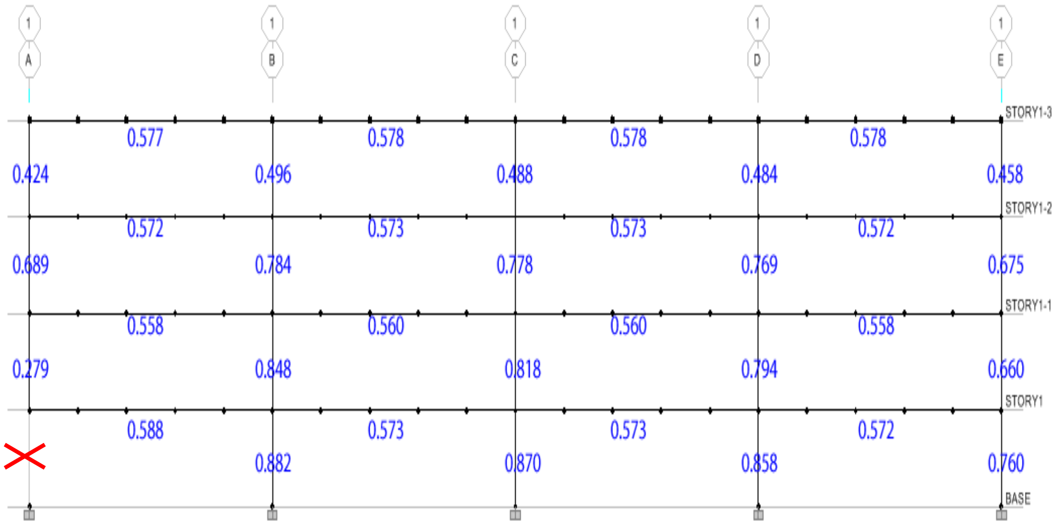

Figure 65: Demand Capacity Ratio (DCR) in the longitudinal side due to the eliminate of the column at the corner of the structure
(15 m span Normal I-beam)

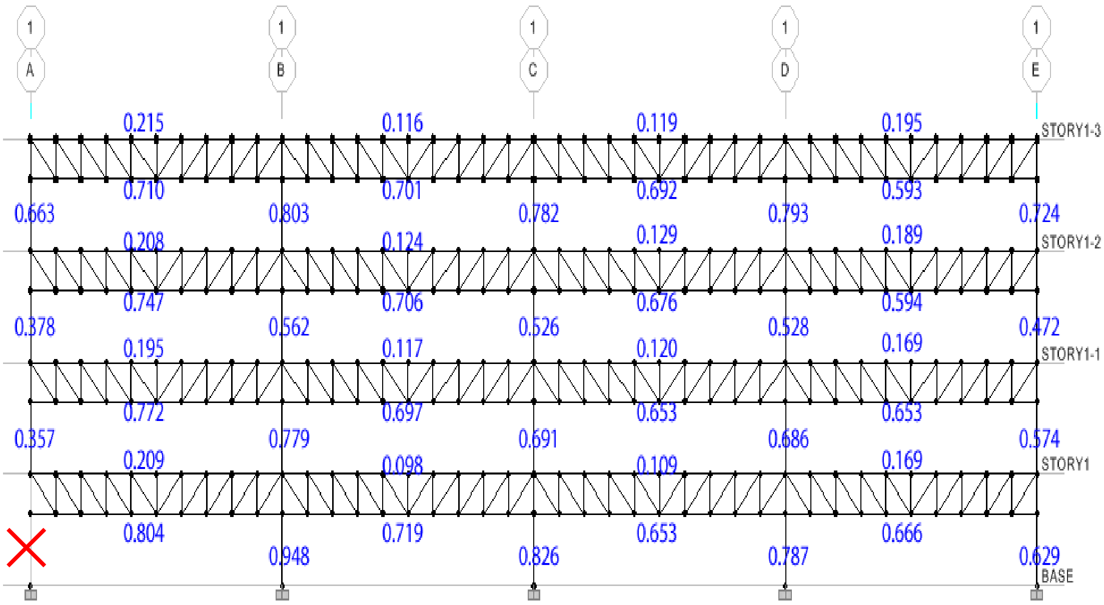

Figure 66: Demand Capacity Ratio (DCR) in the longitudinal side due to the elimination of the column at the corner of the structure (15 m span truss beam)

After removing the corner column all DCR values become less than the limit suggested by GSA. Therefore, the potential of progressive collapse is low. Maximum DCR for normal I-beam structure is 0.882 (Figure 65) and for truss beam structure is 0.948 (Figure 66). Therefore, the susceptibility of structures (beams and columns) against progressive collapse are low.

The removal of the corner column, in the short side of both structures resulted in DCR values that were in the short side, were roughly in the same range of values (Figure 67 and 68).

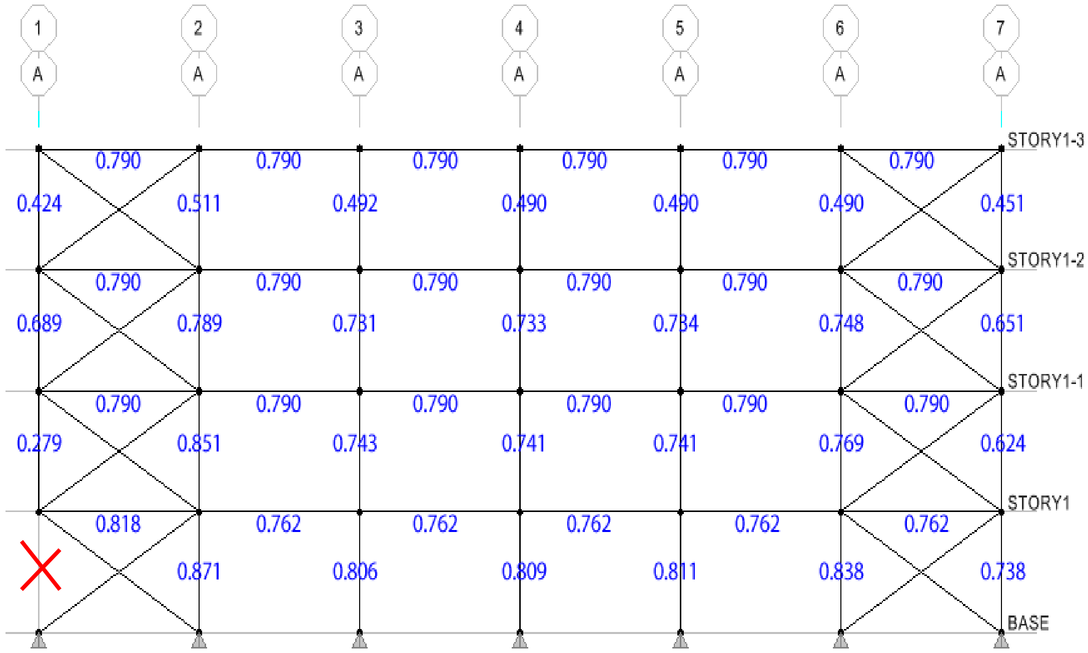

Figure 67: Demand Capacity Ratio (DCR) in transverse side due to eliminate of the column at the corner of the structure (15 m span Normal I-beam).

