T.R.
SAKARYA UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

PROGRESSIVE COLLAPSE RESPONSE OF REINFORCED CONCRETE HIGH-RISE BUILDINGS DESIGNED ACCORDING TO TURKISH EARTHQUAKE CODE

MSc THESIS

Munyaradzi GONDOBWE

Civil Engineering Department

Civil Engineering Program

JUNE 2023

T.R.
SAKARYA UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

PROGRESSIVE COLLAPSE RESPONSE OF REINFORCED CONCRETE HIGH-RISE BUILDINGS DESIGNED ACCORDING TO TURKISH EARTHQUAKE CODE

MSc THESIS

Munyaradzi GONDOBWE

Civil Engineering Department

Civil Engineering Program

Thesis Advisor: Assoc. Prof. Dr. Aydın DEMİR

JUNE 2023

The thesis work titled "PROGRESSIVE COLLAPSE RESPONSE OF REINFORCED CONCRETE HIGH-RISE BUILDINGS DESIGNED ACCORDING TO TURKISH EARTHQUAKE CODE" prepared by Munyaradzi GONDOBWE was accepted by the following jury on 21/06/2023 by unanimously of votes as a MSc THESIS in Sakarya University Graduate School of Natural and Applied Sciences, Civil Engineering department, Civil Engineering program.

Thesis Jury

Head of Jury :	**Title Name SURNAME** Sakarya University	Prof. Dr. Erkan ÇELEBİ
Jury Member :	**Title Name SURNAME** (Advisor) Sakarya University	Doç. Dr. Aydın DEMİR
Jury Member :	**Title Name SURNAME** Sakarya University of Applied Sciences	Döç. Dr. Yusuf SÜMER

STATEMENT OF COMPLIANCE WITH THE ETHICAL PRINCIPLES AND RULES

I declare that the thesis work titled " PROGRESSIVE COLLAPSE RESPONSE OF REINFORCED CONCRETE HIGH-RISE BUILDINGS DESIGNED ACCORDING TO TURKISH EARTHQUAKE CODE ", which I have prepared in accordance with Sakarya University Graduate School of Natural and Applied Sciences regulations and Higher Education Institutions Scientific Research and Publication Ethics Directive, belongs to me, is an original work, I have acted in accordance with the regulations and directives mentioned above at all stages of my study, I did not get the innovations and results contained in the thesis from anywhere else, I duly cited the references for the works I used in my thesis, I did not submit this thesis to another scientific committee for academic purposes and to obtain a title, in accordance with the articles 9/2 and 22/2 of the Sakarya University Graduate Education and Training Regulation published in the Official Gazette dated 20.04.2016, a report was received in accordance with the criteria determined by the graduate school using the plagiarism software program to which Sakarya University is a subscriber, I accept all kinds of legal responsibility that may arise in case of a situation contrary to this statement.

(21/06/2023)

Munyaradzi GONDOBWE

This thesis is dedicated to my lovely parents who invested so much to my education

ACKNOWLEDGMENTS

I would like to express my deep appreciation to everyone who has supported me throughout my academic journey. First and foremost, I am grateful to God for giving me the strength, wisdom, and determination to complete this thesis.

I am also indebted to my family for their unwavering love, encouragement, and moral support. Their sacrifices and endless prayers have been the driving force behind my success.

Special thanks to my supervisor, Assoc. Prof. Dr. Aydin Demir, for his invaluable guidance, feedback, and support throughout my research. His expertise, dedication, and patience have been instrumental in shaping the outcome of this study.

Lastly, I would like to thank my friends for their camaraderie, motivation, and understanding. Their unwavering support and encouragement have been an essential part of my academic journey.

Munyaradzi Gondobwe

TABLE OF CONTENTS

Page

ABBREVIATIONS

VVLOP : Very Low Level of Protection
MLOP : Medium Level of Protection
LLOP : Low Level of Protection
HLOP : High Level of Protection
PC : Progressive Collapse
RC : Reinforced Concrete
FE : Finite Elements
ELR : Enhanced Local Resistance
LRFD : Load and Resistance Factor Design
LSP : Linear Static Procedure
NLS : Nonlinear Static Procedure
NLD : Nonlinear Dynamic Procedure
CR : Column Removal
DCR : Demand Capacity Ratio
WR : Wall Removal
GF : Ground Floor
MF : Middle Floor
TF : Top Floor
IO : Immediate Occupancy
LS : Life Safety
CP : Collapse Prevention

SYMBOLS

Qce	: Expected strength
Qudlim	: Demand
θ	: Plastic rotation
L	: Live load
D	: Dead load
W	: Wind load
S	: Snow load

LIST OF TABLES

Page

LIST OF FIGURES

Page

PROGRESSIVE COLLAPSE RESPONSE OF REINFORCED CONCRETE HIGH-RISE BUILDINGS DESIGNED ACCORDING TO TURKISH EARTHQUAKE CODE

SUMMARY

When a building experiences severe impact loads such as explosions, car or airplane impacts, or similar natural or man-made events on a critical structural member like a column or shear wall, failure of the critical member can lead to the ultimate collapse of the whole building if the remaining members fail to carry the extra load imposed on them due to the failed member. This progression of damage from the member level to the structural level is what is described as progressive collapse. This study investigates the progressive collapse response of tall buildings designed according to the Turkish earthquake code.

In this research, a case study was done on a 40-story tall building designed according to the Turkish earthquake code. The building used was a symmetrical building with a flat plate system, perimeter beams, and shear walls at the core. A 3D model was developed using SAP 2000 version 23 to represent the building. Gravity loads were applied directly to the slabs as uniform loads. Nonlinear behavior on the beams and columns was modeled using the fiber hinge model.

A progressive collapse procedure was performed for a total of 18 scenarios. This was done by removing a critical member using SAP2000 v23 and then reading the displacements on the node directly above the removed member. These results were represented as displacement time histories of the node. Plastic hinge rotations were read on the beams surrounding the removed member, where the hinges formed. The rotations were compared to the plastic hinge rotation limits given in the Turkish Earthquake Code (TEC 2018) and the American Society of Civil Engineers (ASCE/SEI 41-17) for different performance levels. The 18 scenarios were compared in various combinations to get a better insight into the results obtained.

From the results, it was observed that hinges formed on perimeter beams adjacent to the removed column only in the scenarios where the column was removed from the corner and the edge of the bottom floor. In the rest of the 16 scenarios, there were no hinges formed on the perimeter beams or on the columns.

This research gave useful and critical insights into the performance of tall buildings designed according to the Turkish earthquake code, since no study had been done on this subject before. The whole process was described in detail in such a way that it can be replicated by practicing engineers without any prior experience in this subject.

TÜRK DEPREM YÖNETMELİĞİNE GÖRE YAPILAN BETONARME YÜKSEK BİNALARIN İLERLEMELİ ÇÖKMEYE TEPKİSİ

ÖZET

Bir bina, kolon veya perde duvar gibi kritik bir yapısal eleman üzerinde patlamalar, araba veya uçak çarpması veya benzer doğal veya insan yapımı olaylar gibi ciddi darbe yüklerine maruz kaldığında, kritik elemanın hasar alması binanın nihai olarak çökmesine yol açabilir. Kalan elemanlar, hasarlı eleman nedeniyle kendilerine yüklenen ek yükü taşıyamazlarsa bina çöker. Hasarın eleman seviyesinden yapısal seviyeye ilerlemesi, ilerlemeli çökme ifade eder. Bu çalışma, Türk deprem yönetmeliğine göre tasarlanmış yüksek binaların ilerlemeli çökme davranışını incelemektedir.

İlerlemeli çökmenin tek bir tanımı yoktur. Literatürde farklı tanımlar bulunabilir, ancak bunlar ortak bir felsefeyi ima eder. Genel olarak tanımlanan ilerlemeli çökme, dikey bir taşıyıcı elemanın kaybından kaynaklanan hasarın domino benzeri bir etkiyle yayılmasıdır. İlk hasar, binanın bir kısmının veya tümünün maruz kaldığı nihai hasarla orantılı değildir. En elvrişsiz durumda, ilk hasar tüm binanın tamamen çökmesine yol açabilir. İlerlemeli çökme temel olarak, bina kritik bir elemanın kaybından sonra yükleri yeniden dağıtamadığında meydana gelir. İstenen senaryoda, kritik bir taşıyıcı elemanın kaybından sonra, binanın stabil kalmasını ve hasarın diğer elemanlara yayılmamasını sağlamak için bina alternatif yük taşıma yolları bulmalıdır.

Bu çalışma, Türk Deprem Yönetmeliği'ne göre tasarlanmış yüksek bir binanın ilerlemeli çökme davranışını incelemektedir. Tipik bir 40 katlı bina üzerinde bir vaka çalışması yapılmıştır. Bina, çekirdeğinde perde duvarları olan düz bir plaka sistemine sahipti. Binanın taşıyıcı sistemi, X yönünde moment aktaran çerçevelerle perdelerden ve Y yönünde moment aktaran çerçevelerle perdelerden oluşuyordu. Tüm kolonlar için kare kesitler kullanılmıştır. Her katta 3 farklı yerden kolon kaldırıldı; köşe kolon , kenar kolon ve daha büyük iç kolon. Bina, 8 katlı 5 gruba ayrıldı. Aynı gruptaki tüm katlar aynı kolon ve kiriş kesit ölçülerine ve donatıya sahipti. Yüksek yapılarda bina ağırlığını ve maliyeti azaltmak için kullanılan yaygın bir optimizasyon tekniği olan bina yüksekliği boyunca her grup için kesit boyutları küçültüldü.

Binanın doğrusal olmayan bir modeli SAP2000 kullanılarak modellenmiştir. Yapılar için statik ve dinamik bir sonlu elemanlar analiz programı olan SAP2000 v23, üç boyutlu bir sonlu eleman modeli oluşturmak için kullanıldı. Modelde beton ve donatı olmak üzere iki ana malzeme tanımlanmıştır. Beton kalitesi C50 idi. Ancak, analiz mevcut bir bina üzerinde yapıldığından, karakteristik dayanımlar yerine beklenen dayanımlar kullanılmıştır. Betonun karakteristik basınç dayanımı, 65 MPa'lık beklenen bir dayanım elde etmek için 1,3 kat ile çarpılmıştır. Donatı akma dayanımı da 1.2 faktörü ile çarpılarak 504 MPa'lık beklenen akma dayanımı elde edildi. Bu

faktörler, mevcut binaların performansa dayalı analizi için Türk deprem yönetmeliğinden alınmıştır.

Perde duvarlar, donatı detaylarını daha doğru temsil edebilmek için layered shell elemanlar olarak modellenmiştir. SAP2000 v23, perde duvarda sargılı bölgesinin tanımlanmasına izin vermez, bunun yerine genel bir kesite izin verir. Sargılı bölgeyi temsil etmek için her bir duvar için iki malzeme kullanıldı: sargılı beton ve sargısız beton. Sargılı beton dayanımını bulmak için, sargılı bölge için bir kesit analizi yapılmıştır. Bu, sargılı bölgeyi kesit section designer'da bir kolon olarak tanımlayarak yapıldı. Sargılı beton basınç dayanımı, kesit analiz sonuçlarından alınmıştır. Bu basınç dayanımı kullanılarak malzeme tanımında yeni bir beton malzeme tanımlanmıştır. Bu malzeme, layered shell seçeneği kullanılarak duvar kesitinin sargılı bölgesinin tanımlanması için kullanılmıştır. İşlem tüm duvar kesitleri için yapılmıştır.

Döşemeler, shell thin elemanlar olarak modellenmiştir. Literatürde shell thin elemanların en iyi sonuçları verdiği bulunmuştur. Duvarlar ve döşemelerin meshleri tüm model için birbirine geçmiştir. Mesh boyutu, düz elemanlarda 50x50cm ve maksimum 100x100cm arasında olacak şekilde, mesh boyutu mümkün olduğunca tekdüze tutuldu. Duvarlar hem yatay hem de dikey yönde meshlendi. Özeliklæ perdelerin döşemelerle kesiştiği bölgelerde, meshlerin düğüm noktalarında süreklilik olmasına özen gösterilmiştir. Ancak, döşemelerin donatı ve doğrusal olmayan davranışlarının yakalanması bu çalışmanın kapsamı dışındaydı ve bu nedenle ayrıntılı olarak ele alınmadı.

Tüm çerçeve kirişleri aynı kesite ancak farklı donatı detaylarına sahipti. Kirişler, çubuk elemanları olarak tanımlanmıştır. Bu, default menüyü kullanmak yerine yapıldı. Bu, default menü enine donatı tanımına izin vermediği için yapıldı, bu nedenle sargı etkisi doğru bir şekilde yakalanmadı. Ayrıca, kirişin plastik davranışının çok önemli bir parçası olan moment eğrilik analizini vermek için default menüde kesit analizi yapmak mümkün değildir. SAP2000 v23'teki section designer menüsü ise, kullanıcının kirişlerin geometrisini ve donatı detaylandırmasını daha doğru bir şekilde temsil etmesine ve ayrıca kullanıcının ayrıntılı bir kesit analizi yapmasına olanak tanır. Betonun nihai basınç birim şekildeğistirme, deprem yönetmeliğe göre 0,018 olarak tanımlanmıştır. Modelin boyutu göz önüne alındığında, kiriş için fiber boyutları varsayılan değer olan 3x3 olarak bırakılmıştır. Onları daha ince yapmak daha doğru sonuçlar verirdi ama aynı zamanda model büyük olduğu için analiz süresini de uzatırdı.

Yapıdaki tüm kolonlar kare kesitlidir. Kolonlar da çubuk elemanları olarak tanımlandı. Default menü, kolonlar için enine donatı tanımına izin vermesine rağmen, kolon tanımı için yine section designer kullanıldı. Default menüde tanımlanan bir kolonu, Section designer'da tanımlanan bir kolonla karşılaştırmak için parametrik bir çalışma yapıldı. Ankastre mesnetli tek kolon üzerinde itme analizi yapılmıştır. Sonuç, kesit tasarımcısında tanımlanan kolonun, bu tür yük koşulları altında bir kolonun beklenen davranışına daha yakın olan daha sünek bir şekilde davrandığını göstermiştir. Section designer'da kolonların modellenmesi, daha sonra perfomans kriterlerinin hesaplanmasında kullanılacak olan kesit analizine de izin verdi. Tıpkı kirişlerde olduğu gibi, analiz süresini uzatmamak için fiber yerleşimi varsayılan değerlerde bırakılmıştır. Mander beton modeli, betonun sargılı mukavemetini yakalamak için kullanıldı.

Yükler döşemeler üzerine üniform yayılı yükleri olarak uygulanmıştır. Yüklerin değerleri TS498'den alınmıştır.

SAP 2000 sürüm 23'te birçok mafsal seçeneği mevcuttur. Kiriş ve kolonlarda hasar kontrolü yapılırken uçlardaki plastik dönmeler kontrol edilir. Plastik dönüşler, yığılı veya dağıtılmış plastisite modelleri kullanılarak yakalanabilir. Bu çalışmada yığılı plastik mafsal yerine fiber mafsal şeklinde dağıtılmış plastiklik modeli kullanılmıştır. Bunun nedeni, plastik mafsalların aksine, fiber mafsalların mafsalardaki moment, eksenel ve kesme kuvveti (M-N-V) etkileşimlerini dikkate almasıydı. Bu etkileşimin plastik mafsal kapasitesini düşürdüğü gösterilmiştir. Bu etkileşimin göz önünde bulundurulması, binanın daha doğru ilerleyen bir çökme tepkisinin gözlemlenmesine izin verir.

Doğrusal olmayan model tamamlandıktan sonra, ABD Genel Hizmetler İdaresi ve ABD Savunma Bakanlığı tarafından önerilen doğrusal olmayan dinamik alternatif yol yöntemi kullanılarak ilerlemeli çökme değerlendirmesi yapıldı. Dokuz kolon kaldırma senaryosu ve dokuz duvar senaryosu dikkate alınarak toplam on sekiz senaryo yapılmıştır. Her senaryo için, kaldırılan elemanın hemen üzerindeki düğümlerin düşey yer değiştirme zaman geçmişleri çizildi ve sonuçlara ilişkin daha fazla fikir edinmek için grafikler farklı kombinasyonlarda karşılaştırıldı. Plastik mafsalların oluşturulduğu durumlar için plastik mafsal dönüşleri hem TEC-2018 hem de ASCE/SEI 41-17'ye göre hesaplanan performans limitleri ile karşılaştırılmıştır.

Zemin katta kolonun köşeden ve kenardan kaldırıldığı senaryo için plastik mafsallar oluşturulmuştur. Bu iki durumda hasar sınırlıydı. Geri kalan senaryolarda mafsal gözlenmedi.

Bu çalışma için kullanılan bina, TEC-2018'e göre tasarlanmış yüksek bir binaydı ve kabul edilebilir ilerlemeli çökme tepkisine sahipti. Sonuçlar kolon ve perde duvarların kaldırılmasıyla yapı elemanlarında oluşan gerilmelerin yapı elemanlarında sınırlı düzeyde kaldığını ve yapısal bütünlükte herhangi bir bozulmaya neden olmadığını göstermektedir. Ancak, TEC 2018'e göre tasarlanan tüm yüksek binaların iyi bir ilerlemeli çökme tepkisine sahip olduğu sonucuna varmak yanıltıcı olabilir. Bu çalışmadaki sonuçlar sadece söz konusu bina için geçerlidir. Kesin bir sonuca varılmadan önce aynı çalışma farklı taşıyıcı sistemlere sahip farklı yüksek binalar için yapılmalıdır. TEC 2018'e göre tasarlanan yüksek binaların iyi bir ilerlemeli çökme tepkisine sahip olduğunun şüphe gölgesi olmadan kurulabilmesi için sayısal çalışmalara ek olarak, deneysel çalışmalara da ihtiyaç vardır. Şu anda Türkiye'de çok sayıda yıkım yapılıyor. Bu binalar yıkılmadan önce deneysel çalışmalar için kullanılabilir. İlerlemeli çökmeyi daha detaylı incelemek için laboratuvar ortamında ölçekli modeller de yapılabilir. Bununla birlikte, bu çalışma için kullanılan bina, TEC 2018'e göre tasarlanmış yüksek binalar için mükemmel aşamalı çökme tepkisi öneren sonuçlar verdi.

Bina düz levha sistemine sahip olduğu için döşemelerde hasar meydana gelmiş olabilir. Plakalardaki hasarların belirlenmesi basit bir iş değildir ve bu çalışmanın kapsamı dışındadır, dolayısıyla dikkate alınmamıştır. Bununla birlikte, meydana gelebilecek potansiyel kritik hasarlar için plakaların da kontrol edilmesi tavsiye edilir.

1. INTRODUCTION

1.1. Problem Statement

There is no single definition for progressive collapse. Different definitions can be found in the literature, but they allude to a common philosophy. Generally defined, progressive collapse is the spread of damage in a domino-like effect resulting from the loss of a vertical load-bearing element. The initial damage is not proportionate to the ultimate damage experienced by part of the building or the entire building. In the most extreme case, the initial damage can lead to the total collapse of the entire building. Progressive collapse basically occurs when the building fails to redistribute loads after the loss of a critical member. In the desired scenario, after the loss of a critical load-bearing element, the building should find alternative load-carrying paths to ensure that the building remains stable and the damage is not propagated to other members.

