ANALYSIS OF BLAST LOADING EFFECT ON REGULAR STEEL BUILDING STRUCTURES

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ANALYSIS OF BLAST LOADING EFFECT ON REGULAR STEEL BUILDING STRUCTURES

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ABSTRACT

ANALYSIS OF BLAST LOADING EFFECT ON REGULAR STEEL BUILDING STRUCTURES

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Concern about effect of explosives effect on engineering structures evolved after the damage of Second World War. Beginning from 90's with the event of bombing Alfred P. Murrah Federal building located in Oklahoma City this concern deepened and with the attack to World Trade Center twin towers on September 11, 2001 it is peaked. Recent design codes mainly focus on earthquake resistant design and strengthening of the structures. These code design methodologies may sometimes satisfy current blast resistant design philosophy, but in general code compliant designs may not provide recognizable resistance to blast effect. Therefore designer should carry out earthquake resistant design with the blast resistant design knowledge in mind in order to be able to select the most suitable framing scheme that provide both earthquake and blast resistance. This is only possible if designer deeply understands and interprets the blast phenomenon.

In this study, it is intended to introduce blast phenomenon, basic terminology, past studies, blast loading on structures, blast structure interaction, analysis methodologies for blast effect and analysis for blast induced progressive and disproportionate collapse. Final focus is made on a case study that is carried out to

determine whether a regular steel structures already designed according to Turkish Earthquake Code 2007 requirements satisfy blast, thus progressive collapse resistance requirements or not.

Keywords: Blast, Progressive collapse, Earthquake resistance, Steel structure.

ÖZ

DÜZENLİ ÇELİK BİNALARIN PATLAMA YÜKÜ ETKİSİ ALTINDA ANALİZİ

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Patlayıcı maddelerin mühendislik yapılarına yaptığı etki ile ilgili araştırmalar ikinci dünya savaşının yaptığı tahribattan sonra mühendislerin ilgi alanına girmiştir. Bu ilgi 90'ların başında Oklahama Şehrindeki Alfred P. Murrah Federal ofis binasının bombalanması ile daha derinleşmiş ve 11 Eylül 2001'deki Dünya Ticaret Merkezi İkiz Kulelerine yapılan saldırı ile doruk noktasına ulaşmıştır. Güncel tasarım şartnameleri esas olarak yapıların depreme dayanıklı tasarımı veya güçlendirilmesi üzerine yoğunlaşmıştır. Bu şartnamelerin tasarım metodları kimi zaman patlayıcı etkisine dirençli tasarım felsefesi ile uyum göstermekle beraber genellikle depreme dirençli tasarımlar patlayıcı etkisine karşı kayda değer bir direnç sağlamamaktadır. Bu nedenle bir tasarımcı patlayıcı ve deprem etkilerine en fazla direnci sağlayacak çerçeve sistemini seçmek için patlama etkisini de aklında bulundurarak tasarımını gerçekleştirmelidir. Bu ise ancak tasarımcının derin bir patlayıcı etkisi bilgisine sahip olması ve bunu tasarımına yansıtabilmesi ile mümkündür.

Bu çalışmada patlayıcı fenomeni, bununla ilgili temel terminoloji, geçmiş çalışmalar, patlayıcı yüklemesi, patlayıcı ve yapı etkileşimi, patlama kaynaklı tedrici ve orantısız çökme ve bunlara ilişkin analiz yöntemleri tanıtılıp açıklanmaya çalışılmıştır. Nihai vurgu ise daha önceden 2007 Afet yönetmeliği hükümlerine gore tasarlanmış düzenli bir çelik yapının patlama etkisine, dolayısıyla tedrici çökmeye direncini tespite ilişkin bir durum değerlendirmesi üzerine yapılmıştır.

Anahtar Kelimeler: Patlama, Tedrici çökme analizi, Deprem dayanımı, Çelik yapılar.

Her zaman ve her zorlukta yanımda olan abim Ahmet TAHMİLCİ ve biricik Annem'e.

