A RECENT CHALLENGE IN STRUCTURAL STEEL DESIGN: PROGRESSIVE COLLAPSE

by

ERDEM ERGÜÇLÜ

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APPROVED BY:

Prof. Gülay Altay ....
(Thesis Supervisor)

Assist. Prof. Serdar Soyöz ....

Assist. Prof. Cüneyt Vatansever ....

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ABSTRACT

A RECENT CHALLENGE IN STRUCTURAL STEEL DESIGN: PROGRESSIVE COLLAPSE

Progressive collapse is a structural design challenge which was discovered 45 years ago but gained awareness in recent years. It is defined by ASCE 7-05 as the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionate large part of it. In this study two case studies have been considered in order to illustrate progressive collapse analysis of steel braced frames. Relevant design codes from USA, Canada and Europe; and also recent papers have been reviewed in order to provide a broad literature that enhances intelligibility of the case studies. In the first case study, progressive collapse potential of two ten-story prototype steel braced frames have been investigated. These were a Special Concentrically Braced Frame (SCBF) and an Eccentrically Braced Frame (EBF) designed previously for different seismic design categories. Alternate Path Method (APM) introduced by recent progressive collapse specific design codes like UFC 4-023-03 have been utilized. Accordingly, nonlinear dynamic time history analysis have been carried out after removing critical column and associated braces from models at different story levels. Here, element removals simulated initial local failure due to an abnormal event. Analysis results revealed that SCBF has progressive collapse potential at the last two stories whereas EBF has collapse potential at the first three stories. The results of nonlinear dynamic analysis were used in the second case study to investigate the accuracy of Dynamic Increase Factor (DIF) calculated per UFC. This factor is used to represent dynamic nature of progressive collapse in APM with Nonlinear Static Analysis (NSA). The use of the equation suggested by UFC for NSA of braced frames needs further investigation since for some removal cases it has underestimated the DIF.

ÖZET

YAPISAL ÇELİK TASARIMINDA YENİ BİR ZORLUK: AŞAMALI GÖÇME

Aşamalı göçme 45 yıl önce keşfedilen fakat son yıllarda bilinirlik kazanan bir yapısal tasarım zorluğudur. ASCE 7-05 tarafından başlangıçtaki bir bölgesel ve yapısal yıkımın elemandan elemana yayılması sonucunda, bir yapının bütünün çökmesi ya da orantısız olarak büyük bir kısmının çökmesi şeklinde tanımlanır. Bu çalışmada çaprazlı çelik çerçeve sistemlerin aşamalı göçme analizlerini örneklendirmek için iki vaka çalışması ele alınmıştır. Birleşik Devletler, Kanada ve Avrupa'dan ilgili tasarım kodları ve ilgili makaleler vaka çalışmalarının anlaşılırlığını artırır geniş bir literatür sağlamak için özetlenmiştir. İlk vaka çalışmasında on katlı iki prototip çaprazlı çelik çerçevenin aşamalı göçme potansiyelleri araştırılmıştır. Bunlar farklı deprem sınıfları için önceden tasarlanmış özel merkezi çelik çaprazlı çerçeve (ÖMÇÇ) ve dışmerkez çelik çaprazlı çerçevedir (DMÇÇ). UFC 4-023-03 gibi aşamalı göçmeye yönelik güncel tasarım kodlarında ortaya konulan Alternatif Yol Yöntemi (AYY) uygulanmıştır. Bu bağlamda, farklı kat seviyelerindeki kritik kolon ve ilişkili çaprazların modelden çıkartılması sonucunda zaman tanım alanında doğrusal olmayan analiz yöntemi uygulanmıştır. Burada eleman eksiltmeleri olağandışı bir olay sonucunda oluşan başlangıçtaki bir bölgesel ve yapısal yıkımı temsil etmiştir. Analiz sonuçları ÖMÇÇ'nin son iki katta ve DMÇÇ'nin ilk üç katta aşamalı göçme potansiyeline sahip olduğunu göstermiştir. İkinci vaka çalışmasında doğrusal olmayan dinamik analiz sonuçları kullanılarak UFC'e göre hesaplanan ve doğrusal olmayan statik analizde dinamik etkiyi temsil eden dinamik artırım katsayısının doğruluğu araştırılmıştır. Bazı eleman eksiltme durumlarında şartnameye göre hesaplanan kat sayı güvensiz tarafta kaldığından, söz konusu denklemin çaprazlı çerçevelerde kullanımı ileri araştırma gerektirmektedir.

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A	Cross-sectional area
A	Spectral acceleration
A_0	Effective ground acceleration coefficient
A_k	Load due to extraordinary event
A_k	Shearing area
A_n	Net cross-sectional area
A_w	Shearing area
b	Flange width
D	Dead load
D_a	Yield stress increase factor
d_{fi}	Fictive i-th storey displacement
d_i	Lateral displacement of i-th floor
e	Link length
E	Earthquake load
E_s	Modulus of elasticity
F_a	Short period site coefficient
F_{fi}	Fictive i-th storey force
F_i	Required tie strength in the longitudinal or transverse direction is
F_p	Required peripheral tie strength
F_v	Long period site coefficient
F_{ye}	Expected yield strength
G	Gravity loads
G	Shear modulus
G	Dead load
g_i	Total dead load of single story
G_{LD}	Increased gravity loads for deformation-controlled actions
G_{LF}	Increased gravity loads for force-controlled actions
G_N	Increased gravity loads for nonlinear static analysis
G_{ND}	Gravity loads for nonlinear dynamic analysis
h	Height of section

h_i	Storey height of i-th floor
h_{i-1}	Storey height of (i-1)-th floor
I	Importance factor
IF	Load increase factor
I_x	Moment of inertia about x-axis
k	Effective length factor
K_b	Flexural stiffness
K_e	Stiffness of the link beam
K_s	Shear stiffness
L	Live load
L	Bay length
L	Brace length
L_1	Greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the direction under consideration
L_1	For exterior peripheral ties, the greater of the distances between the centers of the columns, frames, or walls at the perimeter of the building in the direction under consideration (m or ft). For peripheral ties at openings, the length of the bay in which the opening is located, in the direction under consideration.
l_b	Beam length
L_{LAT}	Lateral load
m	Component or element demand modifier (m-factor)
M_d	Design moment
m_i	i-th storey mass
m_{LIF}	The smallest m of any primary beam
M_{max}	Maximum moment during nonlinear dynamic analysis
M_p	Bending moment capacity
$M_{p,b}$	Increased moment demand for beam outside of the link element
M_{pe}	Expected plastic moment capacity
n	Live load participation factor
N_{bp}	Axial compression capacity
$N_{çp}$	Axial tension capacity

N_d	Design axial load
$N_{p,b}$	Increased axial load demand
P_{br}	Reaction forces applied to the removal locations for brace
P_{br1}	Reaction forces applied to the removal locations for brace 1
P_{br2}	Reaction forces applied to the removal locations for brace 2
P_{col}	Reaction forces applied to the removal locations for column
P_{cre}	Expected buckling load
P_{max}	Maximum axial load during nonlinear dynamic analysis
P_{sum}	Summation of reaction forces of column and brace
P_{te}	Tensile yielding load
Q	Live load
Q_{CE}	Expected strength of the component or element for deformation-controlled actions from ASCE 41
Q_{CL}	Lower-bound strength of a component or element for force-controlled actions from ASCE 41
Q_i	Load effect
q_i	Total live load of single story
Q_{UD}	Deformation-controlled action, from Linear Static model
Q_{UD}	Acting force (demand) determined in component or connection/joint (moment, axial force, shear, and possible combined forces)
Q_{UF}	Force-controlled action, from Linear Static model
Q_{UF}	Force-controlled action, from Nonlinear Static model
r	Radius of gyration
R_a	Response modification factor
R_n	Nominal tie strength calculated with the appropriate material specific code, including the over-strength factors from Chapters 5 to 8 of ASCE 41.
R_n	Nominal strength calculated from material specific chapters of this UFC, including the over-strength factors where applicable.
R_y	Ratio of the expected yield stress to the specified minimum yield stress
S	Snow load
S_1	Mapped 5 percent damped spectral acceleration at 1-s period

S_a	Spectral response acceleration
SD	Imposed dead load
S_{D1}	Design spectral response acceleration parameter at 1-s period
S_{DS}	Design spectral response acceleration parameter at short periods
S_{M1}	5 percent damped spectral acceleration at 1-s period adjusted for site
S_{MS}	5 percent damped spectral acceleration at short periods adjusted for site
S_S	Mapped 5 percent damped spectral acceleration at short periods
S_x	First moment of area about x-axis
t	Flange thickness
T_1	1. Modal period of the structure
T_{v1}	First modal period associated with the structural response mode for the vertical motion of the bays above the removed column and brace
t_w	Web thickness
T_x	The period of the building in x-direction
V_d	Design shear
V_d	Design shear load
V_p	Plastic shear load capacity
V_t	Total base shear
W	Total weight of the structure defined by Equation 5.8
W	Wind load
w_F	Floor load
w_i	Weight of single story
W_p	Plastic section modulus
W_x	Elastic section modulus
γ_i	Load factor
γ_p	Link element rotation angle
δ	Maximum node displacement
δ_i	Effective relative floor shift
η_{bi}	Torsional irregularity coefficient
η_{ki}	Rigidity irregularity coefficient

θ_p	Floor drift
θ_{pra}	Plastic rotation angle from Table 3.10 or from Table 5-6 in ASCE 41
θ_y	Yield rotation angle
λ	Slenderness ratio
σ_{bem}	Allowable compressive stress
σ_{eb}	Design stress obtained from $N_{p,b}$
ΣP	Sum of the gravity loads
σ_y	Reference stress considering principle stresses
σ_y	Yield stress
τ_{ave}	Average shear stress
ϕ	Strength reduction factor
ϕ_b	Bending strength reduction factor
ϕ_c	Compressive strength reduction factor
ϕ_t	Tensile strength reduction factor
Ω_{LD}	Load increase factor for calculating deformation-controlled actions
Ω_{LF}	Load increase factor for calculating force-controlled actions
Ω_N	Dynamic increase factor

$(\Delta_i)_{ave}$	Average reduced relative floor shift
$(\Delta_i)_{max}$	Maximum reduced relative floor shift
Δ_c	Axial deformation at expected buckling load
Δ_i	Reduced relative floor shift
Δ_t	Axial deformation at tensile yielding load
Δt_{off}	Time step size
$0.002\Sigma P$	Notional lateral load applied at each floor

LIST OF ACRONYMS/ABBREVIATIONS

AISC	American Institute of Steel Construction
ANSI	American National Standard Institute
APM	Alternate Path Method
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
AWF	American Wide Flange
BLCC	Bankers Life and Casualty Company
BSI	British Standard Institute
CP	Collapse Prevention
CSI	Computers & Structures Inc
DCR	Demand to Capacity Ratio
DIF	Dynamic Increase Factor
DL	Deal Load
DoD	Department of Defense
EBF	Eccentrically Braced Frame
ELR	Enhanced Local Resistance
E-W	East-West
FEMA	Federal Emergency Management Agency
GSA	General Service Administration
HHT	Hilber-Hughes-Taylor Alpha
HSS	Hollow Steel Sections
IBC	International Building Code
IF	Load Increase Factor
IO	Immediate Occupancy
ISC	Interagency Security Committee
LIF	Load Increase Factor
LL	Live Load
LRFD	Load and Resistance Factor Design
LS	Life Safety

LS-DYNA	Advanced General-Purpose Multi-physics Simulation Software Package
LSP	Linear Static Procedure
NAC	Nonlinear Acceptance Criteria
NBCC	National Building Code of Canada
NDA	Nonlinear Dynamic Analysis
NDP	Nonlinear Dynamic Procedure
NIST	National Institute of Standards and Technology
N-S	North-South
NSA	Nonlinear Static Analysis
NSP	Nonlinear Static Procedure
OC	Occupancy Category
R/C	Reinforced Concrete
RBS	Reduced Beam Section
RMS	Risk Management Solutions
SAC	Joint Venture Formed In Mid-1994 to Study Welded Connections
SAP	Structural Analysis Program
SCBF	Special Concentrically Braced Frame
TEC 07	Turkish Earthquake Code 2007
TSE	Turkish Standards Institution
UFC	Unified Facilities Criteria
WCPF	Welded Cover Plated Flange
WUF-B	Welded Unreinforced Flange-Bolted Web
WUF-W	Welded Unreinforced Flange- Welded Web

1. INTRODUCTION

1.1. Background and Motivation

It was the partial collapse of Ronan Point Apartment Tower in London which made engineering community first be aware of progressive collapse. Therefore, the first definition of progressive collapse came from the description of the event on the morning of 16 May 1968.

Mrs. Ivy Hodge, a resident on the 18th floor of 22 storey apartment struck a match in her kitchen. This match caused a gas explosion which in return caused loss of load bearing precast concrete panel near the corner of the building. This led to the collapse of the floors above .The additional weight of upper floors from 18th -22nd has started chain reactions and the corner bay collapsed all the way to the ground as it can be seen in Figure 1.1. Mrs. Hodge survived but four other residents were killed and seventeen were injured.

Figure 1.1. Progressive collapse of Ronan Point building (Nair, 2004).

This type of sequential failure which is disproportionate to triggering event was named as “progressive collapse” after this tragedy. After this event, code developers in

Britain have started to work on developing code provisions to mitigate such progressive collapse.

Another milestone attracting attention of engineering communities and public regularity agencies to this collapse type was the Oklahoma City bombing. Murrah Federal Building experienced a terrorist attack and was damaged largely by a bomb on 19 April 1995. Only three columns (G16, G20, and G24) supporting the third-level transfer girder were destroyed directly by blast effect. But, the level of collapse was disproportionate to that as it can be seen in Figure 1.2. 168 fatalities were due to progressive collapse of the structure and not to direct blast effect as stated in UFC 4-023-03 (DoD, 2009).

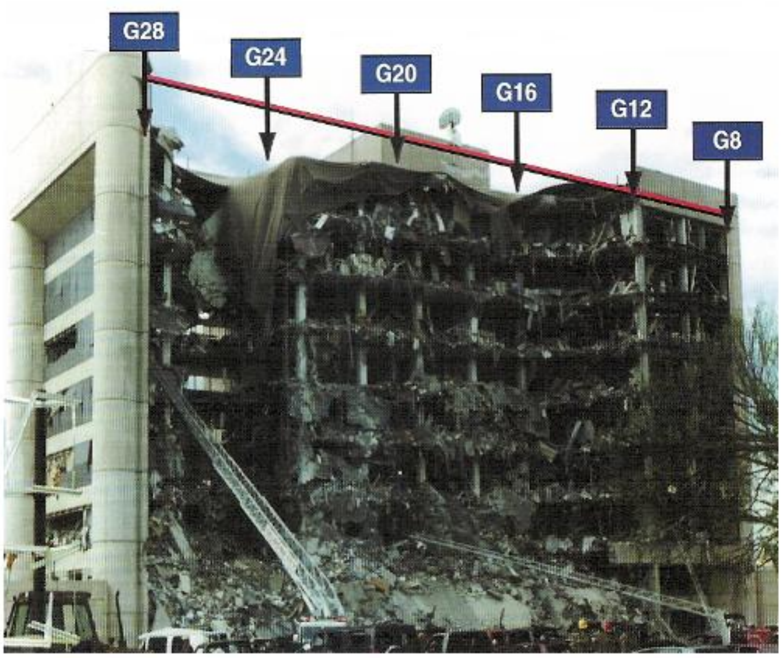


Figure 1.2. Blast induced progressive collapse of Murrah federal office building (Nair, 2004).

In addition to these two well known progressive collapse related cases, there are other structural failures associated to progressive collapse. For more of them, reader may refer to the publication called "Best Practices for Reducing the Potential for Progressive Collapse in Buildings" (NIST, 2007). Although there were such motivating events, engineering community has not interested much in understanding and thus mitigating progressive collapse of structures. There has been some country based studies like the ones in Britain and Canada about this design goal but in global scale progressive collapse has mainly be ignored until 2000s due to the presence of more global system threats like earthquake loading.

Especially, for countries like Turkey where seismic design is the core of structural design due to high seismicity of the land, progressive collapse is not perceived widely by structural engineering. However, according to Gurley (2008), earthquakes can also remove supports by damaging mostly corner columns and thus may lead progressive collapse as shown in Figure 1.3. Therefore, being also be triggered by earthquake loading, progressive collapse should be take into account in Turkey in order to better understand some structural failures after earthquakes. In Turkey, some illegal structural modifications like column removals at base or 1st stories are also present which may result in progressive collapse. In addition, other abnormal events listed in Table 2.2 are widely encountered in Turkey. From these events, blast and impact loading are also valid for Turkey and they need to be considered for design of public and military facilities to reduce possible losses.

Figure 1.3. Earthquake induced progressive collapse from Kocaeli, Turkey (RMS, 1999).

The motivation behind this thesis study is therefore the lack of knowledge about progressive collapse in Turkey in spite of all above statements. Since it is a recent challenge handled globally by structural engineering community, in this thesis progressive collapse is studied with another developing area in Turkey; structural steel design.

2. PROGRESSIVE COLLAPSE BASICS

Most structural collapses can be named as progressive due to inherent redundancy of structures. However, it is usually not possible and feasible to design against general collapse caused by severe abnormal loads acting on large portion of a structure (ASCE 7). Engineering community deals with prevention of progressive collapse which is also disproportionate. In order to have a better understanding of this fact, General Service Administration of USA (GSA, 2003) uses the diagram on Figure 2.1.

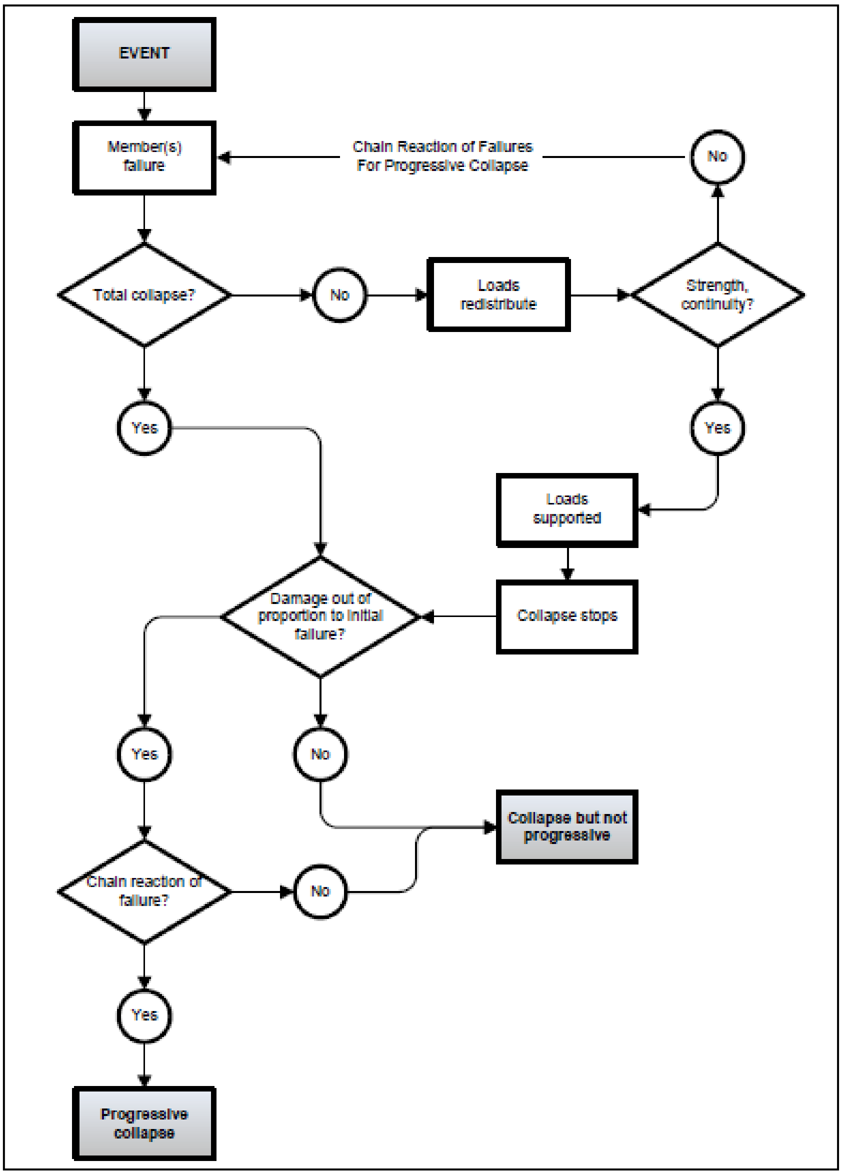


Figure 2.1. Collapse sequence of event diagram (GSA, 2003).

2.1. Definition

In literature there is no unique definition of progressive or disproportionate collapse. Some of them are listed in Table 2.1. Therefore, there is a need to define these two terms separately to prevent possible misusage and misunderstandings.

Disproportionate collapse is the case when there is a prominent disproportion between a relatively minor event and the following collapse of a major part or even the whole of a structure. Progressive collapse is the case when the collapse is due to the failure, induced by a triggering initial event, of one or a few structural components which then in turn triggers a successive failure of other components not directly affected by the initial event (Starossek and Haberland, 2010).

Table 2.1. Definition of progressive collapse and disproportionate collapse.

Source	Definition
Gross and McGuire (1983)	"A progressive collapse is characterized by the loss of load-carrying capacity of a relatively small portion of a structure due to an abnormal load which, in turn, triggers a cascade of failure affecting a major portion of the structure."
GSA (2003)	"A progressive collapse is a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total collapse is disproportionate to the original cause."
UFC 4-010-01 (DoD, 2003)	"Progressive collapse. A chain reaction failure of building members to an extent disproportionate to the original localized damage."
ASCE 7 (2005)	"Progressive collapse is defined as the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionate large part of it."

In spite of different meanings, progressive collapse and disproportionate collapse are often used interchangeably in the literature. In fact, the two terms are related to each other because disproportionate collapse occurs often in progressive manner and progressive collapse can be disproportionate. According to Starossek and Haberland (2010), the term disproportionate collapse is more appropriate in the context of design and performance, and the term progressive collapse is more suitable when referring to the physical phenomenon and mechanism of collapse.

2.2. Abnormal Event

Events triggering progressive collapse are called as abnormal, which are unforeseeable or occur with very low probability. In Table 2.2 some of them are listed. Due to low probability abnormal events are generally not considered during the design of a structure, but they are the cause of initial local damage triggering a progressive collapse.

Table 2.2. Abnormal Events (Starossek and Haberland, 2010).

Faults			Errors
External		Immanent	
Man-made (accidental or intentional)	Environmental		
• Impact (i) Car (ii) Train (iii) Ship (iv) Aircraft (v) Missile • Explosion (i) Gas (ii) Explosive • Fire • Excessive loading	• Earthquake • Extreme wind forces • Heavy snow fall (excessive roof loads) • Floods (scour)	• Lack of Strength • Cracks	• Design errors • Construction errors • Usage errors

2.3. Design Methods for Progressive Collapse Resistant Structures

Before stating available design methods for reducing the risk of progressive collapse of structures or increasing reliability of the system, key elements of failure, i.e. progressive collapse, need to be explained. In Equation 2.1 reliability of a system is defined. Here, P(C) refers to probability of progressive collapse due to an abnormal event and it is defined by Equation 2.2 (NIST, 2007).

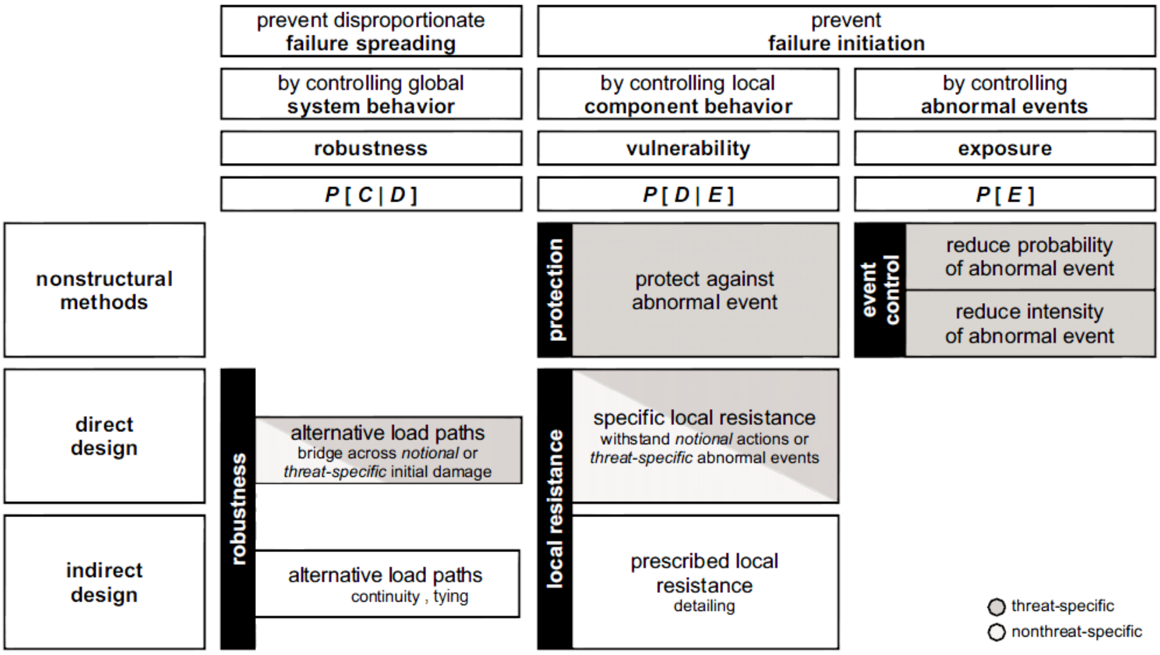
$$L = 1 - P(C) \tag{2.1}$$

$$P(C) = P(C|D)P(D|E)P(E) \tag{2.2}$$

In the Equation 2.2 P(E) refers to the probability of occurrence of an abnormal event, E, affecting structure; $P(D|E)$ refers to the conditional probability of initial damage, D, in consequence of an abnormal event E; and $P(C|D)$ refers to the conditional probability of a progressive spreading of structural failure, C, due to the initial damage D.

Based on Equation 2.2 different design methods are developed in the literature. A comprehensive summary of these methods is in Figure 2.2.

Table 2.3. Summary of design methods (Starossek and Haberland, 2010).



Design methods are divided into two main categories, namely; nonstructural and structural methods.

2.3.1. Nonstructural Methods

Nonstructural methods are based on event control and protection. Both methods are threat-specific. Event control method reduces the exposure by decreasing probability of occurrence of abnormal events. Protection methods refer to additional structures and measures (e.g., impact walls, stand-off distance) to reduce vulnerability of a structure by protecting key elements. Thus, a structure is less vulnerable if possibility of small damage triggering progressive collapse is reduced.

As the name implies, nonstructural methods are generally out of the scope of structural engineering because their implications requires information beyond this field of science. These two methods are generally utilized for blast resistant design.

2.3.2. Structural Methods

Structural methods are also divided into two main categories. These are direct and indirect design approaches. Structural approaches enhance the robustness of a structure by decreasing sensitivity of a structure to initial damage or increase local resistance to mitigate progressive collapse.

One of the prevalent methods in the literature for direct design is Alternate Load Path method (APM). APM provide alternatives for a load to be transferred from a point of application to a point of resistance (Starossek and Haberland, 2010). In other words, this method requires that the structure be capable of bridging over a missing structural element with localized damage (UFC 4-023-03, DoD 2009). APM method is the core of current guidelines for design of buildings to resist progressive collapse and it will be explained in detail in Section 3.5.3. Another direct design approach is specific local resistance. This approach controls local component behavior to decrease vulnerability of a structure. In this method key elements are designed explicitly to withstand specific abnormal loads. Indirect approaches consider resistance to progressive collapse implicitly through the provision of minimum levels of strength, continuity and ductility (UFC 4-023-03, DoD 2009).

2.4. Differences between Seismic Design and Progressive Collapse Design

There is a common perception in the field of structural engineering that ductile detailing stemmed from seismic design will improve progressive collapse resistance of structures. In other words, structures designed against high seismic loads have already inherent robustness against failures caused by abnormal loads. In fact, earthquakes are only one of the environmental events which may cause progressive collapse. For a better understanding of following discussions, differences between seismic and progressive collapse design need to be summarized in the concept of loading, connections and collapse behavior. Since the concern of this thesis work is progressive collapse of steel framed structures, the following discussion is designed accordingly.

Progressive collapse is not a result of an external load applied to the structure as it is the case in earthquake loading. The load triggering collapse is obtained from a sudden local change in building geometry (Marjanishvili, 2004). Therefore, compared to seismic loads causing a structures respond globally, a structure might undergo progressive collapse as a result of an abnormal event which affects only a few elements. Duration of the load that structures exposed depends on duration of the base excitation in a seismic design but progressive collapse occurs only in a matter of seconds. In 1-2 seconds internal forces reach their peak values in dynamic progressive collapse analysis and, connections of steel structures undergo one or two cycles compared to high cycle demands of earthquake forces. Structures resist to seismic forces by moment capacities of connections and cross sections; however, as will be discussed in Chapters 3 and 4, progressive collapse resistance of structures and also force distribution is mainly attained by catenary action. This action is a source of tie forces and means high rotation and tension capacity demands for steel connections. Also, some very efficient ductile connection types like reduced beam section (RBS) is attributed to make steel buildings vulnerable to progressive collapse.

In the open literature there is no explanation about how seismic detailing improves progressive collapse resistance, and many researchers as introduced in Chapter 4 argue that adapting seismic resistant design or rehabilitation rules directly to progressive collapse resistant design cause designs to be both conservative and unconservative depending on the structure.

3. DESIGN CODES AND STANDARDS ADDRESSING PROGRESSIVE COLLAPSE

After progressive collapse of Ronan Point apartment in 1968, engineers have started to develop design guidelines to mitigate progressive collapse of structures. Investigations of the United Kingdom (UK) engineering community led them to develop design approaches addressing the weaknesses in connections between the structural elements. Although this event let American National Standard Institute (ANSI) work on progressive collapse during the 1970s, very few provisions have been developed until ANSI 1982. The Oklahoma city bombing in 1995 caused the rapid development of progressive collapse guidelines issued by federal and defensive bodies (Stevens *et al*., 2011).

Today, many US, Canadian and European codes contain explicit provisions to design against progressive collapse. There are also two complete guidelines addressing analysis and design against progressive collapse in US; namely, Progressive Collapse Analysis and Guidelines issued by General Service Administration, and Unified Facilities Criteria (UFC)-Design of Buildings to Resist Progressive Collapse issued by Department of Defense (DoD). In the following sections summary of code provisions from US, Canadian and European codes adapted from literature as well as detailed assessment of these two explicit guidelines is given.

3.1. British Code

Progressive collapse design requirements of British Standard Institute (BSI) have been developed and instituted shortly after the Ronan Point collapse. Material specific (e.g., reinforced concrete or steel) provisions have been issued and remained unchanged until the advancement of Eurocode Standards. The following Figure 3.1 is a summary of BSI code provisions for progressive collapse, which is adapted from BSI 2006 (Stevens *et al.*, 2011). The top portion of this figure represents threat-specific design approaches like protection whereas the bottom portion stands for nonthreat-specific actions including direct methods (e.g. APM, specific local resistance) and indirect methods.

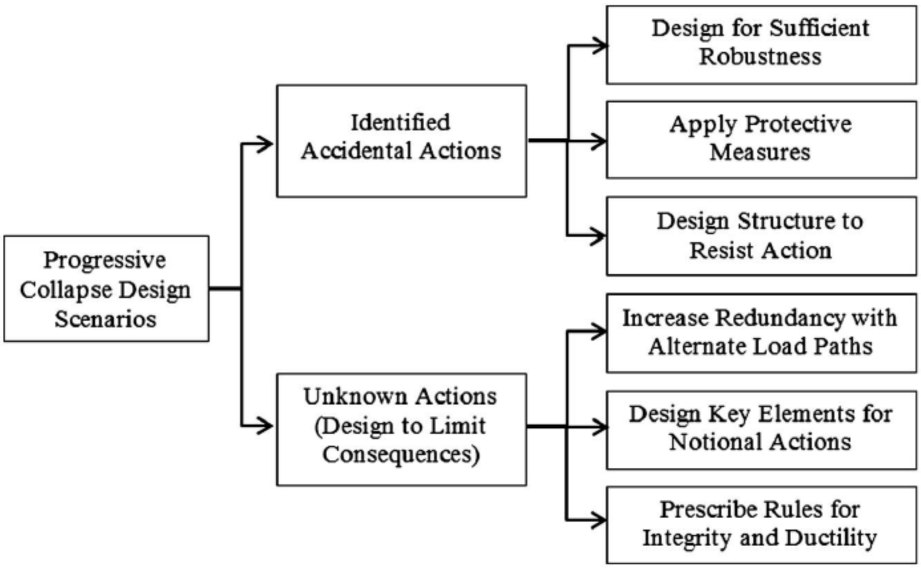


Figure 3.1. Design Strategies for progressive collapse (Stevens *et al.*, 2011).

More specifically, as indirect method BSI requires horizontal (at each floor level) and vertical tying of building components. Years later this method has become the base of so called UFC method “tie forces”, which will be explained in detail in Section 3.5.2. These ties are existing structural elements or additional slab reinforcement bars for reinforced concrete buildings, which need to be designed for the tensile force estimated at 50% of factored gravity floor loads. British code also requires that structural elements have bridging capability. This is achieved by catenary action of horizontal members following notional column removal for a load combination in Equation 3.1. Such a provision can be called as direct design method and similar to APM. According to British code, performance of a structure is satisfactory if the collapse owing to column removal is limited to 15% of the storey area, or 75 m^2. There are also other requirements for moment connections (Mohamed, 2006).

$$1.00\, D + 0.33\, (W + SD + L) \tag{3.1}$$

Where,

D = Dead load

W = Wind load

SD = Imposed dead load

L = Live load

3.2. National Building Code of Canada

National Building Code of Canada (NBCC) is one of the oldest codes which include progressive collapse resistant design guidelines. However, its provisions have been reduced in recent versions.

The 1975 edition of NBCC addressed progressive collapse mitigations under the article "Structural Integrity" as: "Buildings and structural systems shall provide such structural integrity, strength or other defenses that the hazards associated with progressive collapse due to local failure caused by severe overloads or abnormal loads not specifically covered in this Section are reduced to a level commensurate with good engineering practice." Good engineering practices and abnormal loads were explained in the same version to some extent. Some of such practices include providing ductile connections, designing key elements against being removed by abnormal events and generating alternative load paths (Dusenberry and Juneja, 2002).

In the 1977 edition progressive collapse was stated more clearly as "Structural systems for buildings shall be designed to minimize the probability that an initial local failure of a structural element, caused by an abnormal event or severe overload, will spread to other structural members and precipitate the collapse of a disproportionately large portion of the structure.". This edition included definitions for good floor plan, return on walls, strong points, tensile action in floor slabs, bracing of trusses in groups etc., which are recommended for alternative load path establishment. The level of collapse was also limited here like one storey above and below the location of abnormal event for vertical progression (Dusenberry and Juneja, 2002).

Nevertheless, the later editions of NBCC starting from 1980 excluded progressive collapse related design recommendations and stated that the structures in accordance with this code have sufficient integrity to absorb local failure without widespread collapse. Current 1995 version requires considering abnormal loads with probability of occurrence of 10^{-4} / year or more (Dusenberry and Juneja, 2002).

3.3. United States Standards - ASCE 7-05

Minimum Design Loads for Buildings and Other Structures by the American Society of Civil Engineers (ASCE 7, 2005) states under the commentary C1.4 "General Structural Integrity": "... it is usually impractical for a structure to be designed to resist general collapse caused by gross misuse of a large part of the system or severe abnormal loads acting directly on a large portion of it. However, precautions can be taken in the design of structures to limit the effects of local collapse, and to prevent or minimize progressive collapse." ASCE 7 also lists several impressive but important factors that contribute to the risk of damage propagation in modern structures. Some of them are:

- There is an apparent lack of general awareness among engineers that structural integrity against collapse is important enough to be regularly considered in design.
- To have more flexibility in floor plans and to keep costs down, interior walls and partitions are often non-load bearing and hence may be unable to assist in containing damage.
- In attempting to achieve economy in structure through greater speed of erection and less site labor, systems may be built with minimum continuity, ties between elements, and joint rigidity.
- Unreinforced or lightly reinforced load-bearing walls in multistory structures may also have inadequate continuity, ties, and joint rigidity.
- In roof trusses and arches there may not be sufficient strength to carry the extra loads or sufficient diaphragm action to maintain lateral stability of the adjacent members if one collapses.
- In eliminating excessively large safety factors, code changes over the past several decades have reduced the large margin of safety inherent in many older structures. The use of higher-strength materials permitting more slender sections compounds the problem in that modern structures may be more flexible and sensitive to load variations and, in addition, may be more sensitive to construction errors.

Three design approaches are suggested by ASCE 7: indirect design, direct design composed of alternate path method and specific local resistance method.

Indirect design method prescribes implicit consideration of resistance to progressive collapse during the design process through the provision of minimum levels of strength, continuity, and ductility. However, no quantitative requirement is specified for this method. The code defines alternate path method as: “A method that allows local failure to occur, but seeks to provide alternate load paths so that the damage is absorbed and major collapse is averted.” According to APM defined by ASCE 7, selected load-bearing elements should be notionally removed and the capacity of the remaining structure evaluated using the following load combination in Equation 3.2 in which D, L, W and S are specified dead, live, wind and snow loads determined according to Chapters 3, 4, 6 and 7 of ASCE 7-05.

$$(0.9\ or\ 1.2D) + (0.5L\ or\ 0.2S) + 0.2W \tag{3.2}$$

Specific local resistance method is suggested to design some key elements against abnormal loads by providing sufficient strength to them. This approach is useful for retrofitting slabs and columns of first floor against specified maximum pressure to resist progressive collapse (Mohamed, 2006). In order to check the capacity of a structure or structural element to withstand the effect of an extraordinary event, the following load combinations in Equation 3.3 and Equation 3.4 should be used.

$$1.2D + A_k + (0.5L\ or\ 0.2\ S) \tag{3.3}$$

$$(0.9\ or\ 1.2D) + A_k + 0.2W \tag{3.4}$$

In these combinations A_k refers to the load due to extraordinary event A. Due to high uncertainty in this type of load, the load factor is set to unity to be on the safe side. Although ASCE 7 provides very generic guidelines and does not specify analysis methods for designing against progressive collapse, it becomes the base of other detailed guidelines such as discussed in the following sections.

3.4. United States Standards - GSA 2003

After bombing of Alfred P.Murrah Federal Building in 1995 an Interagency Security Committee (ISC) was established, this was responsible for developing long-term construction standards for nonmilitary facilities. ISC required that all newly constructed facilities to be designed, and existing facilities to be checked, with the purpose of mitigating progressive collapse. As a result, the U.S. General Service Administration (GSA) developed and issued "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects" in 2000. GSA updated the guidelines on June 2003 to include new improvements and separate section for structural steel buildings.

3.4.1. Purpose and Philosophy

The purpose of the guideline is to reduce the potential of progressive collapse in new federal office buildings and assess the potential of progressive collapse in existing federal office buildings, and develop potential upgrades. Thus, the aim is to protect human life, to prevent injury and protect buildings function and assets. Being mandatory for GSA facilities, GSA also suggests guidelines be used or adopted by any agency, organization, or private concern. For these purposes the guideline uses a threat independent methodology: APM, which was introduced in Section 2.3.2. and emphasizes that the guideline is not a part of blast design. GSA utilizes liner elastic static procedure for low-to medium rise buildings (up to ten stories) as the analysis method and explains it in detail but for high rise buildings "nonlinear procedure" is proposed with few details. Overall flow for progressive collapse configuration is provided in Figure 3.2. There are three main parts of the guideline which are:

- Exemption process
- R/C buildings analysis and design
- Steel frame buildings analysis and design

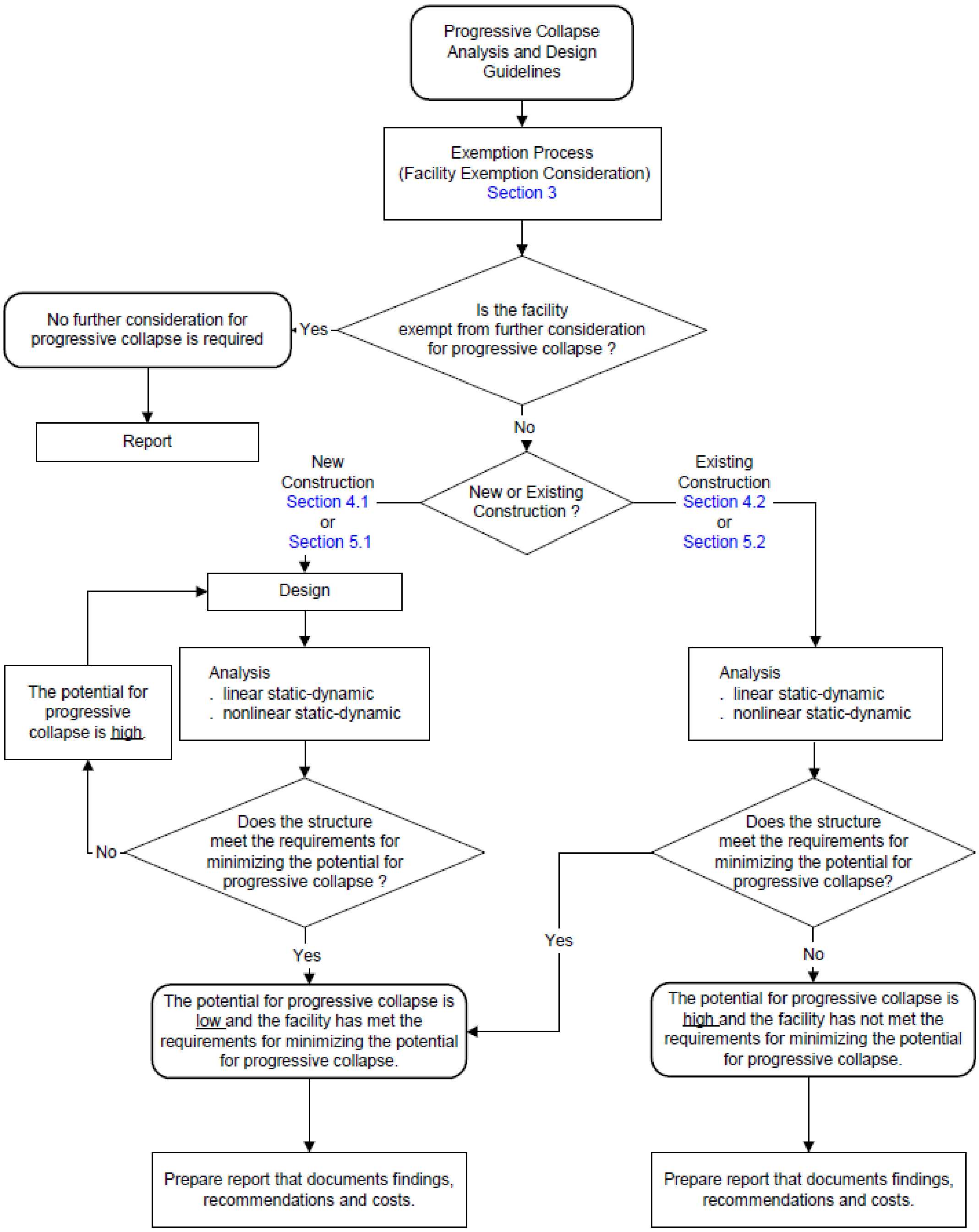


Figure 3.2. Overall flow of GSA 2003 (GSA, 2003).

Since the focus of this thesis is progressive collapse of steel framed structures, details of GSA will be provided from this perspective.

3.4.2. Exemption Process

The first step of GSA's outlined process is to evaluate a facility using a special process whether it might be exempt from further consideration for progressive collapse. After a clearly defined procedure composed of three steps, if the human occupancy is extremely low or if the facility is found to be at an extremely low risk for progressive collapse, the facility may be exempt. Especially for steel framed structures, details of exemption process are very crucial and important because they provide much information regarding the factors contributing progressive collapse resistance of steel structures.

The first step is to determine the type of the structure and the level of occupancy it has. If the structure is

- used for agriculture
- occupied by persons for a total or less than 2 hours per day
- used for up to two family dwelling
- a special structure (i.e., bridge, hydraulic structure)
- one story building of light steel frame with less than 280 m^2
- has useful life of less than 5 years
- designed for progressive collapse with proper documentation

then it is exempted from further consideration. If the structure is not exempted, the second step is applied. The second step depends on the standoff distance of the structure; however, GSA uses this only for exemption process and emphasizes again that the analysis process is threat independent. Illustration of standoff distance is provided in Figure 3.3. For steel construction minimum defended standoff distances are listed in Table 3.1. This step of exemption process might be interpreted that GSA focuses only to manmade abnormal loads, especially to blast loading, which is reasonable considering that GSA was developed after Oklahoma bombing. Having a standoff distance greater than required is not sufficient for a structure to be exempted. In addition to that if the structure:

- has no single point failure mechanism and/or atypical structural condition
- is not over ten stories

- have protected public and/or controlled parking areas and *also is designed consistent with at least seismic Zone 3 defined in the 1997 Uniform Building Code, or Seismic Design Category D or E defined in the 2000 International Building Code*

then it is exempted from further consideration. If standoff distance requirement is not met, step 3 of exemption process is applied.

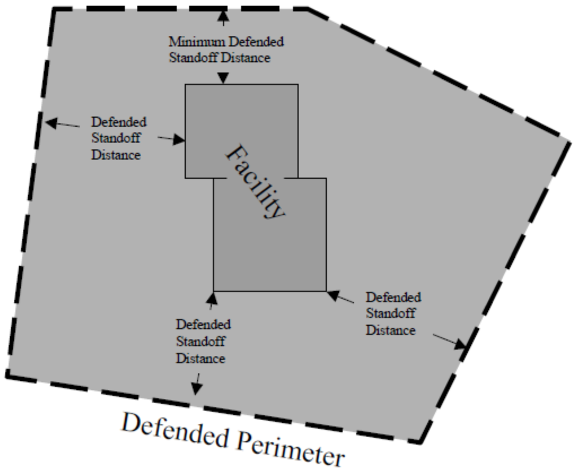


Figure 3.3. Illustration of standoff distance (GSA, 2003).

Table 3.1. Minimum defended standoff distances (GSA, 2003).