Progressive collapse can be caused by a wide range of sudden, unforeseeable impact loads. For example, vehicles, ship traffic, airplanes, and helicopters can cause huge forces over a very short period of time if they crush into buildings. This can cause the local failure of critical structural members that can lead to the progressive collapse of the whole building. A good example of this is the collapse of the World Trade Center in 2001. In this event, terrorists hijacked a commercial plane and crashed it into the building. The impact caused the ultimate collapse of the entire building.

Sudden impact loads can also result from gas explosions. This was the case at the Ronan Point Building in 1968. The Ronan Point Building was a 23-story building with a total building height of 64m. The building was a precast structure with wall panels connected by slots and bolts. The collapse was caused by a gas explosion that occurred on the 18th floor. The explosion blew out an exterior wall panel, which in turn caused the collapse of floors 19 to 23, which were directly above it. The top part of the building collapsed onto the bottom part, leading to the partial collapse of the building (NIST 7396, 2007).

Figure 1.1. Partial coll apse of Ronan Point building (Osteraas, 2006).

Huge loads that result from blasts like bomb explosions can also lead to progressive collapse. An example of this was observed in the collapse of the Alfred P. Murrah Federal Building. The building was a 9-story reinforced concrete building. In this incident, a car bomb detonated, resulting in the destruction of a base column that was supporting three other columns. This led to the overloading of the transfer girder, which ultimately led to the collapse of the building (Osteraas, 2006).

Figure 1.2. Partial collapse of Alfred P. Murrah building in the United States (Stephen, 2017).

Extreme fire can also lead to progressive collapse. Fire is a major cause for concern since structural members lose stiffness and strength over time when exposed to high temperatures. A typical case is the collapse of the World Trade Center is a major cause for concern since structural members lose stiffness and strength over time when exposed to high temperatures. A typical case is the collapse of the World Trade Center. This was a 47-story building located close to the twin towers. An interior column was heated, and consequently, there was a significant loss in gravity load-carrying capacity. This led to the collapse of the entire building.

Progressive collapse can also be caused by gross human error. A classic example of this is the collapse of the Zumrut building in 2004. The Zumrut building was a residential building constructed in Turkey. The building collapsed, killing 92 people. After investigations were done, it was found that there were huge errors made during construction. The strength of the concrete used was less than the minimum code provisions. The transverse reinforcement and reinforcement cover did not comply with the code provision. In addition, errors were found in the design project. There were also alterations that were made to the original approved drawing. For example, the approved plans called for 12 cm thick slabs at the roof level, but instead 32cm ribbed

Figure 1.3. Partial collapse of Zumrut building in Türkiye (Balkaya, 2017).

slabs, which were two times heavier, were constructed. All this and the lack of proper supervision and control over the project led to the collapse of the building (Balkaya, 2017).

Progressive collapse can occur through different mechanisms. Starossek (2007) proposed six mechanisms in his study on the typology of progressive collapse in 2007. According to this study, there is pancake-type collapse, zipper-type collapse, domino-type collapse, section-type collapse, instability-type collapse, and mixed- type collapse.

A good example of the pancake-type mechanism was observed in the World Trade Center towers (the twin towers). After impact, the airplane caused damage that was limited to a few stories but extended across the entire cross-section of the tower. The upper part of the tower separated from the bottom part and fell directly onto the bottom part in a rigid vertical motion. The potential energy of the top part was subsequently changed to kinetic energy as it collapsed onto the bottom part, which was still intact. Upon impact with the bottom part, the bottom part experienced large impact forces, which led to the loss of vertical load-carrying capacity and the collapse of the entire building. The features of the pancake-type of collapse can be summarized as follows: there is the failure of vertical load-carrying elements, then there is partial or complete separation and a collapse in a vertical rigid motion of the top part onto the bottom part of the building; potential energy is converted to kinetic energy; separated elements fall

vertically onto the remaining structure, leading to failure of the remaining structure due to high impact loads; and the collapse is in the vertical direction. Typically, the large impact loads act on the members directly impacted, which also fail and fall onto the members below them, and this continues until the complete collapse of the building. In essence, the direction of the impact load and the direction of collapse are parallel (Starossek, 2007).

Figure 1.4. Attack on the World Trade Centre in the United States (Stephen, 2017).

The zipper-type collapse is most likely to occur in cable-stayed bridges. Cable- stayed bridges can experience the loss of one or more cables, which can lead to the ultimate collapse of the bridge. This was observed in the collapse of the Tacoma Narrows Bridge. The first hangers of the bridge ruptured due to wind-induced oscillations of the girder, which caused the girder to fall off. The features of the zipper-type collapse can be summarized as follows: there is a failure of one or more structural members; loads carried by these members are redistributed throughout the remaining structure; the remaining structure experiences impulsive loading due to the loss of the structural member; the remaining structure responds dynamically to the impulsive load; there is a concentration of forces in the members that are similar to the failed member and also in the members close to the failed member; the combined static and dynamic effects lead to the failure of the remaining members; the collapse is in the transverse direction of the principal forces in the failing members; This mechanism differs from the

pancake mechanism since impact loads do not typically occur and the direction of the principal forces in the failing members is not parallel to the direction of collapse but rather orthogonal.

The mechanism of the domino-type mechanism can be summarized as follows: there is the initial failure and overturning of the structural element, which then falls in an angular rigid motion around one of its bottom edges. The potential energy of the member is converted to kinetic energy, and there is a lateral impact of the upper edge of that member with an adjacent member, resulting in a combined static and dynamic load. The hit object then overturns due to the horizontal loading from the impacting member, leading to the eventual collapse of the whole building in the overturning direction. This type of mechanism can occur in structures where the structures that are at risk of overturning are placed repetitively in a horizontal arrangement. An example of such structures is a row of temporary scaffolding towers. The structures are slender, and they are not braced, which makes them prone to overturning.

The section-type mechanism can be described using a beam under bending moment or a bar under axial tension. When part of the beam or bar is cut, there is a redistribution of the forces across the section, which can lead to damage in other parts of the cross section due to increased stress. This kind of failure is usually called fast failure, but it was included under progressive collapse to make use of the similarities and analogies (Starossek, 2007).

Instability-type collapse occurs when the members that stabilize load-carrying members fail, leading to the members losing stability. This can occur when braces that keep members like columns, beams, or trusses stable fail due to small trigger events. Failure of stiffening plates can lead to local failure, which can spread to the whole member. The mechanism can be summarized as follows: the member that stabilizes the member in compression fails, the member in compression becomes unstable, and there is a sudden failure of the unstable member, which progresses to the collapse of the entire structure.

Other collapse mechanisms that cannot be clearly distinguished using the previous five mechanisms can be classified as mixed-type collapse mechanisms. A good example of

such a mechanism is the collapse of the Alfred P. Murrah Federal Building. The explosion happened about four meters from the column, which was destroyed. The blast wave from the bomb explosion also caused significant damage to the beams and the floors, which left other columns with no lateral support and unstable. A combination of all these extreme loading conditions led to the progressive collapse of the building (Starossek, 2007).

1.2. Progressive Collapse in Codes and Guidelines

In 1970, after the collapse of the Ronan Point building in London, the British Standards (BS) became the first code to address the issue of progressive collapse. The BS 6399 gives a general outline for the design of structures against progressive collapse. On the other hand, BS 5950, BS 8110, and BS 5628 give specific provisions for the design of structures to resist progressive collapse. The British standards require that members be tied at about the same horizontal and vertical elevation. After that, all the members are checked for local damage that can lead to disproportionate collapse. The code also gives the load combination in equation 1.1. The alternate path method is referred to as bridging design (BS 6399, 1996).

$$D + W/3 + L/3 \tag{1.1}$$

Where D is the dead load, W is the wind load and L is the imposed load.

In 1975, the Canadian codes incorporated provisions to ensure that structures could withstand abnormal loading conditions. This move was also hugely influenced by the collapse of the Ronan Point Building. The code basically sought to give regulations on the minimum tensile forces in the ties and the resistance of the members after the loss of a structural member. However, the Canadian code does not give an explicit procedure to design against progressive collapse, nor does it explicitly mention progressive collapse; rather, it addresses the issue of structural integrity (Stephen, 2017).

The American Concrete Institute (ACI) 318 structural integrity provisions were also developed in 1989, with the most recent version being published in 2019 (ACI 318,

2019). Like the Canadian code, the provisions do not explicitly address the issue of progressive collapse but rather aim to provide structural integrity using ties.

The American Society of Civil Engineers also incorporated requirements for structural stability and strength under abnormal loads. Specifications are discussed in general in ASCE 7-05 to reduce the risk of progressive collapse, but no explicit requirements are given. More details on structural integrity are discussed in the commentary to ASCE 7-05 (ASCE 7, 2005). The commentary gives the load combination in equation 1.2, where D is the dead load, L is the live load, S is the snow load, and W is the wind load.

$$(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S) + 0.2W \quad (1.2)$$

The commentary also suggests both direct and indirect methods to mitigate progressive collapse. Indirect methods consider that ensuring continuity, ductility, and a minimum strength in the structural members will reduce the likelihood of progressive collapse occurring. On the other hand, the direct methods are more specific, like the alternate load path method and the specific local resistance method.

The Eurocodes (EN) do not address the issue of progressive collapse explicitly. However, it makes provisions for accidental actions on a building. This is outlined in detail in EN 1991-1-7, but there are other aspects also included in EN 1990 and EN 1991. The EN 1991-1-7 (2006) gives rules for designing structures that can resist accidental actions, although it does not deal specifically with actions caused by external explosions, warfare, or terrorist activities. The code also proposes some strategies to design structures to resist accidental loading. The strategies are shown in the flow chart in figure 1.5.

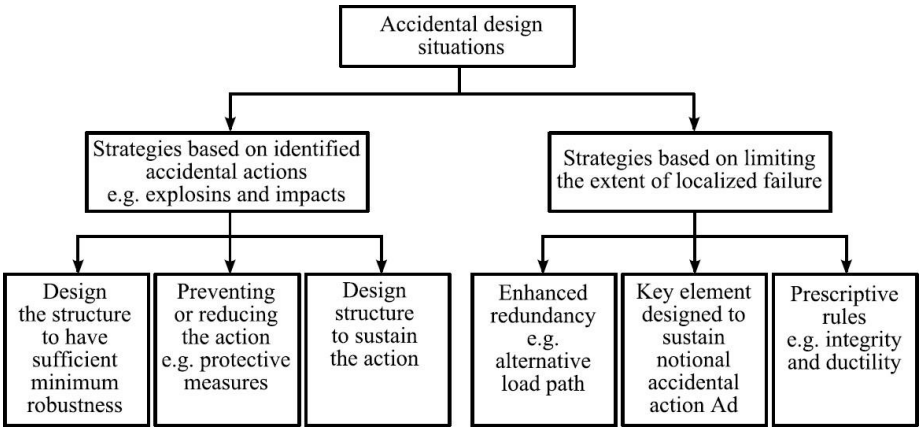

Figure 1.5. Strategies proposed in the EN 1991-1-7 for accidental design situation.

The General Services Administration (GSA) 2016 is one of the most up-to-date guidelines that was specifically designed to address the issue of progressive collapse. The GSA-2016 was developed to assess the risk of progressive collapse in new federal buildings in the United States. The design of key elements and the use of ties are not included in the guidelines. The guideline proposes the use of both linear and nonlinear approaches, although much emphasis is put on linear approaches. This is because of the complexity associated with nonlinear methods; hence, nonlinear approaches are only briefly mentioned without much detail. Performance criteria for nonlinear methods are given in the guidelines. In order to determine whether a building needs extra strength to reduce the risk of progressive collapse, the GSA-2016 gives a flow chart method. Several factors are considered to determine the need for progressive collapse design, including building occupancy, building height, seismic zone, facility security level (FSL), and other structural features. So basically, the guidelines aim to assess the risk of progressive collapse in new buildings.

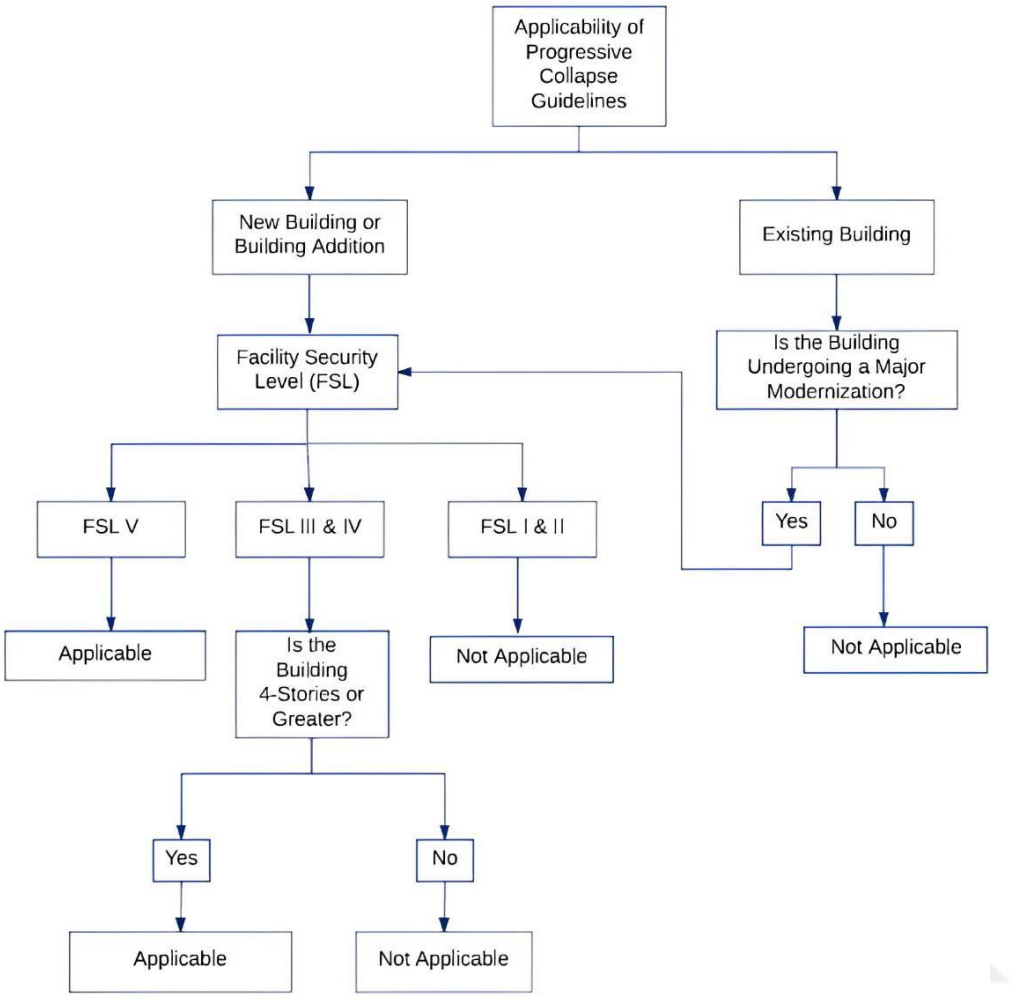

Figure 1.6. Applicability flow chart (GSA-2016, 2016).

Two chapters are used to explain the linear procedure in detail. The linear procedure is applicable for buildings with no more than 10 stories. Taller buildings must be designed according to the nonlinear approach, which is not explained in detail in the guideline. The following load combinations are proposed in the assessment of progressive collapse risk:

$$Ns = 2(DL + 0.25LL) \tag{1.2}$$

$$Nd = (DL + 0.25LL) \tag{1.3}$$

Where Ns is the static load combination, Nd is the dynamic load combination, DL is the dead load and LL is the live load.

The Unified Facilities Criteria (UFC 4-023-03) was developed by the US Department of Defense (DoD) to provide the design requirements to reduce the risk of progressive collapse. They are the most commonly used guidelines, along with the GSA-2016, because they provide detailed procedures for building design against progressive collapse. The first DoD guidelines were published in 2001, with the latest version published in 2016. The guidelines propose two main approaches: providing tie forces and using the alternate load path method. Providing ties is based on the catenary response of the structure, while an alternate load path is based on the ability of the structure to bridge over the element that has failed. The guidelines outline four protection levels that are used in determining the procedure used. The levels are Very Low Level of Protection (VLLOP), Low Level of Protection (LLOP), Medium Level of Protection (MLOP), and High Level of Protection (HLOP). For VLLOP and LLOP, indirect methods can be applied, whereas the alternate path method is used together with the tie force method for MLOP and HLOP. Most of the new and existing buildings fall within the VLLOP and LLOP, so using only the tie force method is sufficient. The UFC 4-023-03 was used for this study, and more details on the procedure are given in the following chapters (UFC 4-023-03, 2016).

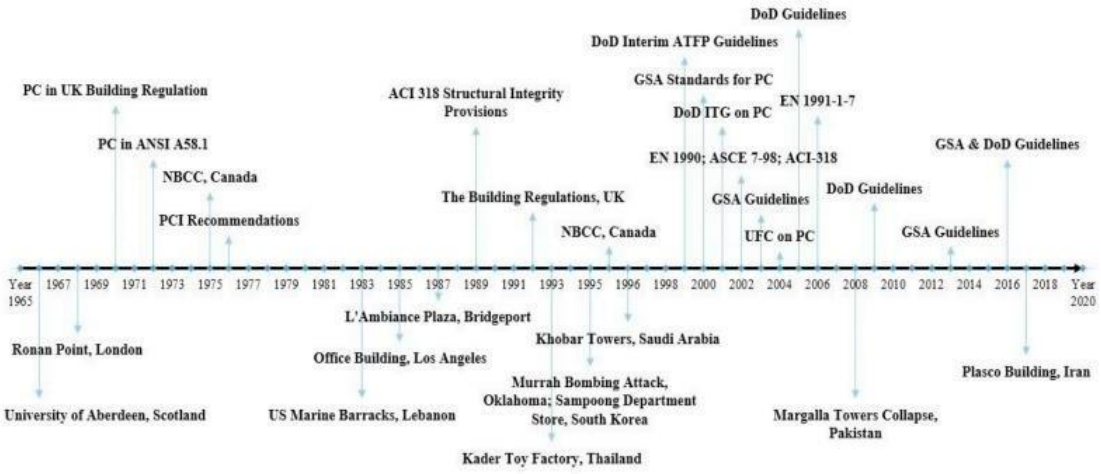

Figure 1.7. Timeline of development of key regulations and guidelines on progressive collapse (Azim et al. 2020).

1.3. Similar Studies in The Literature

Interest in the subject of progressive collapse has been increasing in the academic community. Many studies have been undertaken on the subject using both numerical and experimental methods.

Tavakoli (2013) performed a numerical study in which a new approach was proposed for the dynamic removal of columns. The study was performed on a 5-story steel-framed building. According to the study, the location of the removed column determines the risk of progressive collapse. The authors also concluded that the method they proposed for the dynamic removal of columns offers more practicality and computational simplicity. In another study, Tsai and Lin (2008) investigated the progressive collapse resistance and inelastic response of an earthquake-resistant RC building. According to this study, buildings designed to resist earthquake loads generally have a low potential for progressive collapse. When the progressive collapse response of low and mid-rise buildings designed according to ASCE 41-17 was investigated, it was observed that no significant damage occurred on the building (Gammniouk, 2014; Betit, 2021; Sheik et al., 2021).