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LIST OF SYMBOLS AND ABBREVIATIONS

2D	Two-dimensional
3D	Three-dimensional
AISC	American Institute of Steel Construction
ANFO	Ammonium Nitrate / Fuel Oil
ASCE	American Society of Civil Engineering
ATC	Applied Technology Council
ATF	Bureau of Alcohol, Tobacco and Firearms
BCC	Body-centered cubic
BPRPPCB	Best Practices for Reducing the Potential for Progressive Collapse in Buildings
TS	Turkish Standard
DCR	Demand-Capacity Ratio
DoD	Department of Defence
DIF	Dynamic increase factor
DLF	Dynamic Load Factor
FEMA	Federal Emergency Management Agency
f_y	Yield stresses
GSA	General Services Administration
I	Impulse
i_r	Reflected impulse
LRFD	Load and Resistance Factor Design
NIST	National Institute of Standards and Technology
M	TNT equivalent mass
PCADG	Progressive Collapse Analysis Design Guidelines
PBS	Public Buildings Service
P_r	Reflected pressure
P_o	Ambient pressure
P_{so}	Overpressure (also called incident or peak pressure)
P_{so}^-	Negative Pressure
R	Standoff distance

LIST OF SYMBOLS AND ABBREVIATIONS (CONTINUED)

RDX	Cyclotrimethylene-trinitramine
q_s	Air velocity in front of the explosion
S	Maximum distance from an edge
SDOF	Single Degree of Freedom
SHPB	Split Hopkinton's Pressure Bar
t_d	Time taken for the overpressure to be dissipated.
t_d^-	Time taken for the negative pressure to be dissipated.
TEC	Turkish Earthquake Code
TNT	Trinitro Toluene
U_s	Speed of sound
W	Charge weight
X	Distance in feet to a given overpressure
Z	Scaled Distance

CHAPTER 1

INTRODUCTION

In recent years, a number of tragic terrorist attacks, particularly, in the U.S., have resulted in a number of initiatives to study the resistance of structures to blast. In addition, a number of research projects have been undertaken or are underway to develop mechanisms and systems to reduce the hazard of such attacks. The main aim of these efforts is to protect the safety of the occupants of the building, the rescue workers and those who are around the building whom can be killed or injured by the collapse of the structure and the falling debris. One of the main areas of research and development in this field is the progressive collapse prevention. From structural engineering and construction point of view, of course, one can design a building that can withstand a terrorist bomb attack with minimal or no damage. This has been done for years and continues to be done for militarily sensitive and other critical buildings that are necessary to be functional and occupied even after a bomb attack on them. Of course, designing such a highly protected building requires a significant amount of funding as well as resources. In addition, to achieve the objective of the minimal damage, the designers may end up sacrificing the exterior aesthetics and in some cases the internal functionality of the building. Although in case of military installations, the high cost and bunker like appearance of a building can be justified, however, for civilian buildings, such high costs cannot be afforded and the loss of aesthetics may not always be acceptable. This was because of the assumption that civilian buildings had a very low probability to be a target of terrorist attack. But it is seen that, it is not the case by events of September 11, 2001 bombing of World Trade Center and by bombings of two synagogue, British Embassy and HSBC Bank in Turkey on October, 2004 with total of nearly 3 tons of equal charge of TNT. These recent events show the importance of blast resistant design issues even for Turkey.

In our current codes either no measures are included and no references to documents related to blast effect on structures are made, even in military specifications. But as Turkey is in one of the seismically most active regions of the world and seismic effect together with necessary precautions takes more and more attention day after day. Past studies indicate that seismic precautions taken for reinforced concrete frame structures can result in positive effect for the structures which are subjected to terrorist attack therefore, blast effect. But this is not proven for steel structures through out an analytical study. Therefore, this study is an initial attempt to reveal the relationships between seismic precautions and blast, and its consequent effect of progressive collapse resistance of a steel building.

A structural engineer aiming to provide blast resistance for subject building first of all has to deeply understand and interpret blast phenomenon and its main and secondary effects on an engineering structure.

1.1 BACKGROUND

Explosive loading incidents have become a serious problem that must be addressed quite frequently. Many buildings that could be loaded by explosive incidents are moment resistant frames either concrete or steel structures, and their behavior under blast loads is of great interest. Besides the immediate and localized blast effects, one must consider the serious consequences associated with progressive collapse that could affect people and property. Progressive collapse occurs when a structure has its loading pattern, or boundary conditions, changed such that structural elements are loaded beyond their capacity and fail. The remaining structure has to seek alternative load paths to redistribute the load applied to it. As Krauthammer, 2003 states: “As a result, other elements may fail, causing further load redistribution. The process will continue until the structure can find equilibrium either by shedding load, as a by product of other elements failing, or by finding stable alternative load paths”. In the past, structures designed to withstand normal load conditions were over designed, and have usually been capable of tolerating some abnormal loads. Modern building design and construction practices enabled one to build lighter and more optimized structural systems with considerably lower over design characteristics.