Construction Type	**Minimum Defended Standoff Distance (ft)**		
	ISC Required Level of Protection		
	Low and Medium/low	**Medium**	**Higher**
Steel Construction			
Rigid frame structure with a ***non-frangible facade*** (FEMA 310 Building Type: S4)	25	40	130
Rigid frame structure with a ***frangible facade*** (FEMA 310 Building Type: S1, S5, RM2)	25	35	100
Lightweight steel framed structures (i.e., Butler style buildings, etc.) (FEMA 310 Building Type: S1A, S2, S2A, S3, S5A)	55	105	165

At the step 3, if the primary and secondary structural members of the structure are designed against blast loads together with:

- the facility have protected public and/or controlled parking areas and *also is designed consistent with at least seismic Zone 3 defined in the 1997 Uniform Building Code, or Seismic Design Category D or E defined in the 2000 International Building Code*
- *all the perimeter bays and all affected interior bays part of continuous moment frames*
- the structure possesses all the following structural features (see Section 3.4.3.)
  (i) *discrete beam to beam continuity*
  (ii) *connection redundancy*
  (iii) *connection resilience*
  (iv) *bay width smaller than 30 ft (9.1 m)*
  (v) *story heights smaller than 16 ft (4.9 m)*
- the structure has no single point failure mechanism and/or atypical structural condition
- the primary load bearing structure *does not use the following beam to column connection*
  (i) *partially restrained moment connection*
  (ii) *pre-1995 traditional (Pre-Northridge)*
  (iii) *riveted*
  (iv) *post-1995 without successful American Institute of Steel Construction (AISC) cyclic testing*

then the structure is exempted from further consideration.

At the step 3, if the primary and secondary structural members of the structure are <u>not</u> designed against blast loads but the structure possesses all the requirements above with the strengthened first requirement below:

- the facility have protected public and/or controlled parking areas and *also is designed consistent with at least seismic Zone 4 defined in the 1997 Uniform Building Code, or Seismic Design Category F defined in the 2000 International Building Code* then the structure is also exempted from further consideration.

At the end of exemption process, it is stated that there exists limited test data for steel frame beam-to-column connections subjected to the type of loading followed by column removal as required by APM. Therefore, GSA emphasizes that the exemption process is designed to be conservative such that there will be few exempted steel framed structure.

3.4.3. Local Considerations

The following local consideration are not a requirement of GSA guidelines but recommended to be implemented during the initial phases of structural design in order to increase the probability of achieving a low potential for progressive collapse through providing extra robustness. These are:

- Providing *discrete beam-to-beam continuity* link across a column which is capable of independently redistributing gravity loads for a multiple span condition after a column loss.
- Providing *connection resilience* which is the ability of the connection to withstand rigorous and destructive loading conditions after column removal, without rupture, i.e. providing connection ductility.
- Providing *connection redundancy* which provides direct, multiple load paths through the connection. Here, the guideline addresses to use proper connection types capable of being alternative load paths after a column loss. To illustrate an improper connection type, the guideline uses the Figure 3.4.

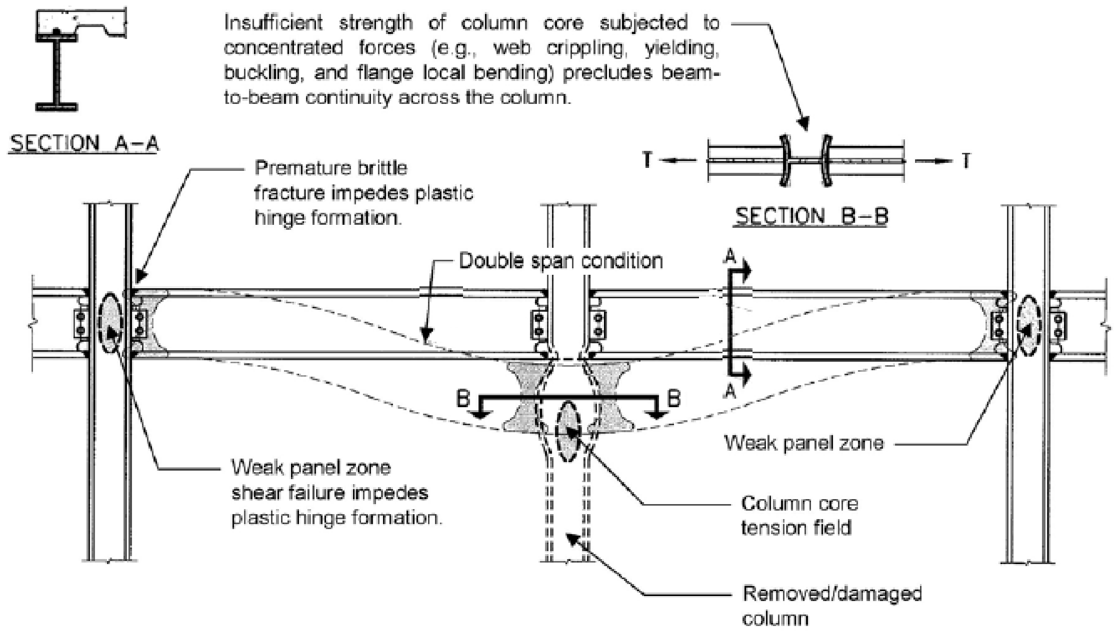


Figure 3.4. Response of traditional moment connection to a primary column loss (GSA, 2003).

- Providing sufficient *connection rotational capacity* to achieve double span condition successfully. For that purpose, it is required that connection types, whose rotational capacities comply with Seismic Provisions for Structural Steel Buildings published by AISC, are utilized.

3.4.4. Analysis Procedure and Loading

GSA 2003 utilizes the direct design approach, Alternate Load Path Method (APM). This method requires that the structure is analyzed for different cases of an instantaneous loss in primary vertical support (column or load bearing wall). A detailed explanation and critics of APM will be provided in Section 3.5.3. when discussing UFC 2009.

The guideline proposes for simple, uniform and repetitive structural layouts the following only first story analysis scenarios in Figure 3.5 and the case in Figure 3.6 for facilities having underground parking and/or uncontrolled public ground floor areas. In addition to these scenarios, different cases need to be considered for atypical structural configurations. Some of which are: vertical discontinuities, variation in bay sizes, plan irregularities, closely spaced columns and etc.

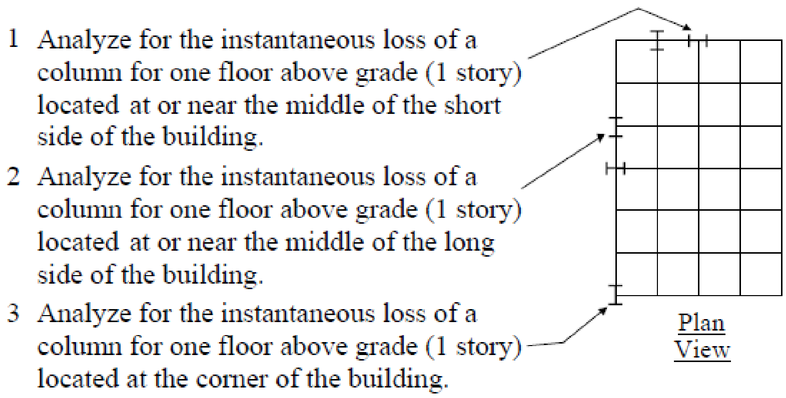


Figure 3.5. Exterior column removal scenarios (GSA, 2003).

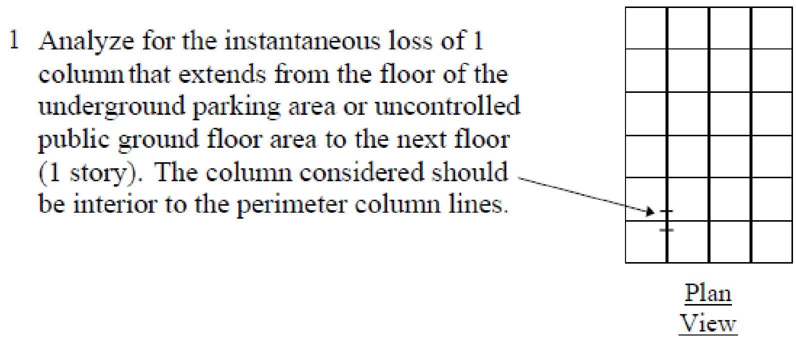


Figure 3.6. Interior column removal scenario (GSA, 2003).

After determining the necessary scenarios, the following gravity load is applied to the structure considered. Equation 3.5 stands for static analysis loading whereas Equation 3.6 is for dynamic analysis loading.

$$Load = 2(DL + 0.25LL) \tag{3.5}$$

$$Load = DL + 0.25LL \tag{3.6}$$

Where,

DL = deal load

LL = live load

3.4.5. Acceptance Criteria

In order to indentify the magnitudes and distribution of potential inelastic demands and displacements, the guideline utilizes a similar approach to the m-factor employed in Prestandard and Commentary for the Seismic Rehabilitation of Building (FEMA 356) issued by Federal Emergency Management Agency (FEMA), November 2000. The magnitude and distribution of demands is indicated by Demand-Capacity-Ratio (DCR) which is defined in Equation 3.7.

$$DCR = \frac{Q_{UD}}{Q_{CE}} \quad (3.7)$$

Where,

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

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

Table 3.2. Summary of DCR limits (GSA, 2003).

Components / Action	**Values for Liner Procedures** **DCR**
Beams-flexure	2<DCR<3 depending on both flange and web slenderness
Columns-flexure	
For $0<P/P_{CL}<0.5$	1.25<DCR<2 depending on both flange and web slenderness
For $P/P_{CL}>0.5$	1
Fully Restrained Moment Connections	2 for both Pre- and Post-Northridge
Partially Restrained Moment Connections	1.5 for tension failures 3 for flexural failures

DCR approach is utilized for linear elastic analysis, and structural members and connections with DCR values greater than the limits defined GSA in detail are regarded as severely damaged or collapsed. For atypical structures, DCR limits are reduced by a factor of 3/4. A brief summary of acceptance criteria for linear procedures is provided in Table 3.2.

According to linear elastic, static analysis procedure explained in GSA 2003, if the DCR for any member end or connection is exceeded based on shear force, the member is considered as failed. Also, if the three hinge mechanism in Figure 3.7 is formed due to exceeding flexural DCR values for both ends and span of a member, the member is considered as failed. Failed members need to be removed from the model, and all dead and live loads remained from these members should be redistributed to other members.

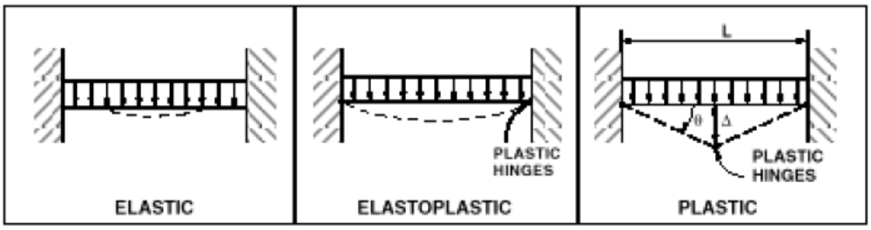


Figure 3.7. Formation of three hinge failure mechanism (GSA, 2003).

On the other hand, for only one member or connection failed due to exceedence in allowable flexural DCR, a hinge is placed at the member end or connection to release the moment. Then, equal-but-opposite moments with a magnitude of expected flexural strength (Q_{CE}) are applied to each side of inserted hinge. This procedure is continued until no DCR values are exceeded. If the DCR values are exceeded beyond the limit of allowable collapse region, which is explained in Figure 3.8, the structure is considered to have a high progressive collapse potential.

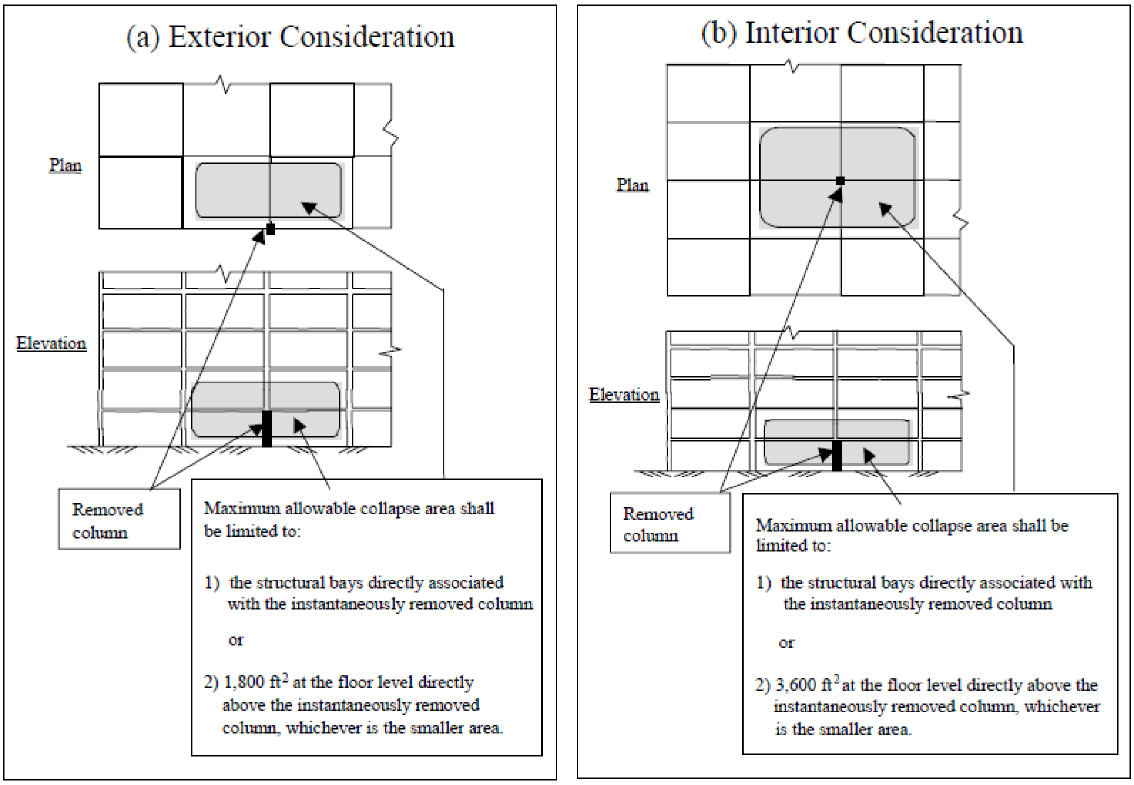


Figure 3.8. An example of maximum allowable collapse areas for a structure that uses columns for the primary vertical support system (GSA, 2003).

3.5. United States Standards - UFC 4-023-03 (DoD, 2009)

Similar to the development of GSA guidelines for progressive collapse analysis and design, motivation behind the Unified Facilities Criteria (UFC) – Design of Buildings to Resist Progressive Collapse was the terrorist attacks on September 11, 2001. This event once again but in a very efficient manner has drawn the attention of governmental and civilian bodies to the progressive collapse. In this context, arguing that there is a lack of adequate design guidance for mitigation of progressive collapse in the U.S. engineering community, U.S. Department of Defense (DoD) has started to implement design requirements. This fact is stated clearly in UFC by DoD as: "... no quantitative requirements for either direct or indirect design to resist progressive collapse are provided in ASCE 7".

For this purpose, Defense Threat Reduction Agency and GSA collaborated on a series of research projects for different type of construction materials (Stevens *et al.*, 2011). A test setup from these projects is seen in Figure 3.9. In addition to these test results, DoD used industry and civilian consensus standards (UK, Eurocode, ASCE 7, ASCE 41-Seismic Rehabilitation of Existing Buildings) to develop specific progressive collapse guideline where there are no available standards. As a result, DoD released a more comprehensive design guideline UFC 4-023-03 related to progressive collapse in 2005 and this was updated again in 2009 on the bases of developments.

Figure 3.9. Push-down test of damaged steel specimen (Stevens *et al.*, 2011).

3.5.1. Purpose and Philosophy

The purpose of this UFC is to provide design requirements necessary to mitigate the potential of progressive collapse for new and existing DoD facilities, which are at risk of getting damaged locally through abnormal events. The guideline is mandatory for the entire of buildings which are occupied by at least 25% with DoD personnel and have three or more stories. Nevertheless, UFC is also recommended for the use of other governmental and civilian organizations which create and implement building codes.

In the guideline, the commentary of ASCE 7 that it is impractical to resist against general collapse caused by the abnormal loads applied on a large portion of a structure, is once again approved. Thus, the aim of this guideline is not to directly limit or eliminate the initial damage; in contrast, the aim is to limit the effects of local collapse and to mitigate progressive collapse. Similar to other progressive collapse related codes, UFC employs threat independent approaches for these purposes. UFC utilizes direct and indirect design approaches introduced by ASCE 7. Table 3.3 provides a summary of design methods of this UFC.

Table 3.3. Summary of design approaches.

Design Approach	**Brief Definition**
Direct Design	
Alternate Path Method (APM)	The building must bridge across a removed element
Enhanced Local Resistance	Shear and flexural capacity of the perimeter columns and walls are increased to increase robustness
Indirect Design	
Tie Forces	A tensile force capacity of the floor or roof system is provided to allow load transfer from damaged portion to undamaged portion

According to this UFC, the level of progressive collapse design is based on the Occupancy Category (OC) defined in UFC 3-301-01, Structural Engineering. Occupancy categories are listed in Table 3.4 and the details are listed in Table 3.5. Specific design requirements for each OC are listed in Table 3.6.

Table 3.4. Occupancy Categories (UFC 4-023-03, 2009).

Nature of Occupancy	**Occupancy Category**
• Building in Occupancy Category 1 in \1\ Table 2-2 of UFC 3-301-01. /1/ • Low Occupancy Buildings	1
• Building in Occupancy Category 2 in \1\ Table 2-2 of UFC 3-301-01. /1/ • Inhabited buildings with less than 50 personal, primary gathering buildings, billeting, and high occupancy family housing	2
• Building in Occupancy Category 3 in \1\ Table 2-2 of UFC 3-301-01. /1/	3
• Building in Occupancy Category 4 in \1\ Table 2-2 of UFC 3-301-01. /1/ • Building in Occupancy Category 5 in \1\ Table 2-2 of UFC 3-301-01. /1/	4

Table 3.5. Details of Occupancy Categories (UFC 3-301-01, 2010).

OC	Nature of Occupancy
1	Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities
2	Buildings and other structures except those listed in Categories 1, 2, 4 and 5
3	Buildings and other structures that represent a substantial hazard to human life or represent significant economic loss in the event of failure, including, but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300 people • Buildings and other structures containing elementary school, secondary school, or daycare facilities with an occupant load greater than 250 • Buildings and other structures with an occupant load greater than 500 • Group I-2 occupancies with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities • Group I-3 occupancies • Power-generating stations; water treatment facilities for potable water, waste water treatment facilities, and other public utility facilities that are not included in Categories 4 and 5 • Buildings and other structures not included in Categories 4 and 5 containing sufficient quantities of toxic, flammable, or explosive substances to be dangerous to the public if released • Facilities having high-value equipment, as designated by the authority having jurisdiction
4	Buildings and other structures designed as essential facilities, including, but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities • Fire, rescue, and police stations, and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers, and other facilities required for emergency response • Emergency backup power-generating facilities required for primary power for Category 4 • Power-generating stations and other utility facilities required for primary power for Category 4, if emergency backup power generating facilities are not available • Structures containing highly toxic materials as defined by Section 307, where the quantity of material exceeds the maximum allowable quantities of Table 307.7(2) • Aviation control towers and air traffic control centers required for post earthquake operations where lack of system redundancy does not allow for immediate control of airspace and the use of alternate temporary control facilities is not feasible. Contact the authority having jurisdiction for additional guidance. • Emergency aircraft hangars that house aircraft required for post-earthquake emergency response; if no suitable back up facilities exist • Buildings and other structures not included in Category 5, having DoD mission-essential command, control, primary communications, data handling, and intelligence functions that are not duplicated at geographically separate locations, as designated by the using agency • Water storage facilities and pump stations required to maintain water pressure for fire suppression
5	Facilities designed as national strategic military assets, including, but not limited to: • Key national defense assets (e.g. National Missile Defense facilities), as designated by the authority having jurisdiction. • Facilities involved in operational missile control, launch, tracking, or other critical defense capabilities • Emergency backup power-generating facilities required for primary power for Category V occupancy • Power-generating stations and other utility facilities required for primary power for Category V occupancy, if emergency backup power generating facilities are not available • Facilities involved in storage, handling, or processing of nuclear, chemical, biological, or radiological materials, where structural failure could have widespread catastrophic consequences, as designated by the authority having jurisdiction

Table 3.6. Design Requirements for each OC (UFC 4-023-03, 2009).

Occupancy Category	**Design Requirements**
1	No specific requirements
2	Option 1: Tie Forces for the entire structure and Enhanced Local Resistance for the corner and penultimate columns or walls at the first story. **OR** Option 2: Alternate Path for specified column and wall removal locations.
3	Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first story columns or walls.
4	Tie Forces; Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first and second story columns or walls.

The UFC includes a wide variety of material specific design requirement including:

- Reinforced Concrete
- Structural Steel
- Masonry
- Wood
- Cold-Formed Steel

In the following sections the required procedures for each of three methods of progressive collapse design will be discussed and summarized giving importance to steel framed structures.

3.5.2. Tie Forces

In this method, the structure is tied mechanically together in order to improve continuity, ductility and development of alternate load paths. Alternate load path is necessary to transfer load from the damaged portion to the undamaged portion through catenary or membrane action. This behavior is illustrated in Figure 3.10.

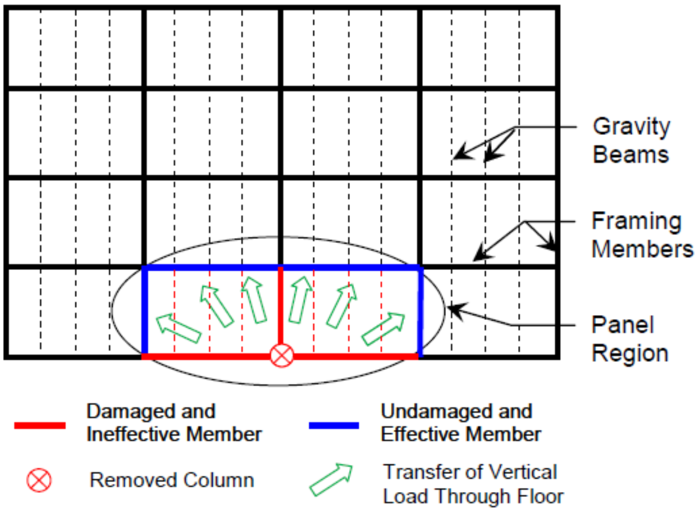


Figure 3.10. Load transfer through floor system (UFC 4-023-03, 2009).

There are two main types of ties or in other words tensile forces to be provided. These are horizontal and vertical ties. Horizontal ties are again divided into three as: longitudinal, transverse and peripheral. In Figure 3.11 tie forces for framed construction are illustrated.

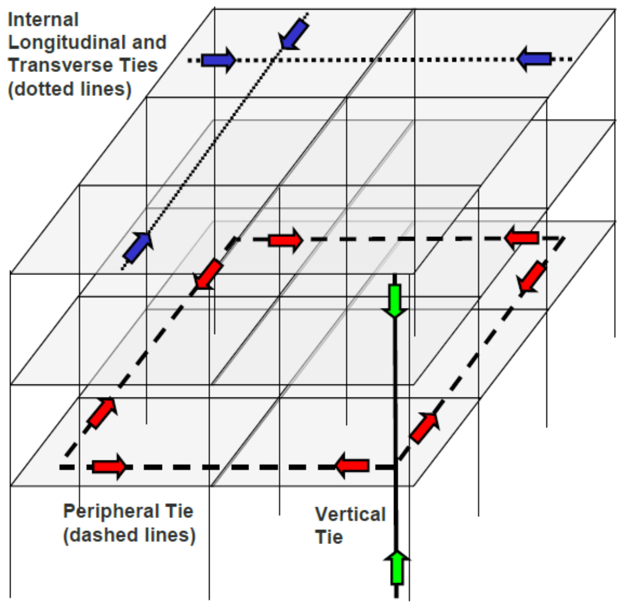


Figure 3.11. Schematic view of tie forces in a frame structure (UFC 4-023-03, 2009).

The vertical ties need to be supplied by columns and walls whereas horizontal ties need to be carried by the floor and roof system. Horizontal ties may also be carried by the structural members (beams, girders, spandrels) and their connections provided that these are capable of carrying required forces while undergoing rotations of 0.20 rad (11.2 deg). However, in the commentary section of the UFC, it is argued that there are few types of connection capable of undergoing large rotations to form catenary action.

Load and Resistance Factor Design (LRFD) approach is utilized for tie forces method and per the LRFD:

$$\emptyset\, R_n \geq \ \gamma_i Q_i \tag{3.8}$$

Where,

$\emptyset\, R_n$ = Design tie strength

$\emptyset$ = Strength reduction factor

R_n = Nominal tie strength calculated with the appropriate material specific code, including the over-strength factors from Chapters 5 to 8 of ASCE 41.

$\Sigma\gamma i\, Q_i$ = Required tie strength

γ_i = Load factor

Q_i = Load effect

Per the UFC, strength reduction factor is applied as specified in material specific codes, and design tie strength of a slab, beam, column or connection with no other load considered for regular design must be greater or equal than the required tie strength calculated using the floor load defined in Equation 3.9.

$$w_F = 1.2D + 0.5L \tag{3.9}$$

Where,

w_F = Floor Load (lb/ft^2 or kN/m^2)

D = Dead Load (lb/ft^2 or kN/m^2)

L = Live Load (lb/ft^2 or kN/m^2)

Then, the required tie strength F_i (lb/ft or kN/m) in the longitudinal or transverse direction is

$$F_i = 3w_F L_1 \tag{3.10}$$

Where,

w_F = Floor Load (lb/ft^2 or kN/m^2)

L_1 = Greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the direction under consideration (ft or m)

And the required peripheral tie strength F_p (lb or kN) is

$$F_p = 6w_F L_1 L_p \tag{3.11}$$

Where,

w_F = Floor Load (lb/ft^2 or kN/m^2)

L_1 = For exterior peripheral ties, the greater of the distances between the centers of the columns, frames, or walls at the perimeter of the building in the direction under consideration (m or ft). For peripheral ties at openings, the length of the bay in which the opening is located, in the direction under consideration.

L_p = 3-ft (0.91-m)

The location restrictions in Figure 3.12 are applied to the internal and peripheral ties if the structural members are not capable of providing tensile strength for required rotation of 0.20 rad. The vertical tie strength is carried by columns and structural walls which are tied continuously from the foundation to the roof level. The required tie force is calculated from the largest vertical load received by the column from any one story, using the tributary area loaded by the floor load w_F. If the vertical design strength of any structural element or its connection is less than the required strength, either the design must be revised or APM is used to prove that the structure can bridge over this deficient element. In contrast, APM is not allowed to be alternative to tie force method for OC 2 if horizontal tie force capacity of structural element or its connection is inadequate.

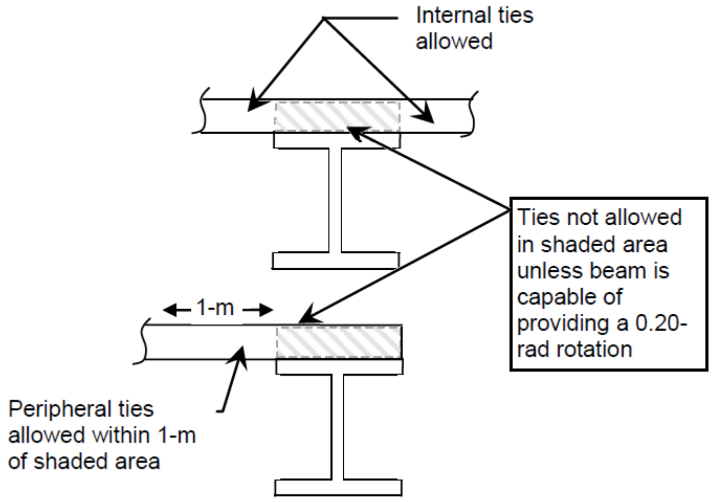


Figure 3.12. Location restrictions for ties parallel to the long axis of structural members (UFC 4-023-03, 2009).

3.5.3. Alternate Path Method (APM)

The most prevalent direct design method in the literature for designing structures against progressive collapse is APM as introduced before in Section 2.3.2. This UFC issued by DoD is the guideline which explains this method in most comprehensive manner. In fact, this UFC improves the APM introduced by ASCE 7-05 by specifying load combinations and acceptance criteria for three different analysis procedures:

- Linear Static Procedure (LSP)
- Nonlinear Static Procedure (NSP)
- Nonlinear Dynamic Procedure (NDP)

For these procedures, the guideline utilizes load factor combinations of ASCE 7-05 for extraordinary events as introduced in Section 3.3 and acceptance criteria of ASCE 41 by employing LRFD philosophy such that:

$$\emptyset R_n \geq \gamma_i Q_i \tag{3.12}$$

Where,

$\emptyset\, R_n$ = Design strength

$\emptyset$ = Strength reduction factor

R_n = Nominal strength calculated from material specific chapters of this UFC, including the over-strength factors where applicable.

$\Sigma\gamma i\, Q_i$ = Required strength

γ_i = Load factor

Q_i = Load effect

<u>3.5.3.1. Column Removal Scenarios.</u> The Alternate Path Method (APM) requires that a structural model is analyzed for different column removal scenarios in order to verify that it has enough flexural resistance to bridge over an element loss due to abnormal event. For this purpose, the UFC requires that the column with insufficient vertical tie strength is removed. In addition to that the guideline defines different internal and external column removal scenarios.

As a minimum, external columns near the middle of the short side, near the middle of the long side and at the corner of the building for each plan location listed below shall be removed as depicted in Figure 3.13:

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

Moreover, the UFC addresses engineering judgment for critical column removal locations such as:

- abrupt decrease in bay size
- re-entrant corners
- lightly loaded adjacent corners
- member frames at different orientation or elevation and etc.

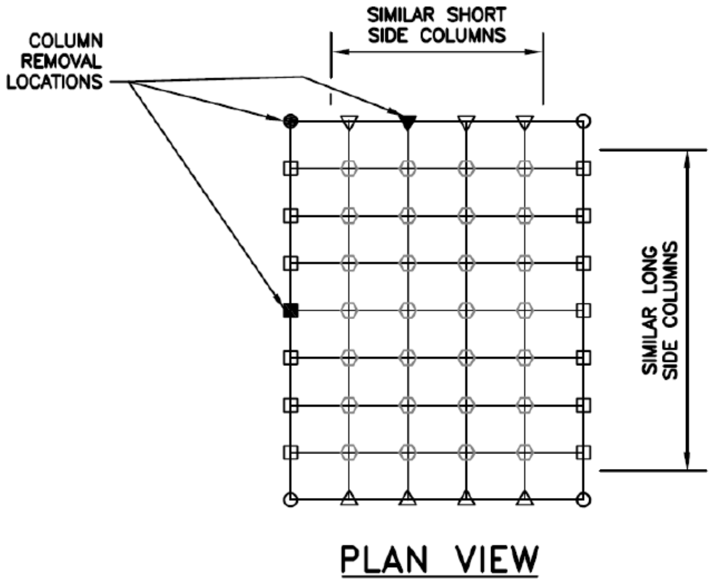


Figure 3.13. Location of external column removal (UFC 4-023-03, 2009).

Per the UFC, internal columns near the middle of the short side, near the middle of long side and at the corner of one story considered, for structures with underground parking or other areas of uncontrolled public access shall be removed as illustrated in Figure 3.14.

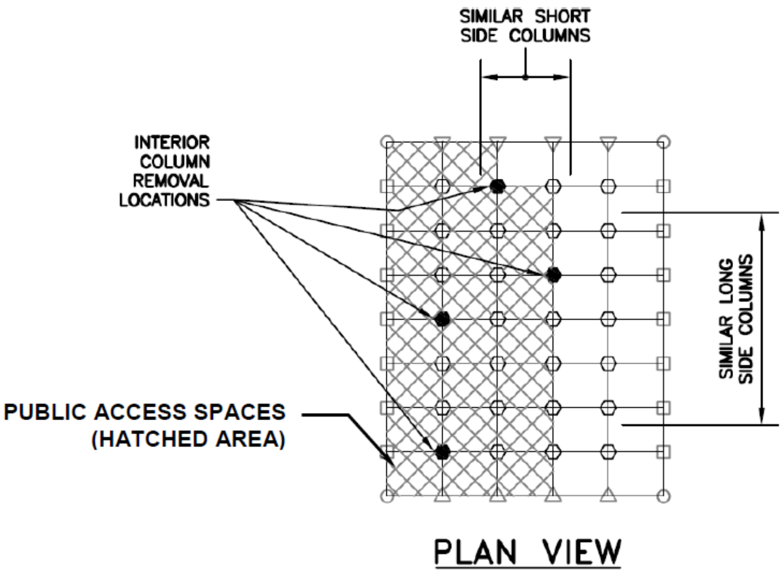


Figure 3.14. Location of internal column removal (UFC 4-023-03, 2009).

When removing columns from the structural model, the guideline requires that the beam to beam continuity is maintained as depicted in Figure 3.15.

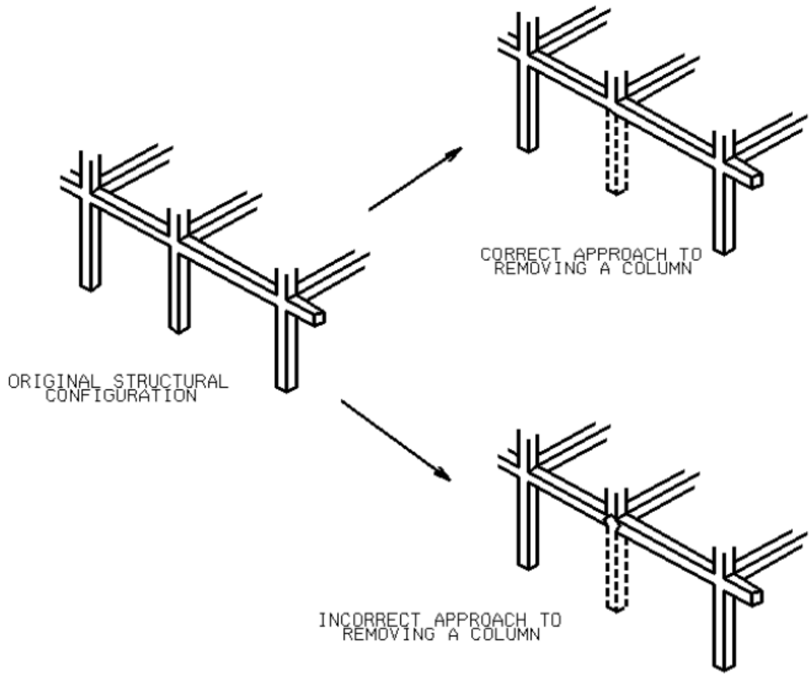


Figure 3.15. Removal of Column for APM (UFC 4-023-03, 2009).

<u>3.5.3.2. Linear Static Procedure.</u> For all analysis procedures, this UFC distinguishes force and deformation controlled actions and specifies different loading and acceptance criteria for each one. A primary component, which provides resistance to collapse, is defined as deformation controlled if it has Type 1 curve or Type 2 curve where $e \geq 2g$ as illustrated in Figure 3.16. In contrast, a primary component with Type 1 or 2 curves where $e < 2g$, or with Type 3 curve is stated as force controlled. A secondary component, which does not contribute to collapse resistance (e.g. flexural strength), is named as deformation controlled if it has a Type 1 curve with any e/g ratio or Type 2 curve with $e \geq 2g$. A secondary component having a force-deformation curve of Type 2 with $e < 2g$ or Type 3 curve is defined as force controlled. Examples of actions types are listed in Table 3.7.

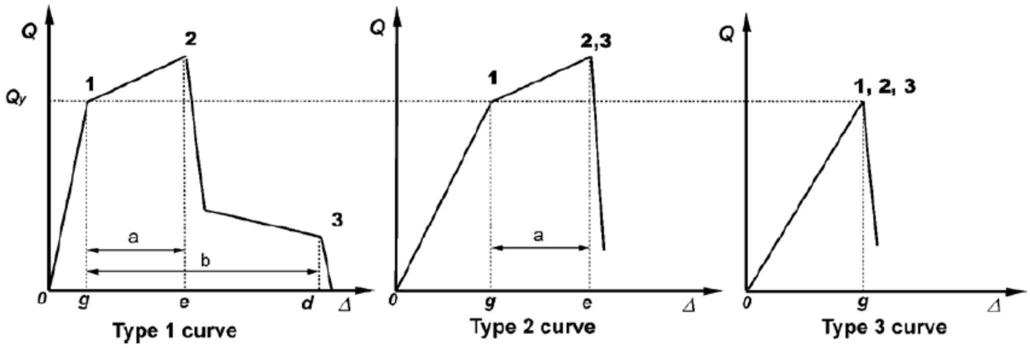


Figure 3.16. Definition of Force-Controlled & Deformation Controlled Actions (UFC 4-023-03, 2009).

Table 3.7. Examples of Force - &Deformation-Controlled Actions (ASCE 41, 2006).

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.

The guideline requires three dimensional structural models, which include stiffness and resistance of only primary elements. Secondary members might also be modeled to check their acceptance criteria provided that their stiffness's are set to zero, rather than performing hand calculations. To calculate deformation controlled actions, the load combination in Equation 3.13 is applied to the floors above removed column and to calculate force controlled actions, the load combination in Equation 3.14 is applied to the floors above removed column. The load combination in Equation 3.15 is the same for the rest of the structure in both action types. Also, the lateral load defined in Equation 3.16 is applied to the each side of the structure one side at a time. See Figure 3.18 for the illustration of the loading procedure. As a result, LSP requires two separate mathematical models for APM to be implemented.

$$G_{LD} = \Omega_{LD} \, [\,(0.9 \; or \; 1.2)D + (0.5 \, L \; or \; 0.2 \, S\,)] \tag{3.13}$$

$$G_{LF} = \Omega_{LF} \, [\,(0.9 \; or \; 1.2)D + (0.5 \, L \; or \; 0.2 \, S\,)] \tag{3.14}$$

$$G = (0.9 \; or \; 1.2)D + (0.5 \, L \; or \; 0.2 \, S\,) \tag{3.15}$$

$$L_{LAT} = 0.002 \, \Sigma P \tag{3.16}$$

Where,

G_{LD} = Increased gravity loads for deformation-controlled actions

G_{LF} = Increased gravity loads for force-controlled actions

Ω_{LD} = Load increase factor for calculating deformation-controlled actions

Ω_{LF} = Load increase factor for calculating force-controlled actions

G = Gravity loads

D = Dead load including façade loads (lb/ft^2 or kN/m^2)

L = Live load including live load reduction per ASCE 7-05 (lb/ft^2 or kN/m^2)

L_{LAT} = Lateral load

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

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

S = Snow load (lb/ft^2 or kN/m^2)

Load increase factors for deformation and force controlled actions in LSP are required to represent dynamic and nonlinear characteristics of progressive collapse. Per this UFC, the load increase factors for framed steel structures are defined below:

$$\Omega_{LD} = 0.9 m_{LIF} + 1.1 \tag{3.17}$$

$$\Omega_{LF} = 2 \tag{3.18}$$

Where,

m_{LIF} = The smallest m of any primary beam, girder, connection but not column which is directly connected to columns above the column removal location. m-Factors for LSP used in this UFC are adapted from Life Safety (LS) performance level values of Table 5-5 of ASCE 41.

The acceptance criterion is defined for deformation-controlled actions as in Equation 3.19 and for force-controlled actions as in Equation 3.20. The criteria shall be satisfied for both primary and secondary components. Linear acceptance criteria for steel frame connection are provided in Table 3.8 and for other structural components such as beam and columns; Table 5-5 of ASCE 41 is addressed.

$$\emptyset\, m\, Q_{CE} \geq Q_{UD} \tag{3.19}$$

$$m\, Q_{CL} \geq Q_{UF} \tag{3.20}$$

Where,

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

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

m = Component or element demand modifier (m-factor)

Φ = Strength reduction factor from the appropriate material specific code

Q_{CE} = Expected strength of the component or element for deformation-controlled actions from ASCE 41

Q_{CL} = Lower-bound strength of a component or element for force-controlled actions from ASCE 41

Structural irregularities are obstruction on the use of LSP. In spite of being irregular, a structure might be analyzed by LSP for APM provided that DCR of any component is less than 2.0. However, LSP is not allowed for buildings with structural irregularities such as the followings:

- significant discontinuity in the gravity load carrying or lateral force resisting system (e.g. non-stacking primary columns)

- the ratio of bay stiffness and/ or strength from one side of the column to the other are less than 50% (e.g. different beam depth on either side of the column)
- the vertical lateral load resisting elements are not parallel to the major orthogonal axes of the lateral force-resisting system (e.g. curved moment frames)

Table 3.8. Linear acceptance criteria for steel frame connections (UFC 4-023-03, 2009).

Connection Type	**Linear Acceptance Criteria**	
	***m*-factors**	
	Primary	**Secondary**
Fully Restrained Moment Connections		
Improved WUF with Bolted Web	2.3 – 0.021d[1]	4.9 – 0.048d
Reduced Beam Section (RBS)	4.9 – 0.025d	6.5 – 0.025d
WUF	4.3 – 0.083d	4.3 -0.048d
SidePlate®	6.7 – 0.039d	11.1 – 0.062d
Partially Restrained Moment Connections (Relatively Stiff)		
Double Split Tee		
• Shear in Bolt	4	6
• Tension in Bolt	1.5	4
• Tension in Tee	1.5	4
• Flexure in Tee	5	7
Partially Restrained Simple Connections (Flexible)		
Double Angles		
• Shear in Bolt	5.8 – 0.107d_{bg}[2]	8.7 – 0.161d_{bg}
• Tension in Bolt	1.5	4
• Flexure in Angles	8.9 – 0.193d_{bg}	13.0 – 0.290d_{bg}
Simple Shear Tab	5.8 – 0.107d_{bg}	8.7 – 0.161d_{bg}

[1]d = depth of beam, inch
[2]d_{bg}= depth of bolt group, inch

3.5.3.3. Nonlinear Static Procedure. There is no DCR or structural irregularity limitations stated for the use of NSP by the guideline. Three dimensional mathematical model is required, which include stiffness and resistance of only primary components. The use of secondary components in the model is optional again. If secondary components are modeled, their stiffness shall be set to zero. Per the NSP, the model shall be discretized to represent the load-deformation response of each component along its length in order to identify location of inelastic (i.e. nonlinear) action. The force-displacement behavior of all components shall be explicitly modeled as in Figure 3.17. Modeling parameters for force-displacement behavior of steel frame connections are listed in Table 3.9 by the guideline. For other structural steel components, Table 5-6 in ASCE 41 is addressed.

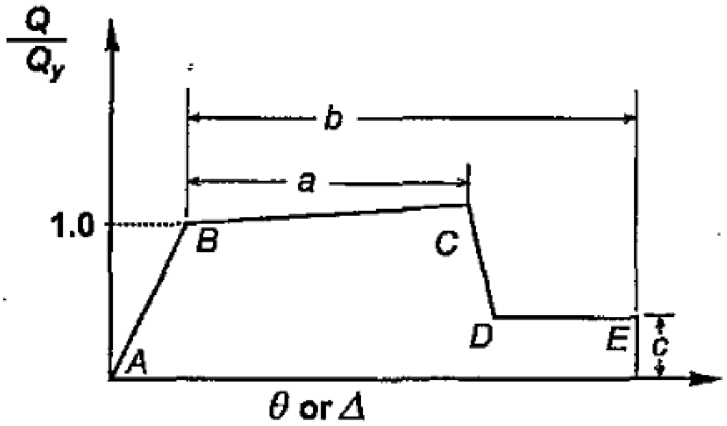


Figure 3.17. Generalized Force-Deformation relation for steel elements and components (ASCE 41, 2006).

The discretized model is then loaded with combination of gravity and lateral loads in order to calculate deformation-controlled and force-controlled actions together using the same model. The load combination in Equation 3.21 is applied to the floors above removed column and the one in Equation 3.15 is the same for the rest of the structure. Also, the same lateral load defined in Equation 3.16 is applied to the each side of the structure one side at a time. See Figure 3.18 for the illustration of the loading procedure. A load controlled procedure is utilized during the application of loads. Per the UFC, at least 10 load steps is required and the software shall be capable of incrementally increasing the load and iteratively reaching convergence before proceeding to the next load increment.

$$G_N = \Omega_N \, [\, (0.9 \; or \; 1.2)D + (0.5 \; L \; or \; 0.2 \; S\,)] \tag{3.21}$$

Where,

G_N = Increased gravity loads for nonlinear static analysis

Ω_N = Dynamic increase factor

D = Dead load including façade loads (lb/ft^2 or kN/m^2)

L = Live load including live load reduction per ASCE 7 (lb/ft^2 or kN/m^2)

S = Snow load (lb/ft^2 or kN/m^2)

Table 3.9. Modeling parameters for nonlinear modeling of steel frame connections (UFC 4-023-03, 2009).