Not many studies have been done on the progressive collapse response of tall buildings. Marchis (2019) performed a numerical assessment of the progressive collapse resistance of RC frames with respect to the number of stories. This study and many other studies found that the progressive collapse (PC) response of buildings improves with the increase in the number of stories. This is mainly because taller buildings have more structural members that can provide an alternative load path. Marchis (2019), also proposed a simplified approach to determine the ultimate load that frame elements can carry without failure. Ren et al. (2014) performed a PC assessment on a typical high-rise building with shear walls. They found that the PC response of buildings with strong walls and weak frame systems was generally inadequate compared to buildings with strong frames and weak wall systems. Gamaniouk (2014) performed a parametric study of progressive collapse in high-rise buildings. Different models with different heights and structural systems were analyzed. The study showed that the ability of a building to resist progressive collapse

is greatly influenced by the number of stories. It was observed that for all the structural systems considered, the taller models showed more resistance to progressive collapse. It was also observed that the number of hinges formed decreased significantly with the increase in the number of stories. Thai et al. (2020) also performed a study on the progressive collapse response of high-rise buildings. The alternative path method was implemented using nonlinear time history analysis on a model designed according to the Chinese building codes. The effects of section size, inter-module connections, and bracing systems on the progressive collapse response of the building were investigated parametrically. This study shows that by distributing redistributed loads and constraining structural parts, bracing systems can improve structural resilience. Thai et al. (2020) came to the additional conclusion that corner members are more important due to the lack of nearby members to share the redistributed load. Tian et al. (2021) explored a new system to design super-tall seismic-progressive collapse-resilient buildings. A 48-story building with a total height of 192 meters, designed according to Chinese building codes, was used for this study. The system proposed was based on the common "frame-truss-core" system. A vibration reduction substructure, seismic-progressive collapse robust composite frames, a composite braced tube with self-centering energy dissipation braces, and truss systems with outriggers and belt trusses were the major components of the suggested system. The suggested approach successfully managed the reaction to column loss and earthquakes.

There are many studies that have been conducted on PC worldwide. However, only a few studies have been done for buildings designed according to the Turkish earthquake code. Demir (2022a) performed a progressive collapse evaluation for low-rise buildings designed for different occupancy classes. The study showed that government buildings have better performance than residential buildings since they are generally designed to be more robust. In another study, Demir (2022b) also evaluated the progressive collapse response for buildings designed according to the Turkish Earthquake Code. The study suggested that there is a need for collapse response evaluation for buildings designed according to the Turkish earthquake code (TEC, 2018). The studies made by Demir (2022a, 2022b, and 2022c) were all performed on low- and mid-rise buildings.

1.4. Thesis Objective and Scope

Türkiye has one of the most modern seismic design codes. It frequently updates its seismic code depending on recent scientific advances due to being in an earthquake-prone zone. However, it also has no special explicit design or evaluation methods against the progressive collapse (PC) of structures built in the country. The last update of the Turkish earthquake code was released in 2018 (TEC, 2018). Like its contemporaries, it is expected to improve structural integrity during an extreme event by providing sufficient structural robustness. Because it also does not include any explicit design prescriptions to reduce the risk of progressive collapse, the response of the buildings designed according to TEC-2018 should be investigated to observe their performance under any extreme event. The studies in the literature for this objective are very scarce, specifically for reinforced concrete (RC) buildings. According to the best knowledge of the author, there is no study in the literature investigating the progressive collapse response of reinforced concrete high-rise buildings designed according to the Turkish earthquake code.

The objective of the study is to investigate the progressive collapse response of reinforced concrete high-rise buildings designed according to the Turkish earthquake code. In this way, the study will make a significant contribution to the literature. A high-rise RC building was designed initially according to TEC-2018. The 40-story high-rise building was numerically modeled using a finite element model in SAP 2000 version 23. The nonlinear dynamic alternate path method specified in the US General Services Administration (GSA-2016) and the US Department of Defense (UFC 4-023-03) was used to evaluate the progressive collapse response of the building. The acceptance criteria were determined according to the TEC-2018 and the ASCE/SEI 41-17. By performing 18 case studies in the finite element program, a preliminary study on the progressive collapse behavior of high-rise buildings designed according to the requirements of the Turkish earthquake code is added to the literature. Based on the results obtained from the analyses, a preliminary assessment of the adequacy of the provisions of the Turkish earthquake code in mitigating the risk of progressive collapse in high-rise buildings is made, and recommendations and discussions are presented accordingly.

2. APPROACHES USED FOR PROGRESSIVE COLLAPSE ANALYSIS

Progressive collapse analysis hasn't made it into many design codes around the world. Currently, the most used guidelines are the GSA-2016 and the UFC 4-023-03. These two guidelines were developed to specifically address the subject of progressive collapse. Existing codes and guidelines do not explicitly make provisions for progressive collapse mitigation. Instead, this issue is addressed implicitly if at all in the building codes. However, as more research is done on this subject, it is becoming more apparent that precautions must be taken to avoid progressive collapse, especially in government buildings. These two guidelines offer a detailed framework that can be implemented to mitigate PC. The UFC 4-023-03 proposes 3 different approaches: alternate path, enhanced local resistance, and tie forces.

2.1. Tie Forces Method

The tie-forces method involves mechanically connecting the building, which improves continuity, ductility, and the creation of alternative load paths. The structural components already in place, which were created using traditional design techniques to support the structure's typical loads, can act as tie forces. Longitudinal, transverse, and peripheral ties are the three types of horizontal ties that must be offered. Columns and load-bearing walls must have vertical ties. These ties are shown in Figure 2.1. as part of the frame structure. These "tie forces" are distinct from "reinforcement ties" according to the ACI 318 Building Code Requirements for Structural Concrete definition. The longitudinal, transverse, and peripheral tie forces are to be carried by the floor and roof system unless the structural members (beams, girders, and spandrels) and their connections can be demonstrated to be capable of carrying the required magnitudes while undergoing rotations of 0.20 rad (11.3 deg) (UFC 4-023-03, 2016). Precast concrete floor boards with concrete topping, reinforcing, and mechanical

anchorage that satisfies the standards of Sections 3-1.4 through 3-1.6 are acceptable floor and roof systems, as are cast-in-place concrete, composite decks, and these types of structures. Other floor or roof systems are acceptable as long as they can support the necessary tie strength during rotations of 0.20 rad (11.3 deg) and are properly proven to and approved by an independent third-party engineer or by a designated facility owner representative.

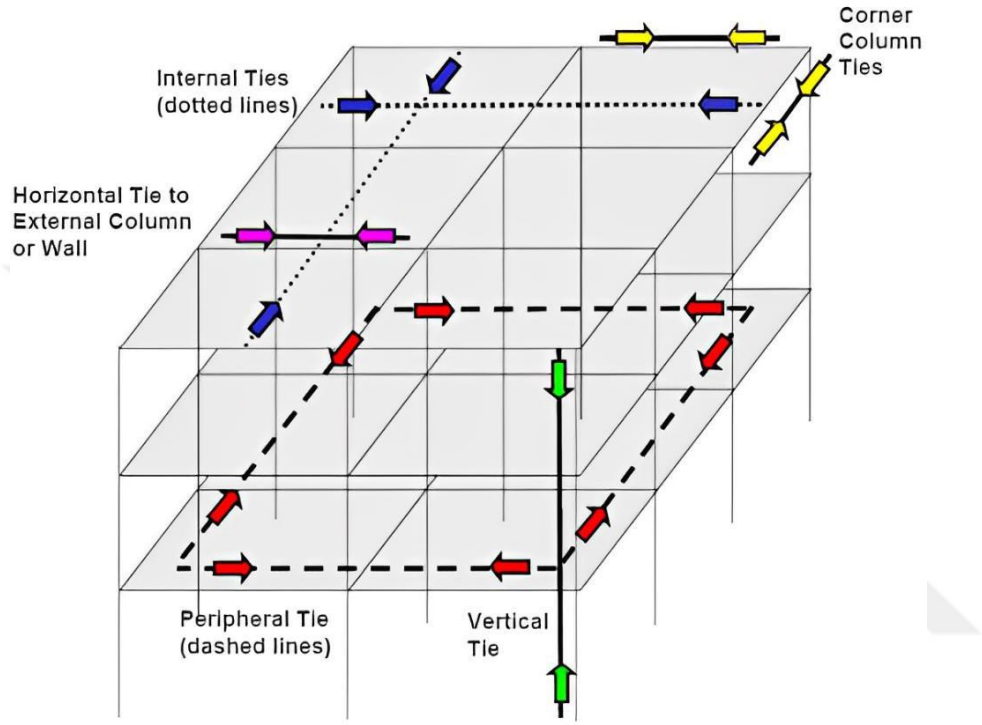

Figure 2.1. Tie forces in a frame structure (UFC 4-023-03, 2016).

2.2. Enhanced Local Resistance (ELR)

This approach can be used in three cases as outlined in the UFC: RC II Option 1 (Tie Forces and ELR), RC III (Alternate Path and ELR), and RC IV (Tie Forces, Alternate Path, and ELR). In other words, the Enhanced Local Resistance (ELR) approach is meant to be used together with either the Alternate Path method or the Tie Forces approach, or both. The aim of implementing this approach is to ensure that when a column or wall experiences a lateral load, the failure mechanism is ductile. This can be achieved only if the column or wall reaches its maximum flexural strength before

it reaches its maximum shear strength. The column or wall and the connection between the column or wall and the lateral supports must meet the ELR requirement (UFC 4-023-03, 2016)

2.3. Alternate Path Method

In this method, the Load and Resistance Factor Design (LRFD) approach outlined in ASCE/SEI 7-16 (2016) is applied with a few modifications. In this method, there are three analysis procedures proposed that are based on ASCE/SEI 41-17 (2017), the general approach. The approach in ASCE/SEI 41-17 is modified to more accurately address specific issues associated with progressive collapse. In addition, some deformation and strength criteria from the UFC 4-023-03 are used instead of the criteria given in the ASCE/SEI 41-17. The three analysis procedures proposed are linear static (LSP), non-linear static (NLS), and non-linear dynamic analysis (NLD).

2.3.1. Linear static procedure

The linear static procedure is a simple approach to implementing the alternate path method. However, there are some limitations associated with this approach. It can only be applied to structures with relatively simple layouts. In other words, the structure should either have none of the irregularities outlined in the UFC 4-023-03 or, in the event that there are irregularities in the structure, the demand capacity ratios in the UFC 4-023-03 cannot be exceeded.

2.3.1.1. Irregularity limitations

There are 4 cases outlined in the UFC 4-023-03 where a structure is considered an irregular structure.

The first case is when there are significant discontinuities in the structural system that carries both the lateral and vertical gravity loads. This can be in the form of out-of-plane offsets in columns or shear walls or in the cases when columns are not placed directly on top of each other but rather placed on the beam span.

The second case is when the two bays on either side of an exterior column have a significant difference in either stiffness or strength or both. That is when the ratio of strength and/or stiffness of the bay on the opposite sides of the column is less than 0.5. This does not apply for corner columns. The ratio can be less than 0.5 because of the difference in the bay lengths, a significant difference in beam depth or strength on either side, or the difference in connection stiffness of the beams on either side of the column. This condition must be satisfied on all the floors.

The third case is like the second but for walls. This is when the ratio of stiffness of intersecting external walls that are load bearing is less than 0.5. This does not apply to the corner.

The fourth case is when the columns, walls, and any other lateral load-resisting elements are not parallel to the major orthogonal axes. This can be the case when the moment frames are skewed or curved (UFC 4-023-03, 2016).

2.3.1.2. Demand capacity ratio (DCR) limitations

In the event that the building has any irregularities, the DCRs have to be calculated. In order to calculate the DCRs, a linear model of the building is created, comprising only primary structural elements. Secondary elements are not included in the model as they cannot redistribute forces. The column or wall is then removed from the building, after which the load case with increased gravity and live loads is applied. The load combination is shown below, where is the amplification factor, usually taken as 2. D is the dead load; L is the live load; and S is the snow load.

$$\mathrm{WF} = \Omega[1{,}2\mathrm{D} + (0{,}5\mathrm{L}) \text{ OR } \mathrm{WF} = \Omega[1{,}2\mathrm{D} + (0{,}2S)] \quad (2.1)$$

Using this load combination, the forces and stresses are determined as the ultimate demand. The DCR is the ratio between this demand (Qudlim) and the expected strength of the member (Qce). This is summarized in the following expression:

$$\mathrm{DCR} = \mathrm{Qudlim}/\mathrm{Qce} \quad (2.2)$$

If the building satisfies the irregularity and the DCR conditions, the linear static procedure may be used. In that case, the demand forces outlined in UFC 4-023-03 that factor in ductility and shear or flexure failure are factored in. The members must be able to meet the demands of the factored loads in order for the building to resist progressive collapse. If they do not meet the demands, the members are redesigned accordingly, and the procedure can be repeated.

2.3.1.3. Linear static analysis procedure

The analysis requires a P-Δ analysis or should be a second order analysis. The procedure can be summarized as follows:

The column or wall is removed, then the load is applied. The analysis is performed, after which the forces and deformations that form in the members are compared to the acceptability criteria given in Table 3-1 in the UFC 4-023-03, 2016, as shown on Figure 2.2. The table gives a generic outline of deformation-controlled actions and force-controlled actions. If all the forces and deformations are in line with the actions outlined in Table 3, the analysis can be considered complete, and it means the building performs in a satisfactory manner against progressive collapse.

Component	Deformation-Controlled Action	Force- Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V[1]
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	P, V, M[2]	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame construction.
2. Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.

Figure 2.2. Deformation-Controlled and Force-Controlled actions (UFC 4-023-03, 2016).

If any of the members violates the criteria outlined in Figure 2.2, the geometry or material properties of the member are modified. The model is then reanalyzed starting from the unloaded state until the load is fully applied. After the full load is applied, the damage states of all the members are checked. If the damage states are violated, the member must be redesigned again. When all the members do not violate the damage states, the member forces and deformations are then compared to the acceptability criteria. If both the damage limits and the acceptability criteria are not violated, the design can be considered satisfactory

2.3.2. Nonlinear static procedure

The nonlinear static procedure has no limitations on DCR or geometric irregularity. The procedure is almost the same as the linear static procedure; however, in the nonlinear static procedure, plastic hinges are added to the elements in order to capture the nonlinear behavior. Only three-dimensional models can be used. The model should be created using the stiffness requirements outlined in ASCE/SEI 41-17 (2017). The force displacement behavior of all the elements must be modeled in detail and should represent strength degradation and residual strength where applicable. The designer can use engineering judgment to decide the location of the plastic hinges, but it is recommended that the hinges be placed at the points of the members where stresses are likely to be highest. Typically, hinges are likely to form at the ends or the midspan of a beam and at the ends of a column. The hinges form when the behavior of the element changes from elastic to inelastic. The hinges allow for rotation without any significant increase in the bending moment. This type of behavior represents the actual behavior that occurs during progressive collapse. The ASCE/SEI 41-17 gives a typical hinge behavior shown in Figure 2.3 that changes with the type of component and the material properties of that component. The hinge properties outlined in ASCE/SEI 41-17 are used by both the UFC 4-023-03 and the GSA-2016 for nonlinear procedures in the alternate path method.

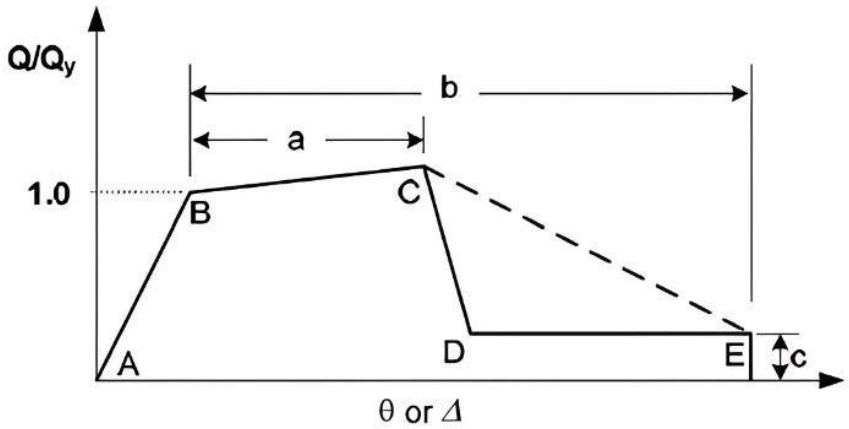

Figure 2.3. Generalized plastic hinge model

The plastic hinge behavior is represented as the relationship between the moment capacity and the rotation of the element. The vertical axis on the generalized plastic hinge definition represents the ratio between the load applied and the yield capacity of that member, while the x axis represents the rotation at a location on the member. Parameters a and b represent the post-yield behavior, while c represents the residual strength ratio of the hinge.

After the plastic hinge definition is done, loads are applied using the same load combination used in the linear static procedure. The load should be applied using a load history from zero until the model is fully loaded. The minimum load steps are 10, and the model must converge before the next load step is applied.

Just as in the linear static procedure, the demand capacities on the members are checked, but this time the hinges formed in all the members have to be checked. The behavior of the hinges must be checked against the acceptability criteria for the collapse prevention or life safety performance levels in the ASCE/SEI 41-17 and the UFC 4-023-03. If any of the members exceed the acceptance criteria, it must be redesigned, and the procedure is repeated until the limits are satisfied

2.3.3. Nonlinear dynamic procedure

The nonlinear dynamic procedure is widely considered the most accurate procedure, and it can be used even for complex systems since there are no geometric or DCR

limitations to its application. It is very similar to the linear static procedure, with the only difference being the dynamic loading conditions. It also requires significantly more computational power and time compared to the previous two methods. It is also very sensitive, as minor changes in assumptions can lead to totally different results. However, it is still the most accurate method, according to the literature.

Only 3-dimensional models can be used in this procedure, and the inclusion of secondary elements in the model is at the discretion of the designer. The stiffness and resistance of all primary members should be accurately represented according to ASCE/SEI 41-17 requirements from Chapters 9 to 15. Care should be taken to apply strength reduction factors to the nonlinear strength of deformation-controlled components. The nonlinear behavior must be accurately modeled to capture the force deformation behavior and the strength degradation and residual strength, if applicable.