Progressive collapse became an issue following the Ronan Point incident in 1968, when a gas explosion in a kitchen on the 18 floor of a precast building caused extensive damage to the entire corner of that building. "The failure investigation of that incident resulted in important changes in the UK building code (Shankar, 2003)." It requires to provide a minimum level of strength to resist accidental abnormal loading "by either comprehensive 'tying' of structural elements, or (if tying is not possible) to enable the 'bridging' of loads over the damaged area or (if bridging is not possible) to insure that key elements can resist 34 kN/m2 (Krauthammer, 2003)." These guidelines have been incorporated in subsequent British Standards. According to Krauthammer, "although many in the UK attribute the very good performance of numerous buildings subjected to blast loads to these guidelines, it might not be always possible to quantify how close those buildings were to progressive collapse."

As stated by Krauthammer, 2003 recent developments in the efficient use of building materials, innovative framing systems, and refinements in analysis techniques could result in structures with lower safety margins. Some of the governmental agencies of U.S. such as Department of Defence (DoD), General Services Administration (GSA), Federal Emergency Management Agency (FEMA) have issued clear guidelines to address this critical problem (DoD 2002, GSA 2003, FEMA 453). Nevertheless, Krauthammer states that, these procedures contain assumptions that may not reflect accurately the actual post attack conditions of a damaged structure, as shown in Figure 1, which is due to the fact that very complicated state of damage must be assessed before the correct conditions can be determined. The structural behavior associated with such events involves highly nonlinear processes both in the geometry and material. One must understand that various important factors can affect the behavior and failure process in a building subjected to an explosive loading event, but these cannot be easily assessed. Another issue Krauthammer criticisizes about these guidelines is "the idea that one might consider the pure removal of a column as a damage scenario, while leaving the rest of the building undamaged, is actually unrealistic" which is the case in GSA 2003 and DoD 2002 .

An explosive loading event near a building will cause extensive localized damage, affecting more than a single column. The remaining damaged structure is expected to behave very differently from the ideal situation. This reveals the importance of assessing accurately the post attack behavior of structural elements that were not removed from the building by the blast loads in their corresponding damaged states. This requires one to perform first a fully-nonlinear blast-structure interaction analysis, determine the state of the structural system at the end of this damaged phase, and then to proceed with a fully nonlinear dynamic analysis for the damaged structure subjected to only gravity loads.

Such comprehensive analyses are very complicated, they are very time consuming and require extensive resources. Due to such reasons currently the best source of easy to use and implicate analysis guideline is GSA's guideline. Actually, accurate analysis of this kind of loading requires nonlinear dynamic analysis software especially developes for blast loading, that implements principles of TM5-1300 "U.S. Departments of the Army, Navy and Air Force (1991) Technical manual, Structures to resist the effects of accidental explosions. Some examples of that software are AUTODYN, DYNA3D, LS-DYNA and ABAQUS. Due to software availability limitations, linear-nonlinear analysis software, SAP 2000 is utilized in this study.

Figure 1: Post-Incident View of Building Damage from the 1992 St. Mary's Axe Bombing Incident in London. (From work of Krauthammer, 2003)

Damaged structures may have insufficient reserve capacities to accommodate abnormal load conditions. Krauthammer states that, there are few numerical examples of computational schemes to analyze progressive collapse. “Typical finite element codes can only be used after complicated source level modification to simulate dynamic collapse problems that contain strong nonlinearities and discontinuities.” Several approaches have been proposed for including progressive collapse resistance in building design and assessment. The alternative load path method is a widely known analytical approach that follows the definition of progressive collapse. It refers to the removal of elements that failed the stress or strain limit state (GSA, 2003).

Structural detailing plays a very significant role during a building’s response to blast. 1994 Northridge earthquake highlighted troublesome weaknesses in design and construction technologies of welded connections in moment-resisting structural steel frames in US. As a result, the US steel construction community launched an extensive research and development effort to remedy the observed deficiencies. (Krauthammer, 2003) During about the same period, domestic and international terrorist attacks have become critical issues that must be addressed by structural engineers.