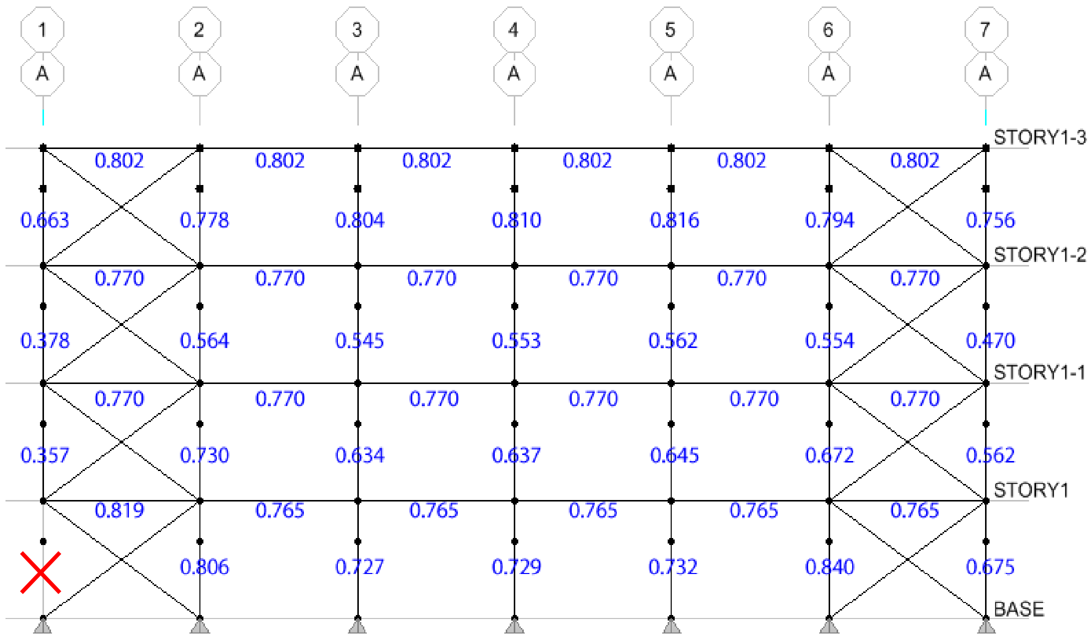

Figure 68: Demand Capacity Ratio (DCR) in the transverse side due to the elimination of the column at the corner of the structure (15 m span truss beam).

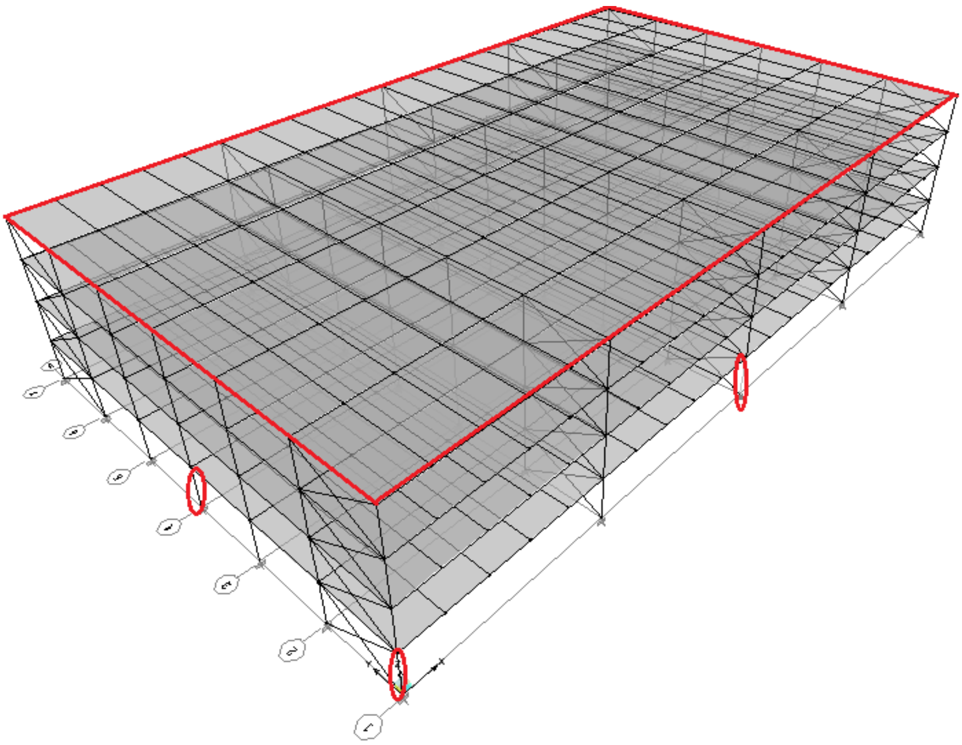

Figure 69: Remove each column and calculate DCR of floor members

Although the DCR of beam and column in 3 cases of removing column (Figure 69), in both structures were roughly in the same range, but in all of the 3 cases, floor members in normal I-beam achieved DCR more than the accepted limits.

By using braces the DCR values of the columns reduce but the DCR values of the beams in the floors do not change. When truss beams were used for floors the maximum DCR of the beams in the floor is 0.971 (Figure 71), when using normal beam the maximum DCR of the floor beams is 4.71(Figure 70). This means that if a column is suddenly removed, the structure could not resist against progressive collapse and it may fail.

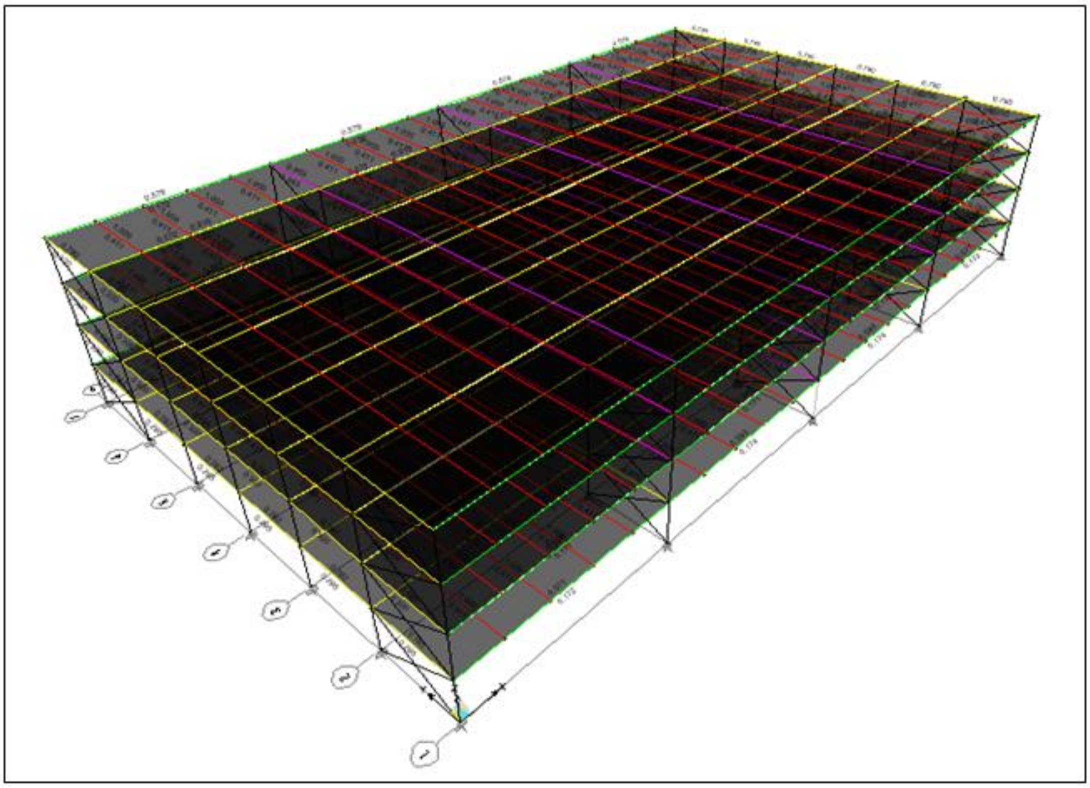

Figure 70: DCR value for floor member is 4.71 (15 m span normal I-beam)