The model is loaded in an optimum manner to best represent the actual loading condition. The load should be distributed evenly throughout the model. Lumped masses are not allowed unless they represent equipment or other similar items. The same load combination used in the previous two approaches is used. This is a two-step process. The first step is the analysis of the model under gravity loads. The loading is done monotonically and proportionately increasing the load from zero until the entire gravity load has been applied and the model has reached equilibrium. When equilibrium is reached, the column or the wall is then removed. It is preferable to remove the column or wall instantaneously; however, the UFC 4-023-03 recommends that the duration of the removal be less than one tenth of the period associated with the mode shape for the vertical motion of the bays above the removed column. After the removal of the column or wall, the analysis is continued until the displacement stabilizes or becomes constant, which is to say there is almost no oscillation of the joint directly above the removed column or wall. Maximum displacement usually occurs during the first cycle, but in buildings that show nonlinear behavior, the displacement can continue increasing before the oscillations stabilize. If the displacement continues to increase, then it is most likely that progressive collapse has occurred. After the analysis, a comparison is made between the element and connection forces and deformations and the acceptability criteria that are generalized

in Figure 2.2 of the UFC 4-023-03. If all the members comply with the outlined limits, the analysis is complete, and the structure shows resistance to collapse prevention. However, if any of the members violates the acceptance criteria, the member has to be redesigned and the analysis process repeated again until all the requirements are met. In the alternate path method, great care should be taken to ensure beam continuity after column removal, as illustrated in Figure 2.4.

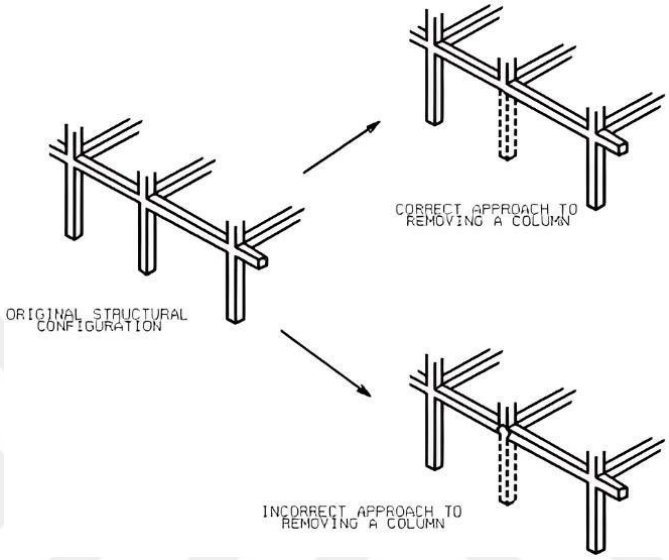

Figure 2.4. Removal of column using the alternate path method UFC.

3. BUILDING DESCRIPTION AND MODELING

3.1. Building Description

The building used in this study was designed as a typical residential tower with a symmetric floor plan. The building location was chosen to be in Istanbul at coordinates 40.99N,28.64E with the soil classified as ZC, which is composed of very tight layers of sand, gravel, and hard clay or weathered, highly fractured weak rocks is composed of very tight layers of sand, gravel, and hard clay or weathered, highly fractured weak rocks. Using these coordinates, the spectral acceleration coefficients for the ZC soil class were obtained from the AFAD website for a DD-2 level earthquake, which is ground motions with a 10% probability of exceeding in 50 years and a recurrence period of 475 years.

The building was designed using a DD-2 ground motion and a strength-based design approach to satisfy the life safety (LS) performance criteria.

The building has 40 floors with the same floor height of 3.2 m for all normal floors and 3 m for the roof floor. A flat plate system was used for this building with perimeter beams. The building's structural system consisted of coupled structural walls combined with moment-resisting frames in the X direction and uncoupled structural walls combined with moment-resisting frames in the Y direction.

Table 3.1. Design parameters for the building.

Parameter	**Value**
Soil class	ZC
Soil shear velocity	500 m/s
Building usage class	3
Building height class	1
Seismic design class	1

Table 3.1. (Continued) Design parameters for the building

Ductility level	High
Ground Motion Level	DD-2
Short period (S_{DS})	1.466
1s period (S_{D1})	0.492
Response modification (R)	8
Overstrength coefficient (D)	2.5

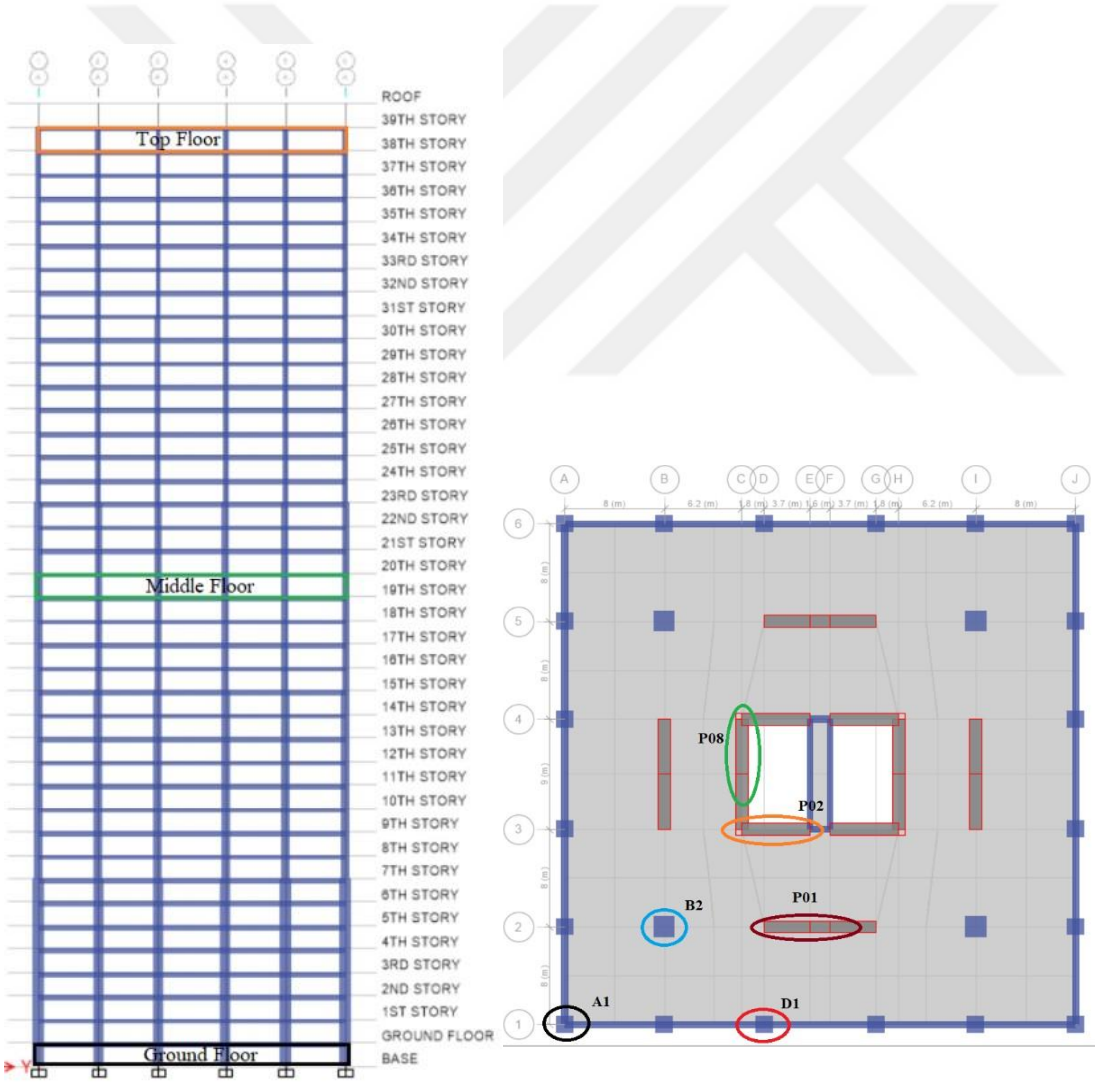

Figure 3.1. Building elevation (left) and plan view (right), (Orakcal, 2022).

Square sections were used for all columns. Each floor had 3 different sections: corner column section, edge column and the larger inner column. The building was divided into 5 groups of 8 floors each. All floors in the same group had the same column and beam section sizes and reinforcement. The section sizes reduced for each group along the height of the building which is a common optimization technique used to reduce building weight and cost in tall structures.

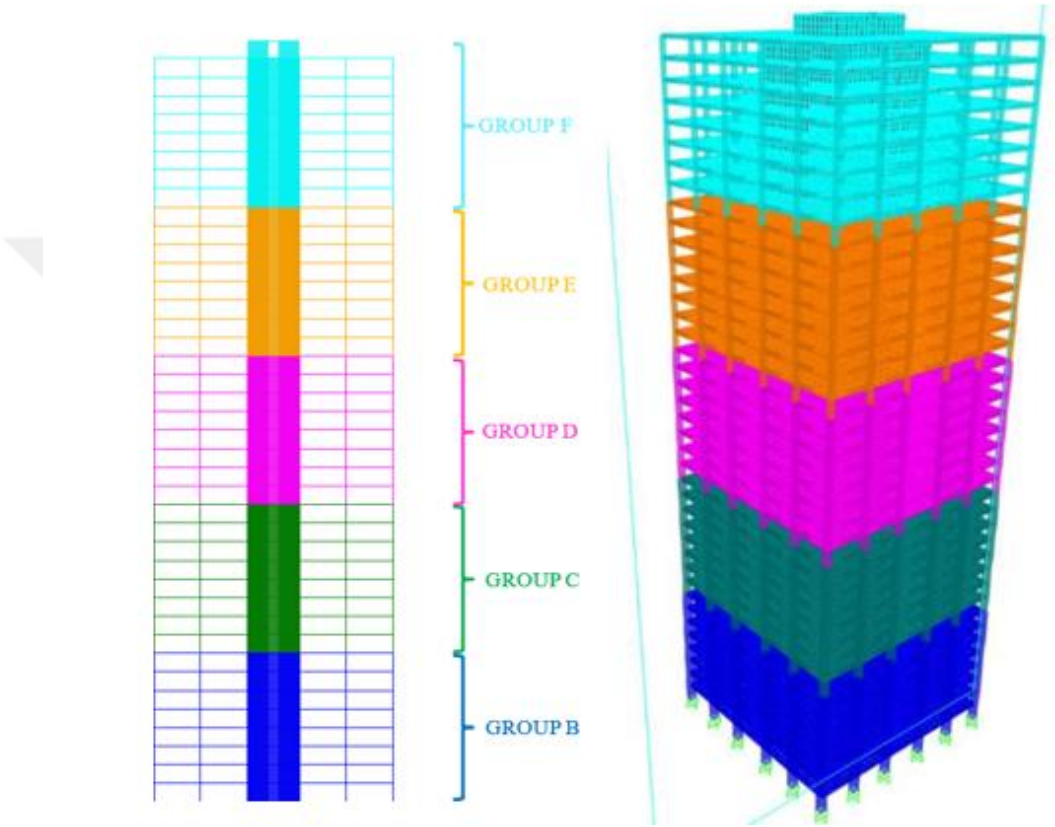

Figure 3.2. Group assignment (left) and 3d finite element model (right).

The same section size was used for all the perimeter beams (50x70 cm), but the reinforcement was different according to the analysis results. The coupling beams were designed with diagonal reinforcement and with a larger section than the perimeter beams (50x80 cm). The structural walls were designed with confinement at the ends of the wall for both C-shaped walls and straight walls. The wall thickness also decreased for each group along the height of the building. The same slab thickness (30cm) was used for all floors. The summary of the section sizes is shown in Table 3.2.

Table 3.2. Cross sectional dimensions of structural members.

	Group B	**Group C**	**Group D**	**Group E**	**Group F**
Perimeter Columns	C140X140	C120X120	C100X100	C80X80	C70X70
Interior Columns	C170X170	C150X150	C130X130	C110X110	C90X90
Perimeter Beams	B50X70				
Secondary Beams	B40X60				
Coupling Beams	B50X80				
Structural Walls	100cm	90cm	80cm	70cm	60cm
Tower Slab	30cm				
Core Wall Slab	15cm				

Table 3.3. Material properties and assigned loads.

Materials		
Concrete grade	:	C50
Modulus of elasticity of concrete	:	37000MPa
Loads		
Self-weight of concrete	:	25.0kN/m^3
Ceiling plaster + Floor finishings + Installments	:	2.0kN/m^2
Infill walls for residential floors	:	1.0kN/m^2
Residential floor live load	:	2.0kN/m^2
Snow load	:	1.0kN/m^2

3.2. Numerical Modeling

SAP2000 v23, a static and dynamic FE analysis program for structures, was used to generate a three-dimensional finite element (FE) model (Figure 3). Since the research was done on an existing building, expected strength was used during material definitions. The C50 concrete was defined as having a compressive strength of 65 MPa, while the S420 rebar was defined as having a 504 MPa yield strength.

Structural walls were modeled as layered shell elements to represent the reinforcement details more accurately. SAP2000 v23 does not allow the definition of the confinement region in the structural wall but rather allows for a general section. In order to represent the confined region, two materials were used for each wall: confined concrete and unconfined concrete. To find the confined concrete strength, a section analysis was done for the confined region. This was done by defining the confined region as a column in the section designer. Confined concrete compressive strength was taken from the section analysis results. Using this compressive strength, a new concrete material was defined in the material definition. This material was used during the definition of the confined region of the wall section using the layered shell option. The process was performed for all the wall sections.

Slabs were modeled as shell-thin elements. In the literature, it has been found that shell-thin paper gives the best results. However, capturing the reinforcement and nonlinear behavior of slabs was outside the scope of this study, and thus it was not addressed in detail.

Columns and beams were modeled as frame elements. In order to represent the reinforcement more accurately and to capture the nonlinear behavior more accurately, the sections were modeled in the section designer menu of SAP 2000 v23.

3.3. Beam and Column Nonlinear Behavior Modeling

To predict the post-yield inelastic behavior of structural load-bearing components, two modeling techniques are used: concentrated (lumped) and continuum plasticity models as shown on Figure 3.3. The lumped plasticity model assumes that deformation above

the elastic limit happens only in discrete areas, with the remaining part of the member remaining elastic. Integrating plastic strain and curvature occurring in a predetermined hinge length will give the inelastic behavior (Demir, 2022c).

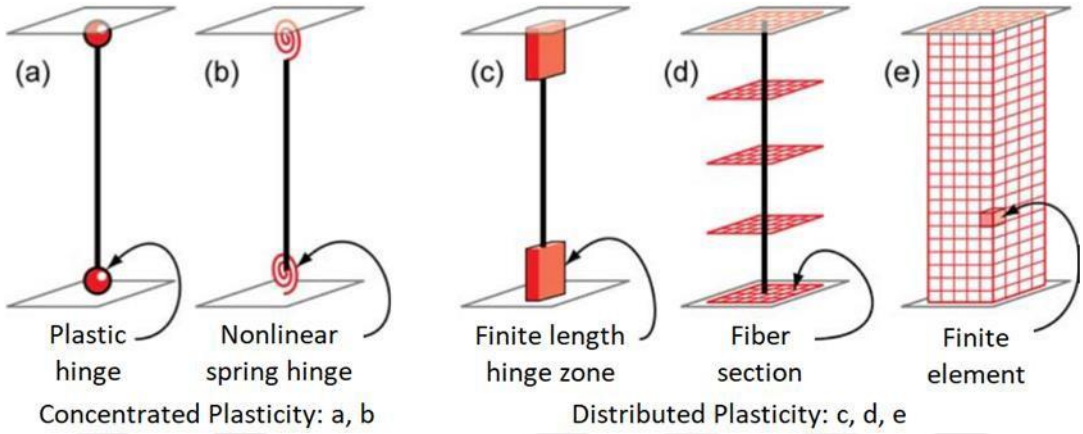

Figure 3.3. Beam-column nonlinear modeling models (ASCE/SEI 7-16, 2016).

Nonetheless, in a distributed plasticity model, the cross-section of a member is discretized into a sequence of representative axial fibers that extend throughout the element or together with a limited-length hinge zone (fiber hinge) (Figures. 3.3c, 3.3d, and 3.3e). Each fiber must have its own stress-strain relationship. Finally, by integrating the behavior throughout the section and multiplying by the hinge length, the axial force-deformation and biaxial moment-rotation correlations are derived.

RC frame structures may withstand excessive collapse by creating two crucial load-resisting processes on their members: Vierendeel (arching) Action and Catenary Action. While fiber components can successfully capture such mechanisms in addition to the flexural behavior of frame members, the usual concentrated plastic hinge technique ignores those aspects (Demir, 2022a). As a result, the nonlinear behavior of the structural load-bearing components was simulated in this study utilizing nonlinear fiber hinges, one of the approaches of the distributed plasticity approach.

4. SAP2000 MODELING PROCEDURE

The nonlinear dynamic method was chosen for this study since tall buildings already fall under a special category, and this method is best suited for this type of building. SAP2000 v23 was the software used in implementing this method, as this version includes a new menu that highly simplifies the analysis. In this chapter, a detailed description will be given for all the modeling procedures up until the implementation of progressive collapse in SAP 2000 version 23 and the collection of results.

The model was created in such a way that nonlinear behavior can be captured accurately. Since the nonlinear dynamic procedure is very sensitive, great care was taken to model all the elements to represent actual members with the geometry and reinforcement as accurately as possible.

4.1. Material Definition

In the model, two main materials were defined, namely, the concrete and the rebar. The concrete grade was C50. However, since the analysis was being done on an existing building, expected strengths were used instead of the characteristic strengths. The characteristic compressive strength of the concrete was multiplied by a factor of 1.3 to give an expected strength of 65 MPa. The yield strength of the rebar was also multiplied by a factor of 1.2 to give an expected yield strength of 504 MPa. These factors were taken from the Turkish seismic provisions for performance-based analysis of existing buildings.

In addition to these two materials there was also definitions of other materials used in the definition of the walls which is discussed in detail in the following sections.

4.2. Section Definitions

The building has a flat-plate structural system with perimeter beams. So, the building basically consists of the perimeter beams, coupling beams, shear walls, and slabs. The sections were defined to accurately capture the nonlinear behavior of all the structural

elements. In order to achieve this, the sections had to be defined in a way that captured the reinforcement and geometric properties as accurately as possible. This was done using tools already available within SAP 2000 version 23.

4.2.1. Beam definition

All perimeter beams had the same cross section but different reinforcement details. Beams were defined as frame elements. This was done instead of using the default menu. This was done because the default menu does not allow the definition of transverse reinforcement, hence the effect of confinement is not accurately captured. In addition, it is not possible to perform section analysis in the default menu to give the moment curvature analysis, which is a crucial part of the plastic behavior of the beam. The Section Designer menu in SAP2000 v23, on the other hand, allows the user to more accurately represent the geometry and reinforcement detailing of the beams and also allows the user to perform a detailed section analysis. The ultimate compressive strain of the concrete was defined as 0.018 according to the TEC-2018. Considering the size of the model, the fiber sizes for the beam were left at the default value of 3x3. Making them finer would give more accurate results, but it would also increase the analysis time since the model was big. A typical section and the fiber layout are shown in Figure 4.1.