In blast resistant design, however, most of the attention during the last half century has been devoted to concrete. Since many buildings, which are highrise and that could be targeted by terrorists are moment-resisting steel frames, their behavior under blast is of great interest, with special attention to connection failure and subsequent progressive collapse. Typical structural steel welded connection details, currently recommended for earthquake conditions, underwent preliminary assessments for their performance under blast effects. The assessments also addressed current blast design procedures to determine their applicability for both the design and analysis of such details. (GSA, 2003) The finding highlighted important concerns about the blast resistance of structural steel details, and about the assumed safety in using current blast design procedures for structural steel details. Obviously, one must address not only the localized effects of blast loads, and the idealized behavior of typical structural elements (e.g., columns, girders, etc.), but also the behavior of structural connections and adjacent elements that define the support conditions of a structural element under consideration. The nature of blast loads, the behavior of structural connections under such conditions, and progressive collapseare discussed in the following chapters of this study to provide clear understanding for current research.

1.2 DEFINITIONS AND TERMINOLOGY

In the following part of this chapter essential definitions and terminology related with explosives and blast will be given which is necessary in comprehensive understanding of the following discussed concepts and procedures. Basic source of information for these definitions and concepts is World Wide Web, especially web sites of Blastgard and Peak Co.'s. In order to avoid confusion definitions and concepts are given in a simple, short and summarized manner. For further details one should easily consult the net and find any detail in more advance of this explanation.

Explosion: Release of energy that causes a pressure discontinuity or blast wave.

High-order explosions: Release a lot of heat and produce shock waves. About 50% of the energy in a blast goes to heat and 50% goes to shock waves.

Ammonium Nitrate / Fuel Oil (ANFO): A crude but effective explosive that is used by farmers to clear stumps and by the mining industry (because it is easy to pump in slurry form) to break up overburden rock and expose ore in open pit mining.

Ammonium nitrate fertilizer is made by chemically combining ammonia with nitric acid in a water solution. Water formed during the reaction is evaporated, leaving a concentrated ammonium nitrate melt. The hot melt is then processed in one of several ways, depending on plant design, into prills or granules. The finished product is then coated with a conditioning agent, usually clay, to prevent it from caking.

Ammonium nitrate fertilizer is an oxidizer, a substance that oxidizes readily to stimulate the combustion of organic matter or other fuels.

While it has a wide use as a fertilizer, ammonium nitrate is also the principal base material in slurry explosives and lower-cost blasting agents. It is converted to an effective blasting agent by properly mixing it with a carbonaceous material such as fuel or ground walnut hulls. Although chemically the same as the fertilizer grade, the ammonium nitrate used for blasting purposes is of a lower density, usually less than 0.85 grams per cubic centimeter and containing small percentages of anti-caking agents. By definition, a blasting agent is any material or mixture consisting of a fuel and oxidizer intended for blasting, not otherwise classified as an explosive, provided that the finished product, as mixed and packaged for use or shipment, cannot be detonated by a No. 8 blasting cap when unconfined. Ammonium nitrate has roughly 50 percent of the strength of TNT when detonated completely. It yields an energy release of approximately 400 calories per gram. TNT when detonated yields an energy release of approximately 750 to 900 calories per gram

More than two million pounds of these mixtures,commonly referred to as ANFO (Ammonium Nitrate Fuel Oil), are consumed each year. They account for approximately 80% of the domestic commercial market.

ANFO products have found extensive use in a variety of blasting applications including surface mining of coal, metal mining, quarrying and construction. Their popularity has increased because of economy and convenience. The most widely used ANFO product is oxygen balanced free-flowing mixture of about 94% ammonium nitrate prills and 6% No. 2 Diesel fuel oil.

C-4: A common variety of military plastic explosive. C-4 is made up of explosive, binder, plasticizer and (latterly) marker or taggant chemicals. As in many plastic explosives the explosive material in C-4 is RDX (Cyclonite, cyclotrimethylene trinitramine) which makes up around 90% of the C-4 by weight. The binder is polyisobutylene (5.5%) and the plasticizer is di (2-ethylhexyl) or dioctyl sebacate (2%). In the U.S., the marker is DMDNB (2, 3-dimethyl-2, 3-dinitrobutane). Another binder used is dioctyl adipate (DOA). A small amount of petroleum oil is also added.