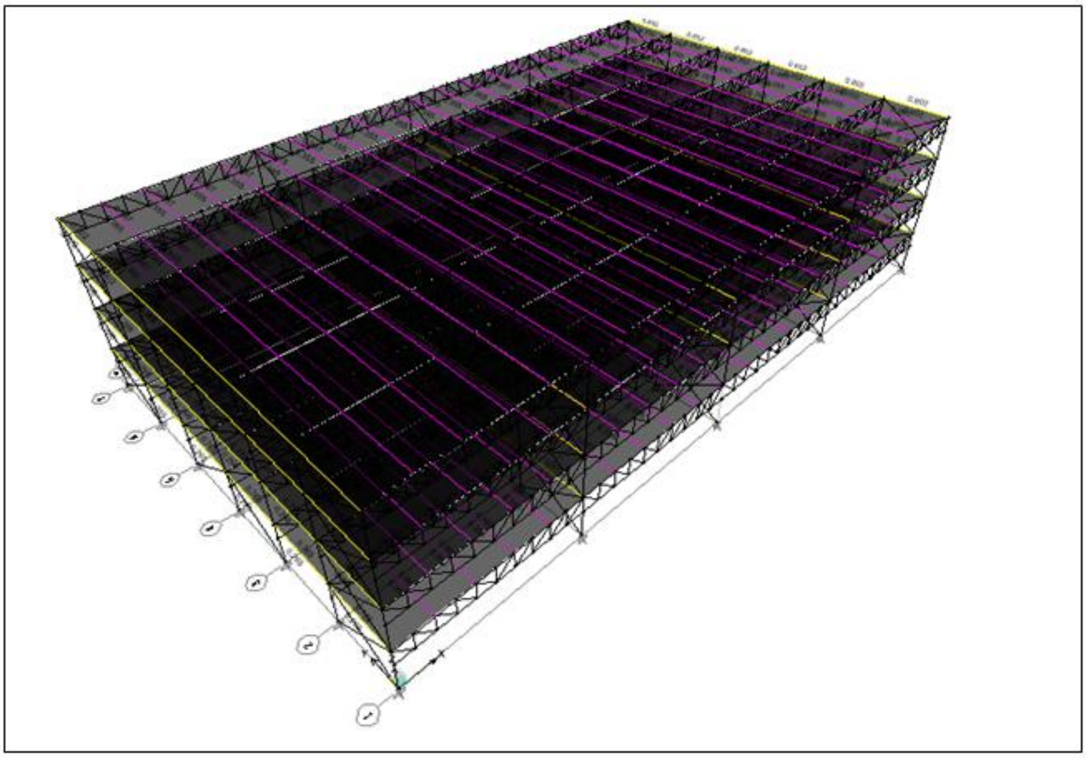

Figure 71: DCR value for floor member is 0.971(15 m span truss beam)

4.6 Vertical Displacements

The loss of columns considerably effect the surrounding structural members, causing deformation of the structure. First floor vertical displacements, directly above the removed columns, of these two buildings were considered at the first floor. In both structures the joints above the columns removed had high displacement values. The largest deformation was obtained in the external columns and beams above and next to the removed columns.

Table 9: Comparison of vertical displacement (m) after removal of column for 9 m span beam.

Joints Above the Column Removed (Span 9 Mete)	Truss Beam	Normal I-Beam
Column Removed on The Long Side	-0.00607 m	-0.0067 m
Column Removed on The short Side	-0.00326 m	-0.00573 m
Column Removed on The Corner Side	-0.00433 m	-0.00479 m

Table 10: Comparison of vertical displacement (m) after removal of column for 12 m span beam.

Joints Above the Columns Removed (Span 12 Mete)	Truss Beam	Normal I-Beam
Column Removed on The Long Side	-0.00830 m	-0.00903 m
Column Removed on The short Side	-0.00521 m	-0.00823 m
Column Removed on The Corner Side	-0.00560 m	-0.00650 m

Table 11: Comparison of vertical displacement (m) after removal of column for 15 m span beams.

Joints Above the Columns Removed (Span 15 Mete)	Truss Beam	Normal I-Beam
Column Removed on The Long Side	-0.01470 m	-0.01630 m
Column Removed on The short Side	-0.01390 m	-0.02180 m
Column Removed on The Corner Side	-0.00983 m	-0.01152 m

4.7 Comparison of Steel Weight of the Structures

Two cases of buildings, one with truss beam floors and the other with normal I-beam floors, were modeled in ETABS software. These models designed in three different span dimension to investigate which one on what condition act better against progressive collapse. By comparing the result of DCR values due to removal of columns on each side and corner of a building, based on GSA guideline, considered the resistance of the structure against progressive collapse. Also, displacement of the joints directly above the removed column, were calculated. The total steel weight of the structure was another paramete, that was considered (Figure 72). Table 12 shows the steel weights of each structure which was resisted against collapsing.

Table 12: The total steel weight of each structure (tonnes).

	Normal I-Beam Structure	Truss Beam Structure
Span 9 m	148.54	163.95
Span 12 m	387.80	341.30
Span 15 m	759.60	647.85