Connection Type	Nonlinear Modeling Parameters			Nonlinear Acceptance Criteria	
	Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians	
	a	***b***	***c***	**Primary**	**Secondary**
Fully Restrained Moment Connections					
Improved WUF with Bolted Web	0.021 - 0.0003d[1]	0.050 - 0.0006d	0.2	0.021 - 0.0003d	0.050 - 0.0006d
Reduced Beam Section (RBS)	0.050 - 0.0003d	0.070 - 0.0003d	0.2	0.050 - 0.0003d	0.070 - 0.0003d
WUF	0.0284 - 0.0004d	0.043 - 0.0006d	0.2	0.0284 - 0.0004d	0.043 - 0.0006d
SidePlate®	0.089 - 0.0005d	0.169 - 0.0001d	0.6	0.089 - 0.0005d	0.169 - 0.0001d
Partially Restrained Moment Connections (Relatively Stiff)					
Double Split Tee					
• Shear in Bolt	0.036	0.048	0.2	0.03	0.040
• Tension in Bolt	0.016	0.024	0.8	0.013	0.020
• Tension in Tee	0.012	0.018	0.8	0.010	0.015
• Flexure in Tee	0.042	0.084	0.2	0.035	0.070
Partially Restrained Simple Connections (Flexible)					
Double Angles					
• Shear in Bolt	0.0502 - 0.0015d_{bg}[2]	0.072 - 0.0022d_{bg}	0.2	0.0502 - 0.0015d_{bg}	0.0503 - 0.0011d_{bg}
• Tension in Bolt	0.0502 - 0.0015d_{bg}	0.072 - 0.0022d_{bg}	0.2	0.0502 - 0.0015d_{bg}	0.0503 - 0.0011d_{bg}
• Flexure in Angles	0.1125 - 0.0027d_{bg}	0.150 - 0.0036d_{bg}	0.4	0.1125 - 0.0027dbg	0.150 - 0.0036d_{bg}
Simple Shear Tab	0.0502 - 0.0015d_{bg}	0.072 - 0.0022d_{bg}	0.2	0.0502 - 0.0015d_{bg}	0.1125 - 0.0027d_{bg}

[1] d = depth of beam, inch
[2] d_{bg} = depth of bolt group, inch

Nonlinear static dynamic increase factor is used to account for dynamic character of progressive collapse. It is calculated using the formula below. Here, the smallest ratio of θ_{pra}/θ_y for any primary element, component or connection in the model within or touching the area loaded with increased load, is used.

$$\Omega_N = 1.08 + \left(\frac{0.76}{\theta_{pra}/\theta_y + 0.83}\right) \tag{3.22}$$

Where,

Ω_N = Dynamic increase factor (DIF)

θ_{pra} = Plastic rotation angle from Table 3.10 or from Table 5-6 in ASCE 41

θ_y = Yield rotation

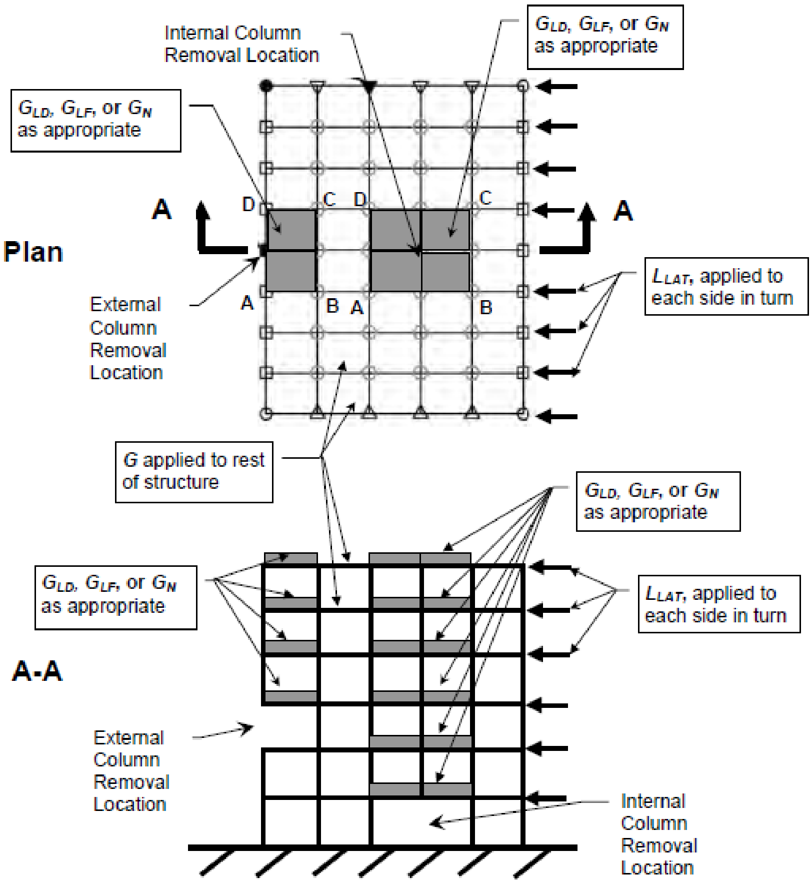


Figure 3.18. Loading procedure for LSP and NSP (UFC 4-023-03, 2009).

Plastic rotation angles of primary and secondary structural components shall be within the limits of nonlinear acceptance criteria listed in Table 3.9 for connections and Table 5-6 of ASCE 41 for other components. For the deformation controlled actions, the acceptance criterion is:

$$\emptyset\, Q_{CL} \geq Q_{UF} \tag{3.23}$$

Where,

Q_{CL} = Lower-bound strength of a component or element for force-controlled actions from ASCE 41

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

$\emptyset$ = Strength reduction factor from the appropriate material specific code

3.5.3.4. Nonlinear Dynamic Procedure. There is no DCR or structural irregularity limitations stated for the use of NDP by the guideline. Three dimensional mathematical model is required, which include stiffness and resistance of only primary components. The use of secondary components in the model is optional again. If secondary components are modeled, their stiffness shall be set to zero. Per the NDP, the model shall be discretized to represent the load-deformation response of each component along its length in order to identify location of inelastic (i.e. nonlinear) action. The force-displacement behavior of all components shall be explicitly modeled as in Figure 3.17. Modeling parameters for force-displacement behavior of steel frame connections are listed in Table 3.9 by the guideline. For other structural steel components, Table 5-6 in ASCE 41 is addressed.

The discretized model is then loaded with combination of gravity and lateral loads in order to calculate deformation-controlled and force-controlled actions together using the same model. The load combination in Equation 3.24 is applied to the entire structure. Also, the same lateral load defined in Equation 3.16 is applied to the each side of the structure one side at a time.

$$G_{ND} = \quad (0.9 \; or \; 1.2)D + (0.5 \; L \; or \; 0.2 \; S\;) \tag{3.24}$$

Where,

G_{ND} = Gravity loads for nonlinear dynamic analysis

D = Dead load including façade loads (lb/ft^2 or kN/m^2)

L = Live load including live load reduction per ASCE 7 (lb/ft^2 or kN/m^2)

S = Snow load (lb/ft^2 or kN/m^2)

Dynamic characteristic of progressive collapse is reflected by the loading procedure itself. Therefore, no dynamic increase factor is needed for NDP. Starting from zero, the loads defined in Equation 3.24 and in Equation 3.16 are applied to the entire structure including columns to be removed. When the static equilibrium is reached, the column is removed from the model instantaneously. In practice, the duration for removal must be less than the one tenth of the period associated with the mode for the vertical motion of the bays above the removed column. In the literature and also in the example provided by the UFC, a step-down force function associated with the internal loads of removed column, like in Figure 3.19 is proposed to initiate dynamic effect of the sudden column removal described above.

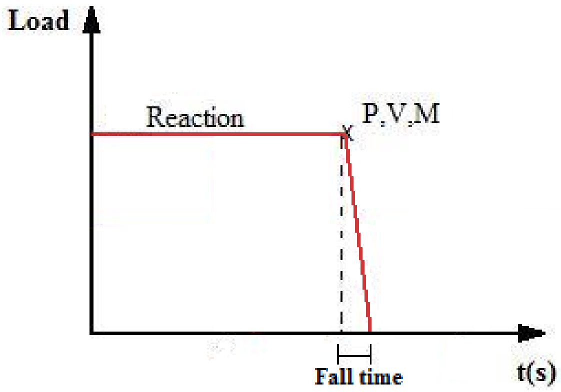


Figure 3.19. Step-down force function.

Plastic rotation angles of primary and secondary structural components shall be within the limits of nonlinear acceptance criteria listed in Table 3.9 for connections and Table 5-6 of ASCE 41 for other components. For the deformation controlled actions, the acceptance criteria are the same in Equation 3.23.

3.5.4. Enhanced Local Resistance

In the UFC 4-023-03 enhanced local resistance (ELR) is provided through the prescribed flexural and shear resistance of perimeter building columns. For OC 2, ELR is applied to the perimeter corner and penultimate columns of the first story above grade whereas for OC 3 ELR is applied to the all perimeter columns of the first story. For OC 4, all perimeter columns of the first two stories are reinforced with ELR.

In this method, enhanced local resistance is provided to the columns and their connections by providing the shear resistance equal or greater than the shear capacity associated with the flexural resistance. In other words, columns are assumed to be loaded with such a uniform load which causes flexural failure (i.e. formation of three hinge mechanism) and columns are designed such that they and their connections do not fail in associated shear. This uniform load is called as flexural resistance and it is calculated considering any effects that may increase flexural capacity (e.g. axial load, end conditions etc.). During the calculation, material over-strength factors are considered whereas strength reduction factors, ϕ are omitted. The flexural resistance shall be determined for the perpendicular direction to the perimeter façade and perimeter columns shall be evaluated for the both perpendicular directions.

For OC 2 the flexural resistance is calculated using the design after APM. For OC 4 two flexural resistances are calculated. First, the base line flexural resistance is calculated using the structural model loaded with only gravity loads. Second, the existence flexural resistance is calculated using the model after APM which includes all applied loads. Thus, the enhanced flexural resistance is the larger of the existing one or 2 times of base line flexural resistance. If 2 times of base line flexural resistance is greater than the existing one, also redesign the column according to this load.

4. RESEARCH AREAS

After the progressive collapse of Ronan Point Apartment Building in 1968 (Figure 1.1), researchers from the field of structural engineering have started to work on designing structures against progressive collapse. Due to structural type of initial event, most of earlier studies and papers were about progressive collapse concerned with panel type precast/prefabricated building systems, flat slab systems and masonry bearing wall structures. The result of early studies was provisions of the British and the Canadian codes concerning progressive collapse. At the time being, steel framed structures assumed to be more robust and ductile; thus, more collapse resistant than their counterparts like reinforced concrete structures (Khandelwal, 2008).

The first study which is also related to steel structures was the "Approaches for Design against Progressive Collapse" by Ellingwood and Leyendecker (1978). In this study, authors recommended three approaches for the mitigation of progressive collapse: event control, indirect design method and direct design method. Their work later became the basis of ASCE 7-05 and thus current design methods as discussed in 3.3. In 1983, Gross and McGuire published the first paper related to analysis for progressive collapse resistant design of steel structures. In this study, they utilized a 2-D computer analysis program, which is capable of modeling inelastic beam-column action, beam-to-column connection behavior, and the effect of shear infill panels in order to carry out APM for four story, three bays steel frames for the first time. Two column removal scenarios were considered: removal of second story external column and second story internal column. For the first case progressive collapse was predicted whereas the structure was capable of bridging over the missing internal column.

The bombing of Murrah Federal Office Building in 1995 (Figure 1.2) and attacks to World Trade Centers in 2001 have intensified research into progressive collapse and current research areas might be listed in the following manner. In the following subsections, recent publications in each research area will be summarized and discussed.

- Fundamental Issues
- Review of Code Provisions
- Modeling and Analysis
- Experimental Studies
- Connection Performance
- Effect of Catenary Action
- Bracing
- Floor Systems
- New Methodologies

4.1. Fundamental Issues

In 2007 U.S. National Institute of Standards and Technology published a very comprehensive and fundamental document named “Best Practices for Reducing the Potential for Progressive Collapse in Buildings”. According to the authors, this document provides owners and practicing engineers with current “best practices” to reduce the potential of progressive collapse of buildings subjected to abnormal loads. The report starts with a discussion of an acceptable risk approach to progressive collapse and continues with review of practical means for reducing vulnerability for new and existing buildings to control initial local failure. Then, an extensive review of current design methods to enhance resistance of structures against progressive collapse is provided. This section is followed by recommended practical design details to mitigate risk. In the appendix section, codes related to progressive collapse are reviewed and also compared. In addition, future research needs are identified. Considering all these contents, this document might be called as a reference book in the literature of progressive collapse.

Starossek and Haberland (2010) presented a detailed summary of terminology and procedures related to disproportionate collapse. Some of the terms; namely, collapse resistance, robustness, vulnerability, redundancy, redundancy and integrity were discussed. Similarities and differences between disproportionate and progressive collapse have been identified providing definitions from the literature. Suggesting working definitions and general performance-based framework for preventing progressive collapse, this paper contributes to the understanding of the literature.

Further discussion on progressive collapse which is more qualitative and fundamental can be found in Shipe and Carter (2004), Ellingwood and Dusenberry (2005), Marchand and Alfawakhiri (2004), and Mohamed (2006).

4.2. Review of Code Provisions

After reviewing codes (ASCE 7, ACI 318) and agency standards (GSA) related to progressive collapse, Nair (2004) argues that none of them would have mitigated progressive collapse of well known cases like Ronan Apartment Building and Murrah Building. Besides, he states that current provisions would have no contribution to the progressive but clearly not disproportionate collapse of World Trade Centers. His main objection to the available methods is about redundancy provided by APM, which treat every column as equally likely to be destroyed.

Ruth *et al.* (2006) examined dynamic increase factor (DIF) using eleven different 2D and 3D analytical models of steel structures and proposed that a DIF of 1.5 could better account for dynamic effects rather than conservative factor of 2.0 proposed by GSA (2003) and UFC 4-023-03 (2005). They presented that this reduced DIF will result in high level of inelasticity thus more economical and reasonable designs. Their research also concluded that parameters such as number of bays, number of stories, member sizes, member lengths, foundation constraints, loading pattern and etc. do not affect DIF noticeably. The work of Marchand *et al.* (2009) supported the findings of Ruth *et al.* (2006) related to factors about DIF. After stating inconsistencies of existing factors, the authors examined DIF and LIF separately for concrete and steel structures taking performance levels into account. As a result, they presented that DIF ranges from 1.0 to 1.4 for RC buildings and 1.2 to 1.8 for steel buildings. They proposed material specific and separate equations for DIF and LIF and the current UFC 4-023-03 (DoD, 2009) uses these equations. In a recent study, McKay *et al.* (2012) revised this paper and changed the range of DIF for RC buildings as 1.05 to 1.75.

J.Kim and T.Kim (2009) investigated progressive collapse potential of steel frames designed per Korean Building Code using both GSA (2003) and UFC 4-023-03 (2005) and observed that linear static approach of UFC is more conservative than the one of GSA. More qualitative discussion about code provisions can be found in Stevens *et al.* (2009).

4.3. Modeling and Analysis

After discussing advantageous and disadvantageous of all analysis methods: linear-elastic static; nonlinear static; linear-elastic dynamic; and nonlinear dynamic, Marjanishvili (2004) proposes an analysis method called progressive analysis method. Per this method, the analysis starts with the most conservative liner-elastic analysis and escalates to increasingly complex methods until the structure meets the evaluation criteria for that considered method. In an another paper, Marjanishvili and Agnew (2006) tested previously proposed method using GSA guidelines and discovered that linear analysis procedure of GSA is unconservative, which was also confirmed later by Marchand *et al.* (2009).

Khandelwal and El-Tawil (2005) showed that seismically designed steel perimeter frames have progressive collapse potential when a gravity frame member is lost. They utilized a 2D finite element model with reduced beam section (RBS) connection and also revealed that RBS connections increases system vulnerability. El-Tawil *et al.* (2007) developed macro models for the nonlinear progressive collapse analysis of steel buildings considering three different connection types: RBS, welded unreinforced flange- welded web (WUF-W) and shear connection. Each connection type was calibrated by finite element models and models were capable of formation of catenary action. In another study Khandelwal *et al.* (2008) used these models to compare collapse potential of two 2D steel buildings: one designed for high seismic risk and the other designed for moderate. The former performed better and the authors explained this with improved layout and stronger moment bays with less gravity columns.

Main and Sadek (2009) developed 3D, 10-story, 5-bay by 5-bay finite element steel structural model with macro modeled connections. Fu (2009) examined the progressive collapse potential of high rise steel buildings with a finite element model of 20 story structure. Unlike the previous works, the author removed not only first floor columns but also intermediate level columns. Fu (2010) used the same model to investigate effect of parameters: strength of structural steel, strength of concrete and reinforcement mesh size. Gerasimidis and Baniotopoulos (2011) studied the influence of time step size during the column loss analysis of APM. The authors utilized two well-known structural dynamics computational algorithms and proposed using a time step size of 1/300 of the period associated with structural response mode for the element removal in order to analysis result not being affected by solution algorithm. More discussion on this area might be found in Alashker *et al.*(2011), Scott and Fenves (2010).

4.4. Experimental Studies

As presented in previous section, current guidelines and standards mostly utilize seismic research data (e.g., ASCE 41-06, 2006) for the evaluation of the progressive collapse performance of structural steel components. However, Sadek *et al.* (2009) showed by experimental setups that rotational capacities of a welded unreinforced flange-bolted web (WUF-B) and RBS connections under monotonic column displacement are about twice as large as those based on seismic (i.e. cyclic) loading conditions. The authors argue that more experiments are needed to reduce conservatism in current guidelines like UFC 4-023-03 (Lew *et al.*, 2013). In another experimental work, Yang and Tan (2012) tested three connections types: web cleat, top and set angle and top and seat with web angles under middle column removal scenario. They presented that angle thickness is an important parameter controlling failure mode and catenary action.

Sezen *et al.* (2010) investigated the progressive collapse potential of two existing steel buildings through both physically testing and analytical modeling. One of the test structures was Ohio Student Union building, which was constructed in 1951 but scheduled for demolition in 2007. It was a steel moment resisting frame building with eight by two bays. The second test structure was Bankers Life and Casualty Company (BLCC) building in Illinois, built in 1968 as nine by eight bays steel moment frame. The researchers removed successively four first story columns and collected data by the means of strain gages installed to the beams and columns near the removal location. The actual observations and strain values are used to verify 2D SAP2000 models, analyzed by linear-static and nonlinear dynamic procedures of GSA (2003). Accordingly, both buildings did not collapse but only Ohio structure satisfied the GSA (2003). The BLCC building failed GSA (2003) criteria even after first column removal. In an another study Sezen *et al.* (2012) developed 3D models of the same buildings and concluded that these models compared better with actual strain data.

Chen *et al.* (2012) tested the progressive collapse performance of a two story, two by two bays laboratory building with concrete slabs. Perimeter first story middle column was removed and displacements were recorded to verify results of two finite element models. One model included slab while the other did not and the displacements from the model with slab complied better with records. Thus, the authors concluded that concrete slabs playing a significant role during load redistribution should be considered during analytical modeling of progressive collapse investigation of further studies and designs.

4.5. Connection Performance

For steel framed structures, mitigating progressive collapse depends mostly on performance (e.g., ductility, rotation capacity) of connection, which is the key for formation of tie forces, i.e. catenary action. Therefore, there are extensive researches on investigating and improving connection details which are directly adapted from seismic design.

Garcia *et al.* (2005) studied the behavior of steel connections: endplates and web cleats on the dynamic tension loading induced by progressive collapse. For this purpose, the authors developed detailed LS-DYNA finite element models and benefited Cowper and Symonds' formulation of the effect of material strain rate sensitivity. They observe that different than the static results, where ductile bearing failure of the endplate occurs; endplates fails due to bolt fracture when subjected to dynamic tension forces. Also, the faster the load is applied the lower the tension force (i.e. capacity) needed to fail the connection. However, web cleats again showed ductile failure mechanism as it is the case during static loading conditions. Khandelwal and El-Tawil (2007) developed detailed finite element subassemblies and studied behavior of connections with and without RBS; in addition, with and without transverse beam. The simulations results showed that out-of-plane pulling action imposed by transverse beam does not noticeably influence system behavior. However, system ductility is adversely influenced by an increase in beam depth and an increase in the yield to ultimate strength ratio. The study also presented that RBS sections are more ductile than subassemblies without RBS.

T.Kim and J.Kim (2009) analyzed the system behavior of steel moment frames designed for moderate and high seismic loads with FEMA connections: RBS, welded unreinforced flange–welded web (WUF-W), welded cover plated flange (WCPF). The authors developed three and six story model structures each of which was designed for moderate and strong earthquake separately. The results of nonlinear dynamic and static analysis showed that although these connections performed similar for moderate and high seismicity, their progressive collapse potential changed significantly. RBS connections of structures designed for moderate seismicity was observed to have high potential for progressive collapse. Both being safe against progressive collapse, WUF–W connection was considered to have higher progressive collapse potential than WCPF connections.

Discussion on the progressive collapse performance of Pre – Northridge connections might be found in Purasinghe *et al.* (2008) and Xu and Ellingwood (2011).

4.6. Effect of Catenary Action

Hamburger and Whittaker (2004) argued that relying on catenary action rather than flexural action of the framing system would provide more efficient design solutions against progressive collapse. Accordingly, cost efficiency might be attained by reducing beam section and number of moment connections. The authors supported their proposals by some previous experimental studies and suggested that a program of research like SAC is necessary in order to determine which type of connections will possess sufficient robustness to permit necessary plastic rotation accompanied with large tensile force.

Liu *et al.* (2005) analyzed 3 and 7 storey ordinary steel framed structures designed according to UK steel design practice using finite element modeling. The authors observed that the peak tying force demand for both analyzed structures is about 2-3 times higher than the design requirements of BS6399. However, the average tying force necessary for the development of catenary action increased by 30% for 7 storey structure compared to 3 storey structure. They argued that the connections designed to the target tying force would be insufficient to resist these forces.

Kim *et al.* (2007) investigated the collapse performance of fully restrained three FEMA connections: WUF-W, RBS, WCPF, with and without considering the effect of catenary action. As the result of actual connection subassembly tests and finite element analysis, they argued that considering catenary action provides additional tensile stress near the beam-column connection. As a consequence, the beam local buckling causing strength degradation in connections might be restrained. This ductile behavior of connections is necessary to mitigate progressive collapse of welded steel moment resisting frames. In another study Kim and Dawoon (2009) investigated the effect of considering catenary action on the global system behavior. They stated that considering catenary action decreased the maximum deflections of the structures obtained from dynamic analysis. In addition, residual capacities of structures obtained from non-linear static push-down analysis increased when it was taken into account.

More theoretical analysis of catenary behavior is provided by Liu (2010) and additional numerical analysis of different connection types subjected to catenary action can be found in Yang and Tan (2012).

4.7. Bracing

Khandelwal *et al.* (2009) investigated the progressive collapse potential of two prototype braced frame buildings designed by the National Institute of Standards and Technology (NIST). The prototype structures were office buildings with the same floor plan and total number of stories, namely 10-story. The main difference was their lateral load resisting system: one of the structures was a Special Concentrically Braced Frame

(SCBF) designed against moderate seismicity and the other was an Eccentrically Braced Frame (EBF) designed for a high seismic load. The 2D macro models of the structures were developed, and first story columns and braces removed at different plan locations considering GSA (2003) loading criteria. According to authors, none of the buildings showed progressive collapse behavior; however, EBF building was less vulnerable to progressive collapse than the SCBF.

Kim *et al.* (2011) studied the progressive collapse potential of braced frames with eight different bracing types using nonlinear static pushdown and dynamic analysis methods. The analyzed structures were all four storey with four bays SCBF designed for the same gravity and earthquake loads. First story interior column was removed from each model and collapse loading was done according to GSA (2003). As a result, all the frames did not collapse progressively and their vertical deflections were less than that of the ordinary moment frame. Also among all bracing types, inverted-V type braced frame showed superior ductile behavior when subjected to static push-down.

Asgarian and Rezvani (2012) analyzed two ten storey concentrically braced frames: one with two braced bays and the other with three. They observed that the frame with two braced bays was more robust and the dynamic amplification factor of 2 suggested by GSA (2003) might be underestimated. Chen *et al.* (2012) showed that horizontal eccentric bracing might be used to enhance progressive collapse resistance of a ten storey steel building subjected to first storey column removal. Fu (2012) developed a 3D detailed finite element model of a 20-storey steel composite building with concentric bracing and applied APM to it. Tsai (2012) proposed a performance-based design approach for retrofitting steel buildings with bracing, which are vulnerable to progressive collapse.

4.8. Floor Systems

Yu *et al.* (2010) established a detailed finite element model of a single story steel frame with composite floor slabs in order to study the influence of some parameters like joint stiffness, concrete tensile and compressive strength, decking profile on the tying performance of structure subjected to column loss. The modal was validated against the test results obtained from a laboratory work at UC Berkeley. Accordingly, the authors stated that more rigid connections and decking profile with higher moment resistance improved collapse resistance by effective tying. Also, due to crack dominant failure mode, compressive strength was ineffective and only a higher tensile capacity of concrete reduced significantly displacement during column loss. They also reported that tensile reinforcement in the vicinity of a joint was more effective than retrofitting with prestressed cable in reducing the risk of progressive collapse.

Alashker *et al.* (2010) studied the effect of key parameters like deck thickness, steel reinforcement and the number of bolts in the shear tab connection influencing the collapse resistance of composite floor systems. The authors built a detailed finite element model of only one quarter of a 2 by 2 bay obtained from a prototype steel framed building designed by NIST. The model was validated by three different test results and analyzed by two different methods: point load applied to column stub incremented with displacement control and uniformly distributed load incremented with force control. They discovered that, not steel reinforcement but steel deck was the main source of floor's tension capacity (i.e., collapse resistance). Increasing deck thickness by 100% resulted in 37% increase in overall floor capacity. The simulation results also revealed that the number of bolts in shear tab connections has little effects on the progressive collapse resistance.

In another study Alashker and El-Tawil (2011) proposed a design oriented model suitable for office use, which might be used to compute load carrying capacity of a composite floor system subjected to column loss.

4.9. New Methodologies

Ellingwood (2005) introduced the concept of structural reliability and probabilistic risk assessment to be used for mitigating the risk of progressive collapse of structures. He defined the conditional failure probabilities (Equation 2.2) on the order of 10^{-2} to 10^{-1}/yr and recommended these to be used in a system reliability evaluation. Szyniszewski (2009) calculated survival probability of occupants after column loss by means of 3D finite element analysis and theorem of total probability.

Ettouney *et al.* (2006) developed a methodology for evaluating the potential of structural instability as a result of APM. According to the authors, component level procedures like APM do not take into consideration the global system stability, which might be threatened due to nonlinearities (e.g., plastic hinges). For sway and non-sway frames, procedures and limitations were provided.

As an alternative to sophisticated dynamic, nonlinear modeling of structural system, Dusenberry and Hamburger (2006) presented two energy based but the most importantly simplified methods which are capable of assessing collapse potential of structural components. These methods are: pushdown analyses and flexural/catenary energy absorption analyses and the authors presented detailed analytical work. The idea behind these methods is simple that if the energy absorbed by the structure after element removal exceeds the change in potential energy, the structure has come to rest and survived. Xu and Ellingwood (2011) introduced an energy based nonlinear static pushdown analysis method and applied it to three steel frames. It was observed that good agreement was attained between the force and deformations obtained by both energy based method and nonlinear dynamic analysis.

Khandelwal and El-Tawil (2008) utilized a new method called pushdown analysis similar to pushover method used for seismic designs in order to assess residual capacity and failure modes of steel structures. Three types of the method were introduced: Uniform pushdown, bay pushdown and incremental dynamic pushdown. Kim *et al.* (2009) used the pushdown analysis method to study the effect of parameters like number of stories, number of spans and span length on the resisting capacity of steel frames against progressive collapse.

Izzuddin *et al.* (2008) proposed a simplified assessment framework for progressive collapse assessment of multistory buildings. Liu (2011) introduced the concept of structural optimization applied to the progressive collapse design. Khalil (2012) used another modeling technique called applied element method to model steel buildings for progressive collapse analysis.

5. CASE STUDY 1: PROGRESSIVE COLLAPSE POTENTIAL OF TWO SEISMICALLY DESIGNED STEEL BRACED FRAMES

5.1. Introduction

The purpose of this case study is to investigate the progressive collapse potential of two different steel braced frames. The braced frames were taken from the prototype steel framed buildings designed by the National Institute of Standards and Technology (NIST) of the United States for the purpose of studying their response to an abnormal event which may result in progressive collapse (Khandelwal *et al.*, 2009).

The buildings are 10 story office buildings with the same plan layout and dimensions of 45.7 x 45.7 m but with different lateral load resisting systems. One of the building is designed with Special Concentrically Braced Frames (SCBFs) as defined in the American Institute of Steel Construction (AISC) Seismic Provisions for Structural Steel Buildings (2010) to comply with seismic design category C (Atlanta, Georgia). The other building is designed with Eccentrically Braced Frames (EBFs) in order to resist lateral loads of seismic design category D (Seattle, Washington). These design categories are defined in ASCE 7-05 as moderate seismic risk and high seismic risk respectively. Therefore, the scope of this study is also observing the effect of both seismic design and bracing type on the progressive collapse behavior steel frames.

For these purposes, the selected buildings are first described in detail in the following section. Then, seismic design compatibility of prototype buildings are checked against provisions of Turkish Earthquake Code (2007) in order to verify that structural components have been designed properly and finally progressive collapse potential of 2D frames is investigated separately using APM defined in UFC 4-023-03 (DoD, 2009).

5.2. Description of Structural Systems

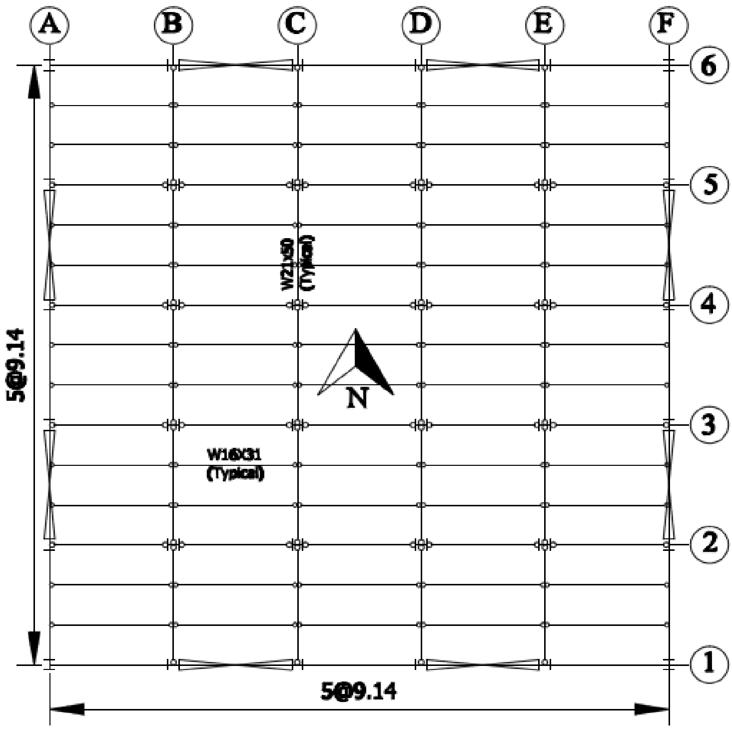
The buildings are ten story steel framed structures designed for office usage. The building with SCBFs is designed to be constructed in Atlanta, Georgia and the one with EBFs is designed to be constructed in Seattle, Washington.

The buildings structural system of both SCBF and EBF consist of perimeter braced frames and internal gravity system. The internal gravity system is same for both building structures such that 5 bays with 5 bays in each perpendicular direction with the bay width 9.14 m. The first story height is 5.33 m and the other remaining 9 story height is 4.2 m thus, the buildings have both 45.7x45.7 m plan dimensions and 43.13 m of total height. Plan and 3D views of buildings with SCBFs and EBFs are shown in Figure 5.1 and Figure 5.2 respectively.

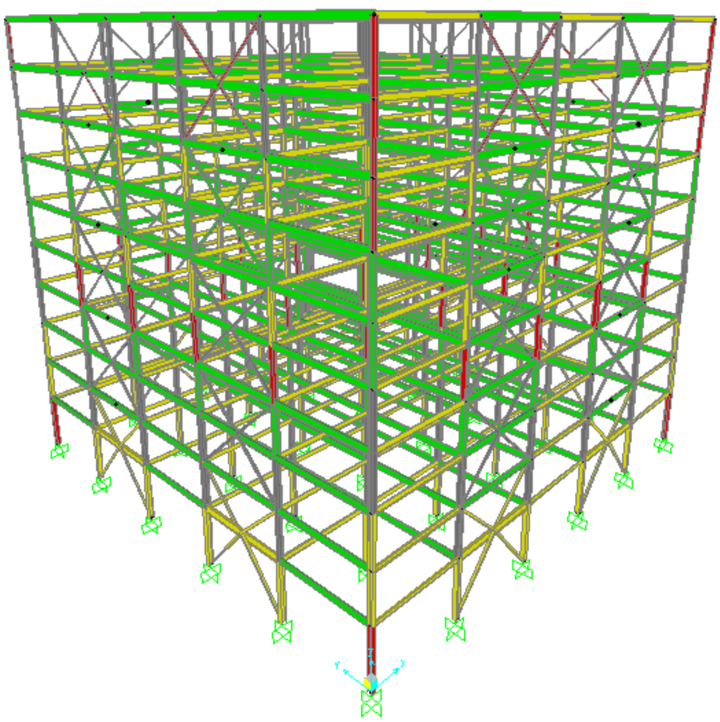
The building with SCBFs has two braced bays in East-West (E-W) and North-South (N-S) elevations. From various concentric bracing types, V + inverted V type bracing is applied to the structure. The building with EBFs has three braced bays with different configuration in each elevation direction. Eccentricity of the braces are determined as e=120 cm from the seismic analysis results presented in Section 5.3.5.

The material and design standards used in the design of structural members and their connections are listed below:

- International Building Code (IBC, 2006)
- AISC – Specifications for Structural Steel Buildings (AISC, 2005)
- AISC – Steel Construction Manual (AISC, 2006)
- AISC – Seismic Provisions for Structural Steel Buildings (AISC, 2005)

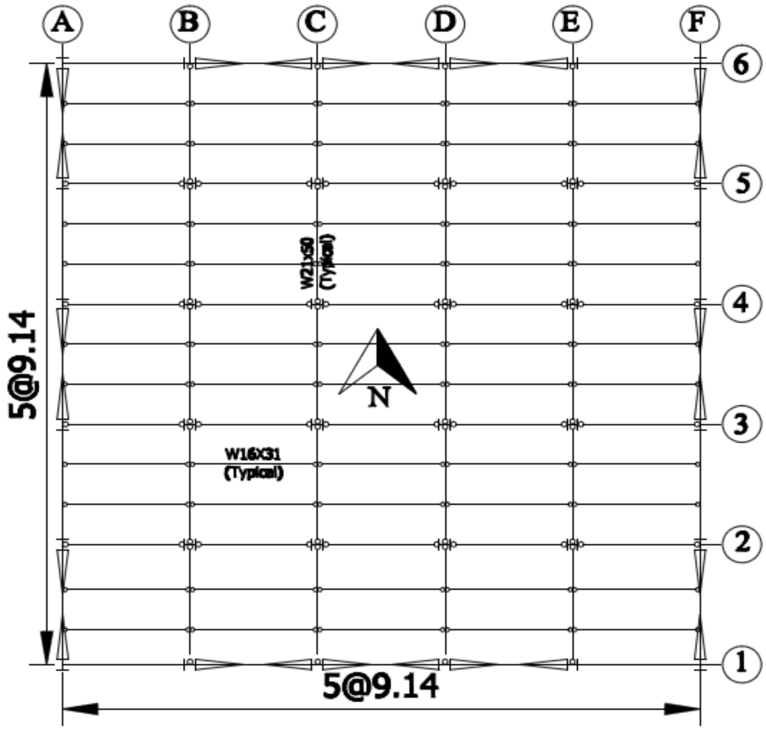


a) Plan view

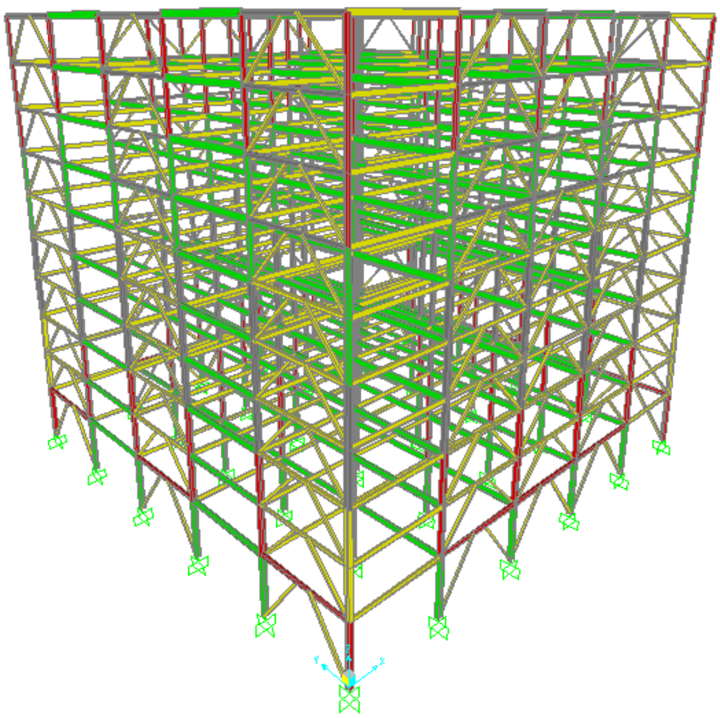


b) 3D view (Secondary beams omitted)

Figure 5.1. Plan and 3D view for the building with SCBFs.



a) Plan view



b) 3D view (Secondary beams omitted)

Figure 5.2. Plan and 3D view for the building with EBFs.

Gravity loads determined according to ASCE 7- 05 are:

For typical floors,

$$g = 2.202 \text{ kN/m}^2 \quad \text{(Self weight of the slab)} \tag{5.1}$$

$$g = 1.436 \text{ kN/m}^2 \quad \text{(Super imposed dead load)} \tag{5.2}$$

$$q = 4.788 \text{ kN/m}^2 \quad \text{(Live Load)} \tag{5.3}$$

For the roof,

$$g = 2.202 \text{ kN/m}^2 \quad \text{(Self weight of the slab)} \tag{5.4}$$

$$g = 0.479 \text{ kN/m}^2 \quad \text{(Super imposed dead load)} \tag{5.5}$$

$$q = 0.958 \text{ kN/m}^2 \quad \text{(Live Load)} \tag{5.6}$$

The braces of both buildings are selected from seismically compact Hollow Steel Sections (HSS) with the material ASTM A500 Grade B Steel (σ_y = 317 MPa). For beam and columns, hot rolled American Wide Flange (AWF) sections with the material A992 structural steel (σ_y = 345 MPa) are utilized. Consequently, sectional dimensions of the building with SCBFs are listed in Table 5.1 and in Table 5.2 with the associated N-S and E-W elevation in Figure 5.3.

Table 5.1. Member sizes of the building with SCBFs – N-S Elevation.

Story	Columns		Beams		Braces
	A/F axes	B/C/D/E axes	A-B/C-D/E-F	B-C/D-E	
10	W14x53	W14x43	W16x31	W21x50	HSS 4-1/2x4-1/2x3/8
9	W14x53	W14x43	W16x31	W21x50	HSS 4-1/2x4-1/2x3/8
8	W14x53	W14x74	W16x31	W21x50	HSS 5-1/2x5-1/2x3/8
7	W14x82	W14x82	W16x31	W21x50	HSS 5-1/2x5-1/2x3/8
6	W14x82	W14x120	W16x31	W21x50	HSS 6x6x1/2
5	W14x99	W14x132	W16x31	W21x50	HSS 6x6x1/2
4	W14x99	W14x176	W16x31	W21x50	HSS 6x6x1/2
3	W14x120	W14x193	W16x31	W21x50	HSS 6x6x1/2
2	W14x120	W14x193	W16x31	W21x50	HSS 7x7x1/2
1	W14x145	W14x233	W16x31	W24x76	HSS 7x7x1/2

Table 5.2. Member sizes of the building with SCBFs – E-W Elevation.

Story	Columns		Beams		Braces
	A/F axes	B/C/D/E axes	A-B/C-D/E-F	B-C/D-E	
10	W14x53	W14x43	W21x50	W21x50	HSS 4-1/2x4-1/2x3/8
9	W14x53	W14x43	W21x50	W21x50	HSS 4-1/2x4-1/2x3/8
8	W14x53	W14x74	W21x50	W21x50	HSS 5-1/2x5-1/2x3/8
7	W14x82	W14x82	W21x50	W21x50	HSS 5-1/2x5-1/2x3/8
6	W14x82	W14x120	W21x50	W21x50	HSS 6x6x1/2
5	W14x99	W14x132	W21x50	W21x50	HSS 6x6x1/2
4	W14x99	W14x176	W21x50	W21x50	HSS 6x6x1/2
3	W14x120	W14x193	W21x50	W21x50	HSS 6x6x1/2
2	W14x120	W14x193	W21x50	W21x50	HSS 7x7x1/2
1	W14x145	W14x233	W21x50	W24x76	HSS 7x7x1/2

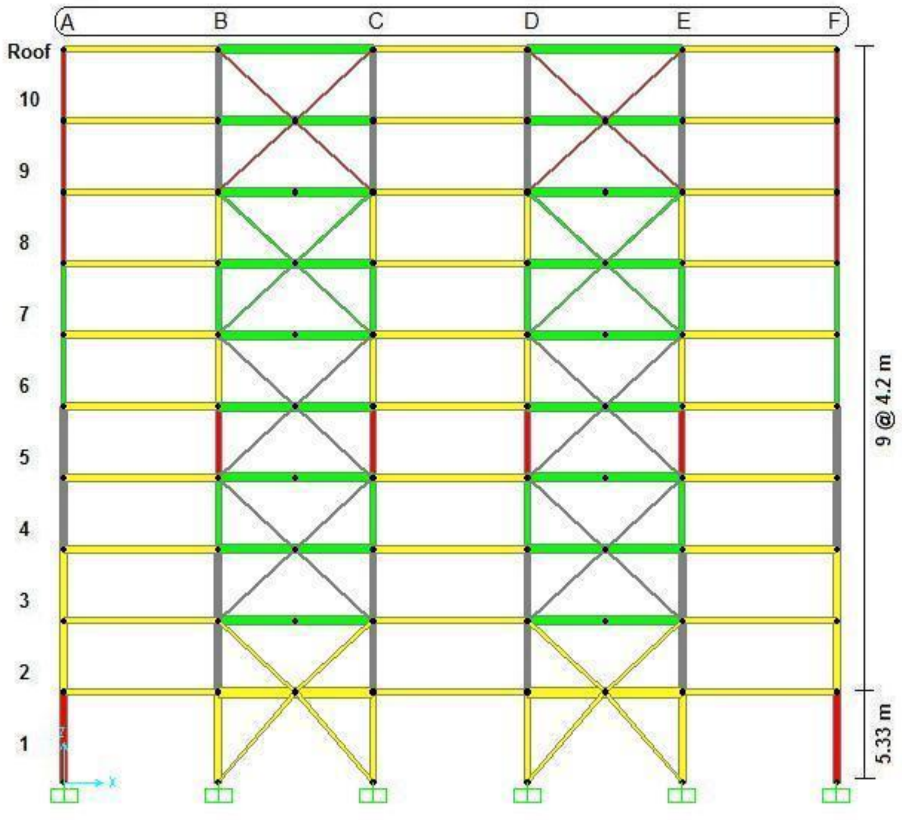


Figure 5.3. N-S & E-W Elevation – SCBF.

Sectional dimensions of the building with EBFs are listed in Table 5.3 and 5.4. The associated N-S elevation, which is used in progressive collapse analysis, is as in Figure 5.4 and the E-W elevation is as in Figure 5.5. Gravity beams are identical for both buildings and they are as shown in plan views.

Table 5.3. Member sizes of the building with EBFs – N-S Elevation.

Story	Columns		Beams		Braces
	A/F axes	B/C/D/E axes	A-B/E-F	B-C/C-D/D-E	
10	W14x53	W14x48	W16x31	W10x39	HSS 7x7x1/2
9	W14x53	W14x48	W16x31	W10x39	HSS 7x7x1/2
8	W14x53	W14x61	W16x31	W10x39	HSS 7x7x1/2
7	W14x82	W14x82	W16x31	W10x39	HSS 7x7x1/2
6	W14x82	W14x109	W16x31	W12x45	HSS 8x8x1/2
5	W14x99	W14x109	W16x31	W12x45	HSS 8x8x1/2
4	W14x99	W14x109	W16x31	W12x45	HSS 8x8x1/2
3	W14x120	W14x132	W16x31	W12x45	HSS 8x8x1/2
2	W14x120	W14x145	W16x31	W12x45	HSS 8x8x1/2
1	W14x145	W14x176	W16x31	W14x48	HSS 8x8x1/2

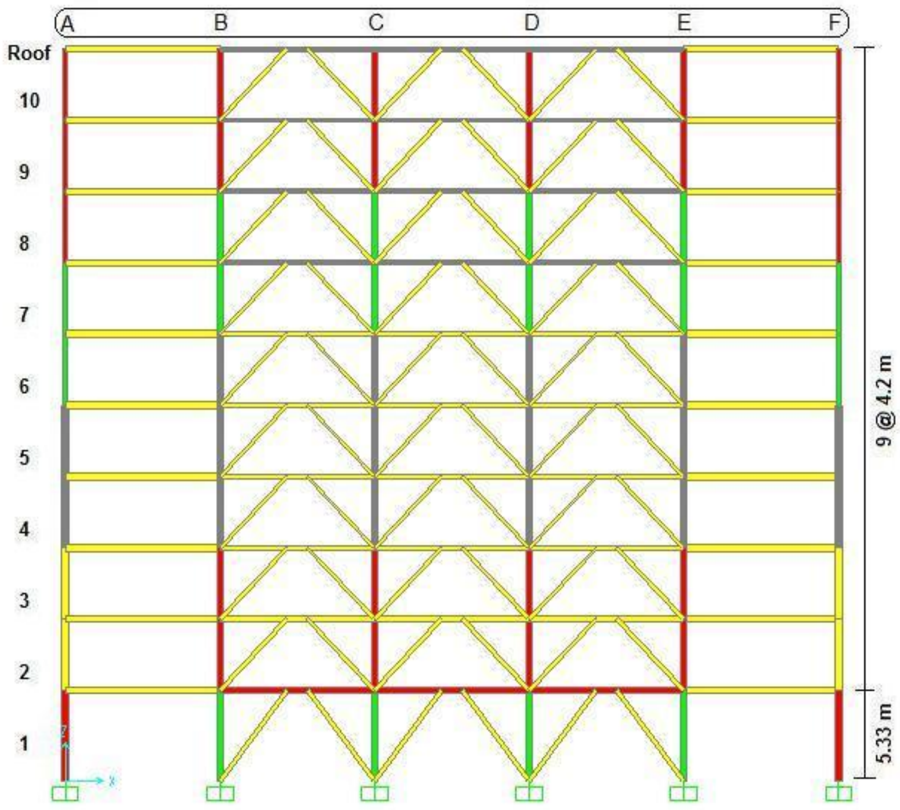


Figure 5.4. N-S Elevation – EBF (Line 1 & 6).

Table 5.4.Member sizes of the building with EBFs – E-W Elevation.