Dynamite: An explosive used in mining, demolitions, and other applications. It was invented by Alfred Nobel in 1867, and rapidly gained popularity as a safer alternative to gunpowder, because it does not explode by accident as easily.

Trinitrotoluene (TNT): A pale yellow crystalline aromatic hydrocarbon compound that melts at 81 °C (178 °F). Trinitrotoluene is an explosive chemical and a part of many explosive mixtures, such as when mixed with ammonium nitrate to form amatol.

Nitroglycerin: A heavy colorless poisonous oily explosive liquid obtained by nitrating glycerol. It is used in the manufacture of explosives, specifically dynamite, and as such is employed in the construction and demolition industries.

RDX: is an explosive nitro amine widely used in military and industrial applications. Nomenclature variants include **cyclotrimethylene-trinitramine**. In its pure, synthesized state RDX is a white, crystalline solid. As an explosive it is usually used in mixtures with other explosives and plasticizers or desensitizers. It is stable in storage and is considered one of the most powerful and brisant of the military high explosives. RDX is also used as a major component of many plastic bonded explosives used in weapons.

Semtex: is a general-purpose plastic explosive. First made by the Semtín East Bohemian Chemical Works (then called VCHZ Synthesia, now called Explosia) in Semtín (a suburb of Pardubice) in the Czech Republic), it is used in commercial blasting, demolition, and in certain military applications. Semtex became notoriously popular with terrorists because it was, until recently, extremely difficult to detect, as in the case of Pan Am Flight 103

Plastic Explosive: A specialized form of explosive material. They are soft and hand malleable and may have the added benefit of being usable over a wider temperature range than the pure explosive. Plastic explosives are especially suited for explosive demolition as they can be easily formed into the best shapes for cutting structural members, and have a high enough velocity of detonation and density for metal cutting work. They are generally not used for ordinary blasting as they tend to be significantly more expensive than other materials that perform just as well in that field. Also, when an explosive is bound in a plastique, its power is generally lower than when it is pure.

Detonation: Release of energy caused by the extremely rapid chemical reaction of a substance in which the reaction front advances into the unreacted substance at equal to or greater than sonic velocity. Detonation is an exothermic reaction characterized by the presence of a shock wave in the material that establishes and maintains the reaction. A distinctive characteristic of detonation is that the reaction zone propagates at a speed greater than the speed of sound.

Detonator: Used to trigger bombs, shape charges and other forms of explosive device. Detonators are often attached to a timer to ensure that the explosion takes place at the desired time, or when the person laying the explosives has reached a safe distance from the blast. Detonators can be chemical, mechanical, or a combination. Many detonators' primary (sensitive to heat and shock) explosive is a material called tetryl.

Deflagration: Chemical reaction of a substance in which the reaction front advances into the unreacted substance at less than sonic velocity. Where a blast wave is produced that has the potential to cause damage, the term explosive deflagration may be used.

Ballistic Impact: Ballistic Impact refers to initiating a unit of ammunition or other energetic material by an impact of a ballistic threat as a bullet or other high velocity projectile.

Overpressure (or peak pressure): Overpressure (or peak pressure) appears approx. 1/10th to 5 milliseconds after detonation, depending on scaled distance. Safety standards for buildings and inhabited areas are typically based on maximum peak pressures.

Impulse: Impulse is the momentum (mass x velocity) imparted in a blast and is determined by the area under the pressure-time curve.

Quasi-Static Pressure: Quasi-static pressure is a major effect in a confined blast. In a room or large space gas pressure will build up to a fairly constant level; however, in a confined space gas pressure just builds until either the walls blow out (vent) or the confined hot gas cools down. The pressure determines required hoop strength in containers and buildings.

Reflected Overpressure: Reflected overpressure theoretically runs from 2 to 8 times incident pressure in free air.

Scaled Distance: Scaled distance is the main way of comparing different blasts. The definition is:

$$\text{Scaled Distance, } Z = \frac{\text{R (Distance from Charge)}}{\text{W (TNT Equivalent Charge Weight)} \sqrt{3}} \qquad (1)$$

Or, Scaled Distance is equal to the Distance from Charge divided by the cubic root of the TNT Equivalent Net Charge Weight. Source: http://www.blastgardintl.com

1.3 OBJECT AND SCOPE

Within the scope of this study, it is intended to:

1. To develop knowledge of explosive materials, blast phenomenon and its effects of regular building type structures based on literature.
2. To introduce basic blast induced damage event, consequently prevention against progressive collapse
3. To summarize analysis approaches and procedures of General Services Administration Progressive Collapse Analysis Design Guidelines, 2003 and nonlinear analysis method for progressive collapse proposed by Guo and Gilsanz, 2003
4. Illustrate the two analysis method that will be outlined through out this work by a case study on a regular steel frame building readily designed according to New Turkish Earthquake Code, 2007, to make deductions on the analyzed particular frame and building type according to these methods.