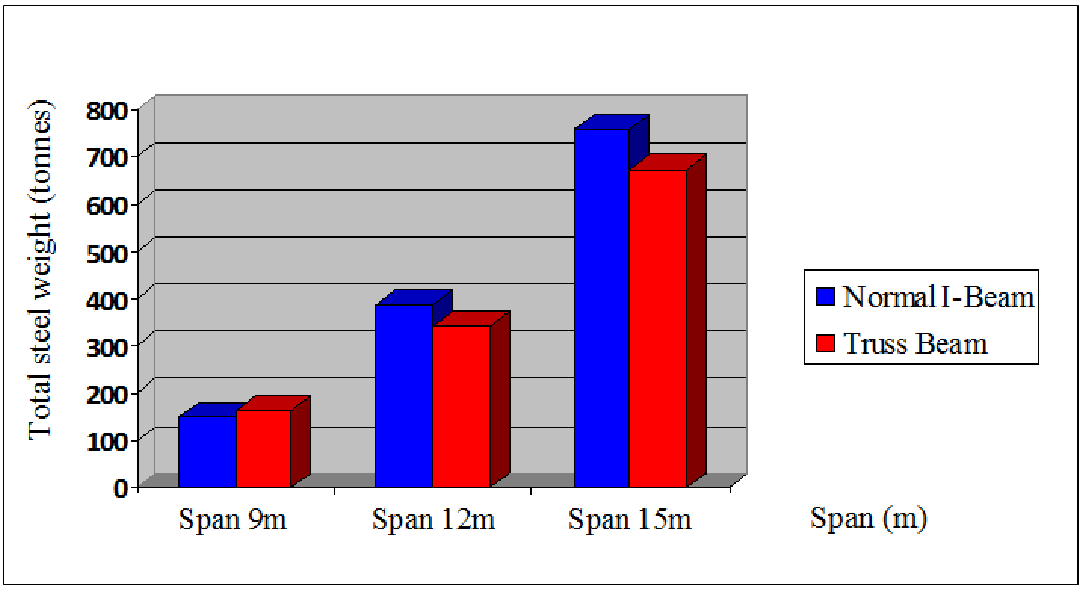

Figure 72: The total steel weight of each structure (tonnes)

Chapter 5

CONCLUSION AND RECOMMENDATION FOR FURTHER INVESTIGATION

5.1 Summary

This chapter summarizes the aim of this research and its findings. It also compares the results. The main aim of this study was to investigate the effect of using normal I-beams and truss beams as floors on the progressive collapse resistance of steel framed structures. While doing this also investigated the effect of beam span length on the progressive collapse resistance. Each building was modeled in ETABS software and their progressive collapse performance was compared due to sudden removal of a column due to car accident or explosion that may cause progressive collapse.

The General Service Administration guidelines (GSA) were used for the investigation of the progressive collapse potential of the structure calculated by ETABS analyzing. Two models of structure that is constructed by normal I-beam and truss beam with three different span dimensions (9 m, 12 m and 15 m). It is necessary to analyze and design a progressive collapse response. Linear static analysis is suggested by GSA guideline and analysis procedure to examine their effectiveness in modeling progressive collapse sequence. The displacement of each joint above the removed column and the demand capacity ratio (DCRs) considered for analysis procedures. The DCR values checked by acceptance criteria that shown in Table 8 for each member.

5.2 Conclusions

Two cases of building with steel frame modeled in ETABS computer program, the analysis procedure used is a linear static method. Major results and objectives from the investigated are presented below:

1. For 9 m span it is more efficient to use normal I-beam, because the total steel weight is less than using truss beams. For 12 m span using both normal and truss beams have approximately same weight. But for 15 metre span it is more efficient to use truss beams. The difference in total weight between these two buildings (normal beam and truss beam) is maximum around 124.2 tonnes.

2. The maximum deflection at the long side of the buildings is found to be in structure with normal I-beams.

3. After removing columns at each side of the structures according to GSA guideline and using load combination 2DL+0.5LL for linear static analysis, the vertical displacement at the column removal location in each truss beam structure is less than the normal I-beam structure. This difference is even bigger when 15 m beam spans are considered and from these results it is expected to get bigger for longer beam spans.

4. By comparing the results due to removing the columns in each side of the the structures, the magnitude of DCRs are suddenly increasing for building with normal I-beams. Whereas the buildings with truss beams, the DCRs are gradually increasing.

5. The DCR values due to sudden removal of column is more for the structure with normal I-beams. The maximum DCR value for the 15 m long normal I-beam is around 4.74 which is more than the limiting value of 2.0. Therefore, the structural member could not tolerate the additional forces and progressive collapse will occur. The DCR values in truss beams are less than the normal I-beams since the additional loads due to the column removal are distributed onto the truss vertical and diagonal members. Most of the excessive loads are absorbed by these members and therefore the potential of progressive collapse is low.

6. In order to improve the progressive collapse resistance of structures in buildings with normal I-beams and reduce the DCR values there are two possible options. One option is to use larger steel cross sections and the other option is to use more bracing. These two suggestions may lead to higher steel weight and may also cause more deformation after the removal of the columns.

5.3 Recommendation for Further Investigations

Computational modeling of progressive collapse is easy to perform through available computer programs, such as ETABS software. According to the considerations and conclusions the following are suggested as possible further work to continue this investigation.

1. This study only explains the removal of external steel frame columns to calculate the potential of progressive collapse. Although in GSA (2003) and DoD (2005) guidelines recommend both exterior and interior removal be considered. Therefore, the future investigation could be performed on structural response of structures due to the removal of interior columns.

2. In this study, steel-frame buildings were evaluated vulnerability to progressive collapse. Researching into the potential of progressive collapse in other structural systems like concrete-frame structure, higher than four floors or atypical structural configurations may be interesting.

3. As explained in Chapter 2, there are four types of analysis methods used to evaluate the progressive collapse. In this research only linear static method used for assessing the potential for progressive collapse. Therefore, it would be interesting to examine the linear dynamic analysis, nonlinear static analysis or nonlinear dynamic analysis results. The results could be compared and the outcome may also indicate the most realistic method to resist against progressive collapse.

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APPENDICES

Appendix A: ETABS Input File For Normal I-Beam (15 m span)

ETABS v9.7.4 File:NORMAL I-BEAM Units:KN-m September 4, 2012 4:04 PAGE 1

S T O R Y   D A T A

STORY	SIMILAR TO	HEIGHT	ELEVATION
STORY1-3	STORY1	3.450	13.800
STORY1-2	STORY1	3.450	10.350
STORY1-1	STORY1	3.450	6.900
STORY1	None	3.450	3.450
BASE	None		0.000

ETABS v9.7.4 File:NORMAL I-BEAM Units:KN-m September 4, 2012 4:04 PAGE 2

C O O R D I N A T E   S Y S T E M   L O C A T I O N   D A T A

NAME	TYPE	X	Y	ROTATION	BUBBLESIZE	VISIBLE
GLOBAL	Cartesian	0.000	0.000	0.00000	1.250	Yes

C O O R D I N A T E   S Y S T E M   G R I D   D A T A

SYSTEM NAME	GRID DIR	GRID ID	GRID TYPE	GRID HIDE	BUBBLE LOC	GRID COORDINATE
GLOBAL	X	A	Primary	No	Top	0.000
GLOBAL	X	B	Primary	No	Top	15.000
GLOBAL	X	C	Primary	No	Top	30.000
GLOBAL	X	D	Primary	No	Top	45.000
GLOBAL	X	E	Primary	No	Top	60.000
GLOBAL	Y	1	Primary	No	Left	0.000
GLOBAL	Y	2	Primary	No	Left	6.000
GLOBAL	Y	3	Primary	No	Left	12.000
GLOBAL	Y	4	Primary	No	Left	18.000
GLOBAL	Y	5	Primary	No	Left	24.000
GLOBAL	Y	6	Primary	No	Left	30.000
GLOBAL	Y	7	Primary	No	Left	36.000