Story	Columns		Beams		Braces
	A/F axes	B/C/D/E axes	B-C/D-E	A-B/C-D/E-F	
10	W14x53	W14x48	W21x50	W10x39	HSS 7x7x1/2
9	W14x53	W14x48	W21x50	W10x39	HSS 7x7x1/2
8	W14x53	W14x61	W21x50	W10x39	HSS 7x7x1/2
7	W14x82	W14x82	W21x50	W10x39	HSS 7x7x1/2
6	W14x82	W14x109	W21x50	W12x45	HSS 8x8x1/2
5	W14x99	W14x109	W21x50	W12x45	HSS 8x8x1/2
4	W14x99	W14x109	W21x50	W12x45	HSS 8x8x1/2
3	W14x120	W14x132	W21x50	W12x45	HSS 8x8x1/2
2	W14x120	W14x145	W21x50	W12x45	HSS 8x8x1/2
1	W14x145	W14x176	W21x50	W14x48	HSS 8x8x1/2

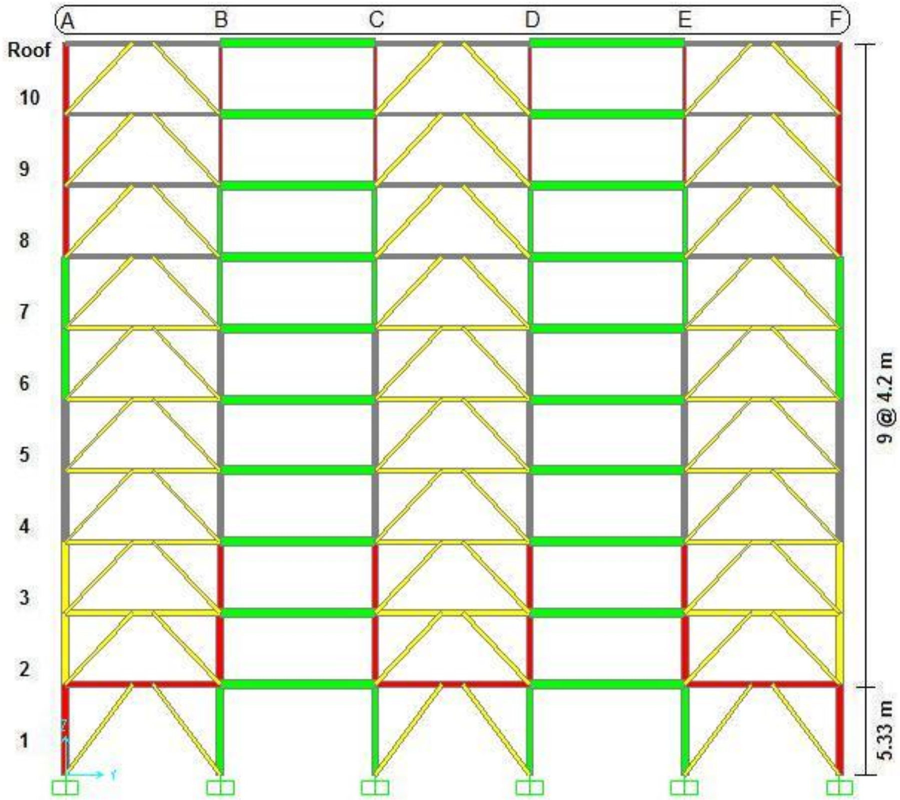
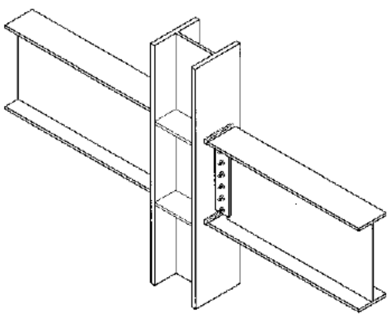
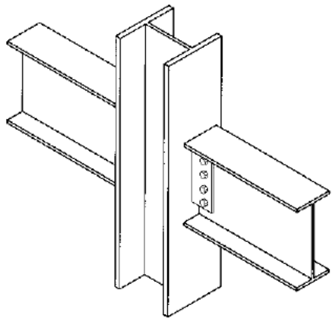


Figure 5.5. E-W Elevation – EBF (Line A & F).

In the internal gravity frame, beams and columns are connected to each other with single plate simple shear tab connections (See Figure 5.7) as depicted in Figure 5.1a and Figure 5.2a. The buildings have limited number of fully restrained welded moment connections (WUF-W, see Figure 5.7) as shown in Figure 5.6.

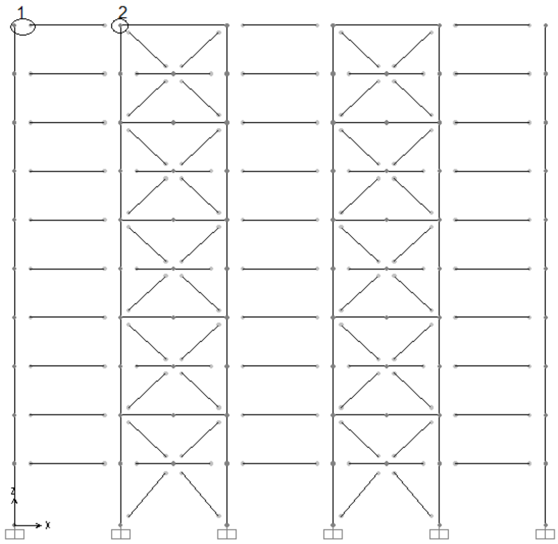


a) WUF-W

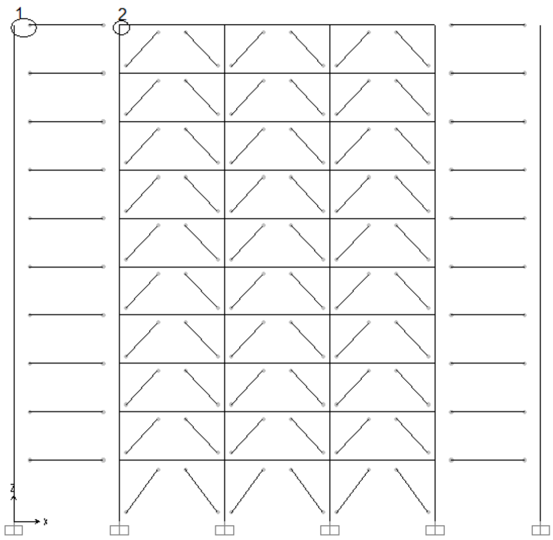


b) Simple shear tab connection

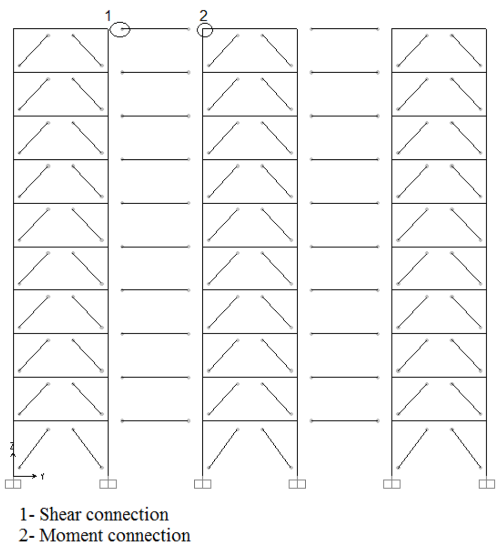
Figure 5.6. Connection details (UFC 4-023-03, 2009).



a) E-W / N-S elevation – SCBF



b) N-S elevation – EBF



c) E-W elevation – EBF

Figure 5.7. Perimeter frames connection detail.

5.3. Checking Seismic Design Compatibility

In this section it is checked whether the selected prototype braced buildings meet the seismic design requirements. For this purpose, 3D and 2D macro elastic models of buildings have been developed using structural analysis program SAP2000 Advanced 14.2.2. As introduced in previous Section 5.2., for the progressive collapse analysis N-S perimeter frames of the buildings were utilized. Therefore, the discussion in this section is mainly concentrated on the seismic performance of SCBF and EBF on x-direction. Codes used in the seismic assessment are listed in Table 5.5.

Table 5.5. Codes used for seismic assessment.

Code	Explanation
Principles For Buildings to be Constructed on Earthquake Zones (2007) Named as Turkish Earthquake Code (TEC 07) from now on	Used as main reference for seismic design requirements
Seismic Provisions for Structural Steel Buildings (AISC, 2010)	Used as reference seismic code when necessary
TS 648-Building Code for Steel Structures (TSE, 1980)	Used as main steel design code
Specifications for Structural Steel Buildings (AISC, 2010)	Used as reference steel design code when necessary
TS 498 -Design Loads for Buildings (TSE, 1997)	Used for live load reduction
ASCE 7-05 - Minimum Design Loads for Buildings and Other Structures (ASCE, 2005)	Used for determining the earthquake loads

5.3.1. Modeling

3D macro elastic models of both buildings have been developed to carry out earthquake analysis required by TEC 07. Structural members have been modeled using frame elements. In the models, secondary beams were omitted. WUF-W connections were modeled as fixed connection with no moment release whereas shear connections were modeled as pin connections. Slabs were also not modeled but diaphragm action has been provided; therefore, systems were loaded with line loads introduced in Section 5.4.1. 3D structural models of the buildings are provided in Figure 5.1 and 5.2. Also, 2D frame models have been produced in order to obtain loads directly carried by braces and EBF links.

5.3.2. Loading Procedure

5.3.2.1. Gravity Loads. Gravity loads applied to the models are as introduced in Section 5.2 with the values 5.1 to 5.6. These area loads have been converted to line loads obeying two way working slab procedure and applied to the 3D models as explained below. Live load reduction was applied per TS498 as shown in Table 5.6. Dead loads of the elements were applied by the analysis program automatically.

For normal floors,

on line A & F , 1 & 6 (See Figure 5.1 and 5.2)

$$g = \frac{3.64x9.14}{4} = 8.32\ \frac{kN}{m} \qquad q = \frac{4.79x9.14}{4} = 10.95\ \frac{kN}{m}$$

on line B-C-D-E , 2-3-4-5

$$g = 2\frac{3.64x9.14}{4} = 16.63\ \frac{kN}{m} \qquad q = 2\frac{4.79x9.14}{4} = 21.89\ \frac{kN}{m}$$

For the roof ,

on line A & F , 1 & 6

$$g = \frac{2.68x9.14}{4} = 6.12\frac{kN}{m} \qquad q = \frac{0.96x9.14}{4} = 2.19\ \frac{kN}{m}$$

on line B-C-D-E , 2-3-4-5

$$g = 2\frac{2.68x9.14}{4} = 12.24\ \frac{kN}{m} \qquad q = 2\frac{0.96x9.14}{4} = 4.38\frac{kN}{m}$$

Table 5.6. Live load reduction factors.

Story	1	2	3	4	5	6	7	8	9	Roof
Reduction factor	1	1	1	0.95	0.88	0.8	0.71	0.65	0.6	1

5.3.2.2. Earthquake Loads. Equivalent earthquake load method introduced in TEC 07 has been utilized in order to define earthquake loads applied to the each floor level. According to TEC 07, this method is allowed for structures having no A1 and B1 type irregularity with total height $H_N \leq 40m$. For each building type it will be shown that these irregularity types; namely, torsional and weak story respectively, are not present. Since both building types have already been designed according to AISC seismic design rules, the fact that $H_N = 43.13\ m \geq 40m$ is assumed not affecting verification significantly.TEC 07 defines total base shear as in Equation 5.7.

$$V_t = \frac{WA(T_1)}{R_a(T_1)} \geq 0.10A_0IW \tag{5.7}$$

$$W = \sum_{i=1}^{N} w_i \tag{5.8}$$

$$w_i = g_i + nq_i \tag{5.9}$$

Where,

V_t = Total base shear

W = Total weight of the structure defined by Equation 5.8

A = Spectral acceleration

R_a = Response modification factor

T_1 = 1. Modal period of the structure

I = Importance factor

A_0 = Effective ground acceleration coefficient

w_i = Weight of single story defined by Equation 5.9

g_i = Total dead load of single story

q_i = Total live load of single story

n = Live load participation factor

Since prototype structures have been designed according to loads defined by ASCE 7-05, spectral acceleration needed for the total base shear calculation will be obtained for each building type accordingly. ASCE 7-05 defines design response spectrum as shown in Figure 5.8. T_0 and T_S periods are determined by Equation 5.10 and Equation 5.11 respectively. Response accelerations S_{D1} and S_{DS} are calculated by Equation 5.12 and Equation 5.14 respectively.

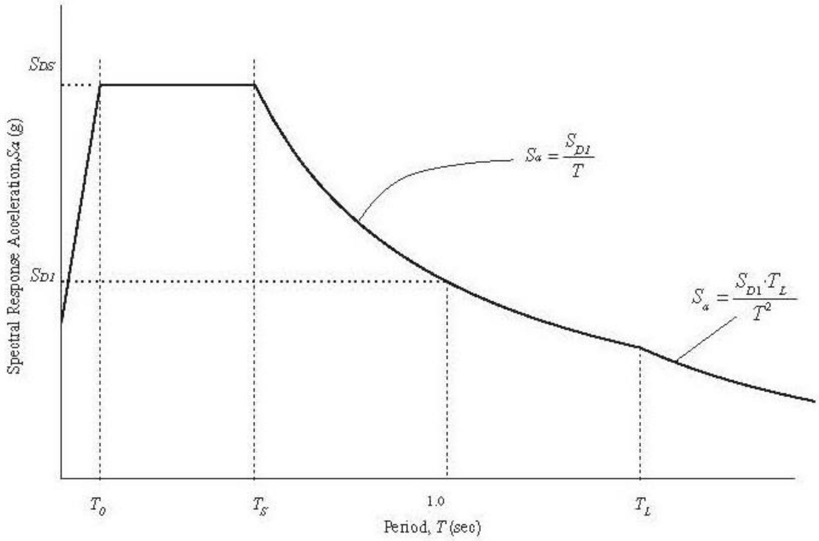


Figure 5.8. Design response spectrum (ASCE 7-05).

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}} \tag{5.10}$$

$$T_S = \frac{S_{D1}}{S_{DS}} \tag{5.11}$$

$$S_{D1} = \frac{2}{3}S_{M1} \tag{5.12}$$

$$S_{M1} = \mathrm{F_v S_1} \tag{5.13}$$

$$S_{DS} = \frac{2}{3}S_{MS} \tag{5.14}$$

$$S_{MS} = \mathrm{F_a S_s} \tag{5.15}$$

Where,

S_a = Spectral response acceleration

S_{D1} = Design spectral response acceleration parameter at 1-s period

S_{DS} = Design spectral response acceleration parameter at short periods

S_{M1} = 5 percent damped spectral acceleration at 1-s period adjusted for site

S_{MS} = 5 percent damped spectral acceleration at short periods adjusted for site

F_v = Long period site coefficient

F_a = Short period site coefficient

S_1 = Mapped 5 percent damped spectral acceleration at 1-s period

S_S = Mapped 5 percent damped spectral acceleration at short periods

5.3.3. Load Combinations

Due to the symmetry present in the structures and considered direction, load combinations used for elastic static analysis of the buildings are reduced to combinations listed in Table 5.7.

Table 5.7. Load combinations.

Combination	Dead	Floor GL	Floor LL	Roof GL	Roof LL	EQ+X
1G+1Q	1.0	1.0	1.0	1.0	1.0	0.0
1G+1Q+1E	1.0	1.0	1.0	1.0	1.0	1.0
0.9G+E	1.0	0.9	0.0	0.9	0.0	1.0
1G+1Q+2E	1.0	1.0	1.0	1.0	1.0	2.0
0.9G+2E	1.0	0.9	0.0	0.9	0.0	2.0
1G+1Q+2.5E	1.0	1.0	1.0	1.0	1.0	2.5
0.9G+2.5E	1.0	0.9	0.0	0.9	0.0	2.5

5.3.4. The Building with SCBFs

5.3.4.1. Equivalent Earthquake Loads. From ASCE 7-05, for the building with SCBFs, which is designed to be located in Atlanta, Georgia: S_S = 0.230 and S_1 = 0.086. Assuming site class E, from table 11.4-1 and 11.4-2 of ASCE 7, F_a = 2.5 and F_v = 3.5, parameters of design spectrum of the building with SCBFs is listed in Table 5.7 .Thus, $0.33 \leq S_{DS} < 0.50$, which means that the structure is assigned to Seismic Design Category C as stated before.

Table 5.8. Design spectrum parameters – SCBF system.

S_{MS}	0.575 g
S_{DS}	0.380 g
S_{M1}	0.301 g
S_{D1}	0.200 g
T_0	0.105 s
T_S	0.530 s

From modal analysis, the period of the building with SCBFs in x-direction is T_x = 2.08s; thus, spectral response acceleration S_a = 0.096 g .Calculation of the total weight of the structure with SCBFs is shown in Table 5.9. Total base shear with the importance factor I = 1 and R = 5 is (Assume A_0 =0.2)

$$V_t = \frac{106106.45x0.096}{5} = 2037.2\ kN \leq 0.1x0.2x1x106106.45 = 2122.1\ kN$$

$$V_t = 2122.1\ kN$$

Table 5.9. Total weight – SCBF system.

SOURCE	CALCULATION	(kN)	
FRAME WEIGHT	SAP2000	6693.90	
DEAD LOAD-9 STORY	3.638x45.7x45.7x9	68381.34	
DEAD LOAD-ROOF	2.681x45.7x45.7	5599.24	
FLOOR BEAM-10 FLOOR	50x10x9.14x0.4513	2062.44	
FLOOR LIVE LOAD	45.7x45.7x4.788x0.3x7.59	22769.29	
ROOF LIVE LOAD	45.7x45.7x0.3x0.958	600.23	
		Σ	**106106.45**

Total base shear is distributed to the each floor level according to Equation 5.16, Equation 5.17 and Equation 5.18 as defined in TEC 07. The procedure is described in Figure 5.9.

$$V_t = \Delta F_N + \sum_{i=1}^{N} F_i \tag{5.16}$$

$$\Delta F_N = 0.0075 N V_t \tag{5.17}$$

$$F_i = (V_t - \Delta F_N)\frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j} \tag{5.18}$$

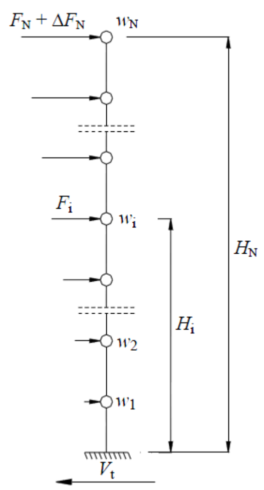


Figure 5.9. Procedure for the equivalent load method.

Distributed earthquake loads to the each floor level are presented in Table 5.10 where,

$$\Delta F_N = 0.0075(10)(2122.1) = 159.2\ kN$$

Table 5.10. Equivalent earthquake loads distributed to each floor level.

Story	w_i (kN)	H_i (m)	$w_i x H_i$	F_i (kN)
Roof	6864.3	43.13	296058.0	397.9
9	10062.7	38.93	391741.5	316.0
8	10281.1	34.73	357063.0	288.0
7	10483.4	30.53	320058.3	258.1
6	10842.4	26.33	285480.3	230.3
5	11109.8	22.13	245859.6	198.3
4	11406.6	17.93	204520.0	165.0
3	11595.2	13.73	159201.8	128.4
2	11604.4	9.53	110589.7	89.2
1	11856.6	5.33	63195.6	51.0
Σ	**106106.4**		**2433767.8**	**2122.1**

5.3.4.2. Drift and Irregularity Check. Equivalent earthquake loads are applied to the 3D model in x-direction with 5% eccentricity requirement. So, $e_x = \pm 0.05x\ 45.7 = 2.285\ m$ The TEC 07 requires that the structure obeys drift requirement defined by Equation 5.21 and calculated by Equation 5.19 and Equation 5.20.

$$\Delta_i = d_i - d_{i-1} \tag{5.19}$$

$$\delta_i = R\Delta_i \tag{5.20}$$

$$\frac{(\delta_i)_{max}}{h_i} \leq 0.02 \tag{5.21}$$

Where,

Δ_i = reduced relative floor shift

d_i = lateral displacement of i-th floor

d_i = lateral displacement of (i-1)th floor

δ_i = effective relative floor shift

From Table 5.11, it can be concluded that the building with SCBFs obeys the drift requirement.

Table 5.11. Drift calculations – SCBF system.

$(d_i)_{max}$	$(\Delta_i)_{max}$	$\delta_i=R(\Delta_i)_{max}$	drift
0.0483	0.0054	0.0270	0.0064
0.0429	0.0059	0.0295	0.0070
0.0370	0.0059	0.0295	0.0070
0.0311	0.0058	0.0290	0.0069
0.0253	0.0054	0.0270	0.0064
0.0199	0.0048	0.0240	0.0057
0.0151	0.0048	0.0240	0.0057
0.0103	0.0040	0.0200	0.0048
0.0063	0.0036	0.0180	0.0043
0.0027	0.0027	0.0135	0.0025

Torsional irregularity is defined by TEC 07 as in Equation 5.22 where the reference coefficient η_{bi} is bigger than 1.2. Soft story is defined by Equation 5.23 where rigidity irregularity coefficient η_{ki} is bigger than 2.0. Table 5.12 summarizes the necessary calculations for η_{bi} and η_{ki} . Results revealed that the building with SCBFs has no torsional and soft story irregularity considering x-direction.

$$\eta_{bi} = \frac{(\Delta_i)_{max}}{(\Delta_i)_{ave}} > 1.2 \tag{5.22}$$

$$\eta_{ki} = \left(\frac{\Delta_i}{h_i}\right)_{ave} / \left(\frac{\Delta_{i-1}}{h_{i-1}}\right)_{ave} > 2.0 \tag{5.23}$$

Where,

η_{bi} = torsional irregularity coefficient

η_{ki} = rigidity irregularity coefficient

$(\Delta_i)_{max}$ = maximum reduced relative floor shift

$(\Delta_i)_{ave}$ = average reduced relative floor shift

h_i = storey height of i-th floor

h_{i-1} = storey height of (i-1)-th floor

Table 5.12. Torsional and soft storey irregularity calculation – SCBF system.

$(d_i)_{max}$	$(d_i)_{min}$	$(\Delta_i)_{min}$	$(\Delta_i)_{max}$	$(\Delta_i)_{ave}$	$(\Delta_i/h)_{ave}$	η_{ki}	η_{bi}
0.0483	0.0436	0.0049	0.0054	0.0052	0.00123	-	1.05
0.0429	0.0387	0.0052	0.0059	0.0056	0.00132	0.93	1.06
0.0370	0.0335	0.0054	0.0059	0.0057	0.00135	0.98	1.04
0.0311	0.0281	0.0053	0.0058	0.0056	0.00132	1.02	1.05
0.0253	0.0228	0.0048	0.0054	0.0051	0.00121	1.09	1.06
0.0199	0.018	0.0044	0.0048	0.0046	0.00110	1.11	1.04
0.0151	0.0136	0.0043	0.0048	0.0046	0.00108	1.01	1.05
0.0103	0.0093	0.0037	0.0040	0.0039	0.00092	1.18	1.04
0.0063	0.0056	0.0032	0.0036	0.0034	0.00081	1.13	1.06
0.0027	0.0024	0.0024	0.0027	0.0026	0.00048	1.69	1.06

Also, TEC 07 requires that fundamental period of the structure considered cannot be bigger than the value calculated by the Equation 5.24. Necessary calculations are listed in Table 5.13. So, $T_x < T_1$.

$$T_1 = 2\pi\left(\frac{\sum_{i=1}^{N} m_i {d_{fi}}^2}{\sum_{i=1}^{N} F_{fi}\, d_{fi}}\right)^{1/2} \tag{5.24}$$

Where,

m_i = i-th storey mass

d_{fi} = fictive i-th storey displacement

F_{fi} = fictive i-th storey force

Table 5.13. Fundamental period check - SCBF system.

F_{fi} (kN)	$(d_i)_{max}$ (m)	m_i (ton)	$m_i{*}d_i^2$	$F_{fi}xd_i$
397.94	0.0483	699.73	1.6324	19.2206
315.96	0.0429	1025.76	1.8878	13.5546
287.99	0.0370	1048.02	1.4347	10.6555
258.14	0.0311	1068.64	1.0336	8.0282
230.25	0.0253	1105.24	0.7075	5.8254
198.30	0.0199	1132.50	0.4485	3.9461
164.95	0.0151	1162.75	0.2651	2.4908
128.40	0.0103	1181.98	0.1254	1.3226
89.20	0.0063	1182.91	0.0469	0.5619
50.97	0.0027	1208.62	0.0088	0.1376
		Σ	7.5908	65.7433
			T_1	**2.13**

<u>5.3.4.3. Columns Stress and Cross-sectional Checks.</u> According to TEC 07, for high ductile systems flange width/flange thickness (b/2t) and height/web thickness (h/t_w) ratios are limited with Equation 5.25 and Equation 5.26 respectively.

$$\frac{b}{2t} \leq 0.3\sqrt{E_s/\sigma_y} \tag{5.25}$$

$$\text{For } \left|N_d/\sigma_y A\right| > 0.10 \qquad \frac{h}{t_w} \leq 1.33\sqrt{E_s/\sigma_y}\left(2.1 - \left|\frac{N_d}{\sigma_y A}\right|\right) \tag{5.26}$$

Where,

b = flange width

t = flange thickness

h = height of section

t_w = web thickness

A = cross-sectional area

N_d = design axial load

E_s = modulus of elasticity (200000 Mpa)

σ_y = yield stress (345 Mpa)

As listed in Table 5.14, three columns of the SCBF do not meet the b/2t requirement. However, h/t_w requirement is attained by all the columns as listed in Table 5.15.

Table 5.14. Columns cross-sectional check - b/2t- SCBF.

Column	b(m)	t(m)	b/2t	Limit	Status
W14x43	0.2031	0.0135	7.52	7.22	fail
W14x53	0.2047	0.0168	6.09	7.22	ok
W14x74	0.2558	0.0199	6.43	7.22	ok
W14x82	0.2573	0.0217	5.93	7.22	ok
W14x99	0.3700	0.0198	9.34	7.22	fail
W14x120	0.3726	0.0239	7.79	7.22	fail
W14x132	0.3740	0.0262	7.14	7.22	ok
W14x145	0.3937	0.0277	7.11	7.22	ok
W14x176	0.3975	0.0333	5.97	7.22	ok
W14x193	0.3990	0.0366	5.45	7.22	ok
W14x233	0.4036	0.0437	4.62	7.22	ok

Table 5.15. Columns cross-sectional check – h/t_w - SCBF.

Column	h(m)	t_w(m)	A(m^2)	N_d (kN)	σ_yxA(kN)	N_d/σ_yxA	h/t_w	Limit	Status
W14x43	0.3470	0.0077	0.00813	495.2	2804.51	0.18	44.79	61.59	ok
W14x53	0.3536	0.0094	0.01010	385.3	3484.50	0.11	37.63	63.71	ok
W14x74	0.3599	0.0114	0.01410	1214.7	4864.50	0.25	31.57	59.25	ok
W14x82	0.3635	0.0130	0.01550	2480.0	5347.50	0.46	27.96	52.40	ok
W14x99	0.3597	0.0123	0.01880	1075.7	6486.00	0.17	29.24	61.94	ok
W14x120	0.3678	0.0150	0.02280	1457.6	7866.00	0.19	24.52	61.31	ok
W14x132	0.3724	0.0164	0.02500	2776.0	8625.00	0.32	22.71	56.94	ok
W14x145	0.3754	0.0173	0.02750	1652.5	9487.50	0.17	21.70	61.67	ok
W14x176	0.3866	0.0211	0.03340	4044.7	11523.00	0.35	18.32	56.01	ok
W14x193	0.3932	0.0226	0.03660	5708.4	12627.00	0.45	17.40	52.77	ok
W14x233	0.4074	0.0272	0.04420	6035.1	15249.00	0.40	14.98	54.57	ok

TEC 07 requires that columns of concentrically braced frames have necessary load capacity against increased load combinations defined by Equation 5.27 and Equation 5.28.

$$1.0G + 1.0Q + 2.0\,E \tag{5.27}$$

$$0.9G + 2.0\,E \tag{5.28}$$

For these increased loads, internal capacities of columns will be obtained by Equation 5.29, Equation 5.30 and Equation 5.31. Here, σ_{bem} is calculated by TS648 provisions for members subjected to concentric compression.

$$N_{bp} = 1.7\sigma_{bem}A \tag{5.29}$$

$$N_{çp} = \sigma_y A_n \tag{5.30}$$

$$M_p = W_p\sigma_y \tag{5.31}$$

Where,

N_{bp} = axial compression capacity

$N_{çp}$ = axial tension capacity

M_p = bending moment capacity

σ_{bem} = allowable compressive stress

σ_y = yield stress

A = cross-sectional area

A_n = net cross-sectional area

W_p = plastic section modulus

Calculation of axial compression capacities of SCBF columns and associated design loads can be found in Table 5.16. Plastic moment capacities of columns are calculated in Table 5.17 in order to check combined action capacity according to Equation 5.32 in Table 5.18. As it can be seen from Table 5.18, columns of SCBF have enough capacity. Axial tension capacity of SCBF columns are listed in Table 5.19.

$$\frac{N_d}{N_{bp}} + \frac{M_d}{M_p} \leq 1 \tag{5.32}$$

Table 5.16. Axial compression capacity - SCBF columns.

Column	Area (m^2)	r_y (m)	L_c (m)	k_y	λ_y	n	σ_{bem} (Mpa)	N_{bp} (kN)	N_d (kN)	Status
W14x43	8.13E-03	0.0481	4.2	1.2	104.8	2.49	72.16	997.2	495.2	ok
W14x53	1.01E-02	0.0488	4.2	1.2	103.3	2.48	74.32	1276.1	385.3	ok
W14x74	1.41E-02	0.0630	4.2	1.2	80.0	2.31	107.41	2574.6	1214.7	ok
W14x82	1.55E-02	0.0629	4.2	1.2	80.1	2.31	107.23	2825.5	1478.8	ok
W14x99	1.88E-02	0.0944	4.2	1.2	53.4	2.07	145.62	4654.1	1075.7	ok
W14x120	2.28E-02	0.0951	4.2	1.2	53.0	2.07	146.20	5666.7	2480.0	ok
W14x132	2.50E-02	0.0955	4.2	1.2	52.8	2.07	146.52	6227.3	2776.0	ok
W14x145	2.75E-02	0.1011	5.33	1.2	63.3	2.17	131.28	6137.5	1652.5	ok
W14x176	3.34E-02	0.1022	4.2	1.2	49.3	2.03	151.62	8609.0	4044.7	ok
W14x193	3.66E-02	0.1028	4.2	1.2	49.0	2.03	152.05	9460.3	5708.4	ok
W14x233	4.42E-02	0.1041	5.33	1.2	61.4	2.15	133.91	10062.2	6035.1	ok

Table 5.17. Moment capacity - SCBF columns.

Column	Area (m^2)	W_p (m^3)	M_p (kN.m)	M_d (kN.m)	Status
W14x43	8.13E-03	1.14E-03	393.6	38.2	ok
W14x53	1.01E-02	1.43E-03	492.3	1.0	ok
W14x74	1.41E-02	2.07E-03	712.4	77.0	ok
W14x82	1.55E-02	2.28E-03	785.9	57.0	ok
W14x99	1.88E-02	2.84E-03	978.1	1.9	ok
W14x120	2.28E-02	3.47E-03	1198.5	87.7	ok
W14x132	2.50E-02	3.84E-03	1323.1	60.7	ok
W14x145	2.75E-02	4.26E-03	1470.0	29.6	ok
W14x176	3.34E-02	5.24E-03	1809.2	107.5	ok
W14x193	3.66E-02	5.82E-03	2006.9	90.8	ok
W14x233	4.42E-02	7.15E-03	2465.0	132.2	ok

Table 5.18. Combined action check - SCBF columns.

Column	N_{bp} (kN)	N_d (kN)	M_p (kN.m)	M_d (kN.m)		Status
W14x43	997.2	495.2	393.6	38.2	0.6	ok
W14x53	1276.1	385.3	492.3	1.0	0.3	ok
W14x74	2574.6	1214.7	712.4	77.0	0.6	ok
W14x82	2825.5	1478.8	785.9	57.0	0.6	ok
W14x99	4654.1	1075.7	978.1	1.9	0.2	ok
W14x120	5666.7	2480.0	1198.5	87.7	0.5	ok
W14x132	6227.3	2776.0	1323.1	60.7	0.5	ok
W14x145	6137.5	1652.5	1470.0	29.6	0.3	ok
W14x176	8609.0	4044.7	1809.2	107.5	0.5	ok
W14x193	9460.3	5708.4	2006.9	90.8	0.6	ok
W14x233	10062.2	6035.1	2465.0	132.2	0.7	ok

Table 5.19. Axial tension capacity - SCBF columns.

Column	Area (m^2)	$N_{çp}$ (kN)	N_d (kN)	Status
W14x43	8.13E-03	2804.5	na	na
W14x53	1.01E-02	3484.5	na	na
W14x74	1.41E-02	4864.5	116.4	ok
W14x82	1.55E-02	5347.5	na	na
W14x99	1.88E-02	6486.0	na	na
W14x120	2.28E-02	7866.0	476.9	ok
W14x132	2.50E-02	8625.0	343.9	ok
W14x145	2.75E-02	9487.5	na	na
W14x176	3.34E-02	11523.0	1000.4	ok
W14x193	3.66E-02	12627.0	1686.5	ok
W14x233	4.42E-02	15249.0	1548.5	ok

5.3.4.4. Beams Stress and Cross-sectional Checks. According to TEC 07, for high ductile systems flange width/flange thickness (b/2t) and height/web thickness (h/t_w) ratios are limited with Equation 5.33 and Equation 5.34 respectively.

$$\frac{b}{2t} \leq 0.3\sqrt{E_s/\sigma_y} \tag{5.33}$$

$$\frac{h}{t_w} \leq 3.2\sqrt{E_s/\sigma_y} \tag{5.34}$$

As listed in Table 5.20 and Table 5.21, these cross-sectional requirements are attained for the beams of SCBF.

Table 5.20. Beams cross-sectional check - b/2t- SCBF.

Beam	b(m)	t(m)	b/2t	Limit	Status
W16x31	0.1403	0.0112	6.26	7.22	ok
W21x50	0.1659	0.0136	6.10	7.22	ok
W24x76	0.2283	0.0173	6.60	7.22	ok

Table 5.21. Beams cross-sectional check – h/t_w – SCBF.

Beam	h(m)	t_w(m)	h/t_w	Limit	Status
W16x31	0.4034	0.00699	57.75	77.05	ok
W21x50	0.5291	0.00965	54.82	77.05	ok
W24x76	0.6076	0.0112	54.25	77.05	ok

TEC 07 requires that beams of concentrically braced bays have the necessary internal capacity against gravity loads when braces are ignored. Lateral torsional buckling is ignored for a system where beams are laterally supported by composite floor system. Thus, using Equation 5.35 and Equation 5.36 stress calculation for load combination 1G+1Q is listed in Table 5.22 and Table 5.23.

$$\sigma = \frac{M_d}{W_x} < 1.33(0.6)\sigma_y \tag{5.35}$$

$$\tau = \frac{V_d \mathrm{x} \mathrm{S_x}}{I_x x t_w} < 1.33\tau_{all}; \;\; \tau_{all} = \frac{0.6\sigma_y}{\sqrt{3}} \tag{5.36}$$

Where,

M_d = design moment

W_x = section modulus

V_d = design shear

S_x = first moment of area about x-axis

I_x = moment of inertia about x-axis

Table 5.22.Normal stress check – SCBF beams.

Beam	W_x (m^3)	M_d (kN.m)	σ (MPa)	σ_{all}	Status
W16x31	7.74E-04	138.8	179.35	275.31	ok
W21x50	1.55E-03	139.3	89.99	275.31	ok
W24x76	2.88E-03	143.3	49.81	275.31	ok

Table 5.23.Shear stress check – SCBF beams.

Beam	t_w(m)	V_d (kN)	S_x (m^3)	I_x (m^4)	τ (MPa)	τ_{all}	Status
W16x31	0.006985	60.7	4.349E-04	1.561E-04	24.21	158.9	ok
W21x50	0.009652	61.1	8.855E-04	4.096E-04	13.68	158.9	ok
W24x76	0.011200	62.9	1.625E-03	8.741E-04	10.44	158.9	ok

According to AISC (2010), beams of braced bays need to be also designed according to post-buckling capacity of braced frame system. This condition is depicted for beam W24x76 in Figure 5.10 and for beam W21x50 in Figure 5.11.

For W24x76,

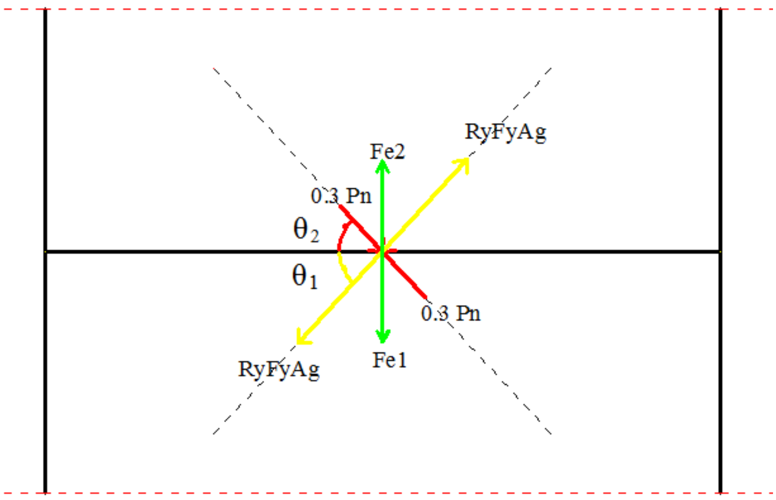


Figure 5.10. Post-buckling force diagram – W24x76.

$$F_{e1} = \left(R_y F_y A_g - 0.3P_n\right) sin\theta_1$$

$$F_{e1} = \left(1.1x317x7.48 - 0.3(1.7x317x7.48)\right) sin49.4 = 1062.2\ kN$$

$$F_{e2} = (R_y F_y A_g - 0.3P_n) sin\theta_2$$

$$F_{e2} = \left(1.1x317x7.48 - 0.3(1.7x317x7.48)\right) sin42.6 = 946.9\ kN$$

$$F_{net} = F_{e1} - F_{e2} = 1062.2 - 946.9 = 115.3\ kN$$

$$M_e = \frac{F_{net} x L}{4} = \frac{115.3x9.14}{4} = 263.5\ kN.m$$

$$M_d = 143.3\ kN.m + 263.5\ kN.m = 406.8\ kN.m$$

Possible axial load assuming braces of 2nd floor remain intact is $N_d = \left(\frac{R_y F_y A_g + 0.3P_n}{2}\right) cos\theta_2$; thus,

$$N_d = \left(\frac{1.1x317x7.48}{2} + \frac{0.3(1.7x317x7.48)}{2}\right) cos42.6 = 1405\ kN$$

$$\sigma = \frac{N_d}{A} + \frac{M_d}{W_x} = \frac{1405x10^{-3}}{0.0145} + \frac{406.8}{2.88} = 238.1 < 1.33(0.6)345 = 273.3\ Mpa\ (ok)$$

$$V_{net} = 115.3 + 62.9 = 178.2\ kN$$

$$\tau = \frac{178.2x1.625x10^{-6}}{8.741x11.2x10^{-7}} = 29.6\ < 159\ MPa\ (ok)$$

For post-buckling case Equation 5.27 is valid in order to check h/t_w ratio; therefore,

For $|N_d/\sigma_y A| = \frac{1405}{345x14.5} = 0.28$ $\quad \frac{h}{t_w} \leq 1.33\sqrt{\frac{200000}{345}}(2.1 - 0.28) = 58.3$

$$\frac{h}{t_w} = 54.3 < 58.3\ (ok)$$

For W21x50,

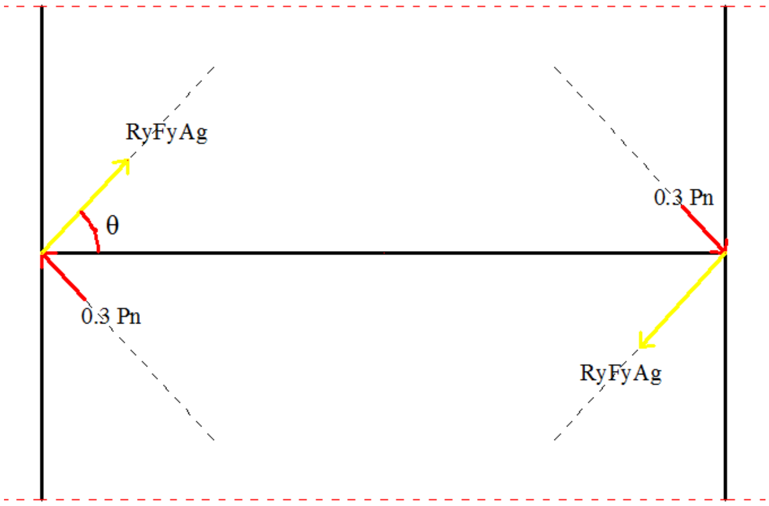


Figure 5.11. Post-buckling force diagram – W21x50.

Possible axial load assuming braces of 3rd floor remain intact is $N_d = \left(\frac{R_y F_y A_g + 0.3P_n}{2}\right) cos\theta$; thus,

$$N_d = \left(\frac{1.1x317x6.28}{2} + \frac{0.3(1.7x317x6.28)}{2}\right) cos42.6 = 1179.6kN$$

$$M_d = 139.3\ kN.m$$

$$\sigma = \frac{N_d}{A} + \frac{M_d}{W_x} = \frac{1179.6x10^{-3}}{0.009484} + \frac{139.3}{1.548} = 214.4 < 1.33(0.6)345 = 273.3\ Mpa\ (ok)$$

$$\tau = \frac{61.1x8.855}{4.096x9.652} = 13.7\ < 159\ MPa\ (ok)$$

For post-buckling case Equation 5.27 is valid in order to check h/t_w ratio; therefore,

For $|N_d/\sigma_y A| = \frac{1179.6}{345x9.484} = 0.36$ $\quad \frac{h}{t_w} \le 1.33\sqrt{\frac{200000}{345}}(2.1 - 0.36) = 55.7$

$$\frac{h}{t_w} = 54.8 < 55.7\ (ok)$$

<u>5.3.4.5. Braces Stress and Cross-sectional Checks.</u> According to TEC 07, for high ductile systems height/web thickness (h/t_w) ratio of square sections is limited with Equation 5.37. As listed in Table 5.24, cross-sectional requirement is attained for braces of SCBF.

$$\frac{h}{t_w} \leq 0.7\sqrt{E_s/\sigma_y} \tag{5.37}$$

Table 5.24. Braces cross-sectional check – h/t_w – SCBF.

Brace	h(m)	t_w(m)	h/t_w	Limit	Status
HSS 7x7x1/2	0.1778	0.01180	15.07	17.58	ok
HSS 6x6x1/2	0.1524	0.01180	12.92	17.58	ok
HSS 5-1/2x5-1/2x3/8	0.1397	0.00887	15.76	17.58	ok
HSS 4-1/2x4-1/2x3/8	0.1143	0.00887	12.89	17.58	ok

TEC 07 requires that slenderness ratio of braces are limited with the Equation 5.38; however, this ratio is specified by AISC as in Equation 5.39 for braces of special concentrically braced frames.

$$\lambda_{TEC} = \frac{kL}{r} \leq 4.0\sqrt{E_s/\sigma_y} \tag{5.38}$$

$$\lambda_{AISC} = \frac{kL}{r} \leq 200 \tag{5.39}$$

Where,

λ = slenderness ratio

k = effective length factor

L = brace length

r = radius of gyration

E_s = modulus of elasticity (200000 MPa)

σ_y = yield stress (317 MPa)

Braces of SCBF obeys slenderness requirement of Seismic Provisions for Structural Steel Buildings (AISC, 2010) according to which they designed. However, TEC 07 requirement is not met as listed in Table 5.25.

Table 5.25. Slenderness ratio – SCBF braces.

Brace	k	l(cm)	r(cm)	λ	λ_{TEC}	λ_{AISC}	Status
HSS 7x7x1/2-First Story	1.0	702.1	6.69	104.9	100.5	200.0	ok
HSS 7x7x1/2	1.0	620.7	6.69	92.8	100.5	200.0	ok
HSS 6x6x1/2	1.0	620.7	5.66	109.7	100.5	200.0	ok
HSS 5-1/2x5-1/2x3/8	1.0	620.7	5.28	117.6	100.5	200.0	ok
HSS 4-1/2x4-1/2x3/8	1.0	620.7	4.24	146.4	100.5	200.0	ok

According to provisions of TS648, except first two story braces of SCBF, braces do not pass stress check as listed in Table 5.26. However, they all have the necessary capacity considering Section E3 of Specifications for Structural Steel Buildings (AISC, 2010) and Section F2 of Seismic Provisions for Structural Steel Buildings (AISC, 2010) as listed in Table 5.27. In order to be consisted with the previous studies used the same models; section of braces has remained unchanged.

Table 5.26. Stress check according to TS648 – SCBF braces.

Brace	Area (m^2)	λ	n	σ_{bem} (Mpa)	N_d (kN)	$\sigma_{eb}=N_d/A$	σ_{eb}/σ_{bem}	Limit	Status
HSS 7x7x1/2-First Story	0.007484	104.9	2.46	71.82	671.9	89.78	1.25	1.33	ok
HSS 7x7x1/2	0.007484	92.8	2.38	87.06	603.7	80.67	0.93	1.33	ok
HSS 6x6x1/2	0.006284	109.7	2.49	65.85	586.7	93.36	1.42	1.33	fail
HSS 5-1/2x5-1/2x3/8	0.004439	117.6	2.50	57.13	373.7	84.19	1.47	1.33	fail
HSS 4-1/2x4-1/2x3/8	0.003535	146.4	2.50	36.84	214.5	60.68	1.65	1.33	fail

Table 5.27. Stress check according to AISC – SCBF braces.

Brace	Area (m^2)	λ	F_e (Mpa)	F_{cre} (Mpa)	N_{all} (kN)	N_d (kN)	N_d/N_{all}	Limit	Status
HSS 7x7x1/2-First Story	0.007484	104.9	179.22	157.18	803.0	671.9	0.84	1.33	ok
HSS 7x7x1/2	0.007484	92.8	229.31	197.42	1008.6	603.7	0.60	1.33	ok
HSS 6x6x1/2	0.006284	109.7	164.13	143.95	617.5	586.7	0.95	1.33	ok
HSS 5-1/2x5-1/2x3/8	0.004439	117.6	142.83	125.27	379.6	373.7	0.98	1.33	ok
HSS 4-1/2x4-1/2x3/8	0.003535	146.4	92.11	80.78	194.9	214.5	1.10	1.33	ok

5.3.5. The Building with EBFs

The reference paper (Khandelwal *et al.*, 2009) has no information about the eccentricity (e) of the braces. Therefore, with a trial error procedure, the eccentricity has been selected as e =120cm, for which the building with EBFs meet all the seismic design requirements of TEC 07 as will be explained in the following sections.

<u>5.3.5.1. Equivalent Earthquake Loads.</u> From ASCE 7-05, for the building with EBFs, which is designed to be located in Seattle, Washington: S_S = 1.551 and S_1 = 0.534. Assuming site class C, from table 11.4-1 and 11.4-2 of ASCE 7-05, F_a = 1.0 and F_v = 1.3, parameters of design spectrum of the building with SCBFs is listed in Table 5.28 .Thus, $0.50 \leq S_{DS}$, which means that the structure is assigned to Seismic Design Category D as stated before.