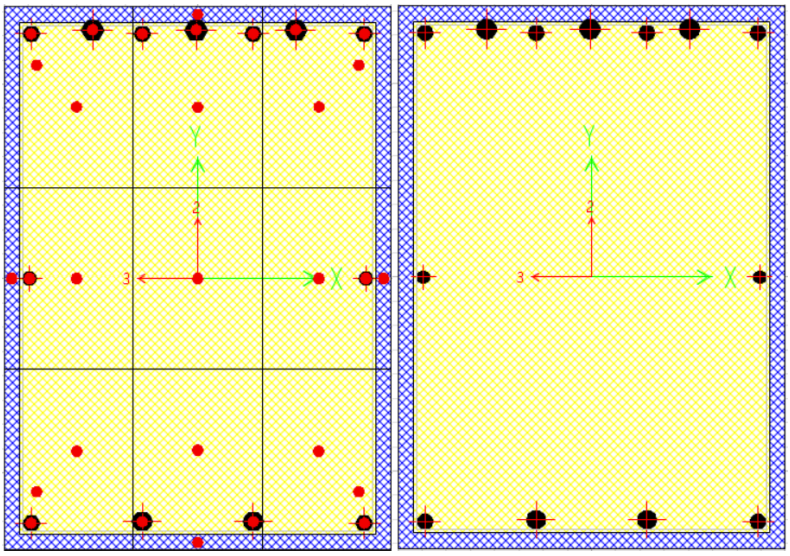

Figure 4.1. Typical beam section showing fiber layout (left) and not showing fiber layout (right).

4.2.2. Column definition

All the columns in the building had a square section. Columns were also defined as frame elements. Although the default menu allows the definition of transverse reinforcement for columns, section designer was again used for column definition. A parametric study was done to compare a column defined in the default menu to a column defined in Section Designer. A push-over analysis was done on a single column with a fixed support. The result showed that the column defined in the section designer behaved in a more ductile manner that is closer to the expected behavior of a column under such load conditions. Modeling the columns in the section designer also allowed for section analysis, which would later be used in the calculation of acceptance criteria. Just like in the beams, the fiber layout was left at default values so as not to increase the analysis time. The Mander concrete model was used to capture the confined strength of the concrete. A typical column section and fiber layout in section designer are shown in Figure 4.2.

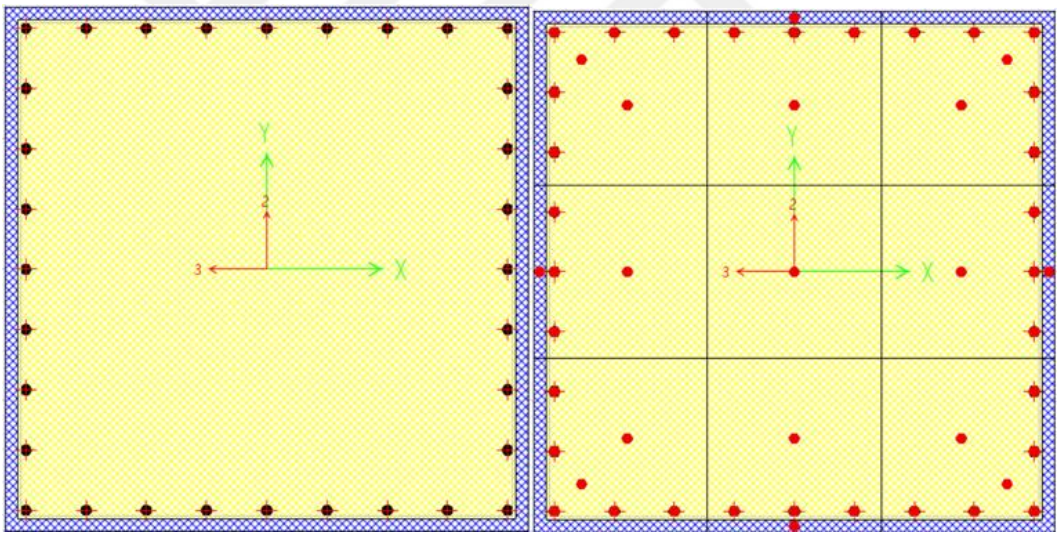

Figure 4.2. Typical column section not showing fiber layout (left) and showing fiber layout (right).

4.2.3. Wall definition

Walls can be modelled as shell-thin, shell-thick, or layered shell elements. In the literature, studies found that shelling out shell-thin gives reasonable results. However, a layered shell gives a more accurate representation of the wall's behavior, although it is a bit more complicated to implement. All the walls in the building have confinement at the ends. In a quest to find a simpler way to model the walls, a wall modeling approach was tried. In this modeling approach, the confinement region of the wall was

modeled as a column section designed in section designer, while the web region was modeled as a shell thin element. The confinement region was joined to the web using rigid link elements. An illustration of the parametric model is shown in Figure 4.3.

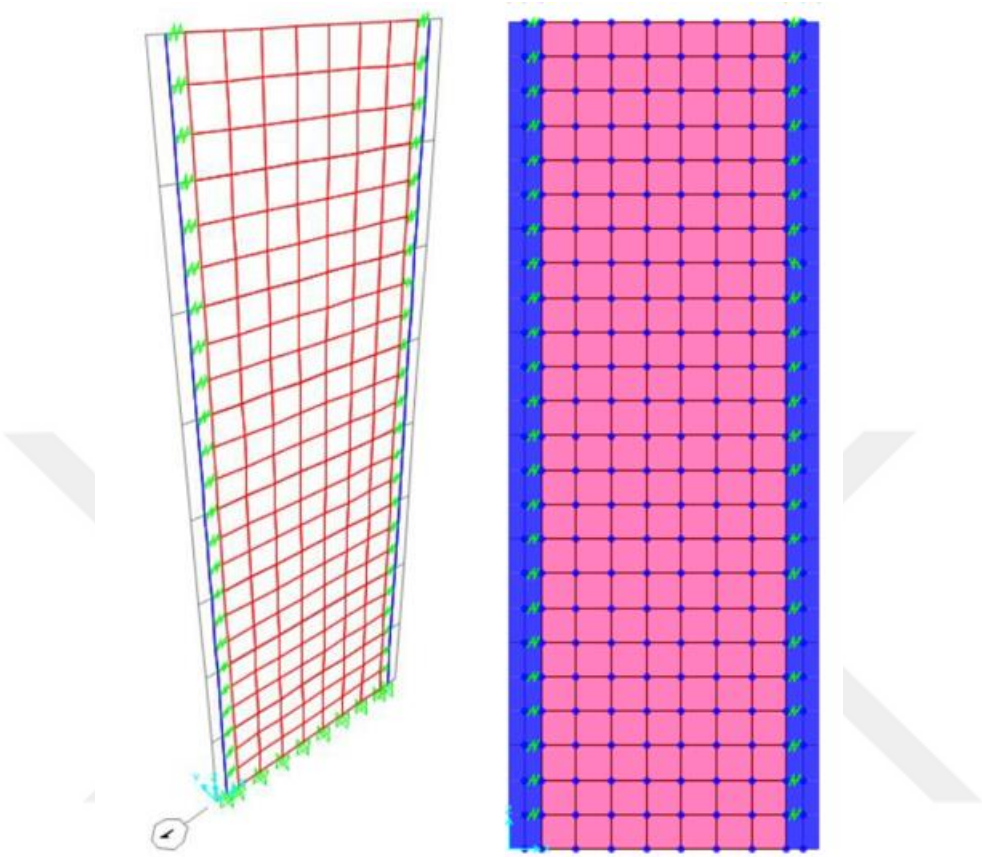

Figure 4.3. Illustration of the shear wall parametric model.

Another model was also made using layered shell elements. A pushover analysis was done on both models under the same loading conditions. The column-wall model showed extremely rigid behavior, and even the period values of these two models were completely different. Given the fact that there are no previous studies on this modeling approach and that it is also equally difficult to implement the layered shell model used for the walls.

The layered shell menu in SAP 2000 v23 has a major limitation. It does not allow the user to draw a section like in Section Designer. That means that the same reinforcement details apply for the entire length of the wall, which is a problem for walls with confinement regions. Even if different wall sections are defined for the confinement region and the web region, it’s not possible to define the cross ties in the

confinement region. The layered shell model only allows for the definition of transverse reinforcement parallel to the length of the wall and not perpendicular to the length of the building. That means that the confinement is not accurately represented.

In order to solve this issue, the confinement regions of all the walls were modeled as frame elements in Section Designer. Section analysis was done on these sections in order to determine the confined concrete strength for all the confinement regions. After that, new concrete materials were defined using the confined concrete strengths for each of the confinement regions. These concrete models were used during the definition of the corresponding walls. In summary, the walls were defined in 3 parts: the i confined region, the web, and the j confined region. Separate wall sections were defined for the three parts.

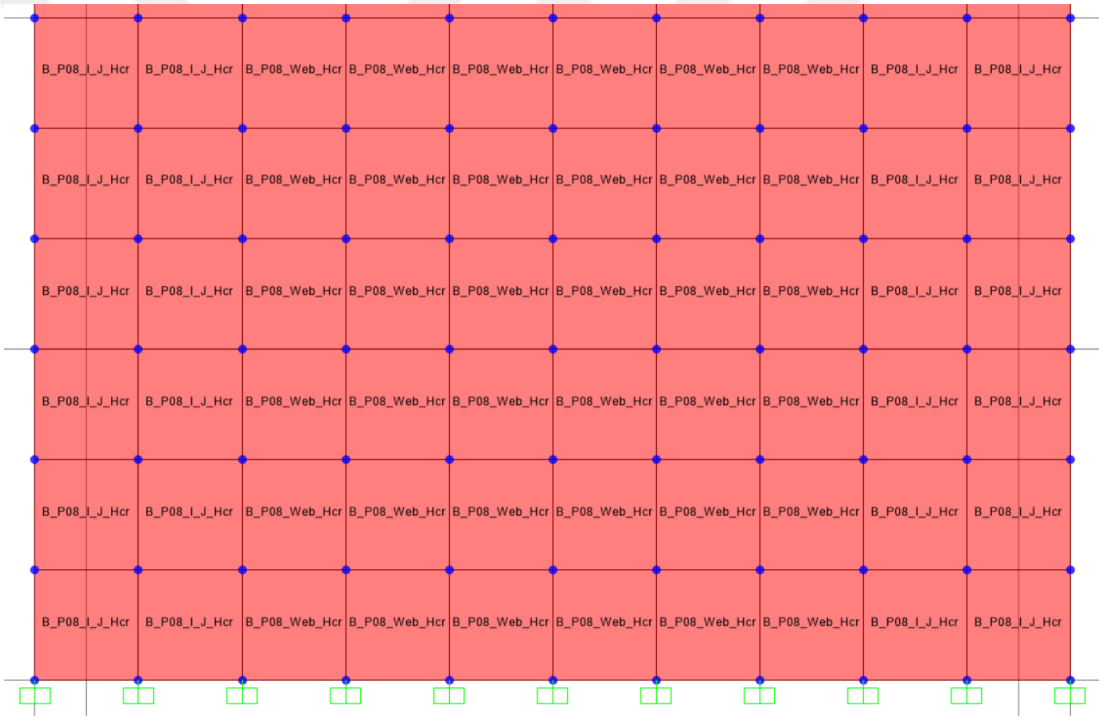

Figure 4.4. Illustration of section assignment on shear walls.

In order to place these sections into the model, first a default wall section was drawn at the wall location. After that, the wall was meshed in such a way that the confinement region could be separated from the web region. The confined region wall sections were then assigned to the fibers in the confined region, while the web wall section was assigned to the web of the wall. Figure 4.4 shows an illustration of a wall with the sections assigned.

4.2.4. Slab definition

Slabs were defined as shell-thin elements. This is because modeling slabs to capture nonlinear behavior is a complex subject that is out of the scope of this study. There are studies on the modeling of slabs for non-linearity, and most of them involve modeling complex finite element models, which would significantly increase the computational demands and analysis time if applied to the model used in this study. However, it is important to note that the slabs are an important issue worth looking at in more detail.

4.3. Meshing

The walls and slabs were meshed for the entire model. The mesh size was kept as uniform as possible, with the mesh size being between 50x50cm and a maximum of 100x100cm on the slabs. The walls were meshed both in the horizontal and vertical directions. Special care was taken to ensure that there was continuity at the nodes of the meshes, especially in regions where shear walls intersected with the slabs.

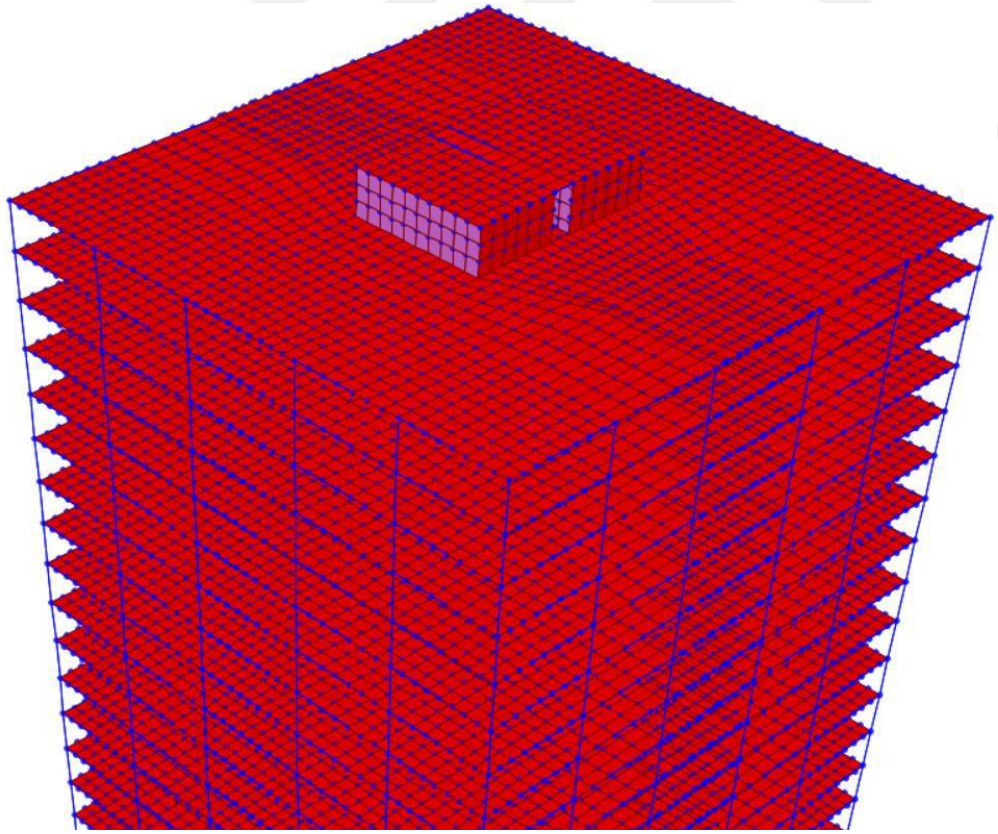

Figure 4.5. Meshing on slabs and wall sections.

Meshing the walls and slabs enables the reading of stresses and forces on the walls more accurately after an analysis is done, and it also allows the model to more accurately represent the behavior of the actual building.

4.4. Loading

Loads were applied as uniform area loads on the slabs. The values of the loads were taken from TS498. The loads applied are summarized in Table 2.3.

4.5. Hinge Definition

There are many hinge options available in SAP 2000 version 23. When checking for damage on the beams and columns, plastic rotations at the ends are checked. The plastic rotations can be captured by using either concentrated or distributed plasticity models. In this study, a distributed plasticity model was used in the form of fiber hinges instead of concentrated plasticity hinges. This was because, unlike plastic hinges, fiber hinges consider moment, axial, and shear force (M-N-V) interactions at the hinges. This interaction has been shown to lower the plastic hinge capacity (Demir, 2022a). Considering this interaction allows for the observation of a more accurate progressive collapse response of the building.

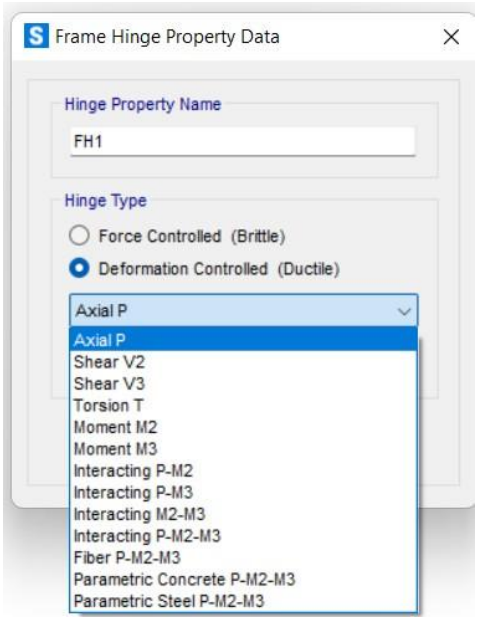

Figure 4.6. Hinge options available in SAP2000 v23.

To incorporate fiber sections in SAP 2000 v23, the sections were initially discretized using an ideal fiber arrangement. The program automatically allocated fibers to the center of each reinforcement, as well as confined and unconfined concrete that was meshed into many square or rectangular sections. The appropriate fibers were afterwards assigned the material characteristics of both concrete and reinforcing bars.

Finally, in a finite-length hinge zone, the fiber hinges were specified as having half the section depth at both ends of the beams and columns (Demir, 2022a). Furthermore, concrete and reinforcing steel constitutive material models were built using the material models provided in TEC 2018. Fiber hinges were assigned at the ends of beams and columns.

4.6. Progressive Collapse Implementation

4.6.1. Current practice

There are several approaches to implementing dynamic analysis in the literature. The column removal must be carried out dynamically, and the guidelines do not recommend the use of dynamic amplification factors. This means the designer must find a way of including the dynamic effects in the modeling process. This can be achieved through two main approaches. The designer can choose to either directly delete the element from the model or use reaction forces in place of the element. The approach of directly deleting the element is not easily implementable in most finite element software. Fu (2009) used this approach in a study on the progressive collapse analysis of high-rise buildings with the 3-D finite element method. In this study, he used the *Remove command in Abaqus to remove the columns.

The reaction forces approach is the most used approach in the literature. Aydin used this approach in a study performed on the progressive collapse of reinforced concrete buildings. In this approach, before the column is removed, the forces acting on the column at equilibrium are first calculated. The forces are calculated using a nonlinear static load combination as defined in UFC 4-023-03 and shown in equation 3.1. These forces are considered the starting conditions for the column removal scenario. The column is then removed from the model, and the forces are then assigned to the node directly above the removed column as concentrated loads. By doing this, the model now represents the loading state of the building after a column has failed. However,

these loads are not all applied instantaneously but rather gradually over time to better represent the actual real-life member loss scenario. In order to do this, a new load pattern is defined, and the loads are applied to the node using opposite signs to the column forces. In SAP 2000 v23, this gradual loading can be done using a ramp function as implemented by Demir (2022b). The total time taken to gradually load the node, or in other words, remove the column, is given as one tenth of the fundamental period of the building in the UFC 4-023-03. In Demir's study, the gradual loading started 0.5 seconds after the model reached equilibrium. After the column was completely removed, the analysis continued for a period of time to allow the model to stabilize, bringing the total removal time to 3 seconds. An example of the ramp function is shown in Figure 4.7. Using the first and second periods of the building, a 5% Rayleigh damping was defined (Demir, 2022b).