ETABS v9.7.4 File:NORMAL I-BEAM Units:KN-m September 4, 2012 4:04 PAGE 3

M A S S   S O U R C E   L O A D S

LOAD	MULTIPLIER
DEAD	1.0000
LIVE	0.1500

ETABS v9.7.4 File:NORMAL I-BEAM Units:KN-m September 4, 2012 4:04 PAGE 4

M A T E R I A L   L I S T   B Y   E L E M E N T   T Y P E

ELEMENT TYPE	MATERIAL	TOTAL MASS tons	NUMBER PIECES	NUMBER STUDS
Column	STEEL	100.35	140	
Beam	STEEL	622.99	616	0

Brace	STEEL	46.34	80
Floor	CONC	3471.70	
Metal Deck	N.A.	82.82	

M A T E R I A L   L I S T   B Y   S E C T I O N

SECTION	ELEMENT TYPE	NUMBER PIECES	TOTAL LENGTH meters	TOTAL MASS tons	NUMBER STUDS
HE300B	Column	35	120.750	14.15	
IPE330	Beam	504	3024.000	146.90	0
TUBO160X160	Brace	20	138.423	12.19	
TUBO140X140	Brace	60	415.270	34.15	
HE600B	Column	35	120.750	25.63	
HE400B	Column	35	120.750	18.80	
HE900B	Beam	112	1680.000	476.08	0
HD400X347	Column	35	120.750	41.77	
DECK1	Floor			3471.70	
DECK1	Metal Deck			82.82	

M A T E R I A L   L I S T   B Y   S T O R Y

STORY	ELEMENT TYPE	MATERIAL	TOTAL WEIGHT tons	FLOOR AREA m2	UNIT WEIGHT kg/m2	NUMBER PIECES	NUMBER STUDS
STORY1-3	Column	STEEL	14.15	2160.000	6.5487	35	
STORY1-3	Beam	STEEL	156.82	2160.000	72.6020	154	0
STORY1-3	Brace	STEEL	11.38	2160.000	5.2701	20	
STORY1-3	Floor	CONC	867.92	2160.000	401.8168		
STORY1-3	Metal Deck	N.A.	20.70	2160.000	9.5853		
STORY1-2	Column	STEEL	18.80	2160.000	8.7023	35	
STORY1-2	Beam	STEEL	156.00	2160.000	72.2239	154	0
STORY1-2	Brace	STEEL	11.38	2160.000	5.2701	20	
STORY1-2	Floor	CONC	867.92	2160.000	401.8168		
STORY1-2	Metal Deck	N.A.	20.70	2160.000	9.5853		
STORY1-1	Column	STEEL	25.63	2160.000	11.8667	35	
STORY1-1	Beam	STEEL	154.37	2160.000	71.4677	154	0
STORY1-1	Brace	STEEL	11.38	2160.000	5.2701	20	
STORY1-1	Floor	CONC	867.92	2160.000	401.8168		

```
 STORY1-1      Metal Deck   N.A.                       20.70      2160.000
9.5853

 STORY1         Column        STEEL                    41.77      2160.000
19.3396          35
 STORY1         Beam           STEEL                  155.79      2160.000
72.1263          154            0
 STORY1         Brace          STEEL                   12.19      2160.000
5.6430           20
 STORY1         Floor          CONC                   867.92      2160.000
401.8168
 STORY1         Metal Deck   N.A.                      20.70      2160.000
9.5853

 SUM             Column        STEEL                  100.35      8640.000
11.6143          140
 SUM             Beam           STEEL                 622.99      8640.000
72.1049          616            0
 SUM             Brace          STEEL                  46.34      8640.000
5.3633           80
 SUM             Floor          CONC                 3471.70      8640.000
401.8168
 SUM             Metal Deck   N.A.                     82.82      8640.000
9.5853

 TOTAL          All            All                   4324.19      8640.000
500.4847          836            0

 ETABS v9.7.4   File:NORMAL I-BEAM   Units:KN-m   September 4, 2012
4:04  PAGE 7

 M A T E R I A L   P R O P E R T Y   D A T A

 MATERIAL      MATERIAL      DESIGN        MATERIAL         MODULUS OF
POISSON'S     THERMAL       SHEAR
 NAME           TYPE          TYPE           DIR/PLANE       ELASTICITY
RATIO        COEFF      MODULUS

 STEEL          Iso           Steel         All            200000000.00
0.3000  1.1700E-05 76923076.92
 CONC           Iso           Concrete      All            24821128.402
0.2000  9.9000E-0610342136.834
 OTHER          Iso           None          All            199947978.80
0.3000  1.1700E-05 76903068.77


 M A T E R I A L   P R O P E R T Y   M A S S   A N D   W E I G H T

 MATERIAL         MASS PER  WEIGHT PER
 NAME             UNIT VOL    UNIT VOL

 STEEL          7.8620E+00  7.7100E+01
 CONC           2.4007E+00  2.3562E+01
 OTHER          7.8271E+00  7.6820E+01


 M A T E R I A L   D E S I G N   D A T A   F O R   S T E E L   M A T
E R I A L S

 MATERIAL             STEEL        STEEL        STEEL
 NAME                    FY           FU     COST ($)

 STEEL          250000.000  400000.000   271447.16


 M A T E R I A L   D E S I G N   D A T A   F O R   C O N C R E T E
M A T E R I A L S

 MATERIAL      LIGHTWEIGHT       CONCRETE          REBAR           REBAR
LIGHTWT
```

NAME	CONCRETE	FC	FY	FYS	REDUC FACT
CONC	No	27579.032	413685.473	413685.473	N/A

ETABS v9.7.4 File:NORMAL I-BEAM Units:KN-m September 4, 2012 4:04 PAGE 8

F R A M E S E C T I O N P R O P E R T Y D A T A

FRAME SECTION NAME	MATERIAL NAME	SECTION SHAPE NAME OR NAME IN SECTION DATABASE FILE	CONC COL	CONC BEAM
HE300B	STEEL	HE300B		
HE360B	STEEL	HE360B		
HE500B	STEEL	HE500B		
HE700B	STEEL	HE700B		
IPE330	STEEL	IPE330		
TUBO60X60X5	STEEL	TUBO60X60X5		
TUBO100X100X10	STEEL	TUBO100X100X10		
TUBO160X160X10	STEEL	TUBO160X160X10		
TUBO160X160X20	STEEL	TUBO160X160X20		
TUBO240X240X20	STEEL	TUBO240X240X20		
HE800B	STEEL	HE800B		
TUBO140X140X22.2	STEEL	TUBO140X140X22.2		
HE280B	STEEL	HE280B		
TUBO140X140X10	STEEL	TUBO140X140X10		
HE600B	STEEL	HE600B		
IPE270	STEEL	IPE270		
HE180B	STEEL	HE180B		
HE200B	STEEL	HE200B		
HE260B	STEEL	HE260B		
TUBO100X100X8	STEEL	TUBO100X100X8		
IPE180	STEEL	IPE180		
TUBO180X180X10	STEEL	TUBO180X180X10		
TUBO80X80X5	STEEL	TUBO80X80X5		
TUBO80X80X10	STEEL	TUBO80X80X10		
TUBO200X200X25	STEEL	TUBO200X200X25		
TUBO120X120X16	STEEL	TUBO120X120X16		
HE240B	STEEL	HE240B		
IPE200	STEEL	IPE200		
TUBO120X120X20	STEEL	TUBO120X120X20		
TUBO120X120X8	STEEL	TUBO120X120X8		
TUBO180X180X22.2	STEEL	TUBO180X180X22.2		
TUBO70X70X10	STEEL	TUBO70X70X10		
IPE240	STEEL	IPE240		
TUBO120X120X22.2	STEEL	TUBO120X120X22.2		
HE400B	STEEL	HE400B		
IPE750X173	STEEL	IPE750X173		
IPE750X147	STEEL	IPE750X147		
HE450B	STEEL	HE450B		
IPE750X137	STEEL	IPE750X137		
IPE600	STEEL	IPE600		
IPE500	STEEL	IPE500		
HE900B	STEEL	HE900B		
IPE400	STEEL	IPE400		
IPE360	STEEL	IPE360		
HD400X347	STEEL	I/Wide Flange		
TUBO160X112X22.2	STEEL	TUBO160X112X22.2		