Table 5.28. Design spectrum parameters – EBF system.

$\mathbf{S_{MS}}$	1.5510 g
$\mathbf{S_{DS}}$	1.0340 g
$\mathbf{S_{M1}}$	0.6940 g
$\mathbf{S_{D1}}$	0.4628 g
$\mathbf{T_0}$	0.0895 s
$\mathbf{T_S}$	0.4480 s

From modal analysis, the period of the building with EBFs in x-direction is T_x = 2.05 s; thus, spectral response acceleration S_a = 0.226 g .Calculation of the total weight of the structure with EBFs is shown in Table 5.29. Total base shear with the importance factor I = 1 and R = 7 is (Assume A_0 =0.3)

$$V_t = \frac{105909.55x0.226}{7} = 3419.4\,kN \leq 0.1x0.3x1x105909.55 = 3177.3\,kN$$

$$V_t = 3419.4\,kN$$

Total base shear is distributed to the each floor level according to Equation 5.16, Equation 5.17 and Equation 5.18 as defined in TEC 07. The procedure is described in Figure 5.9. Distributed earthquake loads to the each floor level are presented in Table 5.30 where,

$$\Delta F_N = 0.0075(10)(3419.4) = 256.5\,kN$$

Table 5.29. Total weight – EBF system.

SOURCE	CALCULATION	(kN)
FRAME WEIGHT	SAP2000	6497.00
DEAD LOAD-9 STORY	3.638x45.7x45.7x9	68381.34
DEAD LOAD-ROOF	2.681x45.7x45.7	5599.24
FLOOR BEAM-10 FLOOR	50x10x9.14x0.4513	2062.44
FLOOR LIVE LOAD	45.7x45.7x4.788x0.3x7.59	22769.29
ROOF LIVE LOAD	45.7x45.7x0.3x0.958	600.23
		Σ **105909.55**

Table 5.30. Equivalent earthquake loads distributed to each floor level.

Story	**w_i (kN)**	**H_i (m)**	**w_ixH_i**	**F_i (kN)**
Roof	6914.9	43.13	298240.4	643.7
9	10113.2	38.93	393707.5	511.3
8	10288.6	34.73	357323.4	464.0
7	10517.1	30.53	321087.2	417.0
6	10862.1	26.33	285999.0	371.4
5	11106.3	22.13	245782.2	319.2
4	11316.4	17.93	202902.7	263.5
3	11516.8	13.73	158125.4	205.3
2	11542.9	9.53	110003.6	142.8
1	11731.3	5.33	62527.7	81.2
Σ	**105909.5**		**2435699.1**	**3419.4**

5.3.5.2. Drift and Irregularity Check. Equivalent earthquake loads are applied to the 3D model in x-direction with 5% eccentricity requirement. So, $e_x = \pm 0.05x\ 45.7 = 2.285\ m$ The TEC 07 requires that the structure obeys drift requirement defined by Equation 5.22 and calculated by Equation 5.20 and Equation 5.21. From Table 5.31, it can be concluded that the building with EBFs obeys the drift requirement.

Torsional irregularity is defined by TEC 07 as in Equation 5.22 where the reference coefficient η_{bi} is bigger than 1.2. Soft story is defined by Equation 5.23 where rigidity irregularity coefficient η_{ki} is bigger than 2.0. Table 5.32 summarizes the necessary calculations for $\eta_{bi}$and η_{ki}. Results revealed that the building with EBFs has no torsional and soft story irregularity considering x-direction.

Table 5.31. Drift calculations – EBF system.

$(d_i)_{max}$	$(\Delta_i)_{max}$	$\delta_i=R(\Delta_i)_{max}$	drift
0.0653	0.0038	0.0266	0.0063
0.0615	0.0052	0.0364	0.0087
0.0563	0.0065	0.0455	0.0108
0.0498	0.0074	0.0518	0.0123
0.0424	0.0066	0.0462	0.0110
0.0358	0.0069	0.0483	0.0115
0.0289	0.0071	0.0497	0.0118
0.0218	0.0073	0.0511	0.0122
0.0145	0.0074	0.0518	0.0123
0.0071	0.0071	0.0497	0.0093

Table 5.32. Torsional and soft storey irregularity calculation – EBF system.

$(d_i)_{max}$	$(d_i)_{min}$	$(\Delta_i)_{min}$	$(\Delta_i)_{max}$	$(\Delta_i)_{ave}$	$(\Delta_i/h)_{ave}$	η_{ki}	η_{bi}
0.0653	0.0581	0.0033	0.0038	0.0036	0.00085	-	1.07
0.0615	0.0548	0.0046	0.0052	0.0049	0.00117	0.72	1.06
0.0563	0.0502	0.0058	0.0065	0.0062	0.00146	0.80	1.06
0.0498	0.0444	0.0065	0.0074	0.0070	0.00165	0.88	1.06
0.0424	0.0379	0.0059	0.0066	0.0063	0.00149	1.11	1.06
0.0358	0.032	0.0061	0.0069	0.0065	0.00155	0.96	1.06
0.0289	0.0259	0.0064	0.0071	0.0068	0.00161	0.96	1.05
0.0218	0.0195	0.0066	0.0073	0.0070	0.00165	0.97	1.05
0.0145	0.0129	0.0066	0.0074	0.0070	0.00167	0.99	1.06
0.0071	0.0063	0.0063	0.0071	0.0067	0.00126	1.33	1.06

Also, TEC 07 requires that fundamental period of the structure considered cannot be bigger than the value calculated by the Equation 5.24. Necessary calculations are listed in Table 5.33. So, $T_x < T_1$.

Table 5.33. Fundamental period check - EBF system.

F_{fi} (kN)	$(d_i)_{max}$ (m)	m_i (ton)	$m_i*d_i^2$	$F_{fi}xd_i$
643.7	0.0653	704.88	3.0057	42.0360
511.3	0.0615	1030.91	3.8992	31.4422
464.0	0.0563	1048.79	3.3243	26.1236
417.0	0.0498	1072.08	2.6588	20.7642
371.4	0.0424	1107.25	1.9906	15.7469
319.2	0.0358	1132.14	1.4510	11.4261
263.5	0.0289	1153.56	0.9635	7.6146
205.3	0.0218	1173.98	0.5579	4.4763
142.8	0.0145	1176.64	0.2474	2.0713
81.2	0.0071	1195.85	0.0603	0.5765
		Σ	18.1586	162.2777
			T_1	**2.10**

5.3.5.3. Columns Stress and Cross-sectional Checks. According to TEC 07, for high ductile systems flange width/flange thickness (b/2t) and height/web thickness (h/t_w) ratios are limited with Equation 5.25 and Equation 5.26 respectively. As listed in Table 5.34, four columns of the EBF do not meet the b/2t requirement. However, h/t_w requirement is attained by all the columns as listed in Table 5.15.

Table 5.34. Columns cross-sectional check - b/2t- EBF.

Column	b(m)	t(m)	b/2t	Limit	Status
W14x48	0.2040	0.0151	6.75	7.22	ok
W14x53	0.2047	0.0168	6.09	7.22	ok
W14x61	0.2539	0.0164	7.74	7.22	fail
W14x82	0.2573	0.0217	5.93	7.22	ok
W14x99	0.3700	0.0198	9.34	7.22	fail
W14x109	0.3710	0.0218	8.51	7.22	fail
W14x120	0.3726	0.0239	7.79	7.22	fail
W14x132	0.3740	0.0262	7.14	7.22	ok
W14x145	0.3937	0.0277	7.11	7.22	ok
W14x176	0.3975	0.0333	5.97	7.22	ok

Table 5.35. Columns cross-sectional check – h/t_w – EBF.

Column	h(m)	t_w(m)	A(m^2)	N_d (kN)	σ_yxA(kN)	N_d/σ_yxA	h/t_w	Limit	Status
W14x48	0.3503	0.0086	0.00910	492.0	3138.47	0.16	40.56	62.23	ok
W14x53	0.3536	0.0094	0.01010	424.7	3484.50	0.12	37.63	63.34	ok
W14x61	0.3528	0.0095	0.01150	985.4	3967.50	0.25	37.04	59.29	ok
W14x82	0.3635	0.0130	0.01550	1595.9	5347.50	0.30	27.96	57.69	ok
W14x99	0.3597	0.0123	0.01880	1319.7	6486.00	0.20	29.24	60.73	ok
W14x109	0.3637	0.0133	0.02060	3968.0	7107.00	0.56	27.35	49.37	ok
W14x120	0.3678	0.0150	0.02280	1563.2	7866.00	0.20	24.52	60.88	ok
W14x132	0.3724	0.0164	0.02500	4915.9	8625.00	0.57	22.71	49.00	ok
W14x145	0.3754	0.0173	0.02750	5910.9	9487.50	0.62	21.70	47.30	ok
W14x176	0.3866	0.0211	0.03340	6976.8	11523.00	0.61	18.32	47.86	ok

TEC 07 requires that columns of EBF have necessary load capacity against increased load combinations defined by Equation 5.40 and Equation 5.41.

$$1.0G + 1.0Q + 2.5\,E \tag{5.40}$$

$$0.9G + 2.5\,E \tag{5.41}$$

For these increased loads, internal capacities of columns will be obtained by Equation 5.29, Equation 5.30 and Equation 5.31. Here, σ_{bem} is calculated by TS648 provisions for members subjected to concentric compression. Calculation of axial compression capacities of EBF columns and associated design loads can be found in Table 5.36. Plastic moment capacities of columns are calculated in Table 5.37 in order to check combined action capacity according to Equation 5.32 in Table 5.38. As it can be seen from Table 5.38, columns of EBF have enough capacity except column W14x176. However, this column posses necessary capacity when effective length is changed to k=1 from conservative value of k=1.2. Therefore, in order to be consistent with the original design, first story column is remained unchanged. Axial tension capacity of EBF columns are listed in Table 5.39.

Table 5.36. Axial compression capacity - EBF columns.

Column	Area (m^2)	r_y (m)	L_c (m)	k_y	λ_y	n	σ_{bem} (Mpa)	N_{bp} (kN)	N_d (kN)	Status
W14x48	0.00910	0.0485	4.2	1.2	103.9	2.48	73.40	1135.1	492.0	ok
W14x53	0.01010	0.0488	4.2	1.2	103.3	2.48	74.32	1276.1	424.7	ok
W14x61	0.01150	0.0621	4.2	1.2	81.2	2.32	105.77	2067.7	985.4	ok
W14x82	0.01550	0.0629	4.2	1.2	80.1	2.31	107.23	2825.5	1595.9	ok
W14x99	0.01880	0.0944	4.2	1.2	53.4	2.07	145.62	4654.1	1319.7	ok
W14x109	0.02060	0.0949	4.2	1.2	53.1	2.07	146.04	5114.2	3968.0	ok
W14x120	0.02280	0.0951	4.2	1.2	53.0	2.07	146.20	5666.7	1563.2	ok
W14x132	0.02500	0.0955	4.2	1.2	52.8	2.07	146.52	6227.3	4915.9	ok
W14x145	0.02750	0.1011	4.2	1.2	49.9	2.04	150.83	7051.2	5910.9	ok
W14x176	0.03340	0.1022	5.33	1.2	62.6	2.16	132.27	7510.0	6976.8	ok

Table 5.37. Moment capacity - EBF columns.

Column	Area (m^2)	W_p (m^3)	M_p (kN.m)	M_d (kN.m)	Status
W14x48	9.097E-03	1.285E-03	443.3	72.0	ok
W14x53	1.010E-02	1.427E-03	492.3	7.9	ok
W14x61	1.150E-02	1.671E-03	576.5	92.0	ok
W14x82	1.550E-02	2.278E-03	785.9	84.7	ok
W14x99	1.880E-02	2.835E-03	978.1	10.0	ok
W14x109	2.060E-02	3.146E-03	1085.4	122.4	ok
W14x120	2.280E-02	3.474E-03	1198.5	13.1	ok
W14x132	2.500E-02	3.835E-03	1323.1	141.4	ok
W14x145	2.750E-02	4.261E-03	1470.0	92.8	ok
W14x176	3.340E-02	5.244E-03	1809.2	311.0	ok

Table 5.38. Combined action check - EBF columns.

Column	N_{bp} (kN)	N_d (kN)	M_p (kN.m)	M_d (kN.m)		Status
W14x48	1135.1	492.0	443.3	72.0	0.60	ok
W14x53	1276.1	424.7	492.3	7.9	0.35	ok
W14x61	2067.7	985.4	576.5	92.0	0.64	ok
W14x82	2825.5	1595.9	785.9	84.7	0.67	ok
W14x99	4654.1	1319.7	978.1	10.0	0.29	ok
W14x109	5114.2	3968.0	1085.4	122.4	0.89	ok
W14x120	5666.7	1563.2	1198.5	13.1	0.29	ok
W14x132	6227.3	4915.9	1323.1	141.4	0.90	ok
W14x145	7051.2	5910.9	1470.0	92.8	0.90	ok
W14x176	7510.0	6976.8	1809.2	311.0	1.10	fail
W14x176	(k=1)8371.6	6976.8	1809.2	311.0	1.0	ok

Table 5.39. Axial tension capacity - EBF columns.

Column	Area (m^2)	$N_{çp}$ (kN)	N_d (kN)	Status
W14x48	9.097E-03	3138.5	na	na
W14x53	1.010E-02	3484.5	na	na
W14x61	1.150E-02	3967.5	na	na
W14x82	1.550E-02	5347.5	14.3	ok
W14x99	1.880E-02	6486.0	na	na
W14x109	2.060E-02	7107.0	872.4	ok
W14x120	2.280E-02	7866.0	na	na
W14x132	2.500E-02	8625.0	1284.8	ok
W14x145	2.750E-02	9487.5	1741.5	ok
W14x176	3.340E-02	11523.0	2269.1	ok

5.3.5.4. Shear Links Stress and Cross-sectional Checks. According to TEC 07, for high ductile systems flange width/flange thickness (b/2t) and height/web thickness (h/t_w) ratios are limited with Equation 5.33 and Equation 5.34 respectively. As listed in Table 5.40 and Table 5.41, only W10x39 fails slightly the first requirement but the other cross-sectional requirements are attained for the links of EBF.

Table 5.40. Shear links cross-sectional check - b/2t- EBF.

Beam	b(m)	t(m)	b/2t	Limit	Status
W10x39	0.2028	0.0135	7.51	7.22	fail
W12x45	0.2043	0.0146	7.00	7.22	ok
W14x48	0.2040	0.0151	6.75	7.22	ok

Table 5.41 Shear links cross-sectional check – h/t_w - EBF.

Beam	h(m)	t_w(m)	h/t_w	Limit	Status
W10x39	0.2520	0.0080	31.50	77.05	ok
W12x45	0.3063	0.0085	36.00	77.05	ok
W14x48	0.3503	0.0086	40.56	77.05	ok

TEC 07 requires that length of the shear links or i.e. eccentricity is limited with the Equation 5.42 where M_p is calculated by Equation 5.31 and V_p is calculated by Equation 5.43. As calculated in Table 5.42, e=120 cm is within the limit required.

$$1.0\frac{M_p}{V_p} \leq e \leq 5.0\frac{M_p}{V_p} \tag{5.42}$$

$$V_p = 0.60\sigma_y A_k \tag{5.43}$$

Where,

M_p = plastic moment capacity

V_p = plastic shear load capacity

e = link length

σ_y = yield stress (345 MPa)

A_k = shearing area

Table 5.42. Shear link length check.

		A_k =(h-2xt)xt_w								
Beam	W_p (m^3)	h(m)	t(m)	t_w(m)	A_k (m^2)	M_p (kN.m)	V_p (kN)	M_p/V_p	e (m)	$5(M_p/V_p)$
W10x39	7.669E-04	0.2520	0.0135	0.008001	0.001800	264.6	372.6	0.71	1.20	3.55
W12x45	1.060E-03	0.3063	0.0146	0.008509	0.002358	365.7	488.1	0.75	1.20	3.75
W14x48	1.285E-03	0.3503	0.0151	0.008636	0.002764	443.3	572.2	0.77	1.20	3.87

TEC 07 requires that shear links have the necessary internal capacity against the load combination 1G+1Q+1E. Lateral torsional buckling is ignored for a system where beams are laterally supported by composite floor system. Thus, using Equation 5.44 and Equation 5.36 stress calculations are done. According to TS648, if the average shear stress in the web exceeds the half value of the allowable shear stress, Equation 5.45 need also be satisfied. For the Equation 5.45, average shear stress defined by Equation 5.46 is also allowed by TS648 provisions.

$$\sigma = \frac{M_d}{W_x} + \frac{\mathrm{N_d}}{\mathrm{A}} < 1.33(0.6)\sigma_y \tag{5.44}$$

$$\sigma_v = \sqrt{\sigma^2 + 3\tau^2} < 0.8\sigma_y \tag{5.45}$$

$$\tau_{ave} = \frac{V_d}{hxt_w} \tag{5.46}$$

Where,

M_d = design moment

W_x = section modulus

N_d = design axial load

A = cross-sectional area

σ_y = yield stress (345 MPa)

σ_y = reference stress considering principle stresses

τ_{ave} = average shear stress

The internal loads imposed to shear links are obtained from 2D model of EBF in N-S elevation, which is shown in Figure 12. In this model gravity beams are omitted and half of the total seismic loads presented in Table 5.30 is applied in order to building to carry lateral loads only by bracing system. As listed in Table 5.43 and 5.44, shear links are able carry axial and shearing stresses imposed on them.

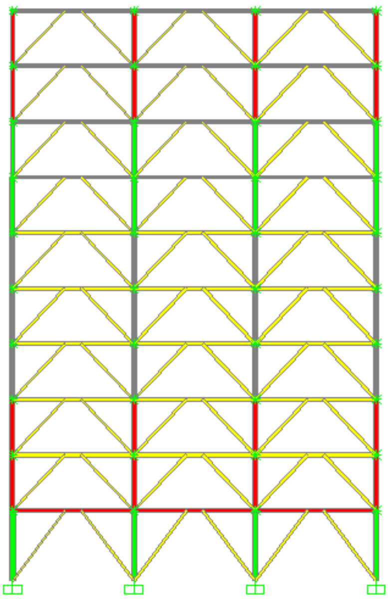


Figure 5.12. 2D Model EBF - N-S Elevation.

Table 5.43. Normal stress check - EBF shear links.

Beam	W_x (m^3)	A (m^2)	M_d (kN.m)	N_d (kN)	σ (MPa)	σ_{all}	Status
W10x39	6.905E-04	7.419E-03	117.8	30.0	174.6	275.3	ok
W12x45	9.512E-04	8.516E-03	183.7	33.9	197.1	275.3	ok
W14x48	1.153E-03	9.097E-03	217.3	32.1	192.0	275.3	ok

Table 5.44. Shear stress check - EBF shear links.

Beam	t_w(m)	I_x (m^4)	V_d (kN)	S_x (m^3)	τ (MPa)	τ_{ave} (MPa)	τ_{all}	Status
W10x39	8.001E-03	8.699E-05	176.8	3.771E-04	95.8	87.7	158.9	ok
W12x45	8.509E-03	1.457E-04	274.2	5.167E-04	114.3	105.2	158.9	ok
W14x48	8.636E-03	2.019E-04	308.0	6.269E-04	110.7	101.8	158.9	ok

The reference stress defined by Equation 5.45 is also below the capacities of shear links as listed in Table 5.45.

Table 5.45. Reference stress check - EBF shear links.

Beam	σ (MPa)	τ_{ave} (MPa)	σ_v (MPa)	σ_{all} (MPa)	Status
W10x39	174.6	87.7	231.4	276.0	ok
W12x45	197.1	105.2	268.4	276.0	ok
W14x48	192.0	101.8	260.7	276.0	ok

TEC07 requires that design shear load, V_d ensures the conditions defined by Equation 5.47 and Equation 5.48 for the limited axial load: $\left|N_d/\sigma_y A\right| < 0.15$. As listed in Table 5.46 and Table 5.47 shear links of EBF obey this requirement.

$$V_d \leq V_p \tag{5.47}$$

$$V_d \leq \frac{2M_p}{e} \tag{5.48}$$

Where,

V_d = design shear load

V_p = plastic shear load capacity

M_p = plastic moment capacity

e = link length

TEC 07 defines the rotation angle of the link elements of EBF as in Equation 5.49 and in Figure 5.13. The rotation angle need to be limited as defined in Equation 5.50 and Equation 5.51. Linear interpolation is allowed in between these equations. As listed in Table 5.48, shear links of EBF obeys rotation limitations of TEC 07.

Table 5.46. Design shear load check 1 - EBF shear links.

Beam	N_d (kN)	A (m^2)	$N_d/\sigma_y xA$	V_d (kN)	V_p (kN)	Status
W10x39	30.0	7.419E-03	0.012	176.8	372.6	ok
W12x45	33.9	8.516E-03	0.012	274.2	488.1	ok
W14x48	32.1	9.097E-03	0.010	308.0	572.2	ok

Table 5.47. Design shear load check 2 - EBF shear links.

Beam	e (m)	M_p (kN.m)	V_d (kN)	$2M_p/e$	Status
W10x39	1.20	264.6	176.8	441.0	ok
W12x45	1.20	365.7	274.2	609.5	ok
W14x48	1.20	443.3	308.0	738.9	ok

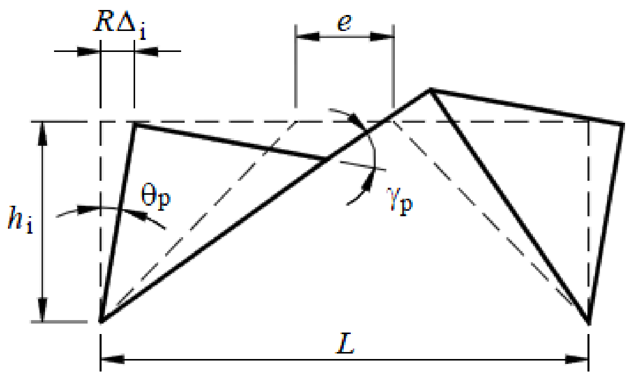


Figure 5.13. Shear link rotation angle definition (TEC 07).

$$\gamma_p = \frac{L}{e}\theta_p \quad where\ \theta_p = R\frac{\Delta_i}{h_i} \tag{5.49}$$

$$\gamma_p \leq 0.10 \quad for\ e \leq 1.6M_p/V_p \tag{5.50}$$

$$\gamma_p \leq 0.03 \quad for\ e \geq 2.6M_p/V_p \tag{5.51}$$

Where,

γ_p = link element rotation angle

L = bay length

e = link length

θ_p = floor drift defined by Equation 5.21

Table 5.48. Rotation angle check - EBF shear links.

Beam	L (m)	$1.6(M_p/V_p)$	e (m)	$2.6(M_p/V_p)$	θ_p	γ_p	Limit	Status
W10x39	9.14	1.14	1.20	1.85	0.0123	0.0937	0.0937	ok
W12x45	9.14	1.20	1.20	1.95	0.0123	0.0937	0.1000	ok
W14x48	9.14	1.24	1.20	2.01	0.0093	0.0708	0.1000	ok

According to TEC 07, beams outside of the link elements should be sized according the loads which cause link elements to reach their plastic capacities. These loads are obtained by multiplying internal design loads of link elements by the minimum of M_p/M_d or V_p/V_d. The internal load capacity of beams outside of the link element will then also be increased again by $1.1D_a$. Thus, beam outside of the link element should be designed according to Equation 5.52.

$$M_{p,b} = 1.1xD_a xIFxM_d \leq \; M_p \;\; where\; IF = min\begin{cases} \mathrm{M_p/M_d} \\ \mathrm{V_p/V_d} \end{cases} \tag{5.52}$$

Where,

D_a = yield stress increase factor (1.1 for ASTM A992, σ_y =345 MPa)

IF = load increase factor

$M_{p,b}$= increased moment demand for beam outside of the link element

As listed in Table 5.49, beams outside of the link element have slightly less capacity than the required. To be consisted with the original design, it is assumed that the headings of beams are supported by rigidity plates as TEC 07 requires.

Table 5.49. Capacity check for beams outside of shear links- EBF.

Beam	M_p/M_d	V_p/V_d	IF	M_d (kN.m)	D_a	$M_{p,b}$	M_p (kN.m)
W10x39	2.25	2.11	2.11	117.8	1.1	300.4	264.6
W12x45	1.99	1.78	1.78	183.7	1.1	395.7	365.7
W14x48	2.04	1.86	1.86	217.3	1.1	488.5	443.3

<u>5.3.5.5. Braces Stress and Cross-sectional Checks.</u> According to TEC 07, for high ductile systems height/web thickness (h/t_w) ratio of square sections is limited with Equation 5.37. As listed in Table 5.50, cross-sectional requirement is attained for braces of EBF.

Table 5.50. Braces cross-sectional check – h/t_w – EBF.

Brace	h(m)	t_w(m)	h/t_w	Limit	Status
HSS 8x8x1/2	0.2032	0.01180	17.22	17.58	ok
HSS 7x7x1/2	0.1778	0.01180	15.07	17.58	ok

TEC 07 requires that slenderness ratio of braces are limited with the Equation 5.38. Braces of EBF obeys slenderness requirement as listed in Table 5.51.

Table 5.51. Slenderness ratio – EBF braces.

Brace	k	l(cm)	r(cm)	λ	λ_{TEC}	Status
HSS 8x8x1/2-First Story	1.0	664.6	7.73	86.0	100.5	ok
HSS 8x8x1/2	1.0	577.9	7.73	74.8	100.5	ok
HSS 7x7x1/2	1.0	577.9	6.69	86.4	100.5	ok

As listed in Table 5.52, braces of EBF have necessary internal capacity against the loads obtained from 1G+1Q+1E combination. The loads are obtained again from 2D model as in Figure 5.12. TEC 07 also requires that braces to be sized according the loads which cause link elements to reach their plastic capacities. These loads is obtained by multiplying internal design loads of link elements by the minimum of M_p/M_d or V_p/V_d. The internal load capacity of beams outside of the link element will then also be increased again by $1.25D_a$. Thus, beam outside of the link element should be designed according to Equation 5.53 and Equation 5.54. As listed in Table 5.53, braces have necessary capacity.

$$\frac{\sigma_{eb}}{1.7x\sigma_{bem}} = \frac{N_{p,b}}{1.7xAx\sigma_{bem}} < 1.0 \tag{5.53}$$

$$N_{p,b} = 1.1xD_a xIFxN_d \;\; where\; IF = min\begin{cases} \mathrm{M_p/M_d} \\ \mathrm{V_p/V_d} \end{cases} \tag{5.54}$$

Where,

D_a = yield stress increase factor (1.1 for ASTM A992, σ_y =345 MPa)

IF = load increase factor

$N_{p,b}$ = increased axial load demand

σ_{eb} = design stress obtained from $N_{p,b}$

σ_{bem} = allowable compressive stress

Table 5.52. Stress check according to TS648 – EBF braces.

Brace	Area (m^2)	λ	n	σ_{bem} (MPa)	N_d (kN)	$\sigma_{eb}=N_d/A$	σ_{eb}/σ_{bem}	Limit	Status
HSS 8x8x1/2-First Story	0.008710	86.0	2.33	95.55	508.1	58.34	0.61	1.33	ok
HSS 8x8x1/2	0.008710	74.8	2.24	109.58	498.0	57.18	0.52	1.33	ok
HSS 7x7x1/2	0.007484	86.4	2.34	95.04	331.5	44.29	0.47	1.33	ok

Table 5.53. Stress check according to increased loads – EBF braces.

Brace	Area (m^2)	IF	D_a	N_d (kN)	$N_{p,b}$ (kN)	σ_{bem} (MPa)		Limit	Status
HSS 8x8x1/2-First Story	0.008710	1.86	1.1	508.1	1299.47	95.55	0.92	1.00	ok
HSS 8x8x1/2	0.008710	1.78	1.1	498.0	1218.86	109.58	0.75	1.00	ok
HSS 7x7x1/2	0.007484	2.11	1.1	331.5	961.76	95.04	0.80	1.00	ok

5.4. Progressive Collapse Analysis

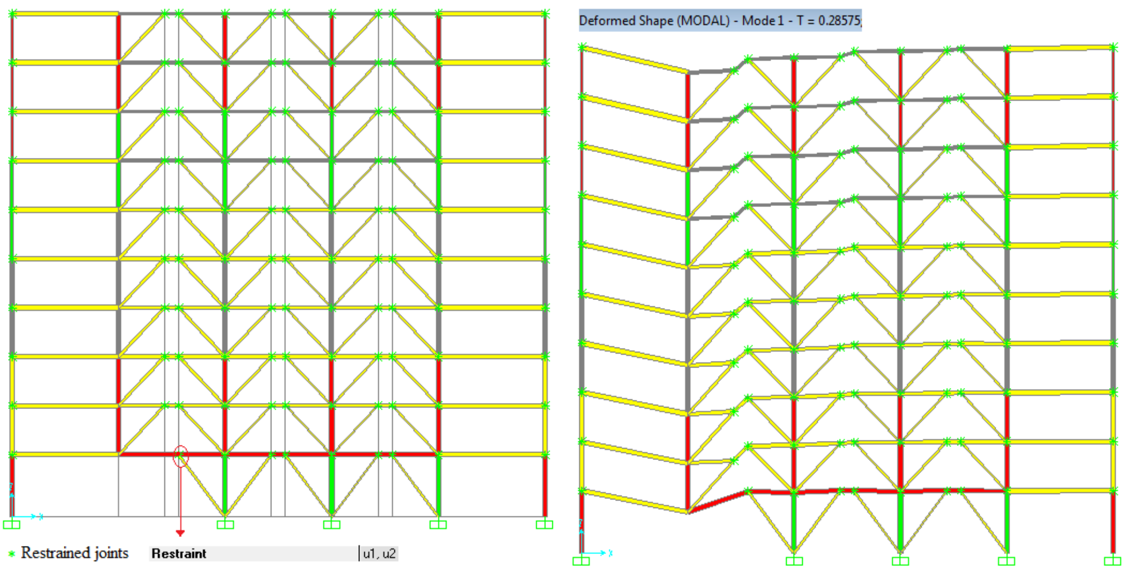
In the previous sections, SCBF and EBF systems have been verified with respect to their compliance with seismic design requirements of TEC 07. This section will investigate further the progressive collapse potential of N-S elevation of two seismically designed steel braced frames using the methodology explained below.

5.4.1. Methodology

Progressive collapse potential of SCBF and EBF systems has been investigated using the APM, which is the prevalent direct design method in the literature for designing structures against progressive collapse. As stated in previous sections, UFC 4-023-03 (DoD, 2009) is the most current progressive collapse guideline explaining this method in detail with all necessary analysis and acceptance criterion. Therefore, APM was applied in accordance with Section 3.2 of this UFC.

From the three different analysis procedures allowed by this UFC, namely; LSP, NSP and NDP, the last one; Nonlinear Dynamic Procedure, have been chosen as the structural analysis method. The reason behind this is the fact that NDP is the method which best reflects the dynamic nature of structural response to a column removal scenario. Also, static analysis methods have been accepted by many researchers as conservative, which is a fact addressed in Chapter 4.

In this method, first 2D linear elastic model of the considered frame was developed as in Figure 5.3 or in Figure 5.4. Then, this model was loaded with the load combination introduced in Equation 3.24. Elastic static analysis was carried out in order to record internal forces of structural elements, which were considered to be removed by APM case. After recording internal forces, elements were removed from the elastic model. All joints except the ones upper the column removal location were restrained in u1 and u2 directions in order to obtain modal periods associated with the structural response mode for the vertical motion of the bays above the removed column and brace or braces. These modal periods have been used then for the duration of the removal and nonlinear dynamic analysis. Determination of vertical mode is described in Figure 5.14.



a) b)

a) Sample column removal and joint movement vertical restriction

b) Sample vertical mode deformation

Figure 5.14. Sample vertical mode determination.

After this step, 2D nonlinear structural model was developed as it will be explained in Section 5.4.2. Both material and geometric nonlinearity have been considered. All structural analyses have been carried out utilizing structural analysis program SAP2000 Advanced 14.2.2. From the nonlinear model, elements considered to be removed according to APM case were deleted. To the removal locations reactions were applied, which are equivalent to the internal forces obtained from the elastic modal. The whole frame was then loaded with the load combination introduced in Equation 3.24 and depicted as in Figure 5.15.

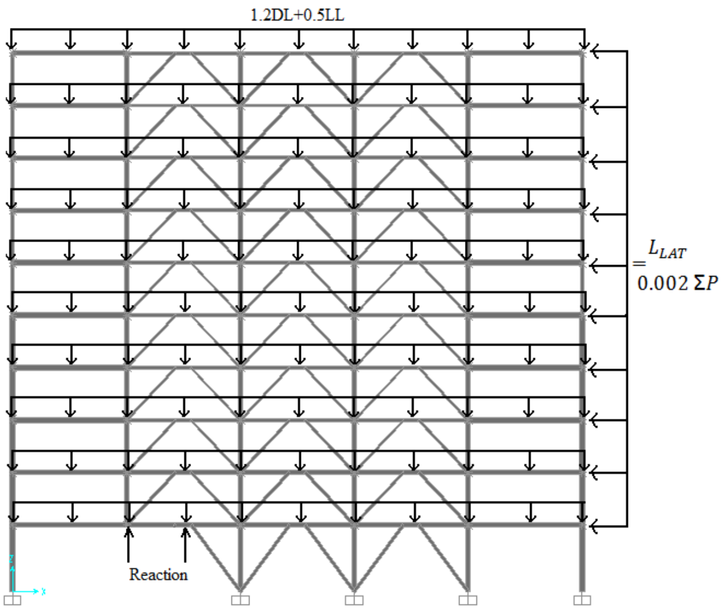


Figure 5.15. Nonlinear dynamic procedure loading.

In order to simulate the phenomenon that the column and the associated brace are suddenly removed, reaction forces were removed within a time step size called Δt_{off} after a time was elapsed as shown in Figure 5.16. Time step size is defined in Equation 5.55 for which the analysis result not being affected by solution algorithm (Gerasimidis and Baniotopoulos, 2011).

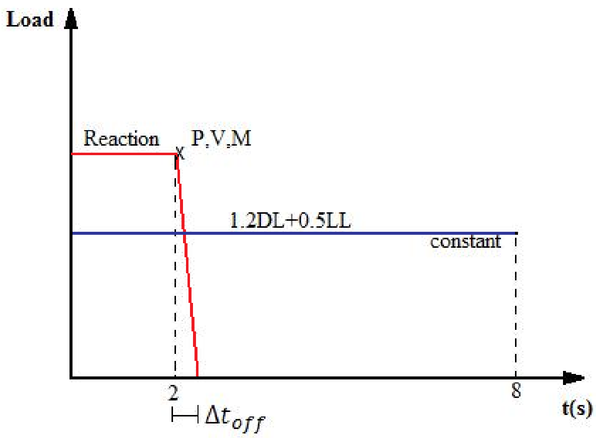


Figure 5.16. Time histories of loads for NDP.

$$\Delta t_{off} \leq \frac{T_{v1}}{300} \tag{5.55}$$

Where,

Δt_{off} = time step size

T_{v1} = first modal period associated with the structural response mode for the vertical motion of the bays above the removed column and brace

The reaction forces were kept unchanged for two seconds in order the system reach a stable condition and to avoid exciting dynamic effects. Then, column and associated brace or braces were removed suddenly simulated with the time step size whereas UFC load combination was kept unchanged as shown in Figure 5.16 until the end of nonlinear dynamic time history analysis. The analysis duration was fixed as eight seconds which has been observed as enough for the system to reach the maximum displacements and static conditions. Details of time history analysis are provided in Section 5.4.5. Finally, analysis results were used to check deformation and force controlled actions in accordance with UFC 4-023-03 (DoD, 2009). This methodology used for both SCBF and EBF is summarized in Figure 5.17.

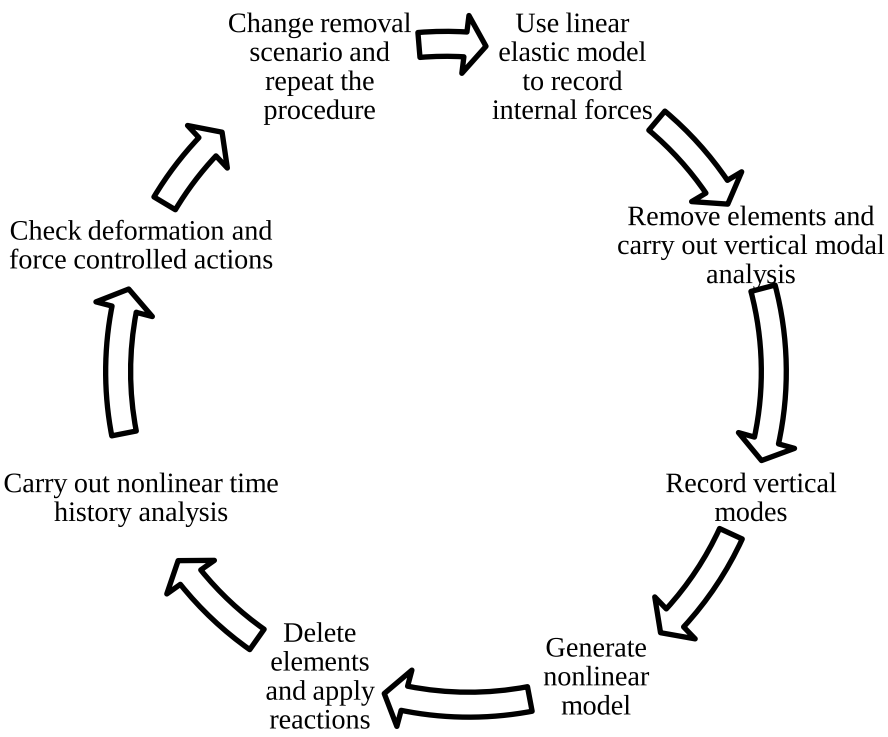


Figure 5.17. Flowchart of the methodology utilized.

5.4.2. Modeling

2D linear elastic models of N-S elevations of SCBF and EBF have been developed in order to obtain internal forces of elements removed. These models have been developed as in Figure 5.3 and in Figure 5.4 using sections defined in Table 5.1 and in Table 5.3 respectively. For each floor level of the models diaphragm action has been defined and all joints have been restrained to move in u2 direction, i.e., in perpendicular y direction to the frame.

Separate 2D linear elastic models of N-S elevations of SCBF and EBF have been developed for each removal scenario. These models have been used for determining modal periods associated with the structural response mode for the vertical motion of the bays above the removed column and brace or braces. These models possess the same properties as described in Figure 5.14 except removal of different elements described in Section 5.4.4. Mass source has been defined as in Equation 5.8 and Equation 5.9 with live load participation factor n=0.3 as defined in TEC 07 in order to carry out modal analysis.

According to nonlinear APM of UFC, the analytical model shall be discretized to represent the load-deformation response of each component along its length in order to identify location of inelastic (i.e. nonlinear) action. In other words, plastic hinges are allowed to form along the members. From the elements of braced frames; braces, shear links and connections are allowed to be checked by deformation controlled action as listed in Table 3.7. However, plastic hinges have been also defined for gravity beams and for beams of braced bays. For beam of a braced bay, plastic hinge has been used when the bay deformed as beam after the removal of a brace. Details of hinge properties are provided for each braced frame separately in the following subsections.

5.4.2.1. Plastic Hinge Properties – SCBF. Plastic hinges based on flexural capacity were allowed for beams of SCBF and their properties have been determined using Table 5.54 adopted from Table 5-6 of ASCE 41. This table provides both modeling parameters as defined in Figure 5.18 and acceptance criteria. Nonlinear acceptance criterion is selected from collapse prevention (CP) level. Intermediate performance levels such as immediate occupancy (IO) and life safety (LS) are also provided.

Table 5.54. Beam hinge modeling parameters (ASCE 41, 2006).

	Modeling Parameters			Acceptance Criteria		
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians		
				Primary		
Component/Action	a	b	c	IO	LS	CP
Beams - Flexure						
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ *and* $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ *and* $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lower resulting value shall be used.					

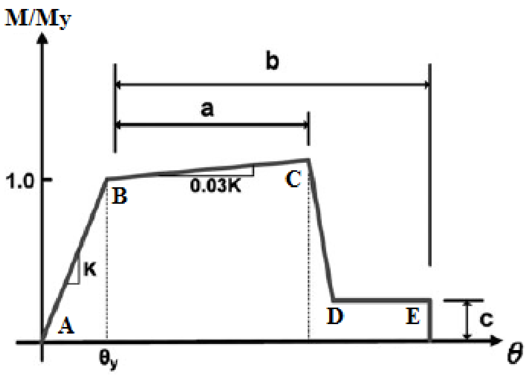


Figure 5.18. Force-deformation relationship for flexural members (Kim *et al.*, 2011).

Modeling parameters for plastic hinges of the beams are listed in Table 5.55 with the necessary intermediate deformation levels and nonlinear acceptance criteria. UFC requires that expected plastic moment capacity (M_{pe}) and associated yield rotation angle (θ_y) of beams are calculated by Equation 5.56 and Equation 5.57 respectively. These equations are defined by ASCE 41;however, bending strength reduction factor (ϕ_b) per AISC is also required by the UFC.

Table 5.55. Beam hinge properties – SCBF.

Beam	$b_f/2t_f$	h/t_w	Plastic Rotation Angle		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
Limit	7.01	56.36	a	b	c	Primary		
W24x76	6.60	54.25	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$
W21x50	6.10	54.82	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$
W16x31	6.26	57.75	$8.77\theta_y$	$10.77\theta_y$	0.58	$0.96\theta_y$	$5.81\theta_y$	$7.77\theta_y$

$$M_{pe} = \phi_b x W_p x F_{ye} \text{ where } F_{ye} = R_y x F_e \tag{5.56}$$

$$\theta_y = \frac{M_p l_b}{6EI_x} \tag{5.57}$$

Where,

M_{pe} = expected plastic moment capacity

ϕ_b = bending strength reduction factor

W_p = plastic section modulus

F_{ye} = expected yield strength

R_y = 1.1 from Table A3.1 of AISC 341 (AISC, 2010)

θ_y = yield rotation angle

l_b = beam length

E = modulus of elasticity

I_x = moment of inertia in x-direction

Yield rotation angle and moment capacities of beams are as calculated in Table 5.56, which are used to define plastic beam hinges as in Table 5.55. As allowed by ASCE 41, a strain hardening slope of 3% of the elastic slope has been used. A screen shot of a hinge property definition in SAP2000 is provided in Figure 5.19. Beam plastic hinges have been allowed to occur at mid-span of beams.

Table 5.56. Beam yield rotation angles- SCBF.

Beam	l_b (m)	W_p (m^3)	F_{ye} (MPa)	I_x (m^4)	M_{pe} (kN.m)	θ_y (rad)
W16x31	9.14	8.849E-04	379.5	1.561E-04	302.2	0.01475
W21x50	9.14	1.803E-03	379.5	4.096E-04	615.8	0.01145
W24x76	9.14	3.277E-03	379.5	8.741E-04	1119.3	0.00975

Plastic hinges based on compression or tension capacity were allowed to form for braces and their properties have been determined using Table 5.57 adopted from Table 5-7 of ASCE 41. This table provides both modeling parameters as defined in Figure 5.20 and acceptance criteria. Nonlinear acceptance criterion is selected from collapse prevention

(CP) level. Intermediate performance levels such as immediate occupancy (IO) and life safety (LS) are also provided.

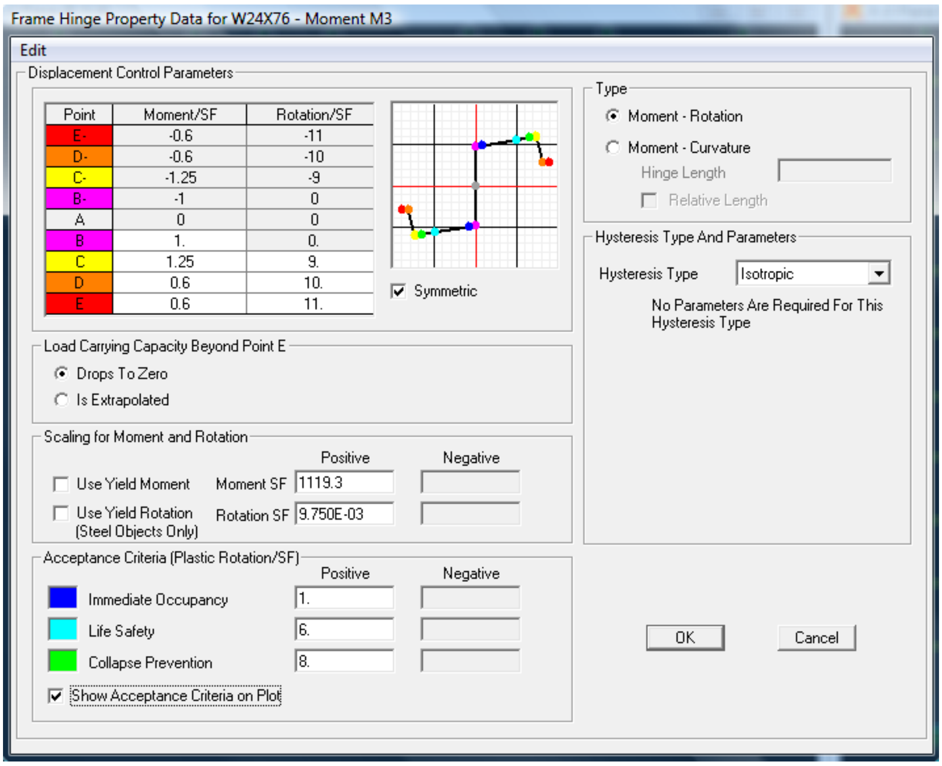


Figure 5.19. Sample hinge property data of SAP2000 – Beam.

Table 5.57. Brace hinge modeling parameters (ASCE 41, 2006).