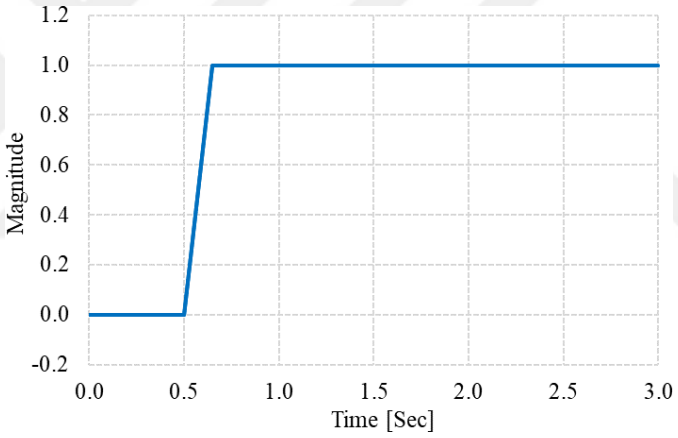

Figure 4.7. Ramp function example (Demir, 2022b).

The Newmark integration method was used with gamma and beta coefficients of 0.5 and 0.25, respectively. This is generally the approach used in the literature where SAP 2000 v23 is used. However, depending on the software used, the procedure can have a few differences, though the concept remains the same. The approach just described is generally complicated if the designer is not properly guided, and it becomes even more difficult when removing structural walls.

4.6.2. Method used in this study

In this study, a new simple method was implemented. The SAP 2000 version 23 update added a new option that allows for easy column and wall removal. In addition to the conventional load cases like the dead and live load cases, a gravity load case was

defined. This load case was defined as a nonlinear static load case. P- delta effects were considered for this case, and the load combination in equation 3.1 was applied. The initial conditions were set to start from zero, that is, from the unstressed state. Figure 4.8 shows the parameters used for the gravity load case.

After this, a new column removal load case was defined using the new option added to SAP2000 v23. This load case was defined as a nonlinear time history using direct integration as the solution method. P-delta effects were also included, considering large displacement, and the option to consider collapse was also activated. Instead of starting the loading from zero as in the gravity load case, the loading continues from the end of the nonlinear gravity load case. SAP2000 v23 now allows the user to select the frame, shell, or group element they want to remove. For columns, object type was selected as frame, and the frame ID was defined.

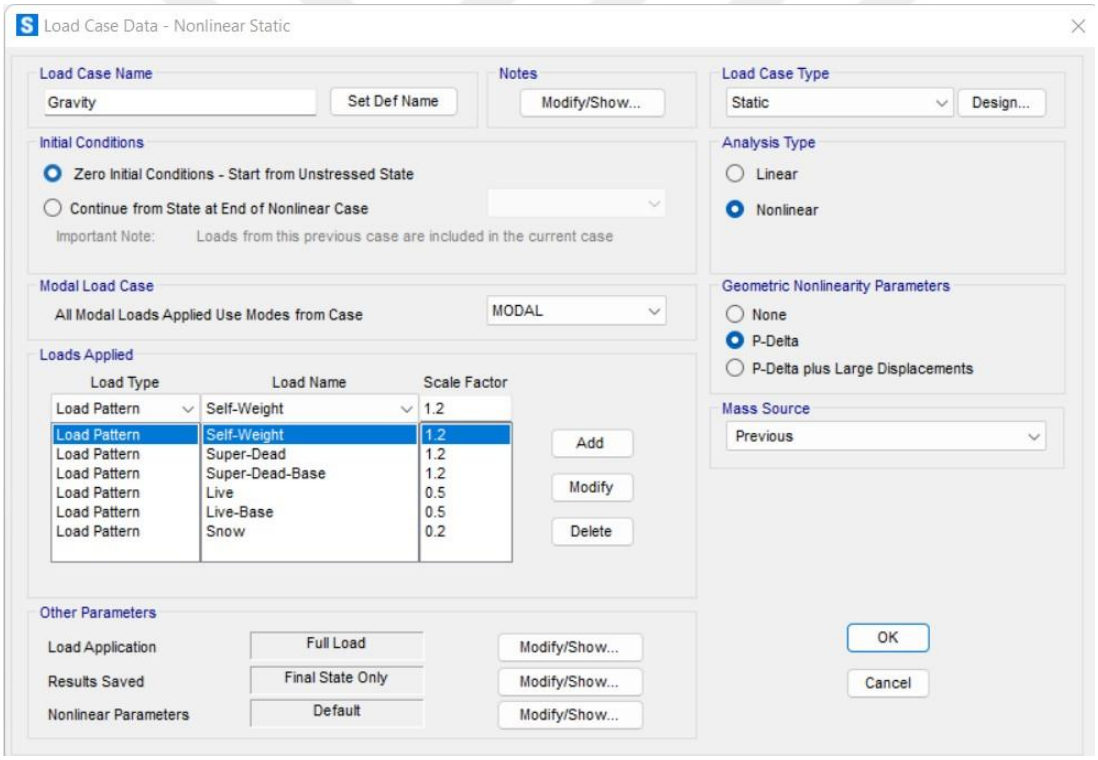

Figure 4.8. Gravity load case definition.

For walls, the program allows the user to input all the shell IDs of the wall to be removed. Since the wall is meshed, there would be more than one shell ID to be added. In this model, the number of shells after meshing was done on both slabs and walls was substantial, so it was not practical to select the specific shell ids to be removed from all the shell elements in the whole model. Instead, the shell elements to be

removed were assigned to a group. This group was then removed in the element removal load case.

The column or wall was set to be removed 0.5 seconds after the nonlinear gravity load case, and the duration of the removal time was taken as one tenth of 0.325 since the fundamental period of the building was 3.25 seconds. The total removal time, including the time before and after the member is removed, was defined as 5 seconds divided into 500 steps of 0.01 seconds. Direct integration damping was used. The mass proportional coefficient and stiffness proportional coefficient were calculated by the program using the first and second periods and the damping ratio. The first period was defined as the fundamental period, and the second period was defined as the first period where the modal participation ratios for the mode in the x and y directions and the torsion mode are all more than 90%. For this building, it occurred at 0.2082. Since the building was a tall building, the damping ratio was taken as 2.5%. For the time integration, the Newmark integration was used, with the gamma and beta coefficients taken as 0.5 and 0.25, respectively. After setting the analysis option to 3-dimensional analysis, the analysis was done.

This procedure was done for 18 scenarios, which were implemented on 18 different models. All the models were exactly the same, with the only difference being the element removal case. Details on the scenarios considered are given in the following chapter. Figure 4.10 shows a visual representation of the model before and after column removal. The removal of the edge column from the ground floor was used as an example. The shear wall removal scenario model illustration is shown in Figure 4.11.

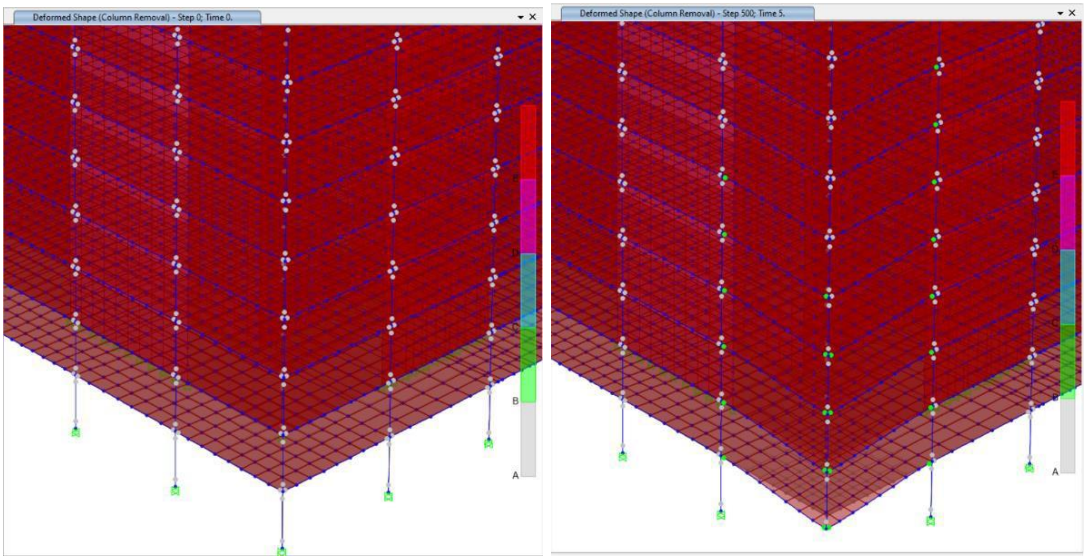

Figure 4.9. 3D illustration of column removal scenario before (left) and after (right) column removal.

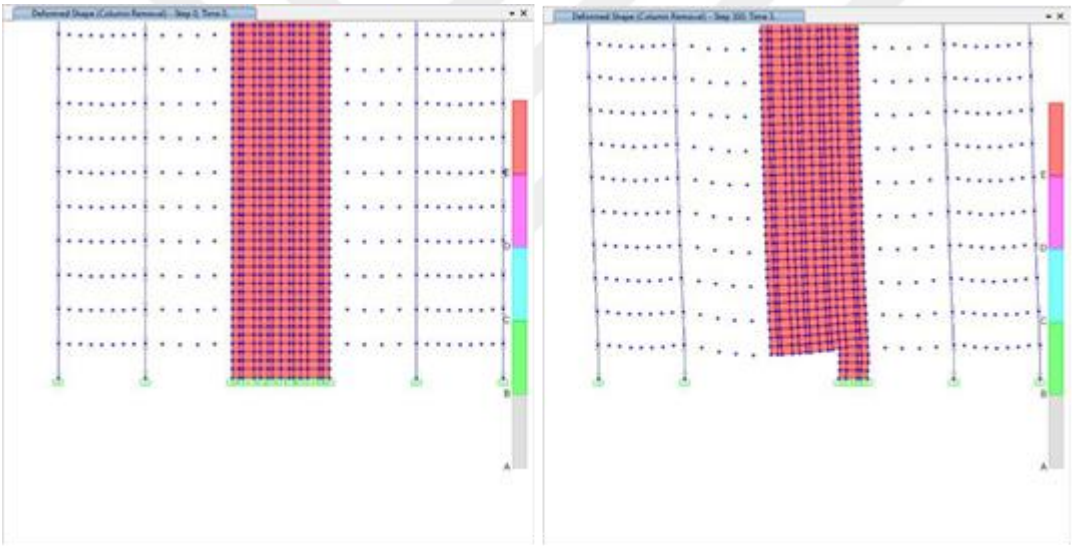

Figure 4.10. Illustration of shear wall removal scenario before (left) and after (right) column removal.

4.7. Collecting Results

The joint displacement time histories were collected using the show plot functions option in SAP2000 v23. The menu allows the user to choose the type of time history they want and specify the joint ID and the direction of the displacement. After the time history is plotted in the program, it is then exported as a text file for further processing using tools like Excel or Python. These tools allow the user to compare different scenarios, as done in this study.

In the cases where hinges were formed on the frame elements, the beam to be checked was selected, and then using the show hinge results menu, the plastic rotations were copied from the program for both ends of the beam. It is also possible to read the strain at any desired fiber. However, plastic rotations were checked for this model since it was a tall building with many members.

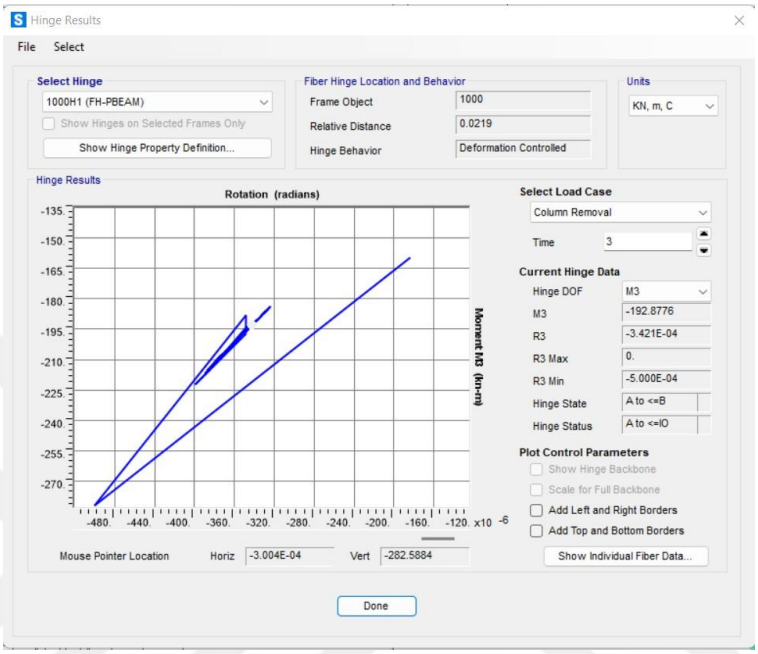

Figure 4.11. Display hinge results menu.

5. RESULTS AND DISCUSSIONS

A total of 18 removal scenarios were considered. The following naming convention was used: CR—Column Removal, WR—Wall Removal, GF—Ground Floor, MF—Middle Floor, and TF—Top Floor.

The location of the removed columns is shown in Figure 5.1. The UFC 4-023-03 proposes three locations for column removal: the corner, the edge, and the inner column. The UFC 4-023-03 also proposes that members be removed at 4 building heights: the first story above grade, the story directly below the roof, the story at mid-height, and the story above the location of a change in wall size (UFC 4-023-03, 2016). In this study, only the first three story heights were considered.

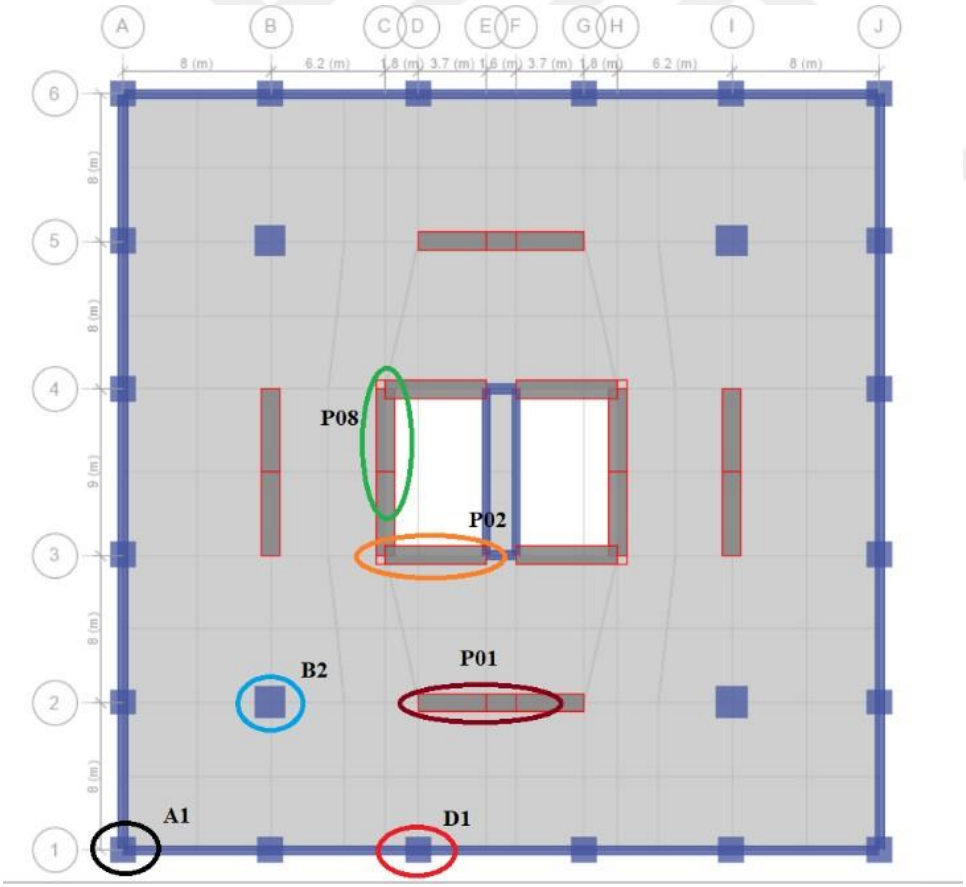

Figure 5.1. Position and name of removed members.

5.1. Analysis Results

Table 5.1 summarizes the joint displacement results for the 18 scenarios giving the maximum displacement and the residual displacements.

Table 5.1. Maximum and residual displacements for the node above the removed member.

Scenario	Maximum Displacement (mm)	Residual Displacement(mm)	Damage State
CR-GF-Corner	-37.47144	-36.5	Minimum Damage
CR-GF-Edge	-32.5236	-31.3	Minimum Damage
CR-GF-Inner	-36.38	-35	No Damage
CR-MF-Corner	-35.1	-34	No Damage
CR-MF-Edge	-24.73	-23.5	No Damage
CR-MF-Inner	-25.85	-25	No Damage
CR-TF-Corner	-36.18	-30	No Damage
CR-TF-Edge	-17.89	-15.7	No Damage
CR-TF-Inner	-17.9	-15	No Damage
WR-GF-P01	-9.9117	-9	No Damage
WR-GF-P02	-3.2166	-3	No Damage
WR-GF-P08	-1.5073	-1.5	No Damage
WR-MF-P01	-7.47	-6.5	No Damage
WR-MF-P02	-2.11	-2	No Damage
WR-MF-P08	-1.01	-0.95	No Damage
WR-TF-P01	-17.21	-15.5	No Damage
WR-TF-P02	-0.77	-0.65	No Damage
WR-TF-P08	-0.17	-0.08	No Damage

The joint displacement time histories were plotted together on the same plot using different combinations and are shown from Figure 5.2 to Figure 5.9.

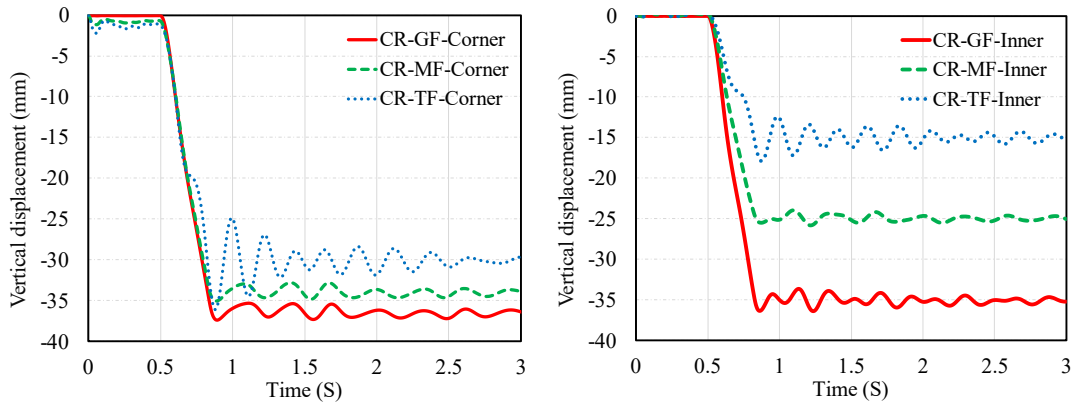

Figure 5.2. Removal of corner column (A1) (left) and inner column (B2) (right) for ground, middle and top floor.