F R A M E S E C T I O N W E I G H T S A N D M A S S E S

FRAME SECTION NAME	TOTAL WEIGHT	TOTAL MASS
HE300B	138.7164	14.1451

HE360B	0.0000	0.0000
HE500B	0.0000	0.0000
HE700B	0.0000	0.0000
IPE330	1440.6404	146.9042
TUBO60X60X5	0.0000	0.0000
TUBO100X100X10	0.0000	0.0000
TUBO160X160X10	0.0000	0.0000
TUBO160X160X20	119.5313	12.1888
TUBO240X240X20	0.0000	0.0000
HE800B	0.0000	0.0000
TUBO140X140X22.2	334.9010	34.1503
HE280B	0.0000	0.0000
TUBO140X140X10	0.0000	0.0000
HE600B	251.3653	25.6321
IPE270	0.0000	0.0000
HE180B	0.0000	0.0000
HE200B	0.0000	0.0000
HE260B	0.0000	0.0000
TUBO100X100X8	0.0000	0.0000
IPE180	0.0000	0.0000
TUBO180X180X10	0.0000	0.0000
TUBO80X80X5	0.0000	0.0000
TUBO80X80X10	0.0000	0.0000
TUBO200X200X25	0.0000	0.0000
TUBO120X120X16	0.0000	0.0000
HE240B	0.0000	0.0000
IPE200	0.0000	0.0000
TUBO120X120X20	0.0000	0.0000
TUBO120X120X8	0.0000	0.0000
TUBO180X180X22.2	0.0000	0.0000
TUBO70X70X10	0.0000	0.0000
IPE240	0.0000	0.0000
TUBO120X120X22.2	0.0000	0.0000
HE400B	184.3345	18.7969
IPE750X173	0.0000	0.0000
IPE750X147	0.0000	0.0000
HE450B	0.0000	0.0000
IPE750X137	0.0000	0.0000
IPE600	0.0000	0.0000
IPE500	0.0000	0.0000
HE900B	4668.7726	476.0816
IPE400	0.0000	0.0000
IPE360	0.0000	0.0000
HD400X347	409.6576	41.7734
TUBO160X112X22.2	0.0000	0.0000

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D E C K   S E C T I O N   P R O P E R T Y   D A T A

DECK SECTION	DECK TYPE	SLAB MATERIAL	DECK MATERIAL	DECK SHEAR THICK	DECK UNIT WT
DECK1	Filled	CONC	N/A	N/A	9.4000E-02

D E C K   S E C T I O N   S H E A R   S T U D   D A T A

DECK SECTION	STUD DIAM	STUD HEIGHT	STUD FU
DECK1	0.0191	0.0700	448159.260

D E C K   S E C T I O N   G E O M E T R Y   D A T A

DECK SECTION	SLAB DEPTH	RIB DEPTH	RIB WIDTH	RIB SPACING
DECK1	0.1300	0.0600	0.1800	0.2900

S T A T I C L O A D C A S E S

STATIC CASE	CASE TYPE	AUTO LAT LOAD	SELF WT MULTIPLIER	NOTIONAL FACTOR	NOTIONAL DIRECTION
DEAD	DEAD	N/A	1.0000		
LIVE	LIVE	N/A	0.0000		
EX	QUAKE	EUROCODE8 20	0.0000		
EY	QUAKE	EUROCODE8 20	0.0000		

L O A D I N G C O M B I N A T I O N S

COMBO	COMBO TYPE	CASE	CASE TYPE	SCALE FACTOR
DCMPC1	ADD	DEAD	Static	1.6000
DCMPC2	ADD	DEAD	Static	1.6000
		LIVE	Static	0.3200
DCMPS1	ADD	DEAD	Static	1.4000
DCMPS2	ADD	DEAD	Static	1.4000
		LIVE	Static	1.6000
DCMPD1	ADD	DEAD	Static	1.0000
DCMPD2	ADD	DEAD	Static	1.0000
		LIVE	Static	1.0000
DSTLS1	ADD	DEAD	Static	1.3500
DSTLS2	ADD	DEAD	Static	1.3500
		LIVE	Static	1.5000
DSTLS3	ADD	DEAD	Static	1.0000
		LIVE	Static	0.3000
		EX	Static	1.0000
DSTLS4	ADD	DEAD	Static	1.0000
		LIVE	Static	0.3000
		EX	Static	-1.0000
DSTLS5	ADD	DEAD	Static	1.0000
		LIVE	Static	0.3000
		EY	Static	1.0000
DSTLS6	ADD	DEAD	Static	1.0000
		LIVE	Static	0.3000
		EY	Static	-1.0000
DSTLS7	ADD	DEAD	Static	1.0000
		EX	Static	1.0000
DSTLS8	ADD	DEAD	Static	1.0000
		EX	Static	-1.0000
DSTLS9	ADD	DEAD	Static	1.0000
		EY	Static	1.0000
DSTLS10	ADD	DEAD	Static	1.0000
		EY	Static	-1.0000
DSTLS11	ADD	DEAD	Static	1.0000
		EX	Static	1.0000
DSTLS12	ADD	DEAD	Static	1.0000
		EX	Static	-1.0000
DSTLS13	ADD	DEAD	Static	1.0000
		EY	Static	1.0000
DSTLS14	ADD	DEAD	Static	1.0000
		EY	Static	-1.0000
DSTLD1	ADD	DEAD	Static	1.0000
DSTLD2	ADD	DEAD	Static	1.0000
		LIVE	Static	1.0000

Appendix B: ETABS Input File For Truss Beam (15 m span)

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 ETABS v9.7.4  File:TRUSS BEAM  Units:KN-m  September 4, 2012 4:05
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 S T O R Y   D A T A

 STORY        SIMILAR TO         HEIGHT   ELEVATION

 STORY1-3     STORY1              4.250      17.000
 STORY1-2     STORY1              4.250      12.750
 STORY1-1     STORY1              4.250       8.500
 STORY1       None                4.250       4.250
 BASE         None                            0.000

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 C O O R D I N A T E   S Y S T E M   L O C A T I O N   D A T A

 NAME          TYPE                         X              Y     ROTATION
BUBBLESIZE  VISIBLE

 GLOBAL        Cartesian                0.000          0.000      0.00000
1.250  Yes