	Modeling Parameters			Acceptance Criteria		
	Plastic Deformation		Residual Strength Ratio	Plastic Deformation		
				Primary		
Component/Action	a	b	c	IO	LS	CP
Braces in Compression						
a. Slender $\frac{Kl}{r} \geq 4.2\sqrt{E/F_y}$ HSS, Pipes, Tubes	$0.5\Delta_c$	$9\Delta_c$	0.3	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$
b. Stocky $\frac{Kl}{r} \leq 2.1\sqrt{E/F_y}$ HSS, Pipes, Tubes	$1\Delta_c$	$7\Delta_c$	0.5	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$
c. Intermediate	Linear interpolation between the values for slender and stocky braces (after application of all applicable modifiers) shall be used.					
Braces in Tension	$11\Delta_T$	$14\Delta_T$	0.8	$0.25\Delta_T$	$7\Delta_T$	$9\Delta_T$

Δ_c is the axial deformation at expected buckling load

Δ_T is the axial deformation at expected tensile yielding load

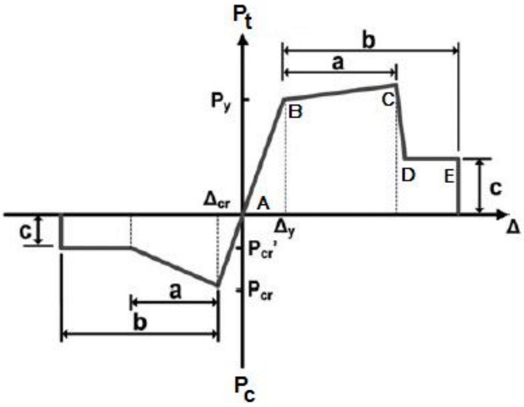


Figure 5.20. Force-deformation relationship for braces (Kim *et al.*, 2011).

Modeling parameters for the braces of SCBF under compression and tension are listed in Table 5.58 and 5.59 respectively with the necessary intermediate deformation levels and nonlinear acceptance criteria.

Table 5.58. Brace hinge properties in compression- SCBF.

Brace	Kl/r	Plastic Deformation		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
Limit: a>105.5, b<52.75		a	b	c	Primary		
HSS 7x7x1/2-First Story	104.9	0.51 Δ_c	8.98 Δ_c	0.30	0.25 Δ_c	4.99 Δ_c	6.99 Δ_c
HSS 7x7x1/2	92.8	0.62 Δ_c	8.52 Δ_c	0.35	0.25 Δ_c	4.76 Δ_c	6.76 Δ_c
HSS 6x6x1/2	109.7	0.50 Δ_c	9.00 Δ_c	0.30	0.25 Δ_c	5 Δ_c	7.00 Δ_c
HSS 5-1/2x5-1/2x3/8	117.6	0.50 Δ_c	9.00 Δ_c	0.30	0.25 Δ_c	5 Δ_c	7.00 Δ_c
HSS 4-1/2x4-1/2x3/8	146.4	0.50 Δ_c	9.00 Δ_c	0.30	0.25 Δ_c	5 Δ_c	7.00 Δ_c

Table 5.59. Brace hinge properties in tension- SCBF.

Brace	Plastic Deformation		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
	a	b	c	Primary		
HSS 7x7x1/2-First Story	11 Δ_t	14 Δ_t	0.8	0.25 Δ_t	7 Δ_t	9 Δ_t
HSS 7x7x1/2	11 Δ_t	14 Δ_t	0.8	0.25 Δ_t	7 Δ_t	9 Δ_t
HSS 6x6x1/2	11 Δ_t	14 Δ_t	0.8	0.25 Δ_t	7 Δ_t	9 Δ_t
HSS 5-1/2x5-1/2x3/8	11 Δ_t	14 Δ_t	0.8	0.25 Δ_t	7 Δ_t	9 Δ_t
HSS 4-1/2x4-1/2x3/8	11 Δ_t	14 Δ_t	0.8	0.25 Δ_t	7 Δ_t	9 Δ_t

According to UFC, expected buckling load (P_{cre}) is calculated by Equation 5.58, Equation 5.59 and Equation 5.60, and associated axial deformation (Δ_c) is calculated by Equation 5.61, which are defined by Specifications for Structural Steel Buildings (AISC, 2010). Different than the AISC equations, UFC requires using expected yield strength R_yF_y (F_{ye}) in lieu of F_y. Here, R_y=1.4 from Table A3.1 of AISC 341 (AISC, 2010). These equations are also addressed by ASCE 41; however, compressive strength reduction factor (ϕ_c=0.90) per AISC is required by the UFC. Expected buckling load and plastic deformations of braces are as calculated in Table 5.60 and in Table 5.61, which are used to define plastic brace hinges in the compression range as in Table 5.58.

$$if \begin{cases} \lambda \leq 4.71\sqrt{\dfrac{E}{F_{ye}}} = 100\,;\;\; F_{cr} = \left[0.658^{\frac{F_{ye}}{F_e}}\right] F_{ye} \\ \lambda \geq 4.71\sqrt{\dfrac{E}{F_{ye}}} = 100\,;\; F_{cr} = 0.877 F_e \end{cases} \tag{5.58}$$

$$F_e = \frac{\pi^2 E}{\lambda^2} \tag{5.59}$$

$$P_{cre} = \phi_c F_{cr} A \tag{5.60}$$

$$\Delta_c = \frac{P_{cre} L}{EA} \tag{5.61}$$

Table 5.60. Expected buckling loads of braces- SCBF.

Brace	Area (m^2)	λ	F_e (Mpa)	F_{cr} (Mpa)	P_{cre} (kN)
HSS 7x7x1/2-First Story	0.007484	104.9	179.22	157.18	1058.7
HSS 7x7x1/2	0.007484	92.8	229.31	197.42	1329.7
HSS 6x6x1/2	0.006284	109.7	164.13	143.95	814.1
HSS 5-1/2x5-1/2x3/8	0.004439	117.6	142.83	125.27	500.5
HSS 4-1/2x4-1/2x3/8	0.003535	146.4	92.11	80.78	257.0

Tensile yielding load (P_{te}) is calculated by Equation 5.62 and associated axial deformation (Δ_t) is calculated by Equation 5.63. Different than the AISC equations, UFC requires using expected yield strength R_yF_y (F_{ye}) in lieu of F_y. Also, tensile strength reduction factor (ϕ_t=0.90) per AISC is required by the UFC. Tensile yielding load and plastic deformations of braces are as calculated in Table 5.62, which are used to define plastic brace hinges in the tension range as in Table 5.59.

Table 5.61. Axial deformations at expected buckling loads- SCBF.

Brace	l(cm)	Area (m^2)	P_{cre} (kN)	Δ_c (m)
HSS 7x7x1/2-First Story	702.1	0.007484	1058.7	0.00497
HSS 7x7x1/2	620.7	0.007484	1329.7	0.00551
HSS 6x6x1/2	620.7	0.006284	814.1	0.00402
HSS 5-1/2x5-1/2x3/8	620.7	0.004439	500.5	0.00350
HSS 4-1/2x4-1/2x3/8	620.7	0.003535	257.0	0.00226

$$P_{te} = \phi_c F_{ye} A \tag{5.62}$$

$$\Delta_t = \frac{P_{te} L}{EA} \tag{5.63}$$

Table 5.62. Tensile yielding loads and associated axial deformations of braces- SCBF.

Brace	l(cm)	Area (m^2)	P_{te} (kN)	Δ_t (m)
HSS 7x7x1/2-First Story	702.1	0.007484	2989.3	0.0140
HSS 7x7x1/2	620.7	0.007484	2989.3	0.0124
HSS 6x6x1/2	620.7	0.006284	2510.0	0.0124
HSS 5-1/2x5-1/2x3/8	620.7	0.004439	1773.0	0.0124
HSS 4-1/2x4-1/2x3/8	620.7	0.003535	1411.9	0.0124

Brace plastic hinges have been allowed to occur at mid-span of braces assuming cross gusset plates are used as brace to beam/column connections. According to AISC 341 (AISC, 2010), this type of connection remain elastic during loading and forces the plastic hinges to form in the braces. A screen shot of a brace hinge property definition in SAP2000 is provided in Figure 5.21.

As stated in Section 5.2, SCBF in N-S elevation has single plate simple shear tab connections and limited number of fully restrained welded moment connections (WUF-W) as shown in Figure 5.6a. Hinge properties of these connections have been determined using Table 5-2 of the UFC (see Table 3.9). Properties of fully restrained and partially restrained connection hinges are calculated as in Table 5.63 and in Table 5.64 respectively. As UFC does not specify intermediate deformation levels; IO and LS, limits for these levels have been assumed as shown in associated tables. Depth of bolt group (d_{bg}) has been assumed same d_{bg}=6 in as for all beam sections.

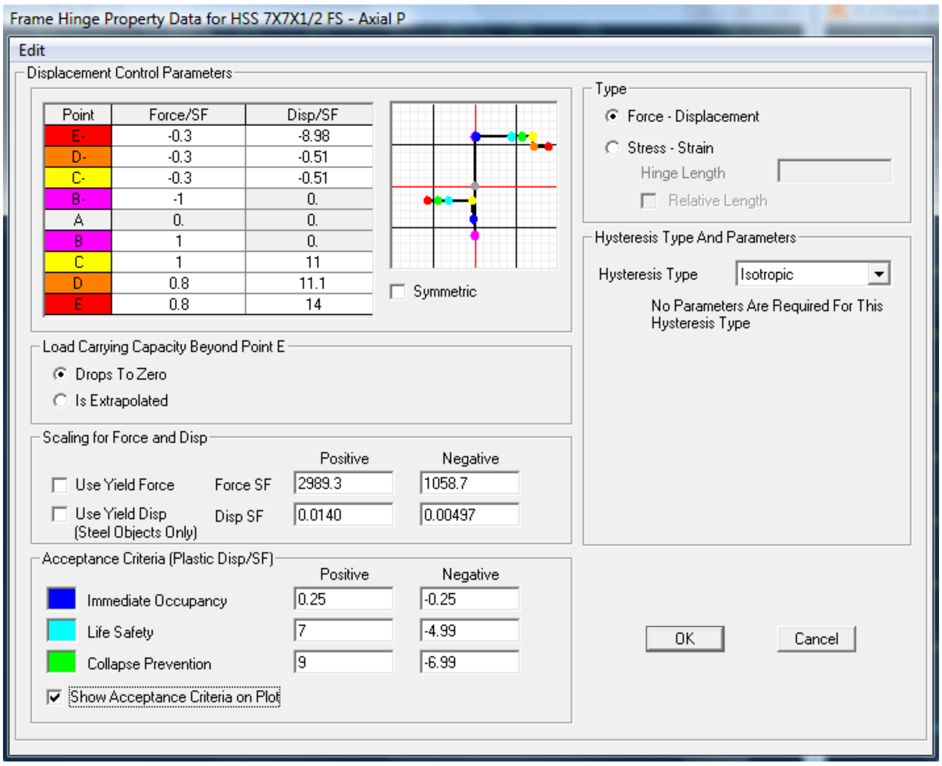


Figure 5.21. Sample hinge property data of SAP2000 – Brace.

Table 5.63. Fully restrained connection hinge properties – SCBF.

Beam	Depth (in) d	Plastic Rotation Angle		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
		a	b	c	Primary		
		rad	rad		rad		
		0.0284-0.0004d	0.043-0.0006d		0.50*CP	0.75*CP	0.0284-0.0004d
W21x50	20.83	0.02007	0.03050	0.2	0.01003	0.01505	0.02007

Connection hinges have been allowed to form at the ends of beam members. Values in Tables 5.63 and 5.64 have been used to define connection hinges as in Figure 5.20 without strain hardening. A screen shot of a connection hinge property definition in SAP2000 is provided in Figure 5.22.

Table 5.64. Partially restrained connection hinge properties – SCBF.

Beam	Depth H (in) d_{bg}	Plastic Rotation Angle		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
		a	b	c	Primary		
		rad	rad		rad		
		$0.0502-0.0015d_{bg}$	$0.072-0.0022d_{bg}$		0.33*CP	0.7*CP	$0.0502-0.0015d_{bg}$
W24x76	6	0.04120	0.05880	0.2	0.01370	0.02880	0.04120
W21x50	6	0.04120	0.05880	0.2	0.01370	0.02880	0.04120
W16x31	6	0.04120	0.05880	0.2	0.01370	0.02880	0.04120

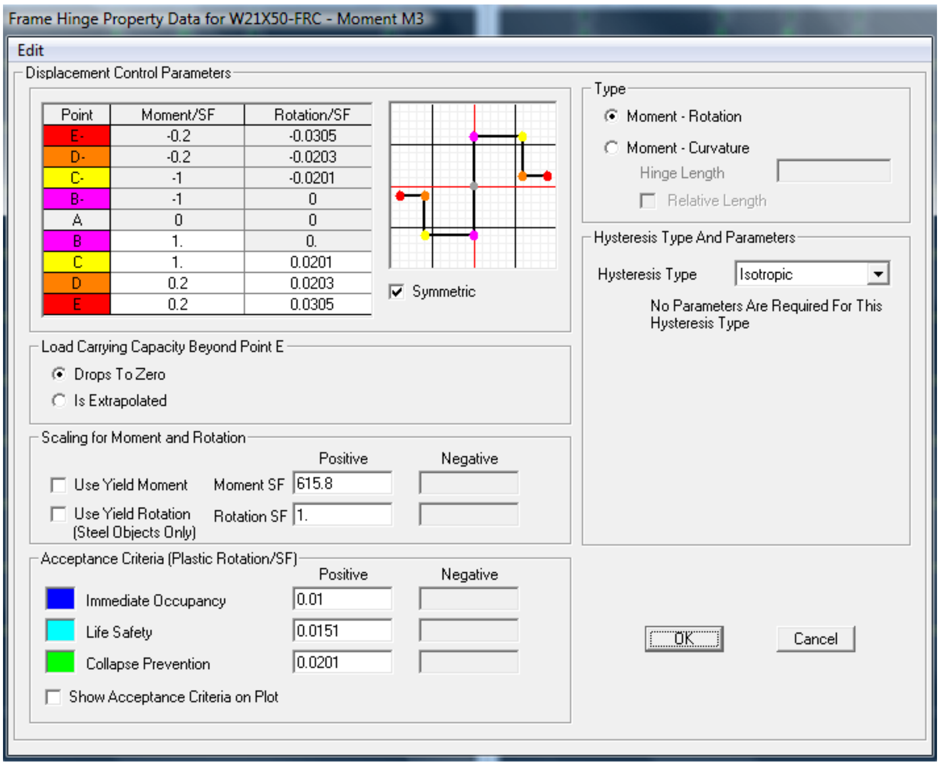


Figure 5.22. Sample hinge property data of SAP2000 – Connection.

<u>5.4.2.2. Plastic Hinge Properties – EBF.</u> Plastic hinges based on flexural capacity were allowed for beams of EBF and their properties have been determined using same the procedure explained in the previous section. Modeling parameters for beam plastic hinges are listed in Table 5.65 and the required yield rotation angle and moment capacities of beams are as calculated in Table 5.66.

Table 5.65. Beam hinge properties – EBF.

Beam	$b_f/2t_f$	h/t_w	Plastic Rotation Angle		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
	7.01	56.36	a	b	c	Primary		
W14x48	6.75	40.6	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$
W12x45	6.99	36	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$
W10x39	7.59	31.5	$7.34\theta_y$	$9.34\theta_y$	0.47	$0.75\theta_y$	$4.67\theta_y$	$6.34\theta_y$
W16x31	6.26	57.75	$8.77\theta_y$	$10.77\theta_y$	0.58	$0.96\theta_y$	$5.81\theta_y$	$7.77\theta_y$

Table 5.66. Beam yield rotation angles- EBF.

Beam	l_b (m)	W_p (m^3)	σ_{ye} (MPa)	I_x (m^4)	M_{pe} (kN.m)	θ_y (rad)
W14x48	9.14	1.285E-03	379.5	2.019E-04	438.9	0.01656
W12x45	9.14	1.060E-03	379.5	1.457E-04	362.0	0.01893
W10x39	9.14	7.669E-04	379.5	8.699E-05	261.9	0.02293
W16x31	9.14	8.849E-04	379.5	1.561E-04	302.2	0.01475

Plastic hinges based on shear capacity were allowed for shear links of EBF and their properties have been determined using Table 5.67 adopted from Table 5-6 of ASCE 41. This table provides both modeling parameters as defined in Figure 5.18 and acceptance criteria. Nonlinear acceptance criterion is selected from collapse prevention (CP) level.

Table 5.67. Link beam hinge modeling parameters (ASCE 41, 2006).

	Modeling Parameters			Acceptance Criteria		
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians		
				Primary		
Component/Action	a	b	c	IO	LS	CP
EBF Link Beam						
a. $e \leq \frac{1.6M_{CE}}{V_{ce}}$	0.15	0.17	0.8	0.005	0.11	0.14
b. $e \geq \frac{2.6M_{CE}}{V_{ce}}$	Same as for beams					
c. $\frac{1.6M_{CE}}{V_{ce}} < e < \frac{2.6M_{CE}}{V_{ce}}$	Linear interpolation shall be used					

Modeling parameters for plastic hinges of the shear links are listed in Table 5.68 with the necessary intermediate deformation levels and nonlinear acceptance criteria. UFC requires that expected shear capacity (V_{ce}) of the shear links are calculated by Equation

5.64 depending on the eccentricity. These equations are defined by ASCE 41; however, shear strength reduction factor (ϕ_v=0.90) per AISC is also required by the UFC.

$$if \begin{cases} e \leq \dfrac{1.6M_{ce}}{V_{ce}}; V_{ce} = 0.6F_{ye}A_w \\ e > \dfrac{2.6M_{ce}}{V_{ce}}; V_{ce} = \dfrac{2M_{ce}}{e} \\ linear\ interpolation\ between\ two \\ for\ intermediate\ e\ values \end{cases} \tag{5.64}$$

Table 5.68. Shear links hinge properties.

Beam	Limits			Plastic Rotation Angle		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
	$1.6(M_{ce}/V_{ce})$	e (m)	$2.6(M_{ce}/V_{ce})$	a	b	c	Primary		
W14X48	1.24	1.20	2.01	0.150	0.170	0.800	0.005	0.110	0.140
W12X45	1.20	1.20	1.95	0.150	0.170	0.800	0.005	0.110	0.140
W10X39	1.14	1.20	1.85	0.152	0.174	0.770	0.005	0.111	0.141

Table 5.69. Expected shear capacities of the shear links.

Beam	$1.6(M_{ce}/V_{ce})$	e (m)	$2.6(M_{ce}/V_{ce})$	A_k (m^2)	V_{ce} (kN)
W14x48	1.24	1.20	2.01	0.002764	566.5
W12x45	1.20	1.20	1.95	0.002358	483.2
W10x39	1.14	1.20	1.85	0.001800	374.6

Plastic hinges at shear links have been allowed to form at the middle of link beams. Values in Tables 5.68 and 5.69 have been used to define connection hinges as in Figure 5.20 without strain hardening. A screen shot of a connection hinge property definition in SAP2000 is provided in Figure 5.23.

Plastic hinges based on compression or tension capacity were allowed to form at braces and their properties have been determined using Table 5.57. Same procedure described in Section 5.4.2.1 has been applied to the hinge property definitions and application of EBF braces. Modeling parameters for the braces of EBF under compression and tension are listed in Table 5.70 and 5.71 respectively with the necessary intermediate deformation levels and nonlinear acceptance criteria.

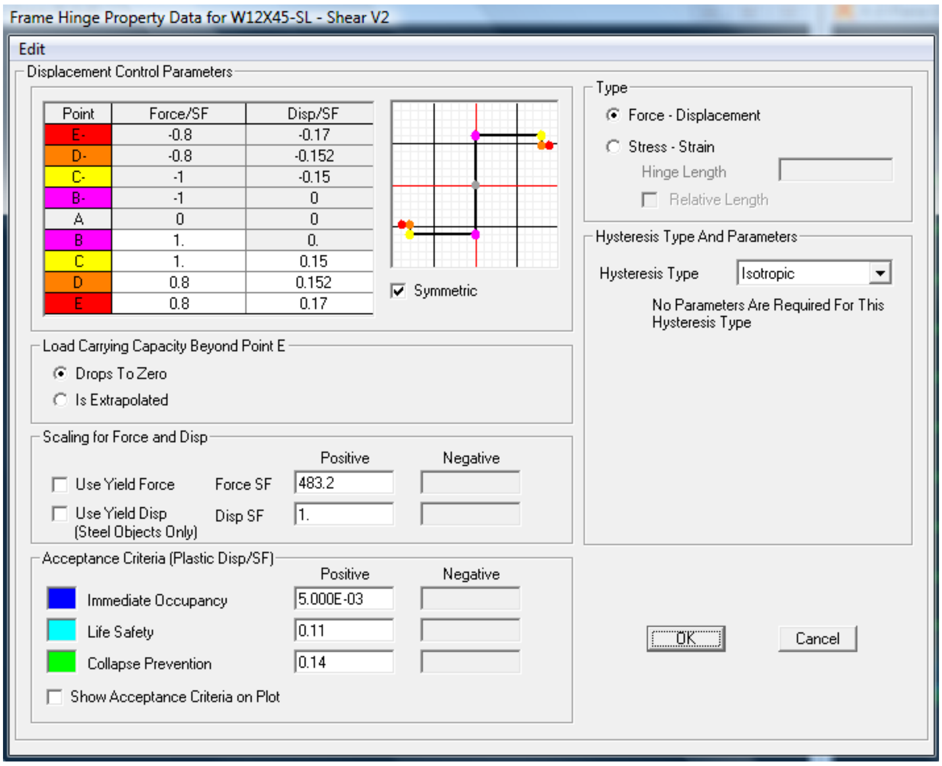


Figure 5.23. Sample hinge property data of SAP2000 – Shear link.

Table 5.70. Brace hinge properties in compression- EBF.

Brace	Kl/r	Plastic Deformation		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
Limit: a>105.5, b<52.75		a	b	c	Primary		
HSS 8x8x1/2-FS	86.0	0.68 Δ_c	8.26 Δ_c	0.37	0.25 Δ_c	4.63 Δ_c	6.63 Δ_c
HSS 8x8x1/2	74.8	0.79 Δ_c	7.84 Δ_c	0.42	0.25 Δ_c	4.42 Δ_c	6.42 Δ_c
HSS 7x7x1/2	86.4	0.68 Δ_c	8.28 Δ_c	0.37	0.25 Δ_c	4.64 Δ_c	6.64 Δ_c

Expected buckling load and plastic deformations of braces are as calculated in Table 5.72, which are used to define plastic brace hinges in the compression range as in Table 5.70. Tensile yielding load and plastic deformations of braces are as calculated in Table 5.73, which are used to define plastic brace hinges in the tension range as in Table 5.71.

Table 5.71. Brace hinge properties in tension- EBF.

Brace	Plastic Deformation		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
	a	b	c	Primary		
HSS 8x8x1/2	11 Δt	14 Δt	0.8	0.25 Δ_t	7 Δ_t	9 Δ_t
HSS 7x7x1/2	11 Δt	14 Δt	0.8	0.25 Δ_t	7 Δ_t	9 Δ_t

Table 5.72. Expected buckling loads and associated axial deformations of braces- EBF.

Brace	l(cm)	Area (m^2)	λ	F_e (MPa)	F_{cr} (Mpa)	P_{cre} (kN)	Δ_c (m)
HSS 8x8x1/2-First Story	664.6	0.008710	86.0	266.89	221.27	1734.5	0.00662
HSS 8x8x1/2	577.9	0.008710	74.8	352.80	262.14	2054.9	0.00682
HSS 7x7x1/2	577.9	0.007484	86.4	264.42	219.84	1480.7	0.00572

Table 5.73. Tensile yielding loads and associated axial deformations of braces- EBF.

Brace	l(cm)	Area (m^2)	P_{te} (kN)	Δ_t (m)
HSS 8x8x1/2-First Story	664.6	0.008710	3478.9	0.0133
HSS 8x8x1/2	577.9	0.008710	3478.9	0.0115
HSS 7x7x1/2	577.9	0.007484	2989.3	0.0115

The same procedure used to define connection hinges of SCBF has been applied to the simple shear tab connections and to the limited number of fully restrained welded moment connections (WUF-W) of EBF in N-S elevation as shown in Figure 5.6b. Properties of fully restrained connection hinges are as calculated in Table 5.74. For the only partially restrained connection hinge, which is used to connect W16x31 beam to columns, the properties calculated in Table 5.64 are valid.

Table 5.74. Fully restrained connection hinge properties – EBF.

Beam	Depth (in) d	Plastic Rotation Angle		Residual Strength Ratio	IO	LS	Nonlinear Acceptance Criteria
		a	b	c	Primary		
		rad	rad		rad		
		0.0284-0.0004d	0.043-0.0006d		0.50*CP	0.75*CP	0.0284-0.0004d
W14x48	13.79	0.02288	0.03473	0.2	0.01144	0.01716	0.02288
W12x45	12.06	0.02358	0.03576	0.2	0.01179	0.01768	0.02358
W10x39	9.92	0.02443	0.03705	0.2	0.01222	0.01832	0.02443

5.4.3. Loading Procedure

The gravity loads introduced in Section 5.2 with Equation 5.1 to Equation 5.6 are also valid for the 2D progressive collapse models. However, in order to reflect a possible collapse behavior of the external braced frames, the presence of secondary beams (W16x31) should also be considered. In a possible collapse scenario, secondary beams will also resist extra dynamic loads and they will transfer these through W21x50 beams to the columns and thus to the external frame. Therefore, a new loading procedure different than the one in Section 5.3.2. is necessary. The previous loading procedure would have overestimated the loads on beams of EBF or SCBF.

These area loads have been converted to line and point loads as shown in Figure 5.24 and have been transferred accordingly. Live load reduction was applied per TS498 as shown in Table 5.6. Dead loads of the elements were applied by the analysis program automatically.

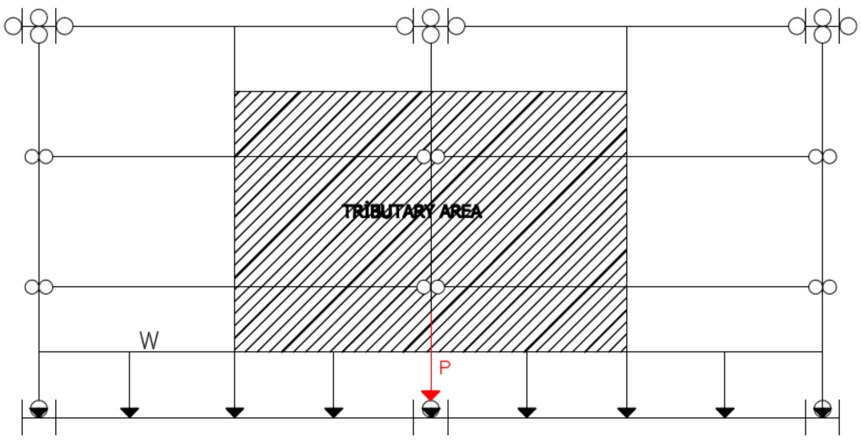


Figure 5.24. Transfer of gravity loads to the braced frames.

For normal floors,

between line A to F (See Figure 5.3 and 5.4)

$$g = 1.52x3.638 = 5.54\ \frac{kN}{m} \qquad q = 1.52x4.788 = 7.28\ \frac{kN}{m}$$

on columns B-C-D-E

$$P_g = \frac{6.093x9.14x3.638}{2} = 101.31\ kN \quad P_q = \frac{6.093x9.14x4.788}{2} = 133.33\ kN$$

on columns A-F

$$P_g = \frac{6.093x9.14x3.638}{4} = 50.65\ kN \quad P_q = \frac{6.093x9.14x4.788}{4} = 66.67\ kN$$

For the roof ,

between line A to F (See Figure 5.3 and 5.4)

$$g = 1.52x2.681 = 4.07\ \frac{kN}{m} \qquad q = 1.52x0.958 = 1.46\ \frac{kN}{m}$$

on columns B-C-D-E

$$P_g = \frac{6.093x9.14x2.681}{2} = 74.66\ kN \quad P_q = \frac{6.093x9.14x0.958}{2} = 26.68\ kN$$

on columns A-F

$$P_g = \frac{6.093x9.14x2.681}{4} = 37.33\ kN \quad P_q = \frac{6.093x9.14x0.958}{4} = 13.34\ kN$$

UFC requires also that the lateral load given in Equation 3.16 is applied to the sides of the structure one at a time. Due to the symmetry present in the structure and APM cases considered, lateral load has been applied to the frames only in negative x-direction as shown in Figure 5.15. As a result, a more severe loading combination has been obtained for bays from which elements had been removed. Lateral loads applied to the SCBF and EBF are as calculated in Table 5.75.

Table 5.75. Lateral loads applied to the frames.

Story	SCBF		EBF	
	ΣP (kN)	0.002ΣP	ΣP (kN)	0.002ΣP
Roof	8264.9	16.5	8315.5	16.6
9	14262.6	28.5	14313.1	28.6
8	14831.0	29.7	14838.5	29.7
7	15453.3	30.9	15487.0	31.0
6	16442.2	32.9	16461.9	32.9
5	17269.6	34.5	17266.1	34.5
4	18056.4	36.1	17966.2	35.9
3	18595.0	37.2	18516.6	37.0
2	18604.2	37.2	18542.7	37.1
1	18856.4	37.7	18731.1	37.5

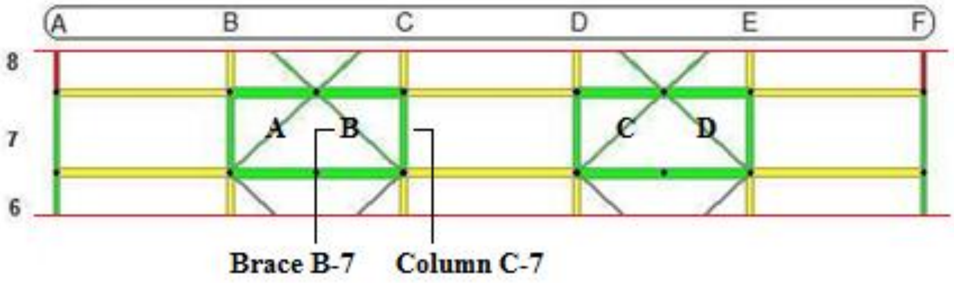
5.4.4. APM – Analysis Scenarios

Alternate Path Method (APM) requires that a structural model is analyzed for different column removal scenarios in order to verify that it has enough flexural resistance to bridge over an element loss due to an abnormal event. For this purpose, the UFC defines different internal and external column removal scenarios as discussed in Section 3.5.3. As the considered models are braced frames, braces connected to the columns specified have been also removed.

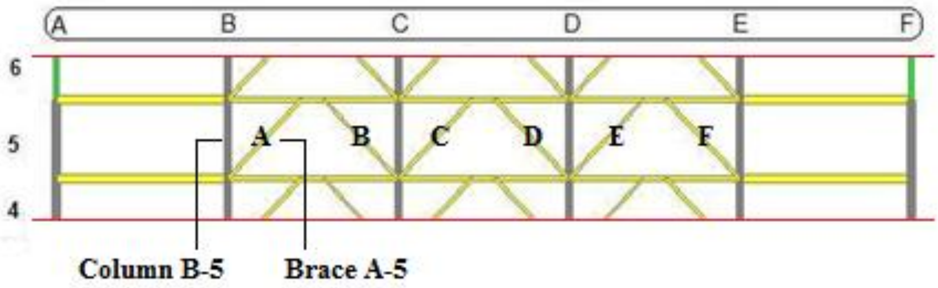
Due to the vertical symmetry present in the braced frames, elements have been removed only from the left hand side of the models. As a result, thirteen different APM scenarios for SCBF and ten scenarios for EBF have been obtained as listed in Table 5.76. The SCBF has three more scenarios than the EBF. Two of these cases are due to change in the section of columns between sixth and seventh floor, which is not the case for EBF. The other one; namely, the first scenario in SCBF, removal of the corner column, has not been considered for the EBF because corner column is also a part of the EBF in E-W elevation. The coding system used for the removed elements is illustrated in Figure 5.25 for a better understanding of Table 5.76.

Table 5.76. APM-Analysis cases (scenarios).

SCBF			EBF		
APM Case/Scenario	Elements Removed		APM Case/Scenario	Elements Removed	
	Column	Brace		Column	Brace/Braces
1	A-1	-	1	B-1	A-1
2	B-1	A-1	2	B-3	A-3
3	B-3	A-3	3	B-5	A-5
4	B-5	A-5	4	B-8	A-8
5	B-7	A-7	5	B-10	A-10
6	B-9	A-9	6	C-1	B-1 & C-1
7	B-10	A-10	7	C-3	B-3 & C-3
8	C-1	B-1	8	C-5	B-5 & C-5
9	C-3	B-3	9	C-8	B-8 & C-8
10	C-5	B-5	10	C-10	B-10 & C-10
11	C-7	B-7			
12	C-9	B-9			
13	C-10	B-10			



a) SCBF – case 11



b) EBF – case 3

Figure 5.25. Example of removal scenarios.

5.4.5. Details of Nonlinear Dynamic Analysis

Progressive collapse analysis procedure based on NDA has been introduced in Section 5.4.1. This procedure was carried out by defining first a nonlinear static analysis case as shown in Figure 5.26. This case was used as the starting condition for the column removal scenarios. In this analysis case, loads were applied according to the UFC load combination in Equation 3.24 to the braced frames. Geometric nonlinearity was considered selecting P-Delta option.

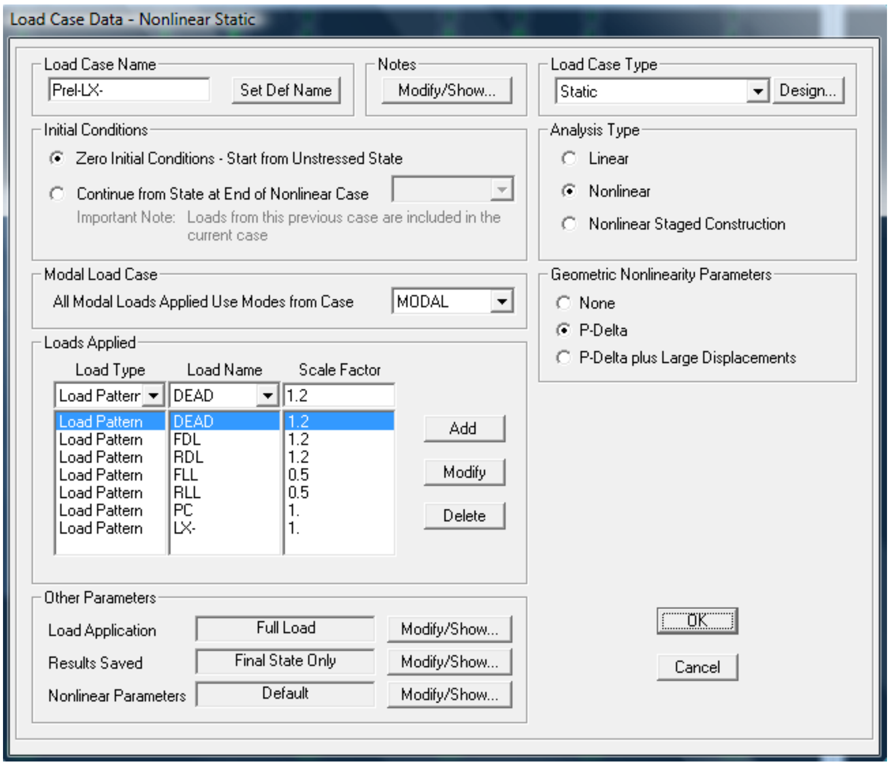


Figure 5.26. Nonlinear static analysis case definition.

The result of the nonlinear static case was used then as the initial condition of nonlinear direct time history analysis case as shown in Figure 5.27. Reaction forces applied to the frames as in Figure 5.15 were removed from the modals within a time step size explained in Figure 5.16 and Equation 5.15. This was attained by assigning a time history function called “Rdown” to the reaction forces (PC) as shown in Figure 5.28, which was allowed to arrive after two seconds. Thus, removal of column and associated brace or braces was simulated. Reaction forces are listed in Table 5.77 for each scenario separately.

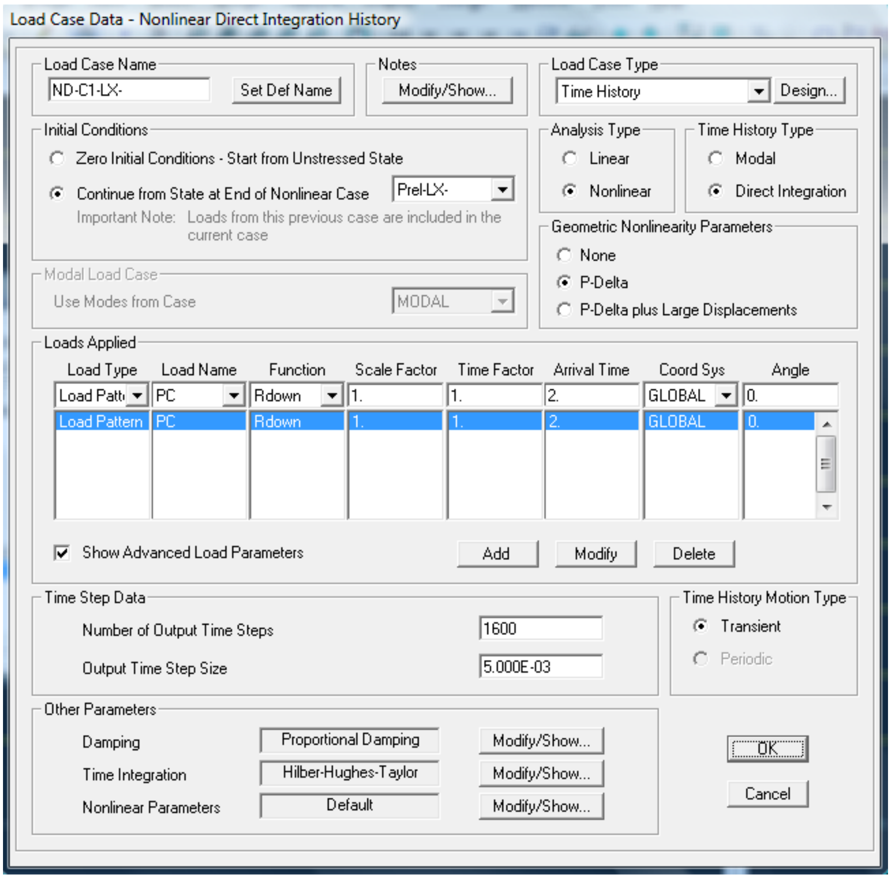


Figure 5.27. Nonlinear dynamic analysis case definition.

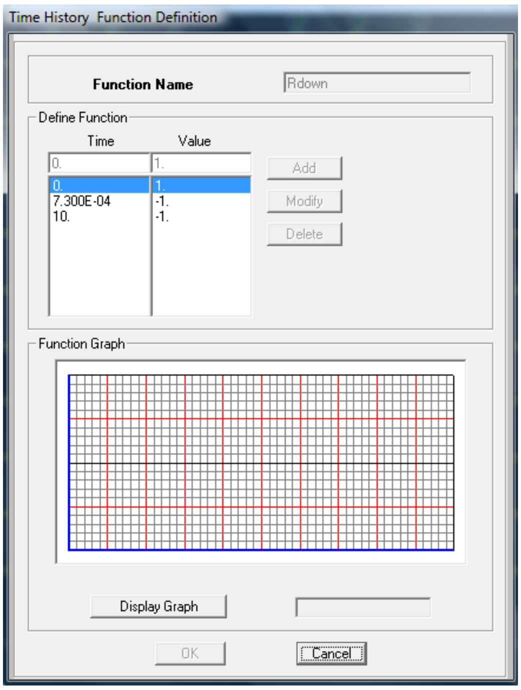


Figure 5.28. Time history function definition.

Damping ratio was assumed to be 5% of the critical mass and stiffness proportional damping, which is usually adopted in literature for the analysis of structures undergoing large deformations. SAP2000 utilizes Rayleigh damping and specifies damping by either period or frequency. In this study, first and second vertical modal periods were used as shown in Figure 5.29. Vertical modal periods of each analysis case and associated time step sizes are listed in Table 5.77. Time step size was chosen as discussed before except for APM case 2 of EBF due to convergence problem with low step sizes. Nevertheless, step size for this case also suitable to UFC criteria.

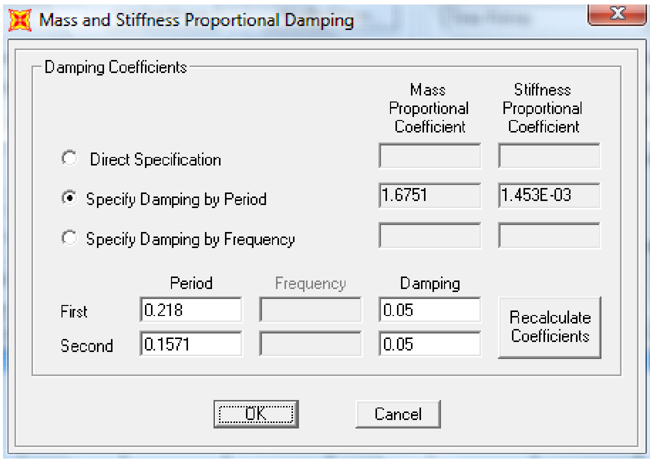


Figure 5.29. Damping definition.

Hilber-Hughes-Taylor alpha (HHT) method was used as the solution algorithm. This method uses a single parameter called alpha (α). This parameter may take values between 0 and -1/3 in order to encourage a nonlinear solution to converge. The method offers higher accuracy for α =0;however, for some APM cases it had to be chosen close to 0 for convergence as show in Table 5.77. A time step size of 0.005s together with total 1600 steps allowed to record nonlinear dynamic behavior of the frames for 8s, which was enough to system reach a stable condition. Table 5.77 summarizes parameters used for the NDA. In this table P_{col}, P_{br}, refers to the reaction forces applied to the removal locations for column and brace of SCBF respectively. P_{sum} refers for the summation of reaction forces of column and brace for some APM case, for which elements are connected to the same joint. P_{col}, P_{br1} and P_{br2}, refers to the reaction forces applied to the removal locations for column and braces of EBF respectively. Sign convention is positive for +z direction. T_1 and T_2 refers to the first and second vertical periods.

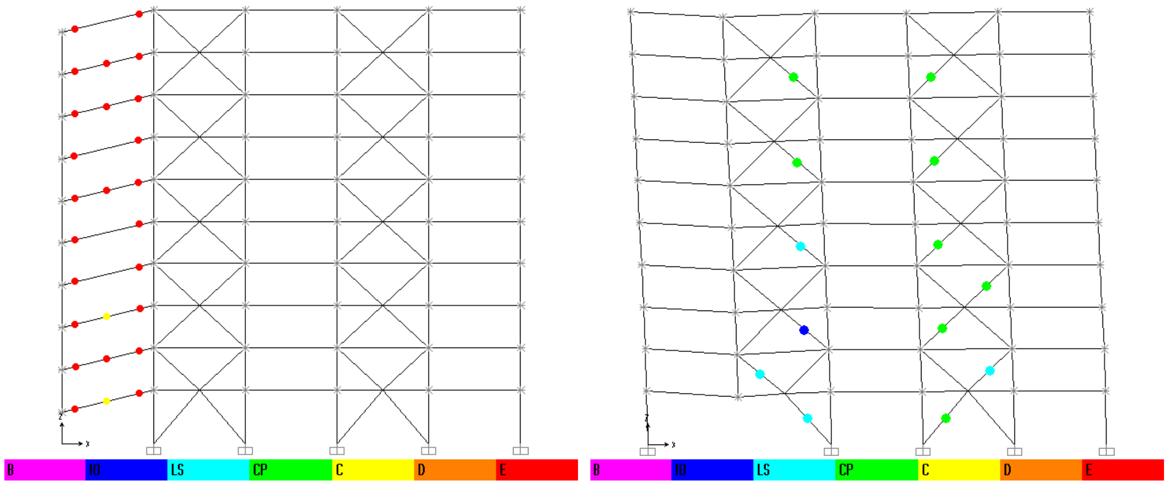
Table 5.77. NDA parameters.

	Elements Removed								
APM Case	Column	Brace/Braces	(kN)			(s)			
SCBF			P_{col}	P_{br}	P_{sum}	T_1	T_2	Δt_{off}	α
1	A-1	-	1357.9			7.3072	0.1571	2.44E-02	0
2	B-1	A-1	2840.3	271.6		0.2180	0.1571	7.30E-04	0
3	B-3	A-3	2165.9	176.0		0.2324	0.1571	7.75E-04	0
4	B-5	A-5	1477.0	158.5		0.2345	0.1571	7.75E-04	0
5	B-7	A-7	889.8	107.4		0.2327	0.1571	7.75E-04	0
6	B-9	A-9	370.2	58.9		0.2230	0.1571	7.40E-04	0
7	B-10	A-10	145.9	15.3	161.2	1.6668	0.1571	5.50E-03	-0.1
8	C-1	B-1	2246.6	96.3		0.2180	0.1571	7.30E-04	0
9	C-3	B-3	1816.9	64.1		0.2324	0.1571	7.75E-04	0
10	C-5	B-5	1303.0	79.5		0.2345	0.1571	7.82E-04	0
11	C-7	B-7	827.4	59.7		0.2327	0.1571	7.76E-04	0
12	C-9	B-9	362.6	38.8		0.2230	0.1571	7.43E-04	0
13	C-10	B-10	138.2	23.1	161.3	1.6668	0.1571	5.56E-03	0
EBF			P_{col}	P_{br1}	P_{br2}	T_1	T_2	Δt_{off}	α
1	B-1	A-1	2860.6	103.3		0.2858	0.1717	1.00E-03	0
2	B-3	A-3	2179.2	78.9		0.2974	0.1716	1.30E-02	0
3	B-5	A-5	1529.1	63.2		0.3071	0.1721	1.00E-03	-0.01
4	B-8	A-8	655.0	40.3		0.3174	0.1738	1.00E-03	0
5	B-10	A-10	143.2	19.2		1.3344	0.1787	4.00E-03	-0.3
6	C-1	B-1 & C-1	2710.9	-27.3	102.6	0.2254	0.1644	7.50E-04	0
7	C-3	B-3 & C-3	2097.6	-11.0	80.8	0.2342	0.1647	7.50E-04	0
8	C-5	B-5 & C-5	1487.1	0.8	66.4	0.2386	0.1655	7.50E-04	0
9	C-8	B-8 & C-8	638.9	15.6	42.6	0.2354	0.1675	7.50E-04	0
10	C-10	B-10 & C-10	128.5	14.8	21.8	0.4876	0.1782	1.50E-03	0

5.4.6. Results of Nonlinear Dynamic Analysis

UFC requires that primary elements and components, which have deformation-controlled internal forces, shall have expected deformation capacities greater than the maximum calculated deformation demands. Expected deformation capacities are listed in Section 5.4.2 for beams, shear links, braces and connections of both braced frames separately as nonlinear acceptance criteria (NAC). NAC was fixed as CP level in SAP2000. Columns of braced frames shall meet the requirement of Equation 3.23. In this section elements and connections of SCBF and EBF will be checked according to this acceptance criterion for each APM case separately.