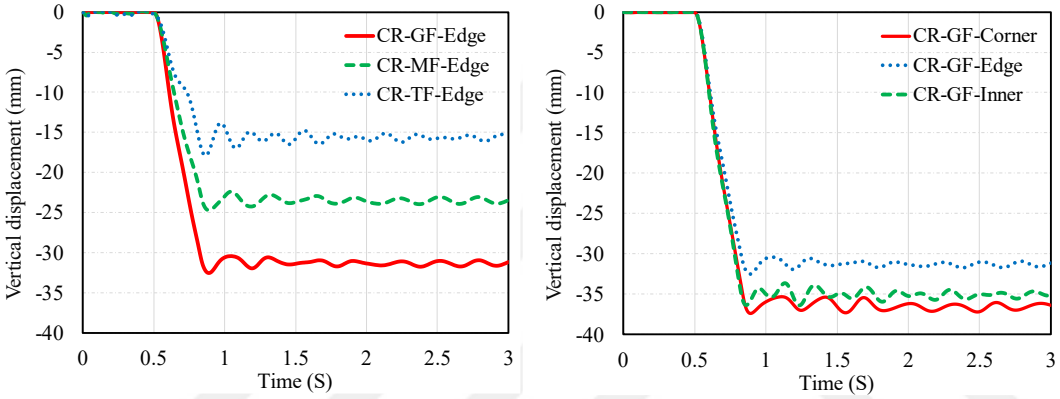

Figure 5.3. Removal of edge column (D1) for ground, middle and top floor (left) and removal of corner (A1), edge (D1) and inner column (B2) on ground floor (right).

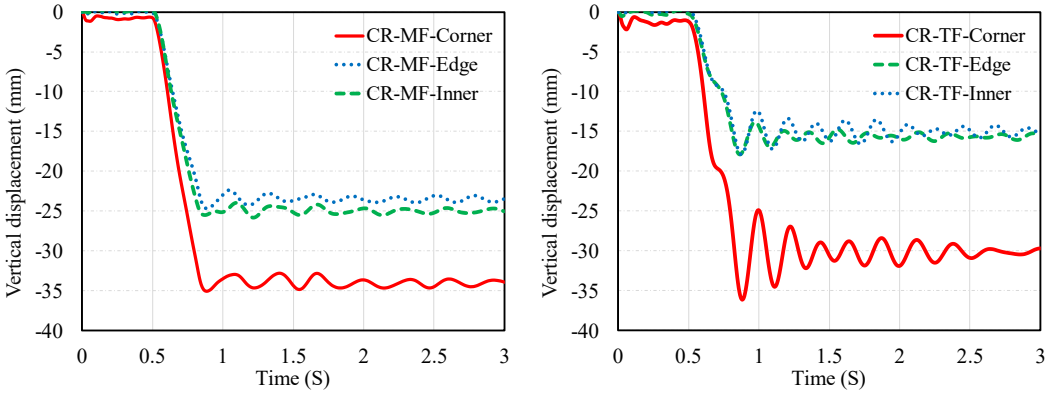

Figure 5.4. Removal of corner (A1), edge (D1) and inner column (B2) on middle (left) and top floor (right).

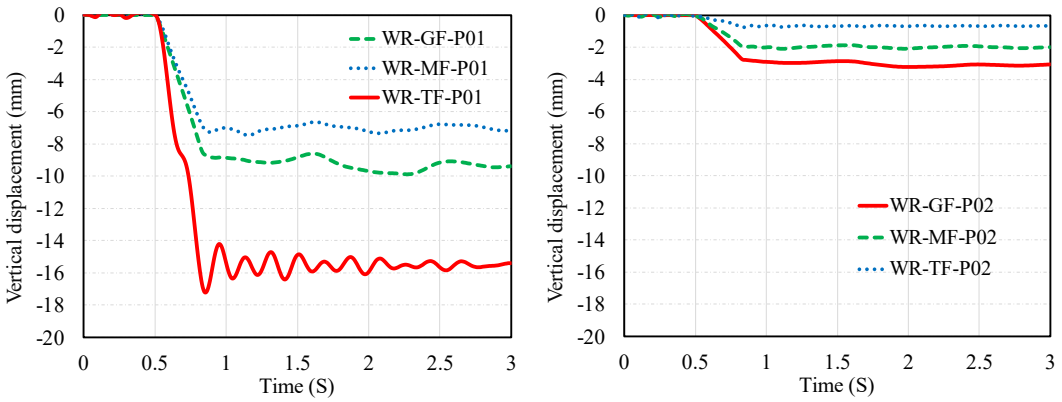

Figure 5.5. Removal of P01 (left) and P02 (right) wall for ground, middle and top floor.

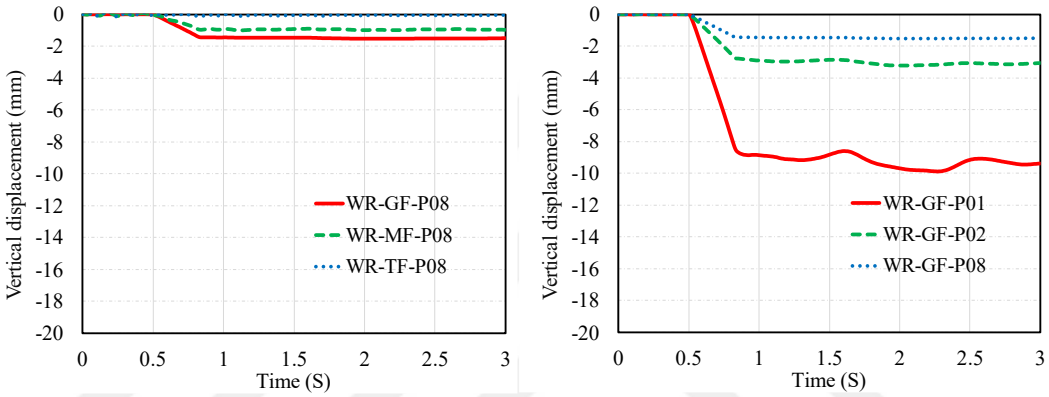

Figure 5.6. Removal of P08 (left) and of wall (P01, P02, P08) on ground floor (right)

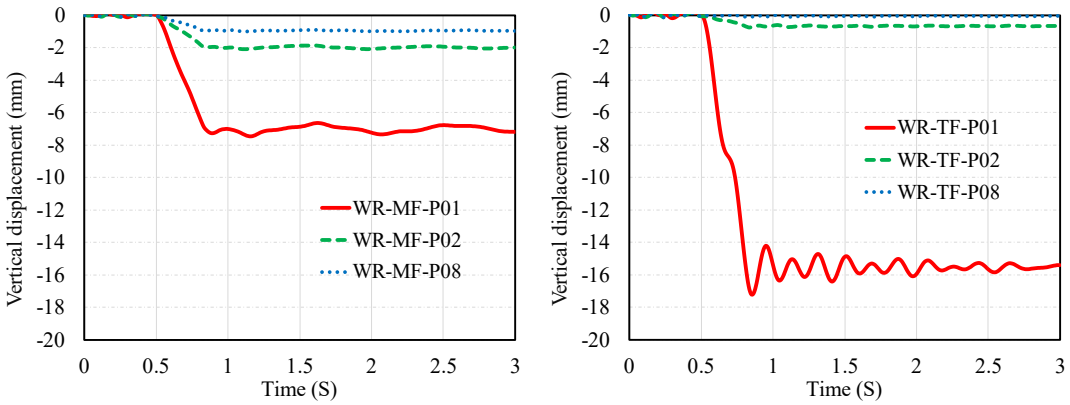

Figure 5.7. Wall (P01, P02, P08) removal on middle floor (left) and top floor (right)

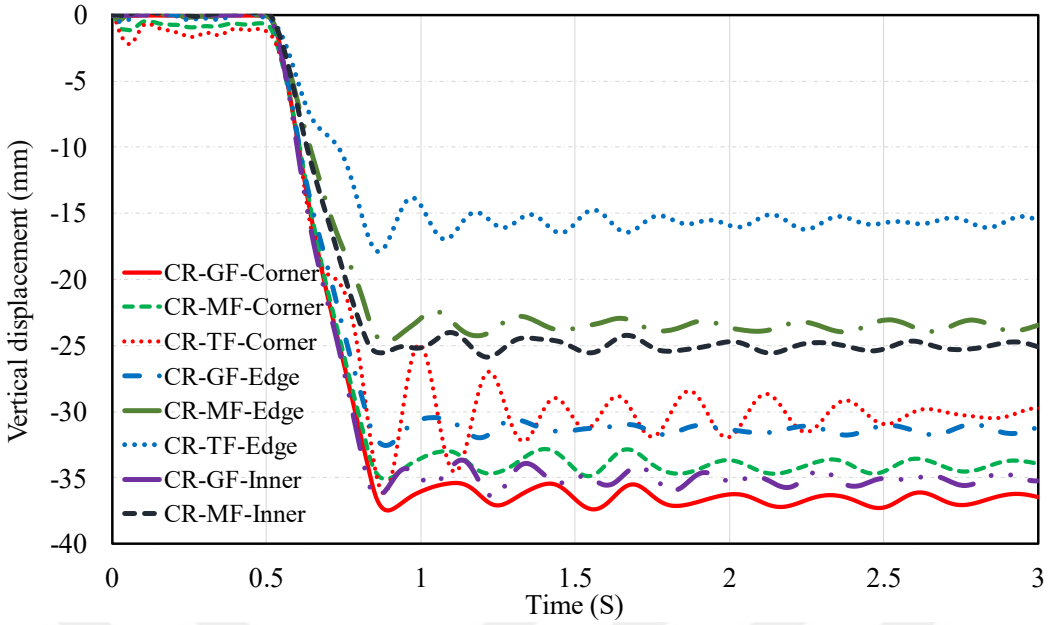

Figure 5.8. All the column removal scenarios

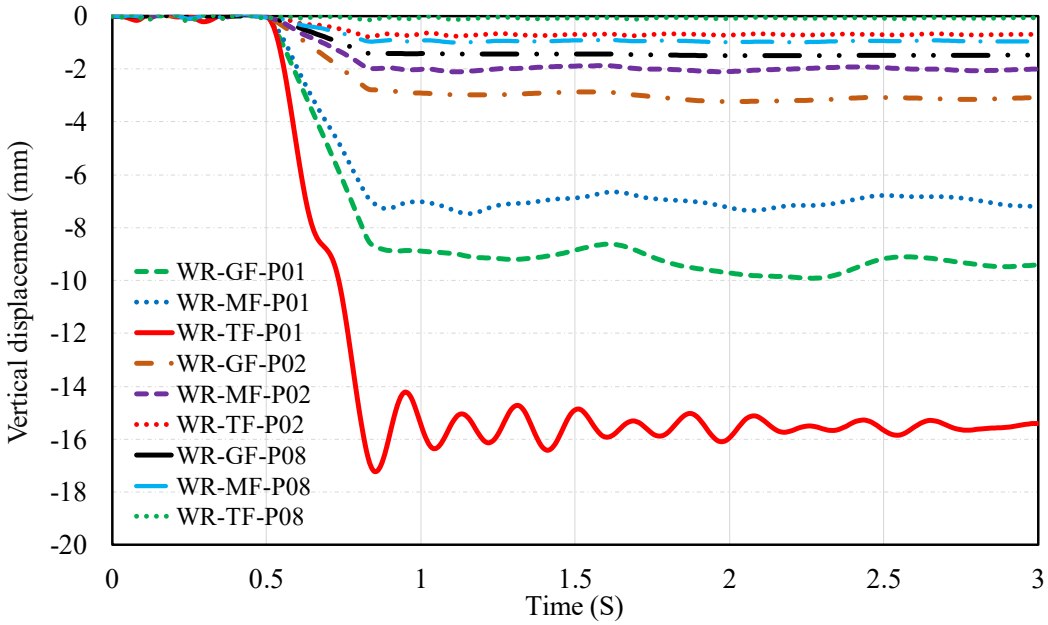

Figure 5.9. All the wall removal scenarios

From the results, it was observed that hinges formed on perimeter beams adjacent to the removed column only in the scenarios where the column was removed from the corner (A1) and the edge (D1) of the bottom floor. For the corner column removal at the ground floor, the maximum displacement was -37.47 mm, and the residual displacement was -36.5 mm. For the edge column removal at the ground floor, the maximum displacement was -32.5 and the residual displacement was -31.3 mm. In the

rest of the 16 scenarios, there were no hinges formed on the perimeter beams or on the columns.

Acceptance criteria were calculated using TEC-2018 and ASCE/SEI 41-17 using the geometric and reinforcement details of the sections. These two guidelines give different approaches to calculating the rotation limits for specific performance levels. The rotation limits were calculated and compared to the plastic rotations taken from the hinge results after the analysis was completed. The rotation limits were calculated for three performance levels: immediate occupancy (IO), life safety (LS), and collapse prevention (CP).

Hinges were formed for only 2 out of the 18 scenarios: the removal of the corner column (A1) at the ground floor and the edge column (D1) at the ground floor. The maximum displacement and residual displacement for the node above the corner column were -37.47mm and -36.5mm, respectively, and for the edge column, they were -32.52mm and -31.3mm, respectively.

For shear wall removal scenarios, the biggest displacement was observed when wall P01 was removed at the ground floor, with a maximum displacement of -9.9117 mm and a residual displacement of -9 mm. The least displacement was observed when wall P08 was removed at the top floor, with a maximum displacement of -0.17 mm and a residual displacement of -0.08 mm.

The damage was assessed by comparing the plastic rotations at the hinges formed to the plastic hinge rotation values calculated according to TEC-2018 and ASCE-41 that give the rotation limits for the three performance levels IO, LS, and CP.

The greatest damage was observed on a perimeter beam in the CR-GF-Corner scenario. The plastic rotation on the beam was 0.02808 rads, which fall under the collapse prevention performance, which has a limit of 0.03446 rads for the given section according to the TEC 2018. The beam also falls under the collapse prevention performance level according to ASCE 41-17, which has a rotation limit of 0.04606 rads for the given section.

The least damage was observed when the shear wall P08 was at the top floor. The maximum plastic rotation on the column near the wall was 0.000012 rads, which falls under the life safety performance level, which has a rotation limit of 0.02756 rads according to the TEC 2018 and 0.02961 rads according to the ASCE 41-17.

After the rotation limits were compared to the rotation on the beams the most extreme case fell under the collapse prevention performance level. This was observed in the two scenarios where hinges formed on the beams. Details about the limits calculated and the comparison of the performance criteria according to TEC-2018 and ASCE/SEI 41-17 are found in appendix A. There were no hinges formed for the remaining column removal scenarios and for all the wall removal scenarios. No collapse was observed in any of the 18 scenarios considered.

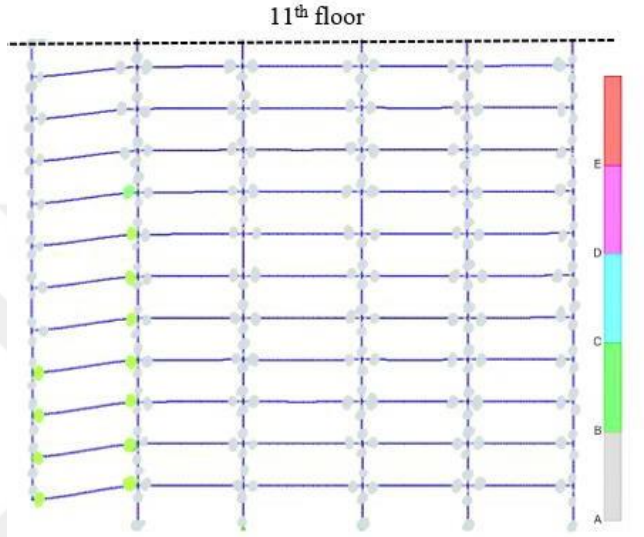

Figure 5.10. Fiber hinge damage results for removal for CR-GR-Corner scenario*

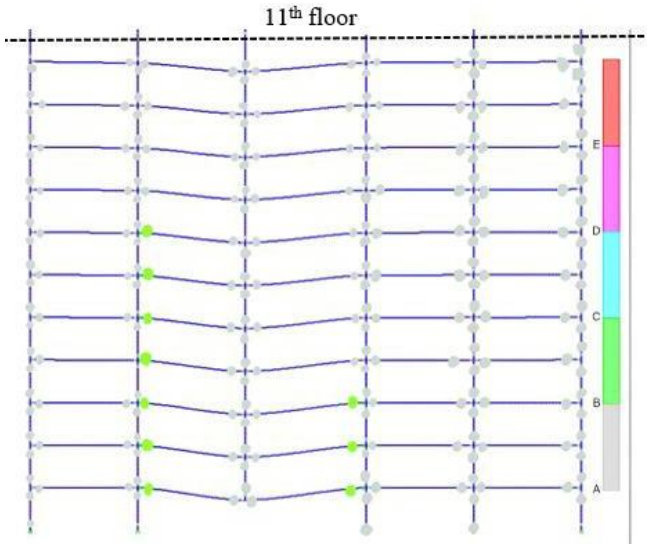

Figure 5.11. Fiber hinge damage results for removal for CR-GR-Edge scenario*

*Gray: no damage, green: limited damage, cyan: moderate damage, pink: severe damage, red: failure.

The results are consistent with the results in the literature. A good example is the parametric study carried out by Gamaniouk (Gamaniouk, 2014). In that study, hinges were formed in an almost identical manner as those observed in this study. Gamaniouk (2014) found that the number of hinges formed decreased as the number of stories increased. So, the level of damage observed in this study is reasonable.

However, there are also other possible reasons for the level of damage observed in this study. The first reason is given in the literature. Basically, taller buildings have more structural members, so the building has more alternative load paths to redistribute the load. In this model, for example, there are large column sections, and the wall sections are large and can carry a substantial amount of gravity loads. The building plan is symmetric, and there are no irregularities, making this particular building less likely to undergo progressive collapse. The large shear walls also have the capacity to carry the excess loads due to the column or shear wall removal.

Another possible reason for the low levels of damage could be overdesign. The structural sections were generalized for each group. This is common practice since it makes construction easier and more cost-effective. This generalization was explained in detail in previous chapters. There is always the possibility of overdesign when designing tall buildings. This is because the design process is long and trying to find the optimum design by using iterative methods can be very time-consuming.

6. CONCLUSION AND RECOMMENDATIONS

This study investigates the progressive collapse response of a tall building designed according to the Turkish Earthquake Code. A case study was performed on a typical 40-story building. The building had a flat plate system with shear walls at the core. A nonlinear model of the building was modeled using SAP2000. Great care was taken to accurately model the nonlinear behavior of the building. This was achieved by using the fiber hinge models for frame elements and applying a fine mesh to the plane elements. After the nonlinear model was completed, the progressive collapse evaluation was performed using the nonlinear dynamic alternate path method prescribed by the US General Services Administration (GSA-2016) and the US Department of Defense (UFC 4-023-03). Nine column removal scenarios and nine wall scenarios were considered, making eighteen total scenarios. For each scenario, the vertical displacement time histories of the nodes directly above the removed element were plotted, and the plots were compared in different combinations in order to get more insight into the results. For the cases where plastic hinges were formed, the plastic hinge rotations were compared to the performance limits calculated according to both TEC-2018 and ASCE/SEI 41-17.

Plastic hinges were formed for the scenario where the column was removed from the corner and from the edge on the ground floor. The damage in these two cases was limited. No hinges were observed in the rest of the scenarios.