 C O O R D I N A T E   S Y S T E M   G R I D   D A T A

 SYSTEM       GRID  GRID    GRID    GRID    BUBBLE          GRID
 NAME         DIR   ID      TYPE    HIDE    LOC       COORDINATE

 GLOBAL       X     A       Primary No      Top            0.000
 GLOBAL       X     B       Primary No      Top           15.000
 GLOBAL       X     C       Primary No      Top           30.000
 GLOBAL       X     D       Primary No      Top           45.000
 GLOBAL       X     E       Primary No      Top           60.000
 GLOBAL       Y     1       Primary No      Left           0.000
 GLOBAL       Y     2       Primary No      Left           6.000
 GLOBAL       Y     3       Primary No      Left          12.000
 GLOBAL       Y     4       Primary No      Left          18.000
 GLOBAL       Y     5       Primary No      Left          24.000
 GLOBAL       Y     6       Primary No      Left          30.000
 GLOBAL       Y     7       Primary No      Left          36.000

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 M A S S   S O U R C E   D A T A

 MASS         LATERAL      LUMP MASS
 FROM         MASS ONLY    AT STORIES

 Loads        Yes          Yes


 M A S S   S O U R C E   L O A D S

 LOAD           MULTIPLIER

 DEAD               1.0000
 LIVE               0.1500

 ETABS v9.7.4  File:TRUSS BEAM  Units:KN-m  September 4, 2012 4:05
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M A T E R I A L   L I S T   B Y   E L E M E N T   T Y P E

ELEMENT TYPE	MATERIAL	TOTAL MASS tons	NUMBER PIECES	NUMBER STUDS
Column	STEEL	143.58	1148	
Beam	STEEL	391.53	728	3408
Brace	STEEL	119.04	1200	
Floor	CONC	3471.70		
Metal Deck	N.A.	82.82		

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M A T E R I A L   L I S T   B Y   S E C T I O N

SECTION	ELEMENT TYPE	NUMBER PIECES	TOTAL LENGTH meters	TOTAL MASS tons	NUMBER STUDS
IPE330	Beam	504	3024.000	144.87	3408
TUBO60X60X5	Beam	112	1680.000	14.20	0
TUBO100X100	Column	1008	1512.000	42.79	
TUBO100X100	Brace	1120	2375.879	67.25	
TUBO160X160	Brace	80	588.218	51.80	
TUBO240X240	Beam	112	1680.000	232.46	0
HD360X196	Column	35	148.750	29.05	
HD360X179	Column	35	148.750	26.47	
HD260X93	Column	35	148.750	13.27	
HD400X237	Column	35	148.750	31.99	
DECK1	Floor			3471.70	
DECK1	Metal Deck			82.82	

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M A T E R I A L   L I S T   B Y   S T O R Y

STORY	ELEMENT TYPE	MATERIAL	TOTAL WEIGHT tons	FLOOR AREA m2	UNIT WEIGHT kg/m2	NUMBER PIECES	NUMBER STUDS
STORY1-3	Column	STEEL	23.97	2160.000	11.0982	287	
STORY1-3	Beam	STEEL	98.04	2160.000	45.3872	182	852
STORY1-3	Brace	STEEL	29.76	2160.000	13.7778	300	
STORY1-3	Floor	CONC	867.92	2160.000	401.8168		
STORY1-3	Metal Deck	N.A.	20.70	2160.000	9.5853		
STORY1-2	Column	STEEL	37.17	2160.000	17.2068	287	
STORY1-2	Beam	STEEL	97.84	2160.000	45.2979	182	852
STORY1-2	Brace	STEEL	29.76	2160.000	13.7778	300	
STORY1-2	Floor	CONC	867.92	2160.000	401.8168		

```
 STORY1-2     Metal Deck   N.A.                          20.70      2160.000
9.5853

 STORY1-1     Column        STEEL                        39.75      2160.000
18.4015         287
 STORY1-1     Beam          STEEL                        97.84      2160.000
45.2967         182         852
 STORY1-1     Brace         STEEL                        29.76      2160.000
13.7778         300
 STORY1-1     Floor         CONC                        867.92      2160.000
401.8168
 STORY1-1     Metal Deck   N.A.                          20.70      2160.000
9.5853

 STORY1        Column       STEEL                        42.69      2160.000
19.7646         287
 STORY1        Beam          STEEL                       97.81      2160.000
45.2827         182         852
 STORY1        Brace         STEEL                       29.76      2160.000
13.7778         300
 STORY1        Floor         CONC                       867.92      2160.000
401.8168
 STORY1        Metal Deck   N.A.                         20.70      2160.000
9.5853

 SUM            Column       STEEL                      143.58      8640.000
16.6178        1148
 SUM            Beam          STEEL                     391.53      8640.000
45.3161         728        3408
 SUM            Brace         STEEL                     119.04      8640.000
13.7778        1200
 SUM            Floor         CONC                     3471.70      8640.000
401.8168
 SUM            Metal Deck   N.A.                        82.82      8640.000
9.5853

 TOTAL         All           All                       4208.66      8640.000
487.1138        3076        3408

 ETABS v9.7.4  File:TRUSS BEAM  Units:KN-m  September 4, 2012 4:05
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 M A T E R I A L   P R O P E R T Y   D A T A

 MATERIAL      MATERIAL     DESIGN       MATERIAL        MODULUS OF
POISSON'S     THERMAL       SHEAR
 NAME           TYPE          TYPE          DIR/PLANE       ELASTICITY
RATIO        COEFF     MODULUS

 STEEL          Iso           Steel        All           200000000.00
0.3000  1.1700E-05 76923076.92
 CONC           Iso           Concrete     All           24821128.402
0.2000  9.9000E-0610342136.834
 OTHER          Iso           None         All           199947978.80
0.3000  1.1700E-05 76903068.77


 M A T E R I A L   P R O P E R T Y   M A S S   A N D   W E I G H T

 MATERIAL         MASS PER  WEIGHT PER
 NAME             UNIT VOL    UNIT VOL

 STEEL          7.8620E+00  7.7100E+01
 CONC           2.4007E+00  2.3562E+01
 OTHER          7.8271E+00  7.6820E+01


 M A T E R I A L   D E S I G N   D A T A   F O R   S T E E L   M A T
E R I A L S
```

MATERIAL NAME	STEEL FY	STEEL FU	STEEL COST ($)
STEEL	250000.000	400000.000	271447.16

M A T E R I A L   D E S I G N   D A T A   F O R   C O N C R E T E   M A T E R I A L S

MATERIAL NAME	LIGHTWEIGHT CONCRETE	CONCRETE FC	REBAR FY	REBAR FYS	LIGHTWT REDUC FACT
CONC	No	27579.032	413685.473	413685.473	N/A