<u>5.4.6.1. SCBF.</u> Removal of corner column (APM Case 1) caused first shear connections of bay AB gravity beams to deform inelastically. Less than 1 second, at time 2.295s, shear connection at the right of roof gravity beam on AB bay reached collapse prevention level and the bay collapsed progressively to the down as shown in Figure 5.30. The axial force in column B-1 increased from 2863 kN to 3692 kN, which is well below the capacity of this column. Axial capacities of columns of SCBF are listed in Table 5.78 and plastic moment capacities are as listed in Table 5.80. Per UFC, column capacities were calculated using lower bound strength reduced by strength reduction factor as in Equation 5.60.



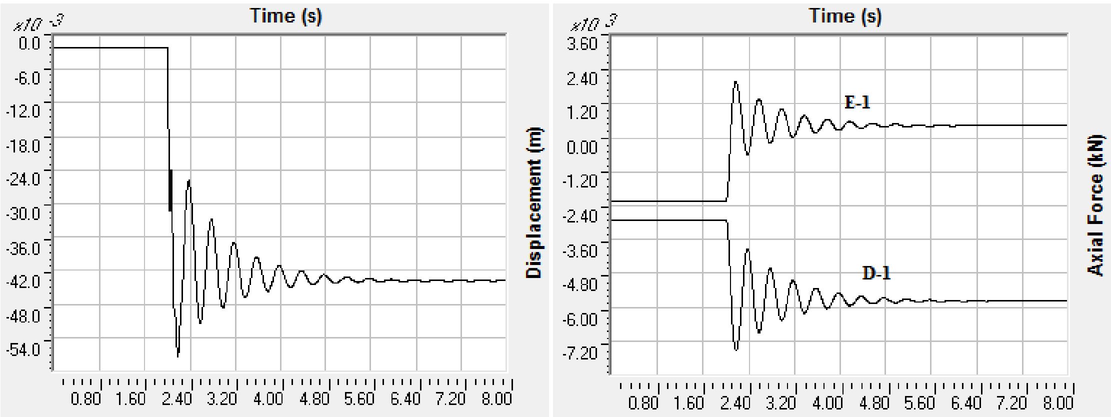
a) Case 1 b)Case 2

Figure 5.30. SCBF deformed shape of APM Case 1&2.

Beams of SCBF were able to redistribute additional loads to the neighbor bays without nonlinear deformation after the removal of column B-1 and brace A-1 (APM Case 2). However, thirteen braces deformed inelastically, i.e. buckled, and eight of them reached CP level as shown in Figure 5.30, which is in the limit of NAC. In the meantime, vertical displacement of the node, from which the elements removed, reached a peak value of 5.74 cm and the response damped out to a steady value of 4.34 cm as shown in Figure 5.32. The force on column C-1increased by a factor of 2.8, from 2325 kN to 6529 kN. Yet, the maximum force was attained on column D-1 by 7536 kN and the axial force on column E-1reversed from compression to tension as in Figure 5.31. These forces are below the capacities of columns. As a result, the system was able to successfully bridge across removed elements within allowed deformation limits.

Table 5.78. Axial compression capacities per AISC 360-10 - SCBF columns.

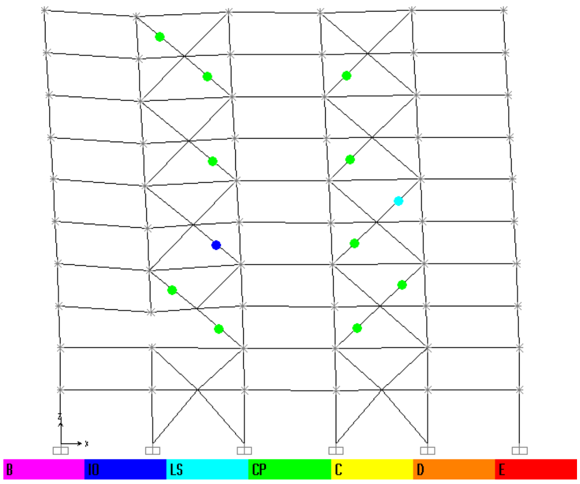
Column	Area (m^2)	r_y (m)	L_c (m)	k_y	λ_y	F_e (MPa)	F_{cr} (MPa)	P_{cr} (kN)
W14x43	8.13E-03	0.0481	4.2	1.0	87.3	258.89	197.51	1445.0
W14x53	1.01E-02	0.0488	4.2	1.0	86.1	266.48	200.67	1824.1
W14x74	1.41E-02	0.0630	4.2	1.0	66.7	444.13	249.24	3162.9
W14x82	1.55E-02	0.0629	4.2	1.0	66.8	442.72	248.98	3473.3
W14x99	1.88E-02	0.0944	4.2	1.0	44.5	997.18	298.49	5050.5
W14x120	2.28E-02	0.0951	4.2	1.0	44.2	1012.03	299.12	6138.0
W14x132	2.50E-02	0.0955	4.2	1.0	44.0	1020.56	299.48	6738.3
W14x145	2.75E-02	0.1011	5.33	1.0	52.7	710.20	281.52	6967.7
W14x176	3.34E-02	0.1022	4.2	1.0	41.1	1168.78	304.90	9165.4
W14x193	3.66E-02	0.1028	4.2	1.0	40.9	1182.54	305.34	10058.0
W14x233	4.42E-02	0.1041	5.33	1.0	51.2	752.97	284.80	11329.2



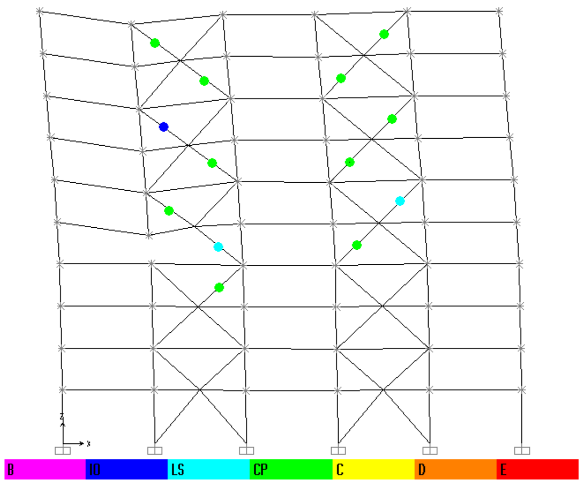
a) Vertical displacement of the node b) Columns axial forces

Figure 5.31. SCBF system response - APM Case 2.

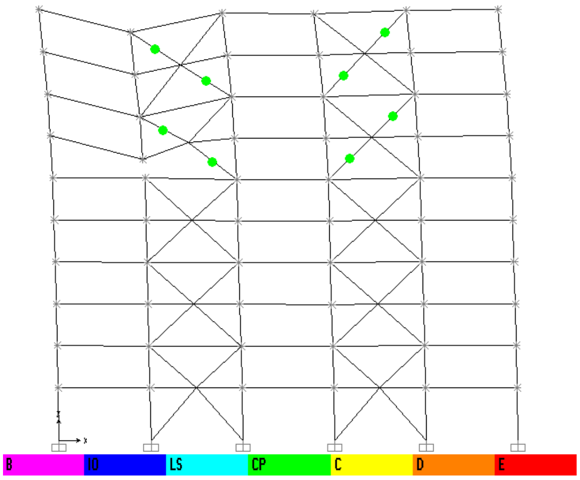
The response of the SCBF system to the removal of column B-3 and brace A-3 (APM Case 3), to the removal of column B-5 and brace A-5 (APM Case 4) and to the removal of column B-7 and brace B-7 (APM Case 5) is similar to the one of APM Case 2. In Case 3 twelve braces, in Case 4 thirteen braces and in Case 5 eight braces deformed inelastically within the limit of NAC as shown in Figure 5.32. Maximum displacement at the node, from which elements removed, is 7.11 cm for Case 3, 7.32 cm for Case 4 and 8.39 cm for Case 5. The maximum axial force attained by the columns of story, from which elements removed, is 5727 kN for Case 3, 3674 kN for Case 4 and 2087 kN for Case 5 on line D. These forces are well below the capacities of the associated columns.



a) APM Case 3



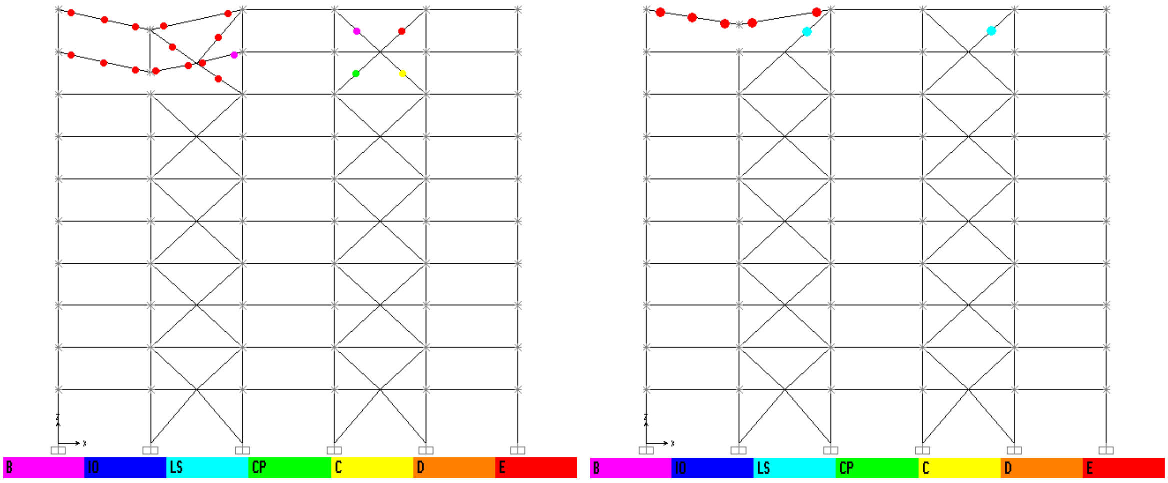
b) APM Case 4



c) APM Case 5

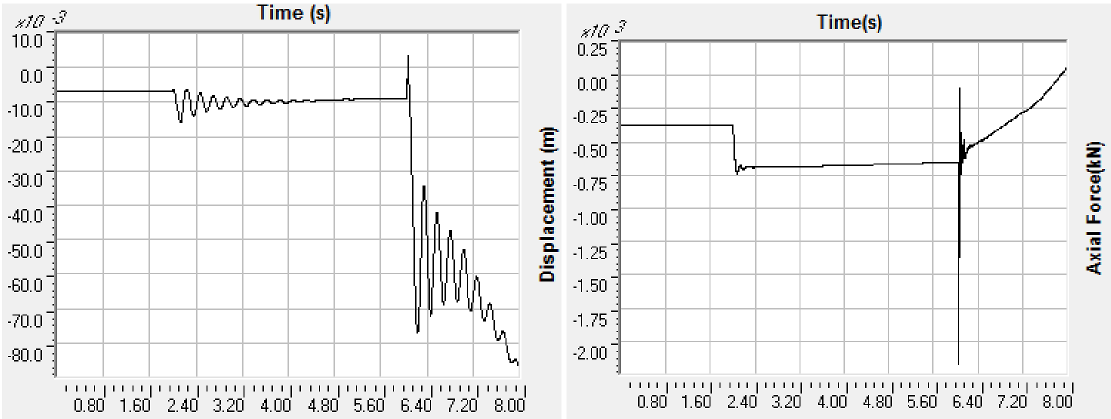
Figure 5.32. SCBF deformed shape of APM Case 3-4-5.

As shown in Figure 5.33, SCBF system was unable to bridge across removed elements; column B-9 and brace A-9 (APM Case 6). Immediate after removal, WUF-W connection on the roof floor together with brace B-9 and A-10 deformed inelastically. When fully restrained connections lost their force carrying capacities at around 6.1 second, shear connections of the bays above removal location started to behave inelastically. In one seconds the mechanism formed and displacement increased rapidly as shown in Figure 5.34. In the meantime, axial force on column C-9 spiked to from 655 kN to 2170 kN as shown in Figure 34, which was more than its capacity. Combined stress on Column D-9 was also more than its capacity as shown in Table 5.80. APM Case 7 caused a local mechanism as shown in Figure 5.33 immediately after element removal. Also, moment demand on column C-10 was 520 kN.m at 2.23 second, which was more than its plastic moment capacity .Together with an axial load of 180.3 kN, demand on this column was more than its capacity. Thus, simulation results showed that the upper two stories are highly vulnerable to progressive collapse.



a) Case 6 b) Case 7

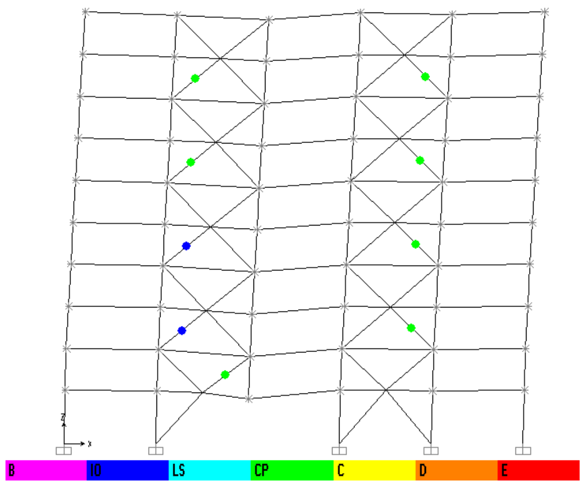
Figure 5.33. SCBF deformed shape of APM Case 6&7.



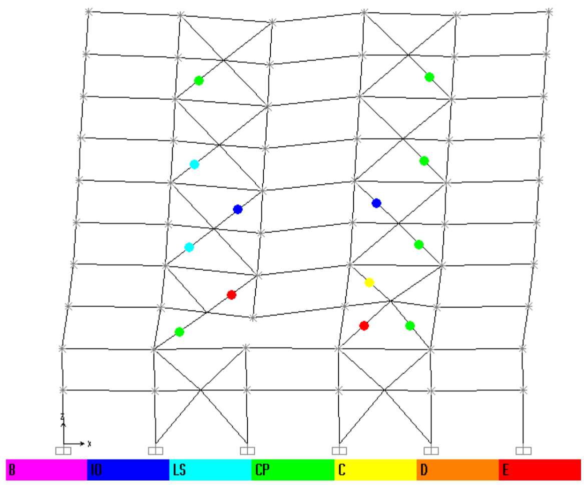
a) Vertical displacement of the node b) Column C-9 axial force

Figure 5.34. SCBF system response - APM Case 6.

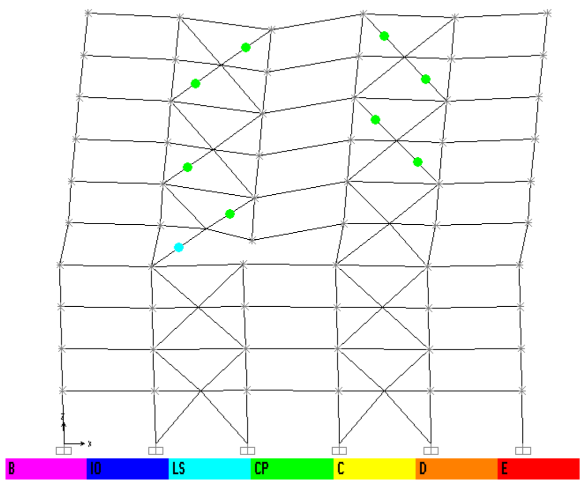
Response of SCBF to the removal of column C-1 and brace B-1 (APM Case 8) was similar to the response of the system to the Case 2. Until the Case 12, nonlinearity was limited to brace elements and the maximum axial force was attained by the columns on line B for cases 8 and 9 as shown in Figure 5.36. Axial force of columns on line D changed from compression to tension as shown in Figure 5.36. Nonlinear deformation of braces was within the allowable limit of UFC except two braces after APM Case 9 as shown in Figure 5.35. Colum axial forces and node displacements are as listed in Table 5.79.



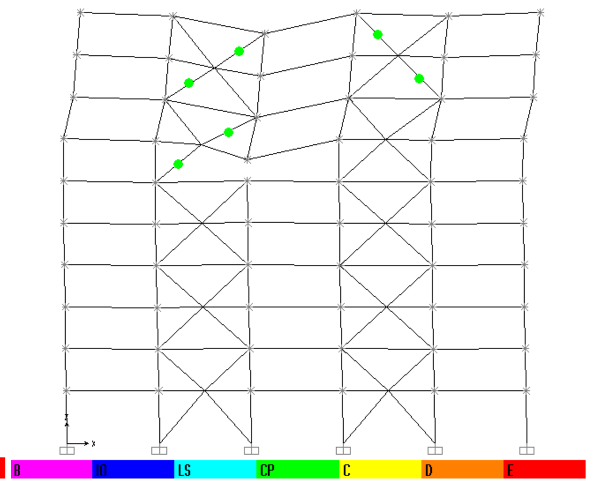
a) Case 8



b) Case 9



c) Case 10



d) Case 11

Figure 5.35. SCBF deformed shape of APM Case 8 to 11.

Immediate after removal of column C-9 and brace B-9 (APM Case 12), braces A-9 and B-10 lost their load carrying capacity. Meanwhile, beam hinge of W21x50 at floor 9, between B-C formed and this cantilever type mechanism increased node displacement excessively. At a node displacement of 57cm, WUF-W connections at 10th floor reached collapse level and together with weak shear connections this increased displacement more.

Thus, at 4 seconds, two bays above the removal location collapsed with a displacement of almost floor height as shown in Figure 5.37. APM Case 13 caused a local mechanism as shown in Figure 5.37 immediately after element removal at 2.385 s.

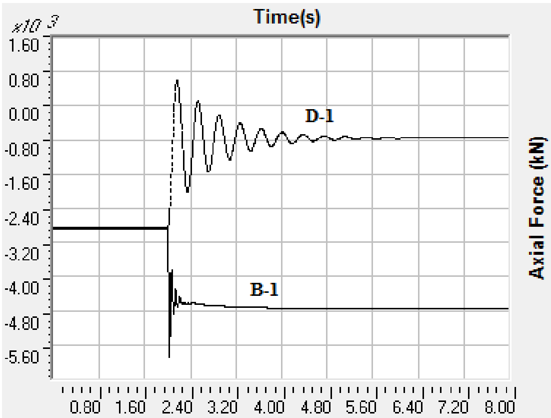
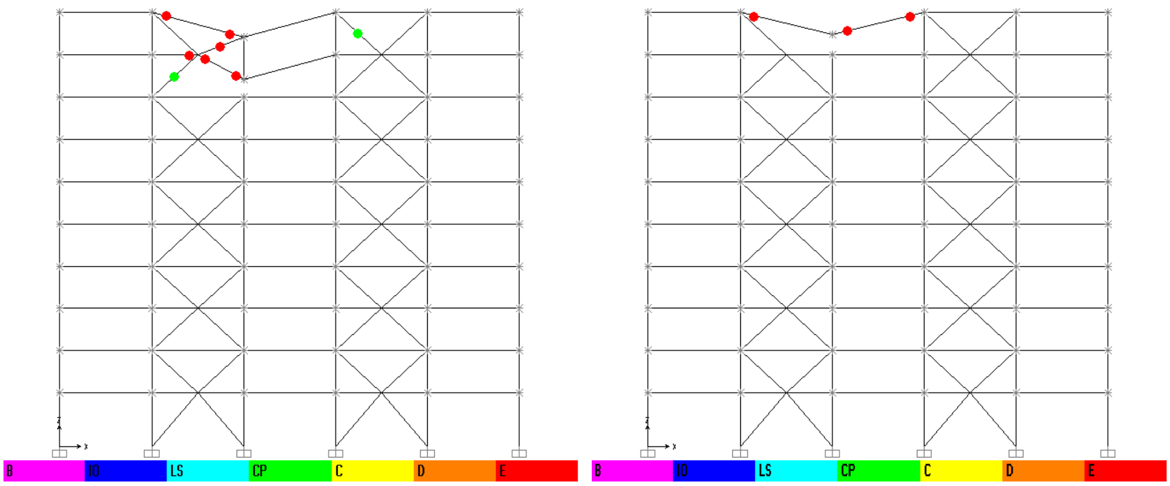


Figure 5.36. SCBF column axial forces - APM Case 8.



a) Case 12 b) Case 13

Figure 5.37. SCBF deformed shape of APM Case 12 & 13.

Table 5.79 summarizes NDA results for SCBF. In this table i refers to the number of story from which elements removed. Collapse is assumed when node displacement was almost equal to the floor height. Maximum axial loads and moments are the ones obtained at the same time. The worst combination with respect to combined stress was taken into consideration. Loads exerted on columns of exterior axes; line A and F, remained almost unchanged during dynamic analysis; as a result, stress check for these columns was omitted in Table 5.80.

Table 5.79. Summary of NDA results for SCBF.

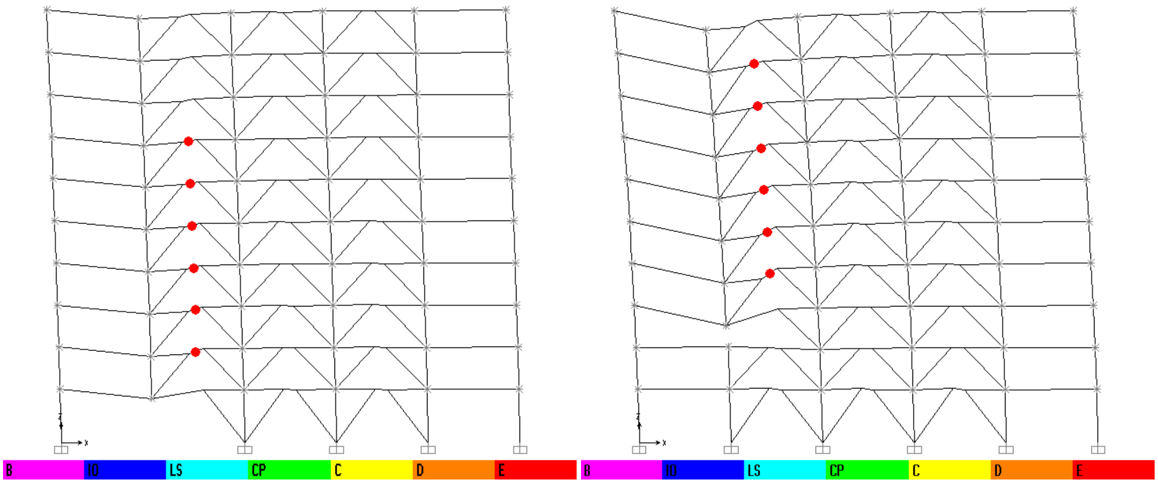
	Elements Removed			Column B-i		Column C-i		Column D-i		Column E-i	
Case	Column	Brace	(m)	(kN)	(kN.m)	(kN)	(kN.m)	(kN)	(kN.m)	(kN)	(kN.m)
SCBF			Node δ	P_{max}	M_{max}	P_{max}	M_{max}	P_{max}	M_{max}	P_{max}	M_{max}
1	A-1	-	collapse	3692	54.3	2359	23.4	3273	21.4	2266	27.0
2	B-1	A-1	0.0574	-	-	6529	33.1	7536	252.2	2226	59.2
3	B-3	A-3	0.0711	-	-	4995	29.6	5727	160.8	1800	2.8
4	B-5	A-5	0.0732	-	-	3237	38.4	3674	136.7	1288	6.0
5	B-7	A-7	0.0839	-	-	1816	13.2	2087	55.0	819	9.4
6	B-9	A-9	collapse	-	-	2170	43.0	989	136.6	258	142.5
7	B-10	A-10	collapse	-	-	180	520.0	168	27.8	151	25.8
8	C-1	B-1	0.0423	5895	40.0	-	-	2854	6.9	5757	113.7
9	C-3	B-3	0.0583	4746	10.3	-	-	2173	22.4	4625	83.2
10	C-5	B-5	0.0626	3114	20.1	-	-	1477	15.5	3126	77.0
11	C-7	B-7	0.0771	1780	2.8	-	-	888	16.4	1923	44.9
12	C-9	B-9	collapse	690	109.7	-	-	369	14.8	946	71.4
13	C-10	B-10	collapse	195	497.0	-	-	179	49.8	155	9.7

Table 5.80. Stress check for SCBF columns.

				Column B-i				Column C-i				Column D-i				Column E-i			
		(kN)	(kN.m)	(kN)	(kN.m)			(kN)	(kN.m)			(kN)	(kN.m)			(kN)	(kN.m)		
Case	Column	P_{cr}	M_p	P_d	M_d	Status		P_d	M_d	Status		P_d	M_d	Status		P_d	M_d	Status	
1	W14x233	11329.2	2218.5	3692	54.3	0.35	ok	2359	23.4	0.22	ok	3273	21.4	0.30	ok	2266	27.0	0.21	ok
2	W14x233	11329.2	2218.5	-	-	-	-	6529	33.1	0.59	ok	7536	252.2	0.78	ok	2226	59.2	0.22	ok
3	W14x193	10058.0	1806.2	-	-	-	-	4995	29.6	0.51	ok	5727	160.8	0.66	ok	1800	2.8	0.18	ok
4	W14x132	6738.3	1190.8	-	-	-	-	3237	38.4	0.51	ok	3674	136.7	0.66	ok	1288	6.0	0.20	ok
5	W14x82	3473.3	707.3	-	-	-	-	1816	13.2	0.54	ok	2087	55.0	0.68	ok	819	9.4	0.25	ok
6	W14x43	1445.0	354.3	-	-	-	-	2170	43.0	1.62	fail	989	136.6	1.07	fail	258	142.5	0.58	ok
7	W14x43	1445.0	354.3	-	-	-	-	180	520.0	1.59	fail	168	27.8	0.19	ok	151	25.8	0.18	ok
8	W14x233	11329.2	2218.5	5895	40.0	0.54	ok	-	-	-	-	2854	6.9	0.26	ok	5757	113.7	0.56	ok
9	W14x193	10058.0	1806.2	4746	10.3	0.48	ok	-	-	-	-	2173	22.4	0.23	ok	4625	83.2	0.51	ok
10	W14x132	6738.3	1190.8	3114	20.1	0.48	ok	-	-	-	-	1477	15.5	0.23	ok	3126	77.0	0.53	ok
11	W14x82	3473.3	707.3	1780	2.8	0.52	ok	-	-	-	-	888	16.4	0.28	ok	1923	44.9	0.62	ok
12	W14x43	1445.0	354.3	690	109.7	0.79	ok	-	-	-	-	369	14.8	0.30	ok	946	71.4	0.86	ok
13	W14x43	1445.0	354.3	195	497.0	1.54	fail	-	-	-	-	179	49.8	0.26	ok	155	9.7	0.13	ok

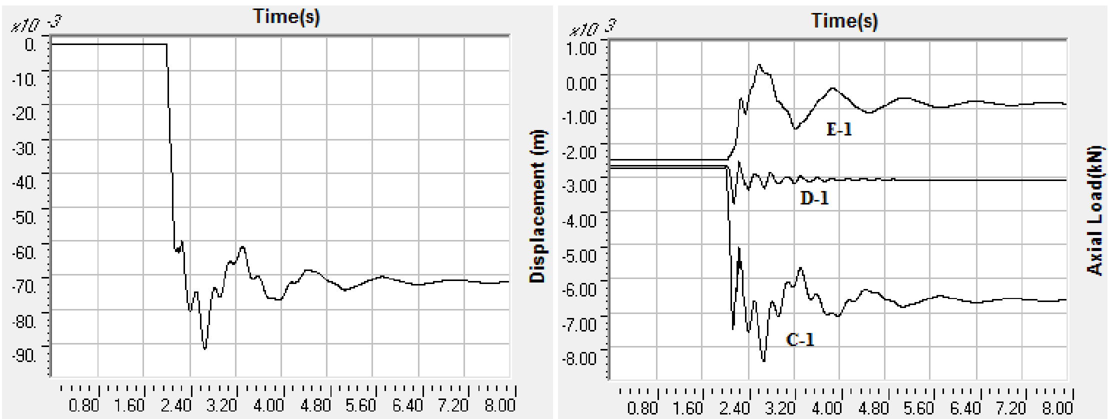
5.4.6.2. EBF. Shear links of EBF from story 2 to 6 were unable to carry high shear demand exerted on them after removal of column B-1 and brace A-1 (APM Case 1) as shown in Figure 5.38. In a few milliseconds after the removal, they deformed more than CP level and at a node displacement of 6.17 cm, beginning from 6^{th} story; shear links lost load carrying capacities progressively to the down stories. Dynamic response of the frame continued and the node displacement at removal location reached a peak value of 9.17 cm.

Finally, response damped out to a steady value of 7.22 cm as shown in Figure 5.39. Meanwhile, the axial force on column C-1increased by a factor of 3.1, from 2715kN to 8457 kN and with a moment of 23.4 kN.m stress demand on this column was more than its capacity. Also, the axial force on column E-1reversed from compression to tension as in Figure 5.39 and brace deformations were within the elastic limits. Axial capacities of columns of EBF are listed in Table 5.81 and plastic moment capacities are as listed in Table 5.83.



a) Case 1 b) Case 2

Figure 5.38. EBF deformed shape of APM Case 1 & 2.



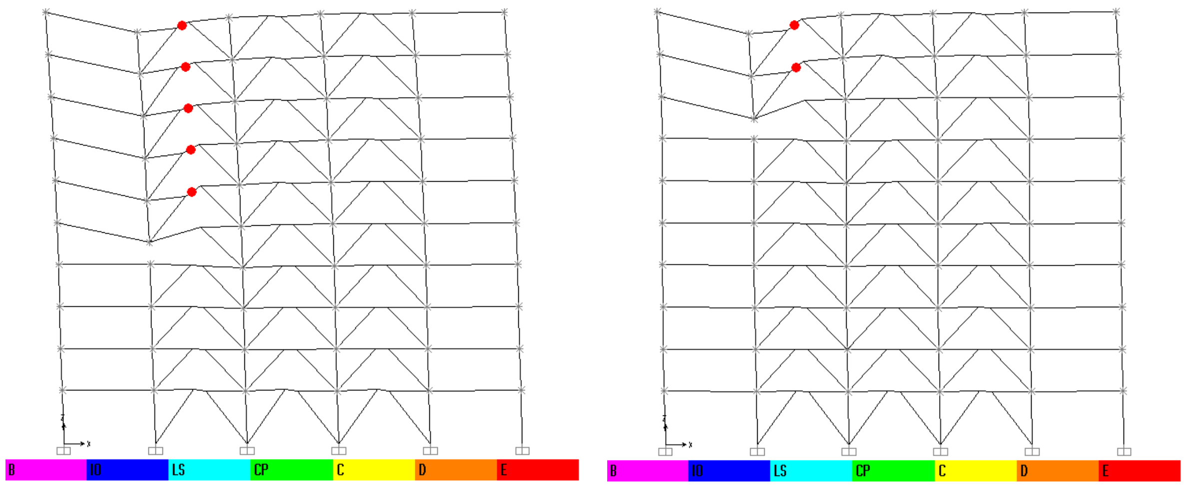
a) Vertical displacement of the node b) Columns axial forces

Figure 5.39. EBF system response - APM Case 1.

Table 5.81. Axial compression capacities per AISC 360-10 - EBF columns.

Column	Area (m^2)	r_y (m)	L_c (m)	k_y	λ_y	F_e (MPa)	F_{cr} (MPa)	P_{cr} (kN)
W14x48	9.10E-03	0.0485	4.2	1.0	86.6	263.22	199.33	1632.0
W14x53	1.01E-02	0.0488	4.2	1.0	86.1	266.48	200.67	1824.1
W14x61	1.15E-02	0.0621	4.2	1.0	67.6	431.53	246.89	2555.3
W14x82	1.55E-02	0.0629	4.2	1.0	66.8	442.72	248.98	3473.3
W14x99	1.88E-02	0.0944	4.2	1.0	44.5	997.18	298.49	5050.5
W14x109	2.06E-02	0.0949	4.2	1.0	44.3	1007.78	298.94	5542.4
W14x120	2.28E-02	0.0951	4.2	1.0	44.2	1012.03	299.12	6138.0
W14x132	2.50E-02	0.0955	4.2	1.0	44.0	1020.56	299.48	6738.3
W14x145	2.75E-02	0.1011	4.2	1.0	41.5	1143.76	304.08	7526.0
W14x176	3.34E-02	0.1022	5.33	1.0	52.2	725.73	282.75	8499.6

The response of the EBF system to the removal of column B-3 and brace A-3 (APM Case 2), to the removal of column B-5 and brace A-5 (APM Case 3) and to the removal of column B-8 and brace B-8 (APM Case 4) is similar to the one of APM Case 1. In Case 2 six, in Case 3 five and in Case 4 two shear links deformed inelastically beyond the limit of NAC as shown in Figure 5.38 and 5.40 respectively. Maximum displacement at the node, from which elements removed and the maximum axial force attained by the columns of story, from which elements removed are listed in Table 5.82. Combined stress on column C-3 was more than its capacity at Case 2. For Case 3&4, column forces are below the capacities of the associated columns.

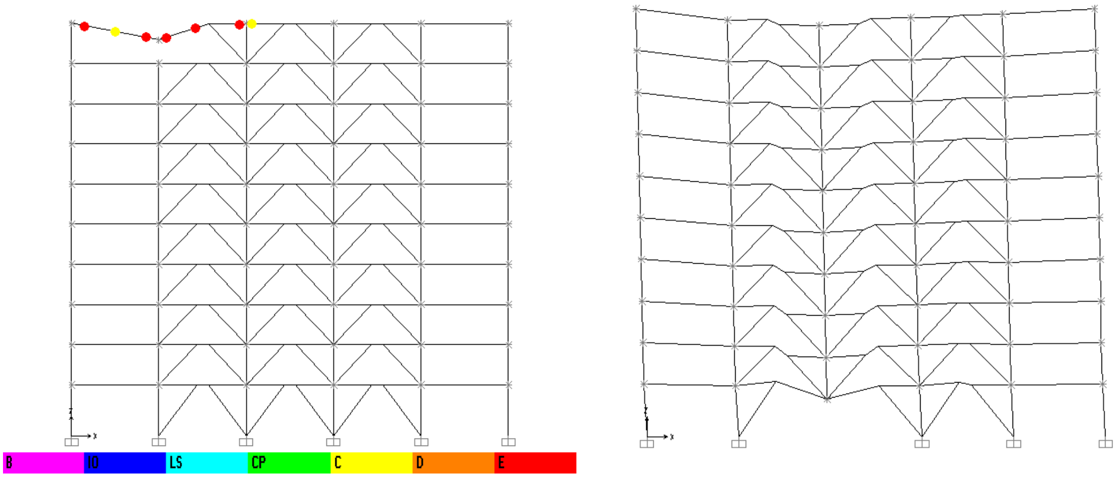


a) Case 3

b) Case 4

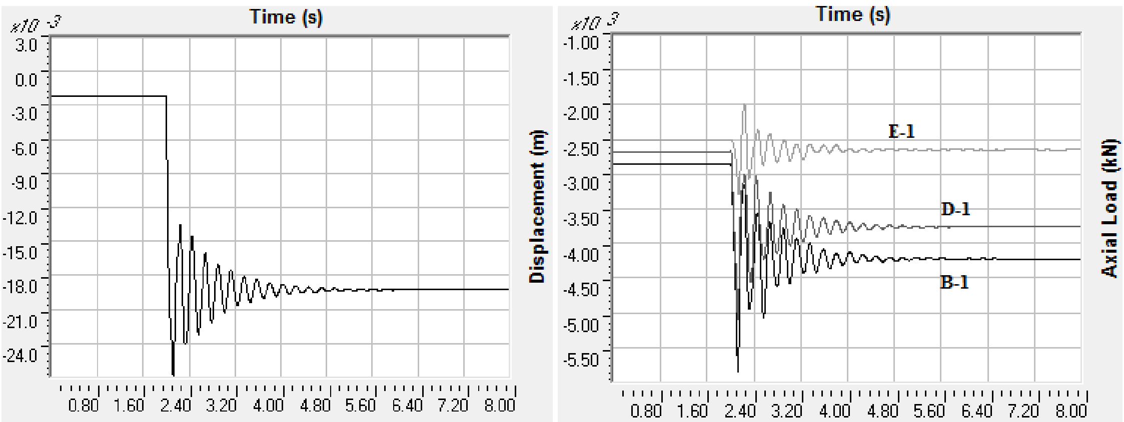
Figure 5.40. EBF deformed shape of APM Case 3 & 4.

As shown in Figure 5.41, removal of column B-10 with brace A-10 (APM Case 5) caused a local mechanism. Immediately after element removal, at time 2.145, mid-hinge of W10x39 at roof level formed. With the inclusion of shear and fix connection hinges, displacement at node increased and the bay above removal location collapsed. At the mean time, column forces remained within capacity limits.



a) Case 5 b) Case 6

Figure 5.41. EBF deformed shape of APM Case 5 & 6.

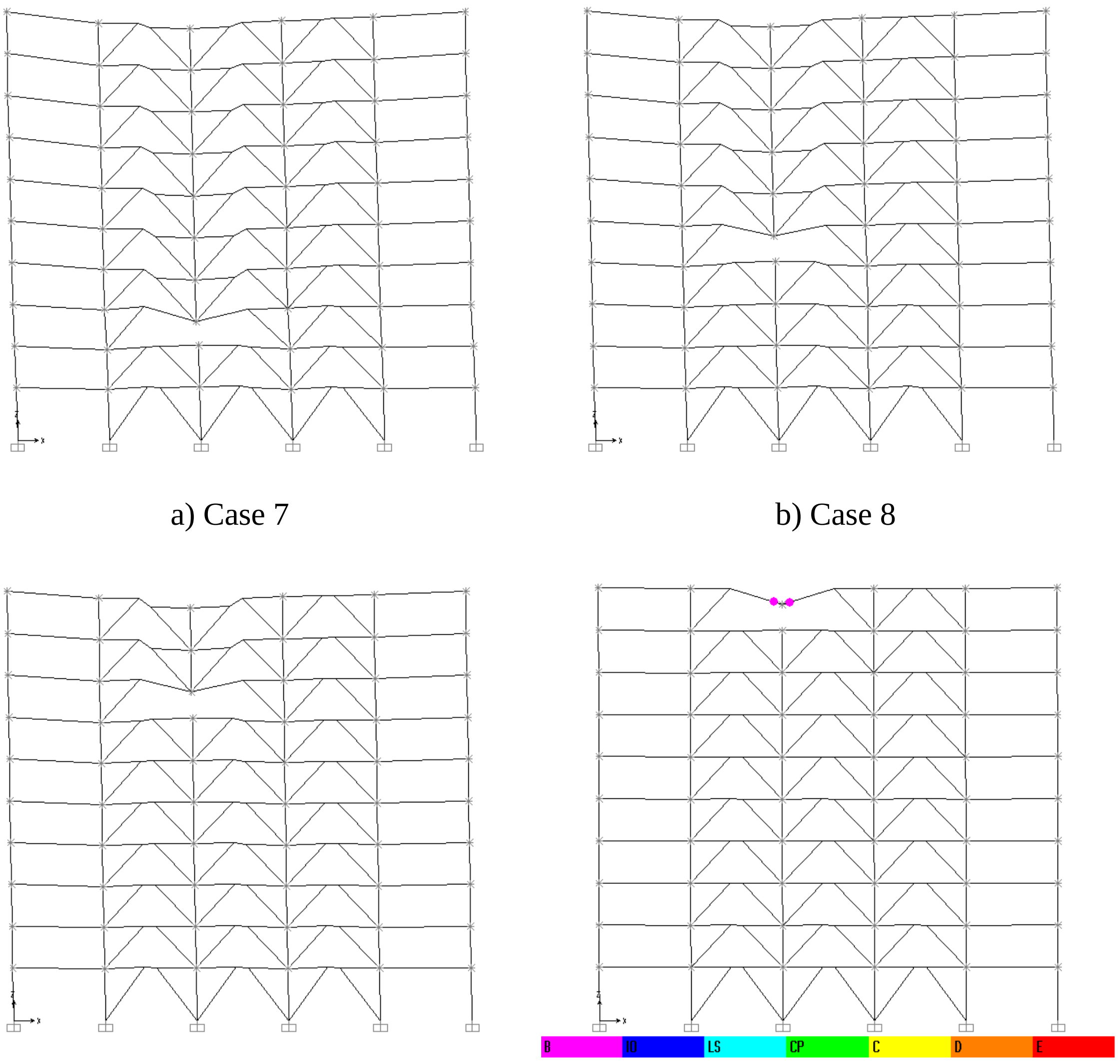


a) Vertical displacement of the node b) Columns axial forces

Figure 5.42. EBF system response - APM Case 6.

EBF was able to redistribute additional loads due to loss of column C-1 and braces B-1&C-1(APM Case 6) to the neighbor bays within elastic deformation limits as shown in Figure 5.41. This behavior of the frame was mainly related to the fact that removal location was near the symmetry axes. This made beams to share extra loads almost half-and-half to right and left columns as shown in Figure 5.42. Thus, load demand on columns remained under their capacities. This symmetric load redistribution might also be observed in node displacement (see Figure 5.42a).

Response of the EBF to the removal of column C-3 and braces B-3&C-3(APM Case 7), column C-5 and braces B-5&C-5(APM Case 8), and column C-8 and braces B-8&C-8(APM Case 9) was similar to the one of Case 6. All the deformations were in elastic limits as shown in Figure 5.43 respectively. Node displacement and axial force demands on column are listed in Table 5.82.



a) Case 7 b) Case 8

c) Case 9 d) Case 10

Figure 5.43. EBF deformed shape of APM Cases 7 to 10.

Immediately after removal of column C-10 and braces B-10&C-10 (APM Case 10) at 2.07s, WUF-W connections of beam W10x39 at roof level reached their plastic moment capacities as shown in Figure 5.43d. Nevertheless, the system damped out and no plastic rotation took place. This was attained mainly by catenary type deformation of beam. Beam W10x39 at roof level transferred tension forces to neighbor column and reduced their axial loads as sown in Figure 5.44.

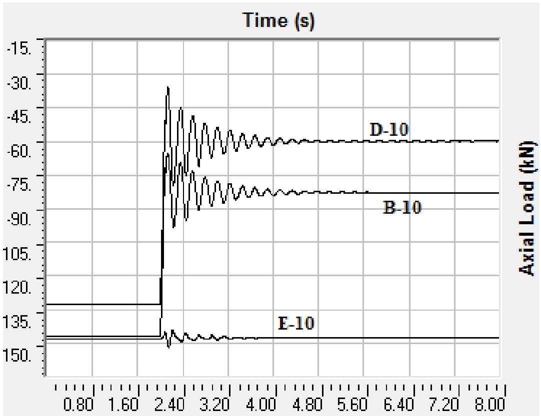


Figure 5.44. EBF column axial forces - APM Case 10.

Table 5.82 summarizes NDA results for EBF. In this table i refers to the number of story from which elements removed. Collapse is assumed when node displacement was almost equal to the floor height. Loads exerted on columns of exterior axes; line A and F, remained almost unchanged during dynamic analysis; as a result, stress check for these columns was omitted in Table 5.83.

Table 5.82. Summary of NDA results for EBF.

	Elements Removed			Column B-i		Column C-i		Column D-i		Column E-i	
Case	Column	Brace&Braces	(m)	(kN)	(kN.m)	(kN)	(kN.m)	(kN)	(kN.m)	(kN)	(kN.m)
EBF			Node δ	P_{max}	M_{max}	P_{max}	M_{max}	P_{max}	M_{max}	P_{max}	M_{max}
1	B-1	A-1	0.0917	-	-	8457	23.4	3774	116.7	2512	44.7
2	B-3	A-3	0.1025	-	-	6498	39.6	2937	41.1	1982	3.1
3	B-5	A-5	0.1048	-	-	4421	43.6	2028	47.7	1440	5.4
4	B-8	A-8	0.1182	-	-	1978	34.7	773	39.6	650	2.8
5	B-10	A-10	collapse	-	-	133	2.6	132	5.0	156	11.5
6	C-1	B-1 & C-1	0.0269	5850	47.9	-	-	5099	65.9	3281	59.8
7	C-3	B-3 & C-3	0.0323	4545	14.8	-	-	3919	16.6	2631	13.7
8	C-5	B-5 & C-5	0.0357	3175	9.7	-	-	2719	10.9	1872	9.9
9	C-8	B-8 & C-8	0.0408	1324	6.9	-	-	1187	7.8	775	3.6
10	C-10	B-10 & C-10	0.1728	65	78.0	-	-	35.6	62.3	152	5.7

Table 5.83. Stress check for EBF columns.

				Column B-i				Column C-i				Column D-i				Column E-i			
		(kN)	(kN.m)	(kN)	(kN.m)			(kN)	(kN.m)			(kN)	(kN.m)			(kN)	(kN.m)		
Case	Column	P_{cr}	M_p	P_d	M_d	Status		P_d	M_d	Status		P_d	M_d	Status		P_d	M_d	Status	
1	W14x176	8499.6	1628.3	-	-	-	-	8457	23.4	1.01	fail	3774	116.7	0.52	ok	2512	44.7	0.32	ok
2	W14x132	6738.3	1190.8	-	-	-	-	6498	39.6	1.00	fail	2937	41.1	0.47	ok	1982	3.1	0.30	ok
3	W14x109	5542.4	976.8	-	-	-	-	4421	43.6	0.84	ok	2028	47.7	0.41	ok	1440	5.4	0.27	ok
4	W14x61	2555.3	518.8	-	-	-	-	1978	34.7	0.84	ok	773	39.6	0.38	ok	650	2.8	0.26	ok
5	W14x48	1632.0	399.0	-	-	-	-	133	2.6	0.09	ok	132	5.0	0.09	ok	156	11.5	0.12	ok
6	W14x176	8499.6	1628.3	5850	47.9	0.72	ok	-	-	-	-	5099	65.9	0.64	ok	3281	59.8	0.42	ok
7	W14x132	6738.3	1190.8	4545	14.8	0.69	ok	-	-	-	-	3919	16.6	0.60	ok	2631	13.7	0.40	ok
8	W14x109	5542.4	976.8	3175	9.7	0.58	ok	-	-	-	-	2719	10.9	0.50	ok	1872	9.9	0.35	ok
9	W14x61	2555.3	518.8	1324	6.9	0.53	ok	-	-	-	-	1187	7.8	0.48	ok	775	3.6	0.31	ok
10	W14x48	1632.0	399.0	65	78.0	0.24	ok	-	-	-	-	35.6	62.3	0.18	ok	152	5.7	0.11	ok

5.4.7. Discussion of Results

In the previous sections, two well known braced steel frame systems, namely; SCBF and EBF, were modeled and analyzed obeying the requirements of UFC 4-023-03 (DoD, 2009). The following observations and discussions might be arisen from the results of NDA.