The building used for this study was a tall building designed according to TEC-2018, and it had an acceptable progressive collapse response. The results show that the stresses that occur in the structural elements by the removal of columns and shear walls remain at a limited level in the structural elements and do not cause any distress to the structural integrity. However, it might be misleading to conclude that all tall buildings designed according to TEC 2018 have a good progressive collapse response. The results in this study apply only to the building considered. The same study has to be done for different tall buildings with different structural systems before a definitive

conclusion can be reached. In addition to numerical studies, experimental studies are also necessary before it can be established without a shadow of doubt that tall buildings designed according to TEC 2018 have a good progressive collapse response. There are currently a lot of demolitions happening in Türkiye. These buildings can be used for experimental studies before they are demolished. Scaled models can also be made in a laboratory environment to investigate progressive collapse in more detail. However, the building used for this study gave results that suggest excellent progressive collapse response for tall buildings designed according to TEC 2018.

It is possible that damage occurred at the slabs since the building has a flat plate system. Determining damages on the slabs is no simple task and is out of the scope of this study, so it was not considered. However, it would be advised to also check the slabs for any potential critical damage that might occur.

It would also be interesting to investigate the progressive collapse response of tall buildings with basement floors, which is the case in most tall buildings.

Lastly it is recommended that progressive collapse procedures be included explicitly in the Turkish building codes especially for government buildings. This can increase safety considering the fact that Türkiye is a frequent target of terrorist attacks.

REFERENCES

ACI 318-19 (2019), Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Farmington Hills, MI, USA.

ASCE/SEI 7-16 (2016), Minimum Design Loads and Associated Criteria for Buildings and Other Structures Minimum Design Loads for Buildings and other Structures, American Society of Civil Engineers; Virginia, USA.

ASCE/SEI 41-17 (2017), Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers; Virginia, USA.

Azim, I., Yang, J., Bhatta, S., Wang, F., & Liu, Q. (2020). Factors Influencing the Progressive Collapse Resistance of RC Frame Structures. *Journal of Building Engineering*, *27*, 1-20. https://doi.org/10.1016/j.jobe.2019.100986

Balkaya, C., (2017). Lessons Learned from Collapse of Zumrut Building under Gravity Loads. *International Journal of Engineering Technologies-IJET*, Volume 3. Issue 2., 44-49. https://doi.org/10.19072/ijet.296655

Betit, B.A., (2021). *Experimental and Analytical Assessment on the Progressive Collapse Potential of a Reinforced Concrete Building* [Master's Thesis]. The Ohio State University

BS 5628, (1978), Code of practice for use of masonry: Part 1: Structural use of unreinforced masonry, British Standards Institute, London, U.K.

BS 5950, (2000), BS 5950: Structural use of steelwork in building: Part 1: Code of practice for design – Rolled and welded sections, British Standards Institute, London, U.K.

BS 6399, (1996), Loading for buildings: Part 1: Code of practice for dead and imposed loads. British Standards Institute, London, U.K.

BS 8110, (2000), Structural use of concrete: Part 1: Code of practice for design and construction. British Standards Institute, London, U.K.

CSI Knowledge Base (2021), Technical Knowledge Base, Computers and Structures, Inc. https://wiki.csiamerica.com.

Demir, A. (2022), "Progressive collapse response of reinforced concrete buildings designed according to Turkish Earthquake Code", *Bitlis Eren Univ. J. Sci*., 11(2), 694-705. https://doi.org/10.17798/bitlisfen.1094003.

Demir, A. (2022), "Progressive collapse resistance of low and mid-rise RC mercantile buildings subjected to a column failure", *Structural Engineering and Mechanics*, Volume.83 No.4 ,563-576. https://doi.org/10.12989/sem.2022.83.4.563.

Demir, A. (2022), "Progressive collapse evaluation of low-rise reinforced concrete buildings designed for different occupancy classes", *Journal of Civil Engineering and Urbanism*, Volume 12. Issue 3., 36-43. https://dx.doi.org/10.54203/jceu.2022.6.

Eurocode 2 (2004), Design of Concrete Structures: Part 1-1: General Rules and Rules for Buildings, European Committee for Standardization, Brussels, Belgium

Eurocode 1 (2006), EN 1991-1-7: Actions on structures - Part 1-7: General Actions-Accidental actions, European Committee for Standardization, Brussels, Belgium

Fu, F., (2009). Progressive collapse analysis of high-rise building with 3-D finite element modeling method. *Journal of Constructional Steel Research*, 1269-1278. https://doi.org/10.1016/j.jcsr.2009.02.001

Gamaniouk, Taras., (2014). *Parametric Analysis of Progressive Collapse in High-Rise Buildings* [Master's Thesis]. Massachusetts Institute of Technology

GSA-2016 (2016), Alternate Path Analysis and Design Guidelines for Progressive Collapse Resistance, General Services Administration, Washington, DC, USA.

Kokot, S., and Solomons, G. (2012). *Progressive collapse risk analysis: Literature survey, relevant construction standards and guidelines*. JRC Technical Reports, *16*(2), http://www.jrc.ec.europa.eu/

Marchis, A.G., and Botez, M.D. (2019). A Numerical Assessment of the Progressive Collapse Resistance of RC Frames with Respect to the Number of Stories. *Procedia Manufacturing*, 32, 136-143. https://doi.org/10.1016/j.promfg.2019.02.194

Marjanishvili, S., and Agnew, E. (2016). Comparison of Various Procedures for Progressive Collapse Analysis. *Journal of Performance of Constructed Facilities*, Volume 20 Issue 4, https://doi.org/10.1061/(ASCE)0887-3828(2006)20:4(365)

Morone, D.J., (2012). *Progressive Collapse: Simplified Analysis Using Experimental Data*, [Master's Thesis]. The Ohio State University

NISTIR 7396 (2007), Best Practices for Reducing the Potential for Progressive Collapse in Building, U.S. National Institute of Standards and Technology (NIST), Washington D.C., USA.

Orakcal, K., (2022). *Performance Based Seismic Design of Tall Buildings-CE58K* [Lecture Notes]. Bogazici University

Osteraas, J.D., (2006). Murrah Building Bombing Revisited: A Qualitative Assessment of Blast Damage and Collapse Patterns. *Journal of Performance of Constructed Facilities*, Volume 20 Issue 4., 330-335. https://doi.org/10.1061/(ASCE)0887-3828(2006)20:4(330)

Ren, P., and Li, Y., Guan, Hong., & Lu, X. (2014). Progressive Collapse Resistance of Two Typical High-Rise RC Frame Shear Wall Structures. *Journal of Performance of Constructed Facilities*, 1-9. https://doi.org/10.1061/(ASCE)CF.1943-5509.0000593

Sagiroglu, S., (2012). *Analytical and Experimental Evaluation of Progressive Collapse Resistance of Reinforced Concrete Structures* [PhD Thesis]. Northeastern University

SAP2000 (2000), Three-Dimensional Static and Dynamic Finite Element Analysis and Design of Structures, Computer and Structures, Inc., California, USA.

Sheikh, T.A., Banday, J.M., and Hussain, M.A. (2021). Progressive collapse study of seismically designed low rise reinforced concrete framed structure. *Civil Engineering and Architecture*, Volume 9 No. 5, 1327-1338. https://doi.org/10.13189/cea.2021.090506

Starossek, U., (2007). Typology of Progressive Collapse. *Engineering Structures,* Volume 29. Issue 9., 2302-2307. https://doi.org/10.1016/j.engstruct.2006.11.025

Stephen, D.O., (2017). *Progressive Collapse Assessment of Structures* [PhD Thesis]. The University of Leeds

TEC-2018 (2019), Turkish Seismic Code for Buildings, Ministry of Environment and Urbanization, Ankara, Türkiye.

Thai, H., Ho, Q.V., Li, and Ngo, T. (2020). Progressive collapse and robustness of modular high-rise buildings. *Structure and Infrastructure Engineering,* Volume 19 No. 3, 302-314. https://doi.org/10.1080/15732479.2021.1944226

Tian, Y., Lin, K., Zhang, L., Lu, X., and H. Xue. (2021). Novel seismic-progressive collapse resilient super-tall building system. *Journal of Building Engineering*, Volume 41, 1327-1338. https://doi.org/10.1016/j.jobe.2021.102790

Travakoli, H.R., and Klavkojouri, F. (2013). Numerical Study of Progressive Collapse in Framed Structures: A New Approach for Dynamic Column Removal. *International Journal of Engineering*, Volume 26 No. 7, 685-692. https://doi.org/10.5829/idosi.ije.2013.26.07a.02

Tsai, M., and Lin, B. (2008). Investigation of progressive collapse resistance and inelastic response for an earthquake-resistant RC building subjected to column failure. *Engineering Structures*, *30*, 3619-3628. https://doi.org/10.1016/j.engstruct.2008.05.031

TS 498 (1997), Design Loads for Buildings, Turkish Standards Institute, Ankara, Türkiye.

TS 500 (2002), Requirements for Design and Construction of Reinforced Concrete Structures, Turkish Standards Institute, Ankara, Türkiye.

UFC 4-023-03 (2016), Design of Building to Resist Progressive Collapse, US Department of Defense, Washington, DC, USA.

APPENDICES

APPENDIX A

Table A.1. Performance criteria checks according to TEC-2018 on hinges formed for the ground floor corner column removal scenario.

Corner Column Story	Corner Column Frame ID	CR-GF-Corner Joint ID (Above Column)
1	231	2720

Members that formed hinges

			Plastic Rotation Limits			Analysis Results (Max)		Performance Level	
Story	**Frame ID**	**Assigned Section**	**Immediate Occupancy**	**Life Safety**	**Collapse Prevention**	**H1**	**H2**	**H1**	**H2**
1	267	D_B002	0	0.025939	0.034585	0.01989	0.02466	Life Safety	Life Safety
1	255	B_B001	0	0.025845	0.03446	0.01983	0.02821	Life Safety	Collapse Prevention
2	313	D_B002	0	0.025939	0.034585	0.02137	0.0245	Life Safety	Life Safety
2	303	B_B001	0	0.025845	0.03446	0.02109	0.02808	Life Safety	Collapse Prevention
3	359	D_B002	0	0.025939	0.034585	0.01996	0.02331	Life Safety	Life Safety
3	349	B_B001	0	0.025845	0.03446	0.01963	0.02688	Life Safety	Collapse Prevention
4	405	D_B002	0	0.025939	0.034585	0.01882	0.02221	Life Safety	Life Safety
4	395	B_B001	0	0.025845	0.03446	0.0185	0.02574	Life Safety	Life Safety
5	451	D_B002	0	0.025939	0.034585	0.01786	0.02113	Life Safety	Life Safety
5	441	B_B001	0	0.025845	0.03446	0.01755	0.02464	Life Safety	Life Safety
6	497	D_B002	0	0.025939	0.034585	0.01706	0.0201	Life Safety	Life Safety
6	487	B_B001	0	0.025845	0.03446	0.01672	0.02361	Life Safety	Life Safety
7	533	B_B001	0	0.025845	0.03446	0.01591	0.02261	Life Safety	Life Safety
8	579	B_B001	0	0.025845	0.03446	0.01517	0.0215	Life Safety	Life Safety

Table A.2. Performance criteria checks according to TEC-2018 on hinges formed for the ground floor edge column removal scenario.

Edge		CR-GF-Edge						
Story	**Frame ID**	**Joint ID (Above Column)**						
1	242	2731						
Members that formed hinges								
			Plastic Rotation Limits			**Analysis Results (Max)**		**Performance Level**	
Story	**Frame ID**	**Assigned Section**	**Immediate Occupancy**	**Life Safety**	**Collapse Prevention**	**H1**	**H2**	**H1**	**H2**
1	275	FB001	0	0.025981	0.034642	0.02556	0.01605	Life Safety	Life Safety
1	274	FB003	0	0.026076	0.034768	0.01145	0.02141	Life Safety	Life Safety
2	321	FB001	0	0.025981	0.034642	0.0244	0.01556	Life Safety	Life Safety
2	320	FB003	0	0.026076	0.034768	0.01088	0.02039	Life Safety	Life Safety
3	367	FB001	0	0.025981	0.034642	0.02328	0.01479	Life Safety	Life Safety
3	366	FB003	0	0.026076	0.034768	0.01036	0.01954	Life Safety	Life Safety
4	413	FB001	0	0.025981	0.034642	0.02223	0.01409	Life Safety	Life Safety
5	459	FB001	0	0.025981	0.034642	0.02124	0.01343	Life Safety	Life Safety
6	505	FB001	0	0.025981	0.034642	0.02031	0.01281	Life Safety	Life Safety
7	551	FB001	0	0.025981	0.034642	0.01946	0.01222	Life Safety	Life Safety

Table A.3. Comparison of acceptance criteria using TEC-2018 vs. ASCE 41 for removal of ground floor corner column.

Corner Column		**CR-GF-Corner**						
Story	**Frame ID**	**Joint ID (Above Column)**						
1	231	2720						
Members that formed hinges					**ASCE 41**		**TEC 2018**	
			Analysis Results (Max)		**Performance Level**		**Performance Level**	
Story	**Frame ID**	**Assigned Section**	**H1**	**H2**	**H1**	**H2**	**H1**	**H2**
1	267	D_B002	0.01989	0.02466	Life Safety	Collapse Prevention	Life Safety	Life Safety
1	255	B_B001	0.01983	0.02821	Life Safety	Collapse Prevention	Life Safety	Collapse Prevention
2	313	D_B002	0.02137	0.0245	Life Safety	Collapse Prevention	Life Safety	Life Safety
2	303	B_B001	0.02109	0.02808	Life Safety	Collapse Prevention	Life Safety	Collapse Prevention
3	359	D_B002	0.01996	0.02331	Life Safety	Life Safety	Life Safety	Life Safety
3	349	B_B001	0.01963	0.02688	Life Safety	Collapse Prevention	Life Safety	Collapse Prevention
4	405	D_B002	0.01882	0.02221	Life Safety	Life Safety	Life Safety	Life Safety
4	395	B_B001	0.0185	0.02574	Life Safety	Collapse Prevention	Life Safety	Life Safety
5	451	D_B002	0.01786	0.02113	Life Safety	Life Safety	Life Safety	Life Safety
5	441	B_B001	0.01755	0.02464	Life Safety	Collapse Prevention	Life Safety	Life Safety
6	497	D_B002	0.01706	0.0201	Life Safety	Life Safety	Life Safety	Life Safety
6	487	B_B001	0.01672	0.02361	Life Safety	Life Safety	Life Safety	Life Safety
7	533	B_B001	0.01591	0.02261	Life Safety	Life Safety	Life Safety	Life Safety
8	579	B_B001	0.01517	0.0215	Life Safety	Life Safety	Life Safety	Life Safety

Table A.4. Comparison of acceptance criteria using TEC-2018 vs. ASCE 41 for removal of ground floor edge column

Edge		CR-GF-Edge					
Story	**Frame ID**	**Joint ID (Above Column)**					
1	242	2731					
Members that formed hinges			**Analysis Results (Max)**		**ASCE 41 Performance Level**		**TBDY 2018 Performance Level**	
Story	**Frame ID**	**Assigned Section**	**H1**	**H2**	**H1**	**H2**	**H1**	**H2**
1	275	FB001	0.02556	0.01605	Collapse Prevention	Life Safety	Life Safety	Life Safety
1	274	FB003	0.01145	0.02141	Life Safety	Life Safety	Life Safety	Life Safety
2	321	FB001	0.0244	0.01556	Collapse Prevention	Life Safety	Life Safety	Life Safety
2	320	FB003	0.01088	0.02039	Life Safety	Life Safety	Life Safety	Life Safety
3	367	FB001	0.02328	0.01479	Life Safety	Life Safety	Life Safety	Life Safety
3	366	FB003	0.01036	0.01954	Life Safety	Life Safety	Life Safety	Life Safety
4	413	FB001	0.02223	0.01409	Life Safety	Life Safety	Life Safety	Life Safety
5	459	FB001	0.02124	0.01343	Life Safety	Life Safety	Life Safety	Life Safety
6	505	FB001	0.02031	0.01281	Life Safety	Life Safety	Life Safety	Life Safety
7	551	FB001	0.01946	0.01222	Life Safety	Life Safety	Life Safety	Life Safety

Table A.5. Shear checks at the hinges for removal of corner column at the ground floor

Corner Column		**CR-GF-Corner**					
Story	**Frame ID**	**Joint ID (Above Column)**					
1	231	2720					
			Vc = Vcapacity				
Members that formed hinges			**Vd = Vdemand**				
Story	**Frame ID**	**Assigned Section**	**Vc (kN)**	**Vd-H1**	**Vd-H2**	**DCR-H1**	**DCR-H2**
1	267	D_B002	1436.0	482.15	19.215	34%	1%
1	255	B_B001	1436.0	11.775	458.43	1%	32%
2	313	D_B002	1436.0	465.74	22.682	32%	2%
2	303	B_B001	1436.0	51.544	441.36	4%	31%
3	359	D_B002	1436.0	467.25	20.089	33%	1%
3	349	B_B001	1436.0	39.348	430.55	3%	30%
4	405	D_B002	1436.0	465.54	23.605	32%	2%
4	395	B_B001	1436.0	53.468	418.41	4%	29%
5	451	D_B002	1436.0	463.64	29.589	32%	2%
5	441	B_B001	1436.0	60.313	406.47	4%	28%
6	497	D_B002	1436.0	461.65	37.494	32%	3%
6	487	B_B001	1436.0	68.267	394.91	5%	28%
7	533	B_B001	1436.0	75.715	383.98	5%	27%
8	579	B_B001	1436.0	84.023	379.14	6%	26%

Table A.6. Shear checks at the hinges for removal of edge column at the ground floor

Edge Column		**CR-GF-Edge**					
Story	**Frame ID**	**Joint ID (Above Column)**					
1	242	2731					
			Vc = Vcapacity				
Members that formed hinges			**Vd = Vdemand**				
Story	**Frame ID**	**Assigned Section**	**Vc (kN)**	**Vd-H1**	**Vd-H2**	**DCR-H1**	**DCR-H2**
1	275	FB001	1436.0	432.03	56.742	30%	4%
1	274	FB003	1436.0	161.26	466.89	11%	33%
2	321	FB001	1436.0	427.08	68.484	30%	5%
2	320	FB003	1436.0	167.23	459.06	12%	32%
3	367	FB001	1436.0	426.90	72.383	30%	5%
3	366	FB003	1436.0	168.54	452.93	12%	32%
4	413	FB001	1436.0	426.99	76.264	30%	5%
5	459	FB001	1436.0	426.91	79.993	30%	6%
6	505	FB001	1436.0	426.50	83.248	30%	6%
7	551	FB001	1436.0	425.23	87.068	30%	6%

CURRICULUM VITAE

Name Surname : Munyaradzi GONDOBWE

EDUCATION:

- **Undergraduate** : 2021, Sakarya University, Faculty of Engineering, Civil Engineering Department

PROFESSIONAL EXPERIENCE AND AWARDS:

- He worked as a civil engineer at Olmez Celik from 2023-Present.
- He worked as a procurement specialist at Makestruct Private Limited between 2021-2023