F R A M E   S E C T I O N   P R O P E R T Y   D A T A

FRAME SECTION NAME	MATERIAL NAME	SECTION SHAPE NAME OR NAME IN SECTION DATABASE FILE	CONC COL	CONC BEAM
HE300B	STEEL	HE300B		
HE360B	STEEL	HE360B		
HE500B	STEEL	HE500B		
HE700B	STEEL	HE700B		
IPE330	STEEL	IPE330		
TUBO60X60X5	STEEL	TUBO60X60X5		
TUBO100X100X10	STEEL	TUBO100X100X10		
TUBO160X160X10	STEEL	TUBO160X160X10		
TUBO160X160X20	STEEL	TUBO160X160X20		
TUBO240X240X20	STEEL	TUBO240X240X20		
HE800B	STEEL	HE800B		
TUBO140X140X22.2	STEEL	TUBO140X140X22.2		
HE280B	STEEL	HE280B		
TUBO140X140X10	STEEL	TUBO140X140X10		
HE600B	STEEL	HE600B		
IPE270	STEEL	IPE270		
HE180B	STEEL	HE180B		
HE200B	STEEL	HE200B		
HE260B	STEEL	HE260B		
TUBO100X100X8	STEEL	TUBO100X100X8		
IPE180	STEEL	IPE180		
TUBO180X180X10	STEEL	TUBO180X180X10		
TUBO80X80X5	STEEL	TUBO80X80X5		
TUBO80X80X10	STEEL	TUBO80X80X10		
TUBO200X200X25	STEEL	TUBO200X200X25		
TUBO120X120X16	STEEL	TUBO120X120X16		
HE240B	STEEL	HE240B		
IPE200	STEEL	IPE200		
TUBO120X120X20	STEEL	TUBO120X120X20		
TUBO120X120X8	STEEL	TUBO120X120X8		
TUBO180X180X22.2	STEEL	TUBO180X180X22.2		
TUBO70X70X10	STEEL	TUBO70X70X10		
IPE240	STEEL	IPE240		
TUBO120X120X22.2	STEEL	TUBO120X120X22.2		
HE400B	STEEL	HE400B		
HD360X196	STEEL	I/Wide Flange		
HD400X592	STEEL	I/Wide Flange		
HD400X314	STEEL	I/Wide Flange		
HD360X179	STEEL	I/Wide Flange		
HD260X93	STEEL	I/Wide Flange		
HD400X187	STEEL	I/Wide Flange		
HD400X237	STEEL	I/Wide Flange		

F R A M E   S E C T I O N   W E I G H T S   A N D   M A S S E S

FRAME SECTION NAME	TOTAL WEIGHT	TOTAL MASS
HE300B	0.0000	0.0000
HE360B	0.0000	0.0000
HE500B	0.0000	0.0000
HE700B	0.0000	0.0000
IPE330	1420.7023	144.8711
TUBO60X60X5	139.2156	14.1960
TUBO100X100X10	1079.1196	110.0394
TUBO160X160X10	0.0000	0.0000
TUBO160X160X20	507.9377	51.7952
TUBO240X240X20	2279.6928	232.4636
HE800B	0.0000	0.0000
TUBO140X140X22.2	0.0000	0.0000
HE280B	0.0000	0.0000
TUBO140X140X10	0.0000	0.0000
HE600B	0.0000	0.0000
IPE270	0.0000	0.0000
HE180B	0.0000	0.0000
HE200B	0.0000	0.0000
HE260B	0.0000	0.0000
TUBO100X100X8	0.0000	0.0000
IPE180	0.0000	0.0000
TUBO180X180X10	0.0000	0.0000
TUBO80X80X5	0.0000	0.0000
TUBO80X80X10	0.0000	0.0000
TUBO200X200X25	0.0000	0.0000
TUBO120X120X16	0.0000	0.0000
HE240B	0.0000	0.0000
IPE200	0.0000	0.0000
TUBO120X120X20	0.0000	0.0000
TUBO120X120X8	0.0000	0.0000
TUBO180X180X22.2	0.0000	0.0000
TUBO70X70X10	0.0000	0.0000
IPE240	0.0000	0.0000
TUBO120X120X22.2	0.0000	0.0000
HE400B	0.0000	0.0000
HD360X196	284.8696	29.0486
HD400X592	0.0000	0.0000
HD400X314	0.0000	0.0000
HD360X179	259.5625	26.4680
HD260X93	130.1689	13.2735
HD400X187	0.0000	0.0000
HD400X237	313.7435	31.9929

D E C K   S E C T I O N   P R O P E R T Y   D A T A

DECK SECTION	DECK TYPE	SLAB MATERIAL	DECK MATERIAL	DECK SHEAR THICK	DECK UNIT WT
DECK1	Filled	CONC	N/A	N/A	9.4000E-02

D E C K   S E C T I O N   S H E A R   S T U D   D A T A

DECK SECTION	STUD DIAM	STUD HEIGHT	STUD FU
DECK1	0.0191	0.0700	448159.260

D E C K   S E C T I O N   G E O M E T R Y   D A T A

DECK SECTION	SLAB DEPTH	RIB DEPTH	RIB WIDTH	RIB SPACING
DECK1	0.1300	0.0600	0.1800	0.2900

S T A T I C   L O A D   C A S E S

STATIC CASE	CASE TYPE	AUTO LAT LOAD	SELF WT MULTIPLIER	NOTIONAL FACTOR	NOTIONAL DIRECTION
DEAD	DEAD	N/A	1.0000		
LIVE	LIVE	N/A	0.0000		
EX	QUAKE	EUROCODE8 20	0.0000		
EY	QUAKE	EUROCODE8 20	0.0000		

L O A D I N G   C O M B I N A T I O N S

COMBO	COMBO TYPE	CASE	CASE TYPE	SCALE FACTOR
DCMPC1	ADD	DEAD	Static	1.6000
DCMPC2	ADD	DEAD	Static	1.6000
		LIVE	Static	0.3200
DCMPS1	ADD	DEAD	Static	1.4000
DCMPS2	ADD	DEAD	Static	1.4000
		LIVE	Static	1.6000
DCMPD1	ADD	DEAD	Static	1.0000
DCMPD2	ADD	DEAD	Static	1.0000
		LIVE	Static	1.0000
DSTLS1	ADD	DEAD	Static	1.3500
DSTLS2	ADD	DEAD	Static	1.3500
		LIVE	Static	1.5000
DSTLS3	ADD	DEAD	Static	1.0000
		LIVE	Static	0.3000
		EX	Static	1.0000
DSTLS4	ADD	DEAD	Static	1.0000
		LIVE	Static	0.3000
		EX	Static	-1.0000
DSTLS5	ADD	DEAD	Static	1.0000
		LIVE	Static	0.3000
		EY	Static	1.0000
DSTLS6	ADD	DEAD	Static	1.0000
		LIVE	Static	0.3000
		EY	Static	-1.0000
DSTLS7	ADD	DEAD	Static	1.0000
		EX	Static	1.0000
DSTLS8	ADD	DEAD	Static	1.0000
		EX	Static	-1.0000
DSTLS9	ADD	DEAD	Static	1.0000
		EY	Static	1.0000
DSTLS10	ADD	DEAD	Static	1.0000
		EY	Static	-1.0000
DSTLS11	ADD	DEAD	Static	1.0000
		EX	Static	1.0000
DSTLS12	ADD	DEAD	Static	1.0000
		EX	Static	-1.0000
DSTLS13	ADD	DEAD	Static	1.0000
		EY	Static	1.0000
DSTLS14	ADD	DEAD	Static	1.0000

		EY	Static	-1.0000
DSTLD1	ADD	DEAD	Static	1.0000
DSTLD2	ADD	DEAD	Static	1.0000
		LIVE	Static	1.0000