As it was observed by previous researchers (e.g. J. Kim and T. Kim, 2009; Khandelwal *et al.*, 2009), potential for progressive collapse is high when a corner column of a steel frame is lost and this is the fact in APM Case 1 for SCBF. Although shear tab connections have great ductility, they do not posses enough strength, which is necessary to transfer additional loads to neighbor bays. In other words, they do not mitigate progressive collapse when a first story corner column is lost due to some abnormal event. Therefore, their usage in perimeter frames, i.e. in perimeter gravity beams, need to be restrained when progressive collapse is of concern.

Ineffectiveness of shear connections to transfer additional loads might also be observed in distribution of axial forces after a removal scenario. If NDA results of APM cases from 8 to 12 is observed in Table 5.79, it might be traced that additional loads is mainly carried by columns on line B of SCBF. However, for the removal cases from the same line, distribution of extra loads is almost symmetric for the columns of EBF on line B and D. This is mainly due to load redistribution capacity of WUF-W connections or considering SCBF, due to ineffectiveness of shear tab connections.

The NDA results demonstrated that SCBF was able to bridge across element losses for APM Cases 2 to 5 and 8 to 11 within allowable deformation limits and column capacities. The only exception within these cases was Case 9 in which two braces lost totally load carrying capacity due to excessive deformation. Nevertheless, not being gravity loads carrying elements; their collapse did not lead the system to progressive collapse. In the lower stories, massive columns designed to carry earthquake loads have enough capacity to mitigate collapse.

However, SCBF had high progressive collapse potential after the removal of upper story column and braces. Especially, APM Case 6 caused two columns to be loaded more than their capacities, and a possible debris impact due to the loss of these columns might result in additional collapse down to the lower stories. This threat with a reduced amount was also valid for Cases 7 and 13. This vulnerability of the frame to progressive collapse is mainly due to high structural optimization and reducing sectional dimensions excessively with height. ASCE 07 also point to this factor that contributes to the risk of damage propagation in modern structures in Section 3.3. Moreover, this vulnerability of the frame proves the importance of the APM case requirement of UFC. Removal scenarios based on only first story columns, which is accepted widely by open literature, may disregard a possible progressive collapse potential at the upper stories.

Progressive collapse potential of EBF was high when the first three story columns and braces were removed. For APM Case 1 and 2, demand on columns C-1 and C-3 was higher than their capacities. Collapse of these columns might progressively damage the entire frame to the down. Considering 3D structure and effect of slab, a Murrah federal building type collapse is possible by propagation of failure in to the building, especially in to the gravity frames. However, for other removal scenarios EBF was able to successfully absorb element losses within the allowable limits. One exception was Case 5 which resulted in a local collapse.

NDA results revealed also the dynamic nature of progressive collapse. Column forces near the removal locations increased up to 3.1 times of the static values. To illustrate, in APM Case 1 of EBF, axial force of column C-1 spiked from 2715 kN to 8457 kN in almost 0.65 s before settling down at a static value of 6630 kN. This dynamic response of the frame complies with the statements in Section 2.4.

It is also observed from analysis results that node displacements and number of nonlinearly deformed elements increased when structural elements were removed from higher story levels. As the removal location was from upper stories, proportionally more braces from remained ones buckled, and similarly more shear links failed. To illustrate, in APM Case 2 of SCBF, 13 braces from 39 remained upper story braces buckled, i.e. 1/3 of the remained ones. This ratio was more than 1/2 for the APM Case 3. The reason behind this response was the fact that less structural elements involved in load redistribution with the increase in removal height. Thus, it was observed that progressive collapse potential increases with the height of removal location considering these two frames only.

The outcome of this work does not comply with the perception that the seismic design ensures progressive collapse resistance of structures. Both SCBF designed for moderate seismic risk and EBF designed for high seismic risk possess progressive collapse potential at different element removal location. Nonetheless, it might be noted that high seismic loads resulted in more braces and less partial connections at the outer frames of EBF system, which in return improved load redistribution within the frame. However, if progressive collapse is a concern of structural design, it should be evaluated separately once again not relying completely on seismic design.

6. CASE STUDY 2: INVESTIGATION OF DYNAMIC INCREASE FACTOR (DIF) FOR ANALYSED STEEL BRACED FRAMES

6.1. Introduction

The objective of this case study is to investigate the accuracy of DIF (Dynamic Increase Factor) proposed by UFC 4-023-03 (DoD, 2009). UFC requires DIF as explained in Section 3.5.3.3 to increase applied loads on the bays above the removal location during a nonlinear static analysis. Although NSA involves nonlinear behavior through hinge definitions along structural elements, it is not able to represent dynamic response after an element removal case. Therefore, this factor is used to account for dynamic character of progressive collapse.

At this point NDA comes into prominence by reflecting most effectively dynamic and nonlinear characteristics of progressive collapse behavior of a structural system. However, this analysis type has also disadvantages like high time requirement for computation and verification compared to static analysis types. As a result, NSA is also widely preferred in engineering community to analyze response of a system after element losses. Yet, accuracy of these analyses mainly depends on the accuracy of DIF used.

The Eqn.(3.22) proposed by UFC to calculate DIF was developed originally by Marchand *et al.* (2009) and during this study researchers have considered only steel moment frames. Therefore, this case study will also investigate the compliment of this equation by braced frames.

6.2. Methodology

The accuracy of DIF required for NSA of SCBF and EBF were investigated using the methodology explained below.

First, 2D nonlinear structural model of each braced frame type was developed as it was explained in Section 5.4.2. . Both material and geometric nonlinearity was considered again. Then the load combination in Equation 3.21 was applied to the bays above the removed column and braces. However, here a trial DIF was utilized in order to increase loads. The remaining parts of the frame were loaded with the load combination in Equation 3.15 as depicted in Figure 6.1.

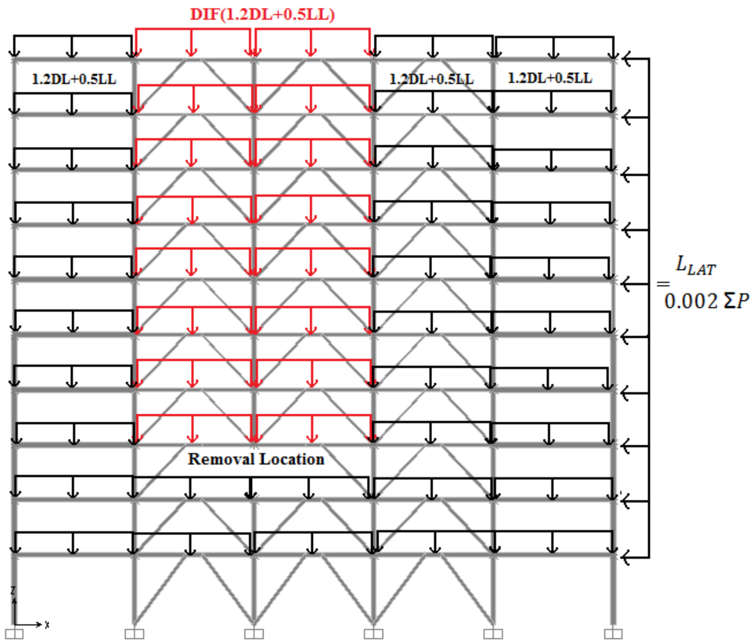


Figure 6.1. Nonlinear static procedure loading.

Secondly, NSA, details of which are explained in Section 6.4., was carried out using the structural analysis program SAP2000 Advanced 14.2.2. The node displacement at removal location was recorded and if it did not match with the one obtained from NDA of the associated APM Case, DIF was adjusted and the analysis was repeated. When the maximum node displacements measured according to NSP and NDP matched within few percent differences, analysis was stopped and the final value of DIF was recorded as required DIF. Meanwhile, axial forces of critical columns, which were also loaded at maximum during NDA, were recorded for further discussions.

Finally, DIF was calculated per UFC 4-023-03 and it was compared with the required DIF obtained by equating results of NSA with NDA. This procedure was applied to the APM cases of SCBF and EBF separately, one by one. However, only the APM cases, which were not resulted in collapse as defined in Tables 5.79 and 5.82, were considered. This was because of the fact that it was not feasible to carry out NSA with SAP2000 for highly nonlinear APM cases.

6.3. Loading Procedure.

The loading procedure introduced in Section 5.4.3 was modified slightly in order to adapt it to the requirements of NSA procedure. Line loads remained the same but point loads on the border of bays above removal locations have been divided into two as shown in Figure 6.2. In this figure P_N represents point loads transferred from the bays above removal location and P_S is the one transferred from undamaged bays.

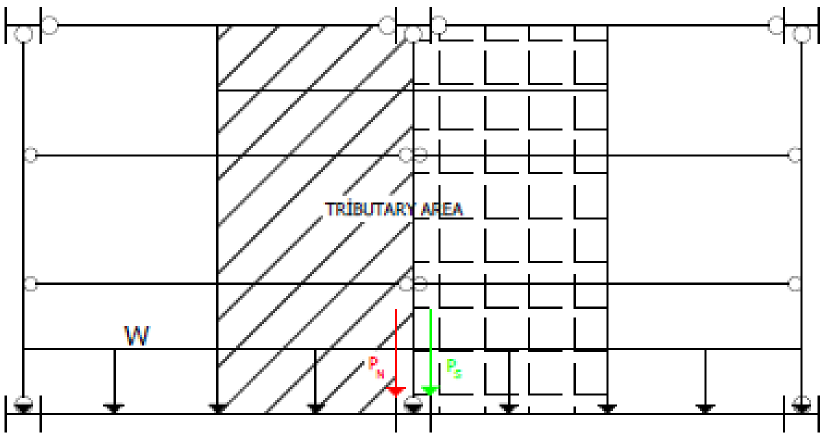


Figure 6.2. Transfer of gravity loads to the braced frames.

To illustrate, during APM Case 9 of SCBF and Cases 7 of EBF, for normal floors 1 and 2, same loading described in Section 5.4.3 was applied.

For the upper remaining normal floors, directly above removal location (See Figure 6.1): $= DIFx(5.54\ \frac{kN}{m})$ & $q = DIFx(7.28\ \frac{kN}{m})$. For the roof floor, directly above removal location: $g = DIFx(4.07\ \frac{kN}{m})$ & $q = DIFx(1.46\ \frac{kN}{m})$. Remaining undamaged bays were loaded with unfactored line loads.

To the columns, directly above removal location, on line B&D: $P_{Ng} = DIFx50.65\ kN$, $P_{Nq} = DIFx66.67\ kN$. On line C: $P_{Ng} = DIFx101.31\ kN$, $P_{Nq} = DIFx133.33\ kN$.For roof story, on line B&D: $P_{Ng} = DIFx37.33kN$, $P_{Nq} = DIFx13.34\ kN$. On line C: $P_{Ng} = DIFx74.66\ kN$, $P_{Nq} = DIFx26.68\ kN$.

To the columns of remaining undamaged bays above removal location, on line A,B,D,F: $P_{Sg} = 50.65\ kN$, $P_{Sq} = 66.67\ kN$. On line E: $P_{Sg} = 101.31\ kN$, $P_{Sq} = 133.33\ kN$.For roof story, on line A, B, D, F: $P_{Sg} = 37.33kN$, $P_{Sq} = 13.34\ kN$. On line E: $P_{Sg} = 74.66\ kN$, $P_{Sq} = 26.68\ kN$.

The loading procedure described above was applied to the each frame of considered APM case with required modifications. Live load reduction was applied per TS498 as shown in Table 5.6. UFC requires also that the lateral load given in Equation 3.16 is applied to the sides of the structure one at a time. Due to the APM cases considered in previous case study, lateral loads in Table 5.75 was applied to the frames only in negative x-direction as shown in Figure 6.1.

6.4. Details of Nonlinear Static Analysis

Progressive collapse analysis procedure based on NSA has been introduced in Section 6.2. In this procedure, first structural elements, which were required to be removed according to APM case considered, were assigned to a group called “Case”. Then, the “Nonlinear Staged Construction” option in SAP2000 was utilized in order to automate removal of elements. In the first stage, all structural elements were added to the analysis and in the second stage required elements were removed as shown in Figure 6.3.

The results of the nonlinear staged construction were used then as the initial condition of nonlinear static analysis case definition as shown in Figure 6.4. Thus, elements were removed automatically from the analytical modal before loading the framing system. In the nonlinear static case geometric nonlinearity was considered by selecting P-Delta plus large displacement options.

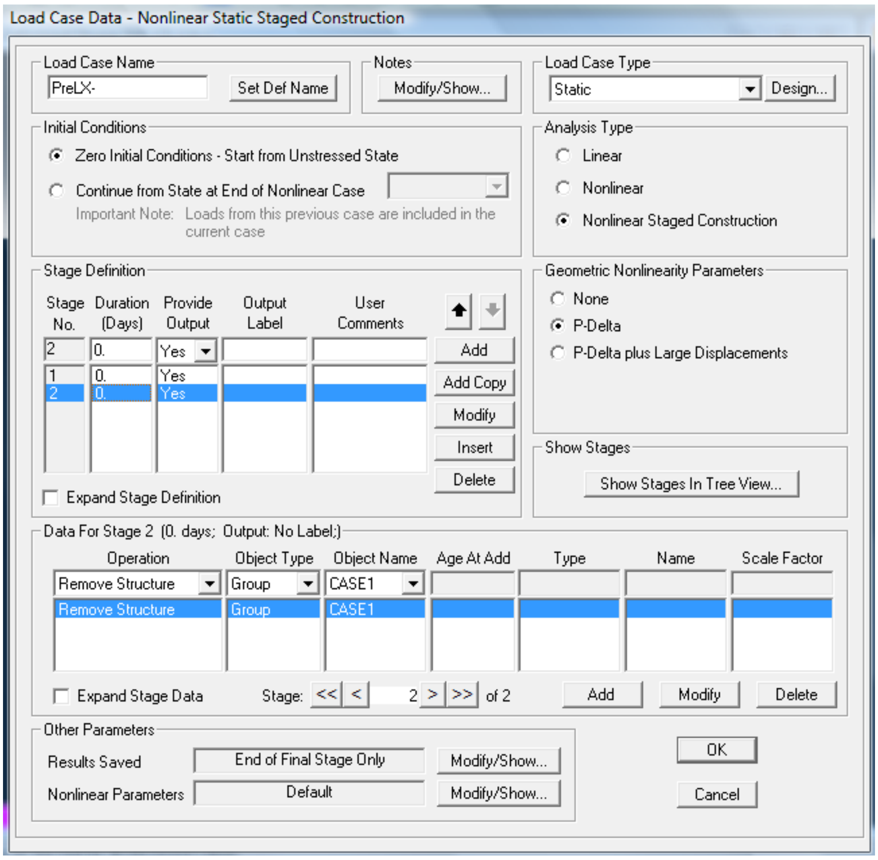


Figure 6.3. Nonlinear staged construction definition.

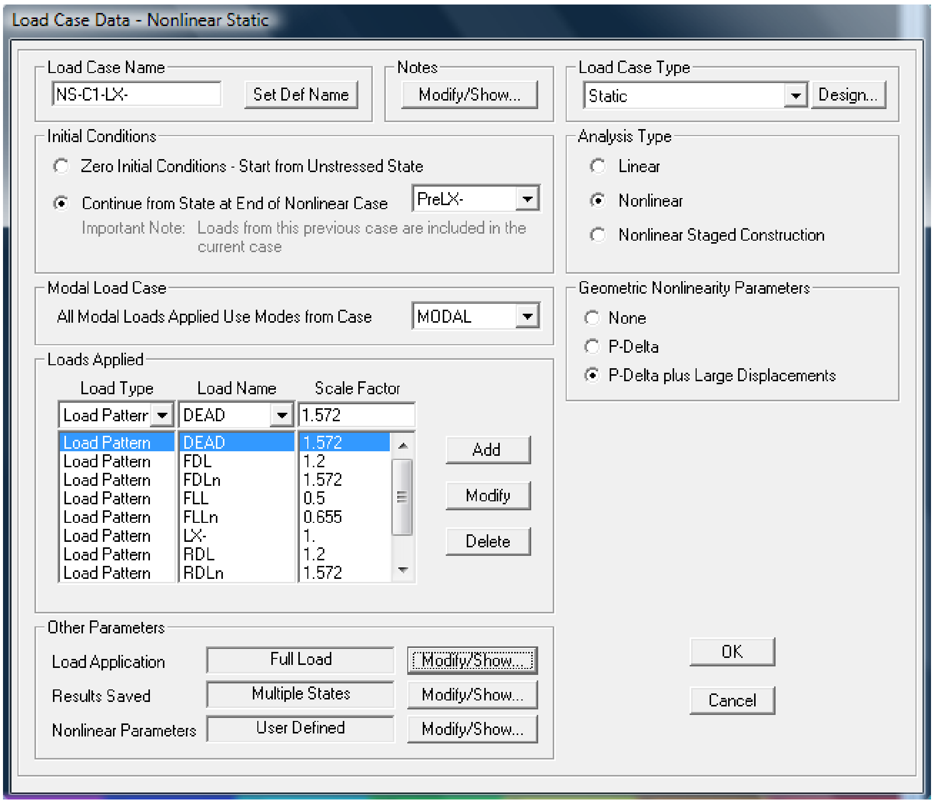


Figure 6.4. Nonlinear static analysis case definition.

Loading procedure defined in previous section was applied by assigning scale factor to each load type separately. Scale factors were adjusted by trial and error in order to determine required DIF. This nonlinear static analysis was load controlled and minimum 10 and maximum 100 steps were defined as UFC required. As the hinge unloading method local redistribution was used. In this method, when a hinge is on a negative sloped portion of the stress-strain curve and the applied load causes the strain to reverse, the program unload only the element under this behavior and transfer the removed load to the neighbor elements(CSI, 2010). This situation was more suitable to the localized damage and load transfer mechanism of progressive collapse situation. Also, if needed during a nonlinear time history analysis, SAP2000 will use the same method (CSI, 2010).

6.5. Results of Nonlinear Static Analysis

The required DIF for each framed type was calculated using the above explained procedures. As a result, for each APM Case considered a unique DIF was obtained, which equated the node displacement to the one obtained from NDA. The results were presented for SCBF and EBF separately.

6.5.1. DIF for SCBF

The required DIF for the cases of SCBF are listed in Table 6.1 with the maximum axial loads on critical columns, which have high force demand according to NDA. The values of moments demanded from the columns at the maximum axial load are also listed. The results of combined stress check on critical columns are listed in Table 6.2.

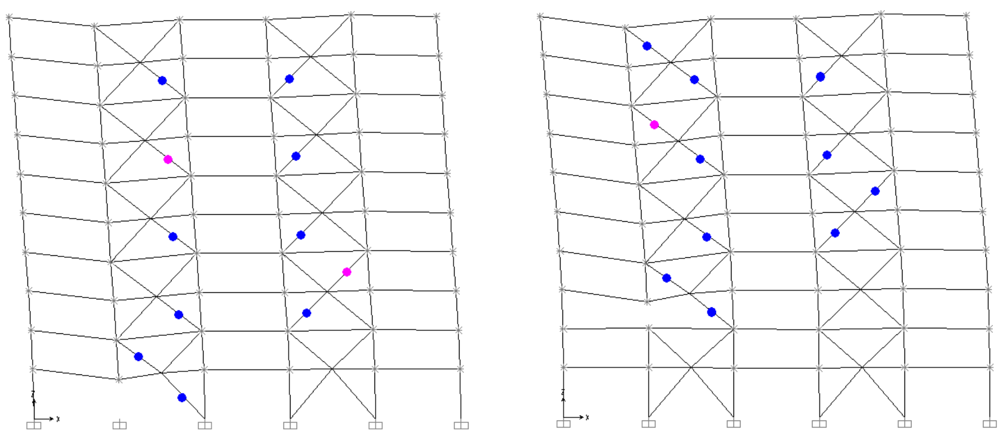
The deformed shapes of considered APM cases of SCBF are as given in Figure 6.5. Although the node displacements match with the results of NDA, nonlinear deformations at structural elements differ with the previous results.

Table 6.1. Required DIFs for APM cases of SCBF.

	Elements Removed				Column C-i		Column D-i	
Case	Column	Brace		(m)	(kN)	(kN.m)	(kN)	(kN.m)
SCBF			DIF	Node δ	P_{max}	M_{max}	P_{max}	M_{max}
1	A-1	-	-	collapse	-	-	-	-
2	B-1	A-1	1.362	0.0574	6240	101.1	6615	93.0
3	B-3	A-3	1.360	0.0713	4867	87.6	5043	120.0
4	B-5	A-5	1.265	0.0733	3100	70.9	3248	91.3
5	B-7	A-7	1.180	0.0840	1819	39.0	1881	56.0
6	B-9	A-9	-	collapse	-	-	-	-
7	B-10	A-10	-	collapse	-	-	-	-
SCBF			DIF	Node δ	Column B-i		Column E-i	
8	C-1	B-1	1.310	0.0423	6011	52.2	5223	95.6
9	C-3	B-3	1.338	0.0583	4790	48.1	4266	86.8
10	C-5	B-5	1.255	0.0626	3074	48.2	2869	73.8
11	C-7	B-7	1.208	0.0771	1857	23.4	1775	42.6
12	C-9	B-9	-	collapse	-	-	-	-
13	C-10	B-10	-	collapse	-	-	-	-

Table 6.2. Stress check for SCBF columns – NSA.

				Column C-i				Column D-i			
		(kN)	(kN.m)	(kN)	(kN.m)			(kN)	(kN.m)		
Case	Column	P_{cr}	M_p	P_d	M_d	Status		P_d	M_d	Status	
2	W14x233	11329.2	2218.5	6240	101.1	0.60	ok	6615	93.0	0.63	ok
3	W14x193	10058.0	1806.2	4867	87.6	0.53	ok	5043	120.0	0.57	ok
4	W14x132	6738.3	1190.8	3100	70.9	0.52	ok	3248	91.3	0.56	ok
5	W14x82	3473.3	707.3	1819	39.0	0.58	ok	1881	56.0	0.62	ok
				Column B-i				Column E-i			
8	W14x233	11329.2	2218.5	6011	52.2	0.55	ok	5223	95.6	0.50	ok
9	W14x193	10058.0	1806.2	4790	48.1	0.50	ok	4266	86.8	0.47	ok
10	W14x132	6738.3	1190.8	3074	48.2	0.50	ok	2869	73.8	0.49	ok
11	W14x82	3473.3	707.3	1857	23.4	0.57	ok	1775	42.6	0.57	ok



a) Case 2 b) Case3

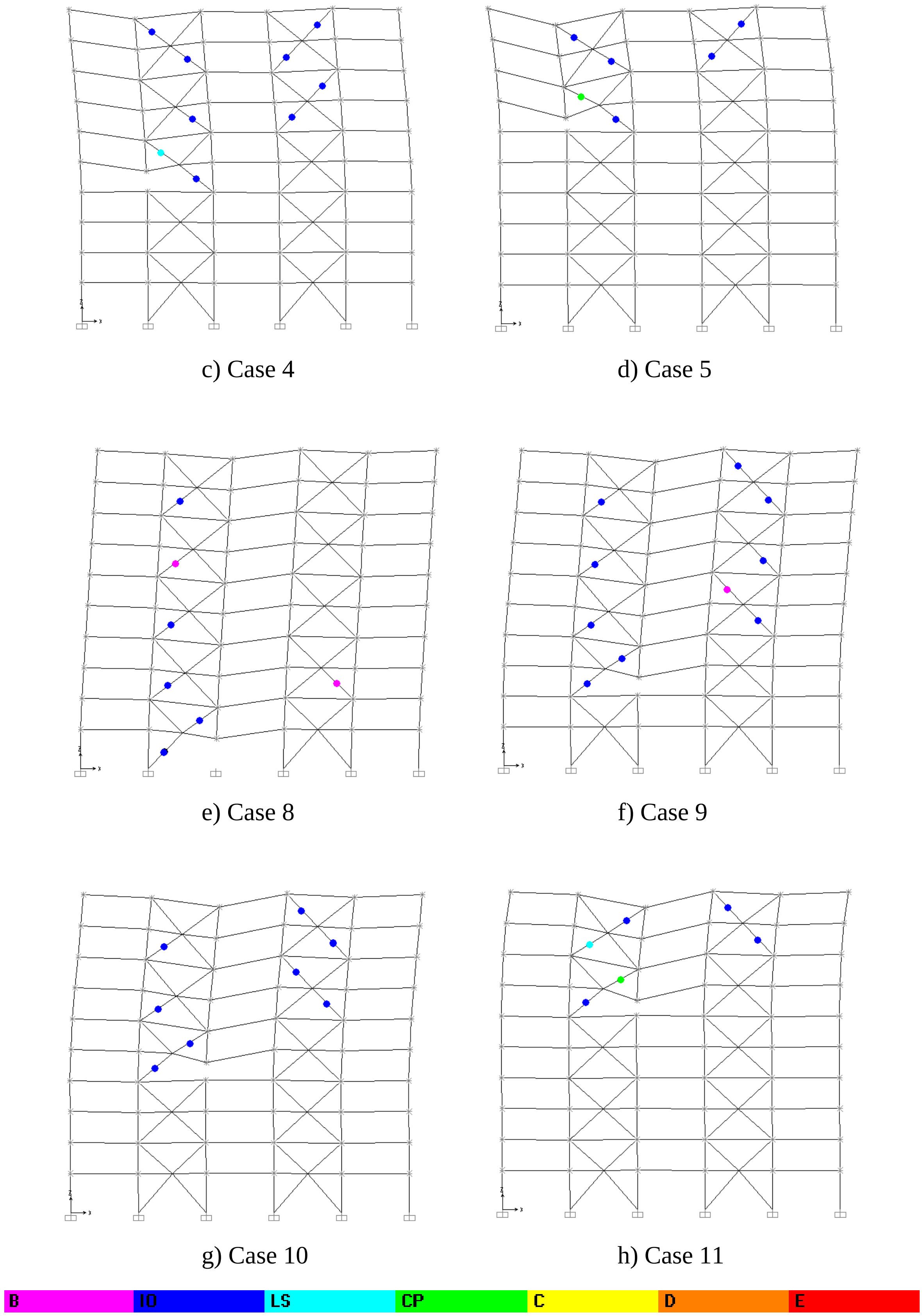


Figure 6.5. SCBF deformed shape of APM cases considered for NSA.

6.5.2. DIF for EBF

The required DIF for the cases of EBF are listed in Table 6.3 with the maximum axial loads on critical columns, which have high force demand according to NDA. The values of moments demanded from the columns at the maximum axial load are also listed. The results of combined stress check on critical columns are listed in Table 6.4.

The deformed shapes of considered APM cases of EBF are as given in Figure 6.6. Since the deformed shapes of EBF for APM cases 6 to 10 were the same with previous NDA results, deformed shapes of the remaining cases were provided. Although the node displacements match with the results of NDA, nonlinear deformations at structural elements differ with the previous results.

Table 6.3. Required DIFs for APM cases of EBF.

	Elements Removed				Column B-i		Column C-i		Column D-i	
Case	Column	Brace&Bra		(m)	(kN)	(kN.m)	(kN)	(kN.m)	(kN)	(kN.m)
EBF			DIF	Node δ	P_{max}	M_{max}	P_{max}	M_{max}	P_{max}	M_{max}
1	B-1	A-1	1.310	0.0919	-	-	8078	81.0	3337	58.7
2	B-3	A-3	1.320	0.1035	-	-	6229	40.6	2627	47.2
3	B-5	A-5	1.310	0.0953	-	-	4344	34.9	1809	46.0
4	B-8	A-8	1.200	0.0814	-	-	1755	13.5	712	18.3
5	B-10	A-10	-	collapse	-	-	-	-	-	-
6	C-1	B-1 & C-1	1.480	0.0269	5620	65.6	-	-	4872	79.1
7	C-3	B-3 & C-3	1.510	0.0323	4406	23.7	-	-	3752	18.3
8	C-5	B-5 & C-5	1.500	0.0358	3076	20.0	-	-	2648	12.5
9	C-8	B-8 & C-8	1.590	0.0409	1356	12.3	-	-	1222	5.6
10	C-10	B-10 & C-1	1.660	0.1741	105	68.9	-	-	81	57.2

Table 6.4. Stress check for EBF columns – NSA.

				Column B-i				Column C-i				Column D-i			
		(kN)	(kN.m)	(kN)	(kN.m)			(kN)	(kN.m)			(kN)	(kN.m)		
Case	Column	P_{cr}	M_p	P_d	M_d	Status		P_d	M_d	Status		P_d	M_d	Status	
1	W14x176	8499.6	1628.3	-	-	-	-	8078	81.0	1.00	fail	3337	58.7	0.43	ok
2	W14x132	6738.3	1190.8	-	-	-	-	6229	40.6	0.96	ok	2627	47.2	0.43	ok
3	W14x109	5542.4	976.8	-	-	-	-	4344	34.9	0.82	ok	1809	46.0	0.37	ok
4	W14x61	2555.3	518.8	-	-	-	-	1755	13.5	0.71	ok	712	18.3	0.31	ok
6	W14x176	8499.6	1628.3	5620	65.6	0.70	ok	-	-	-	-	4872	79.1	0.62	ok
7	W14x132	6738.3	1190.8	4406	23.7	0.67	ok	-	-	-	-	3752	18.3	0.57	ok
8	W14x109	5542.4	976.8	3076	20.0	0.58	ok	-	-	-	-	2648	12.5	0.49	ok
9	W14x61	2555.3	518.8	1356	12.3	0.55	ok	-	-	-	-	1222	5.6	0.49	ok
10	W14x48	1632.0	399.0	105	68.9	0.24	ok	-	-	-	-	81	57.2	0.19	ok

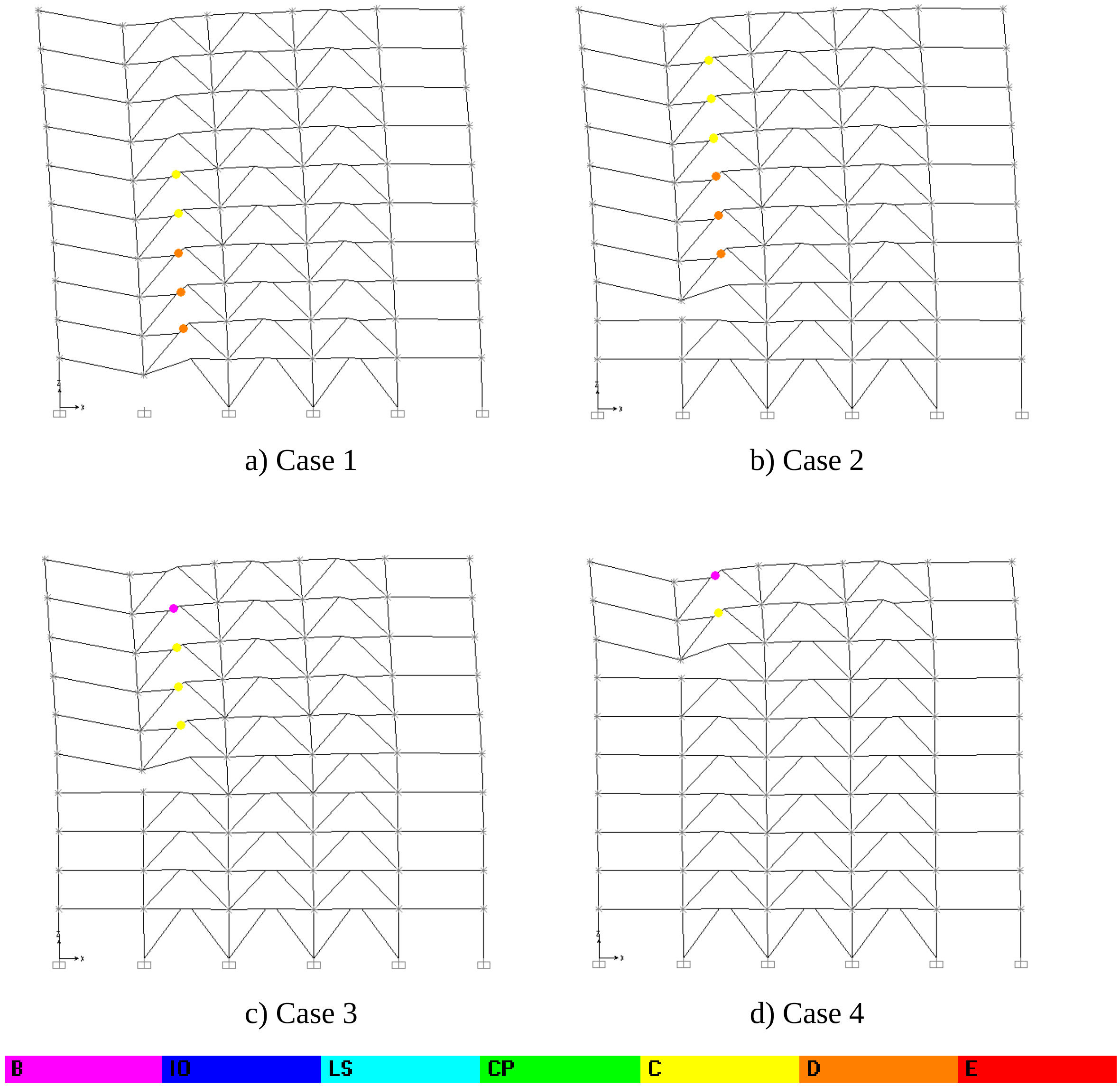


Figure 6.6. EBF deformed shape of APM cases considered for NSA.

6.6. DIF According to UFC

According to UFC 4-023-03, DIF is calculated using the formula in Equation 3.22. Here, the smallest ratio of θ_{pra}/θ_y for any primary element, component or connection in the model within or touching the area loaded with increased load, is used. For any structural element, θ_y is the yield rotation computed as in Section 5.4.2.1 or 5.4.2.2, and θ_{pra} is the NAC for that particular element. A special application is valid for connections such that θ_y is the yield rotation angle of the structural element that is being connected like beam.

Calculations of DIF for SCBF and EBF are shown in Table 6.5 and 6.6 respectively. These values are valid for all APM cases of associated frame because the minimum θ_{pra}/θ_y value was resulted from WUF-W connections and this connection type was present above the all removal locations. Thus, DIF for SCBF is 1.37 and DIF for EBF is 1.48 per UFC.

Table 6.5. Calculation of DIF for SCBF per UFC 4-023-03.

Element Type	θ_{pra}	θ_y	θ_{pra}/θ_y	**Element Type**	θ_{pra}	θ_y	θ_{pra}/θ_y
Braces at compression				**Beams**			
HSS 7x7x1/2-First Story	6.99 Δ_c	Δ_c	6.99	W24x76	$8\theta_y$	θ_y	8.00
HSS 7x7x1/2	6.76 Δ_c	Δ_c	6.76	W21x50	$8\theta_y$	θ_y	8.00
HSS 6x6x1/2	7.00 Δ_c	Δ_c	7.00	W16x31	$7.77\theta_y$	θ_y	7.77
HSS 5-1/2x5-1/2x3/8	7.00 Δ_c	Δ_c	7.00	**Braces at tension**			
HSS 4-1/2x4-1/2x3/8	7.00 Δ_c	Δ_c	7.00	HSS 7x7x1/2-First Story	9 Δ_t	Δ_t	9.00
WUF-W Connection of				HSS 7x7x1/2	9 Δ_t	Δ_t	9.00
W21x50	0.02007	0	1.75	HSS 6x6x1/2	9 Δ_t	Δ_t	9.00
Shear Tab Connection of				HSS 5-1/2x5-1/2x3/8	9 Δ_t	Δ_t	9.00
W24x76	0.04120	0	4.23	HSS 4-1/2x4-1/2x3/8	9 Δ_t	Δ_t	9.00
W21x50	0.04120	0	3.60	$\Omega_N = 1.08 + \left(\frac{0.76}{\theta_{pra}/\theta_y + 0.83}\right)$		**Minimum**	1.75
W16x31	0.04120	0	2.79			**1.37**	

Table 6.6. Calculation of DIF for EBF per UFC 4-023-03.

Element Type	θ_{pra}	θ_y	θ_{pra}/θ_y	**Element Type**	θ_{pra}	θ_y	θ_{pra}/θ_y
Beams				**WUF-W Connection of**			
W14x48	$8\theta_y$	θ_y	8.00	W14x48	0.02288	0.01656	1.38
W12x45	$8\theta_y$	θ_y	8.00	W12x45	0.02358	0.01893	1.25
W10x39	$6.34\theta_y$	θ_y	6.34	W10x39	0.02443	0.02293	1.07
W16x31	$7.77\theta_y$	θ_y	7.77	**Shear Tab Connection of**			
Braces at tension				W16x31	0.04120	0.01475	2.79
HSS 8x8x1/2	9 Δ_t	Δ_t	9.00	**Shear Link**			
HSS 7x7x1/2	9 Δ_t	Δ_t	9.00	W14x48	0.140	0.0043	32.19
Braces at compression				W12x45	0.140	0.0047	30.08
HSS 8x8x1/2-First Story	6.63 Δ_c	Δ_c	6.63	W10x39	0.141	0.0053	26.65
HSS 8x8x1/2	6.42 Δ_c	Δ_c	6.42	$\Omega_N = 1.08 + \left(\frac{0.76}{\theta_{pra}/\theta_y + 0.83}\right)$		**Minimum**	1.07
HSS 7x7x1/2	6.64 Δ_c	Δ_c	6.64			**1.48**	

Yield rotations of shear links were calculated using Equation 6.1 to Equation 6.4 obtained from ASCE 41-06 and the details of calculation are listed in Table 6.7.

$$K_s = \frac{GA_w}{e} \tag{6.1}$$

$$K_b = \frac{12EI_b}{e^3} \tag{6.2}$$

$$K_e = \frac{K_s K_b}{K_s + K_b} \tag{6.3}$$

$$\theta_y = \frac{Q_{CE}}{K_e e} \tag{6.4}$$

Where,

A_w = shearing area

e = link length

G = shear modulus

K_e = stiffness of the link beam

K_b = flexural stiffness

K_s = shear stiffness

Q_{CE} = expected strength as defined in Equation 5.64

Table 6.7. Calculation of θ_y for shear links of EBF.

BEAM	G	A_w	e	E	I_b	Q_{CE}	K_s	K_b	K_e	θ_y
	MPa	m^2	m	MPa	m^4	kN	kN/m	kN/m	kN/m	rad
W14x48	76903	2.76E-03	1.2	200000	2.02E-04	566.5	1.77E+05	2.80E+05	1.09E+05	0.0043
W12x48	76903	2.36E-03	1.2	200000	1.46E-04	483.2	1.51E+05	2.02E+05	8.65E+04	0.0047
W10x39	76903	1.80E-03	1.2	200000	8.70E-05	374.6	1.15E+05	1.21E+05	5.90E+04	0.0053

6.7. Discussion of Results

In the previous sections, DIF required for NSA of APM cases of two different braced frame types, namely; SCBF and EBF, was investigated. The following observations and discussions might be arisen from the comparison of required DIF with the calculated DIF per UFC 4-023-03.

The calculated DIF for the cases of SCBF was higher than the required values of DIF for considered cases of the same braced frame. In Table 6.1, the maximum DIF required to equate the node displacement result of NSA to the one of NDA is 1.362. Therefore, using the calculated value of DIF, that is 1.37, would have allowed the results of NSA to be on the safe side. The same situation was valid for the first four APM cases of EBF. For these cases, the calculated value of DIF, that is 1.48, was higher than the required values of DIF.

For APM Case 6 of EBF, both values of DIF were equal; however, for the remaining last four cases the required DIF was higher than the calculated one per UFC. Therefore, a progressive collapse analysis with NSA approach of UFC would have resulted in underestimated results for these APM cases. Nonetheless, EBF had no progressive collapse potential considering these cases and such an underestimation would have not affected much the overall progressive collapse assessment.

From Table 6.8 it can be observed that, required DIF demanded almost the same stress from critical columns near removal location. Only, a DIF of 1.32 for APM Case 2 of EBF demanded less stress than the capacity of the column W14x132. Yet, DIF of 1.48 per UFC would have overloaded this column during a possible nonlinear static progressive collapse analysis. For the columns away from removal location, stress demand on columns was less than the ones obtained during NDA. The reason behind this was the fact that only loads directly above removal location were increased with DIF; as a result, it was not possible to observe global system behavior during NSA like the case in NDA. Also, the difference between stress demands of columns on line D was higher for SCBF and shear tab connections were unable to transfer additional loads to neighbor bays effectively as addressed in NDA results. For APM Cases 6 to 10 of EBF, demands on columns of line B and D were close during nonlinear static and dynamic analysis.

Table 6.8. Comparison of combined stress on columns.

EBF						SCBF					
		Column C-i		Column D-i				Column C-i		Column D-i	
Case	Column	NDA	NSA	NDA	NSA	Case	Column	NDA	NSA	NDA	NSA
1	W14x176	1.01	1.00	0.52	0.43	2	W14x233	0.59	0.60	0.78	0.63
2	W14x132	1.00	0.96	0.47	0.43	3	W14x193	0.51	0.53	0.66	0.57
3	W14x109	0.84	0.82	0.41	0.37	4	W14x132	0.51	0.52	0.66	0.56
4	W14x61	0.84	0.71	0.38	0.31	5	W14x82	0.54	0.58	0.68	0.62
		Column B-i		Column D-i				Column B-i		Column E-i	
6	W14x176	0.72	0.70	0.64	0.62	8	W14x233	0.54	0.55	0.56	0.50
7	W14x132	0.69	0.67	0.60	0.57	9	W14x193	0.48	0.50	0.51	0.47
8	W14x109	0.58	0.58	0.50	0.49	10	W14x132	0.48	0.50	0.53	0.49
9	W14x61	0.53	0.55	0.48	0.49	11	W14x82	0.52	0.57	0.62	0.57
10	W14x48	0.24	0.24	0.18	0.19						

Most braces of SCBF have buckled during NSA but the level, and the number of buckled braces was less compared to NDA. During NDA, most braces reached CP level whereas the common deformation level was IO for NSA. Nonetheless, nonlinear deformations were within the allowable limits for each analysis type. Shear links of EBF showed less inelastic deformations with required DIF. However, for APM Case 1 and 2, the level of deformation was again more than the NAC for shear links.

On the other hand, the compatibility of NSA is questionable for the progressive collapse analysis of these frames. Especially, for highly nonlinear cases it was not always possible to reach desired level of deformation as it was the case in APM Case 3 and 4 of EBF. Also, the increase in required DIF does not allow more nonlinear deformations at the braces of SCBF. These were due to not only local loading in NSA but also due to the fact that unloading along a negative slope may be unstable in static analysis, and a unique solution is not always mathematically guaranteed. However, NDA provides stability and unique solution via inertial forces (CSI, 2010).

To summarize, for most cases, calculated DIF was higher than the required one and taking into account that static analysis are generally desired to be on the safe side; DIF calculated according to Equation 3.22 of UFC 4-023-03 seems also valid for braced frames. Nevertheless, a generalization is not possible with the results of only two frames and also considering the fact that DIF of 1.48 was an underestimation for some cases of EBF, further investigation for DIF of braced frames is necessary.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. Summary

In this study basics of progressive collapse have been provided in order to make further studies about this research area more understandable. Also, a brief comparison between seismic and progressive collapse design was introduced in order to clarify basic differences between these two design goals. Then, summary of design codes and standards addressing progressive collapse resistant design methods of structures was provided considering steel framed structures. This section was followed by the review of recent publications about mitigating progressive collapse of steel structures.

Two case studies were taken in to account. In the first case study, progressive collapse potential of two ten-story seismically designed steel braced frames was investigated separately. Since the buildings with SCBFs and EBFs were adopted from a research project of NIST of the USA, their compliance with TEC 07 was discussed first. Then, APM method with the NDP as defined in UFC 4-023-03 (DoD, 2009) has been carried out for detailed assessment of progressive collapse potential of these frames at different story levels. In the second case study, the results of NDP were utilized in order to investigate the accuracy of DIF calculated per UFC for NSA of these braced frames.

7.2. Conclusions

Although structural engineering community has recognized progressive collapse as a design goal at the second half of 20th century, studies to understand and to mitigate progressive collapse has intensified mostly in the first years of 21st century, which resulted in specific design codes to resist progressive collapse of structures. Today, progressive collapse is an up-to-date and a worldwide research area in the structural engineering community, which has wide range of sub-research areas.

In the light of the works carried out, the following conclusions are obtained:

- Potential for progressive collapse is high when a corner column of a steel frame is lost. Having high ductility, shear tab connections do not posses enough strength, which is necessary to transfer additional loads to neighbor bays. Therefore, gravity frames should not be allowed at the perimeter frames when progressive collapse is of concern.
- Progressive collapse potential increases with the height of removal location based on these two frames.
- Both braced frames types have progressive collapse potential but at different story levels. SCBF has progressive collapse potential at the last two stories whereas EBF has collapse potential at the first three stories. Therefore, APM cases based on only first story element removal may underestimate progressive collapse potential of structures.
- During reduction of sectional sizes at higher stories with the permission of design loads, progressive collapse needs also be considered for structures having this design goal in order not to increase structures vulnerability at high story levels.
- Compared to SCBF, EBF has less progressive collapse potential due to more braces and less partial connections or more moment connections, which in return improved load redistribution within the frame. However, it cannot be stated that seismic design ensures completely progressive collapse resistance of structures.
- The equation recommended by UFC 4-023-03 seems also valid for DIF calculation of steel braced frames for most of the APM cases considered. However, further investigation is also necessary due to some underestimated values of DIF for EBF.
- The dynamic nature of progressive collapse makes nonlinear dynamic analysis procedures be more realistic and representative compared to static analysis. In spite of high computational and modeling time requirements, it is recommended that NDA procedure is used for progressive collapse analysis of structures.

7.3. Recommended Future Works

In the open literature there is not enough study about the progressive collapse potential of steel braced frames. The most of the available ones utilize 2D macro models without slab modeling. Also, catenary action has been not considered in many of them, which was reported as improving the progressive collapse resistance of steel structures by many researchers. These deficiencies are also valid for the case studies of this thesis work. Therefore, it is recommended to investigate progressive collapse potential of different braced frames considering 3D models with slab and catenary action behavior together. The accuracy of DIF for the NSA of braced frames should be also further investigated considering different braced frame types.

In this study, nonlinear modeling of brace and link hinges were directly based on ASCE 41-06, which has been developed for seismic rehabilitation of existing structures. However, as it is the case for connections, nonlinear modeling and acceptance criteria of braces and shear links need to be modified or developed according to demands during a progressive collapse case.

It should be also noted that for some cases progressive collapse may also be a cause of collapse during an earthquake. Therefore, progressive collapse should also be considered in Turkey in addition to seismic design in order to reduce possible losses after abnormal events. Especially, public buildings like schools, hospitals, governmental and military buildings which have high occupancy levels need to be taken into account.

In this context, the awareness of Turkish structural engineering society should be increased by developing a national progressive collapse resistant design code including both steel and concrete material specific sections.

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