CHAPTER 2

EXPLOSION, BLAST, BLAST STRUCTURE INTERACTION

2.1 EXPLOSIONS AND BLAST PHENOMENON

There are multiple definitions of an explosion in fact describing the same effect defined by Mendis, Gupta and Ramsay as “large-scale, rapid and sudden release of energy.” Explosions can be categorized on the basis of their nature as physical, nuclear or chemical events. In physical explosions, energy may be released from the catastrophic failure of a cylinder of compressed gas, volcanic eruptions or even mixing of two liquids at different temperatures. In a nuclear explosion, energy is released from the formation of different atomic nuclei by the redistribution of the protons and neutrons within the interacting nuclei, whereas the rapid oxidation of fuel elements (carbon and hydrogen atoms) is the main source of energy in the case of chemical explosions. (Smith and Hetherington, 1994) Explosive materials can be classified according to their physical state as solids, liquids or gases. Solid explosives are classified as mainly high explosives for which blast effects are best known. They can also be classified on the basis of their sensitivity to ignition as secondary or primary explosive. (Mendis, Gupta, Ramsay, 2007) The latter is one that can be easily detonated by simple ignition from a spark, flame or impact. Secondary explosives when detonated create blast (shock) waves which can result in widespread damage to the surroundings. Examples include trinitro-toluene (TNT) and ANFO.

Sometimes explosions are classified as thermal explosions and non-thermal explosions. (Longinow, 2003) A thermal explosion is one which burns suddenly (detonates) resulting in a violent expansion of gases with great disturbing force and a loud noise. (Smith, Hetherington) The detonation of an explosive device made up of ammonium nitrate/fuel oil (ANFO), such as the explosions in Istanbul in 2004, is widely known as an example of a thermal explosion. A non-thermal explosion describes a sudden bursting because of buildup of pressure within a container. An example is the filling of a tank with air under pressure, and the tank suddenly bursts producing an explosion. (Longinow, 2003)

Longinow further defines an explosive as a “device that involves the use of a solid or liquid that explodes if ignited, shocked, or subjected to heat or friction”. Examples are nitroglycerine, ammonium nitrate/fuel oil mixtures, TNT, dynamite, lead azide, RDX, gunpowder, and dynamite.

Debate is ongoing in the issue whether something will explode or not and it requires investigation on a case-by-case basis. Some materials such as copper azide will detonate at the slightest shock or movement whereas others such as TNT or RDX may require another explosive (called a primary explosive, or a blasting cap) to detonate the material (PEAK Inc., www.peak.com.). Therefore, there is no easy way of predicting whether a particular material is explosive; a case-by-case investigation is required. However if an oxidizing material (e.g. ammonium perchlorate, potassium permanganate, ammonium nitrate, etc.) can be placed in intimate contact with a fuel source this is a basic recipe for an explosive material. Longinow states that “if the oxidizing part can be incorporated into the molecule itself (e.g. nitric acid plus glycerin to yield nitroglycerine), a powerful explosive is produced. A very well known example is trinitrotoluene, also called 2, 4, 6-trinitrotoluene, or “TNT” for short, which is manufactured from toluene (toluene is the fuel part of the molecule; three “nitro-” groups are the oxidizing part).” If certain combustible metal powders such as aluminum can also be mixed in with the material, the explosive capability may be enhanced. Many explosive chemicals have nitrogen in the form of nitrate (a nitrogen atom linked to three oxygen atoms) or nitro- (a nitrogen atom linked to two oxygen atoms) or azide (two nitrogen atoms linked together) incorporated as part of the organic molecule (Smith and Hetherington, 1994).

Dynamite is a detonating explosive containing a liquid explosive ingredient (usually nitroglycerine or a similar organic nitrate ester or both) that is uniformly mixed with an adsorbent material such as wood pulp and usually contains materials such as nitrocellulose, sodium and/or ammonium nitrate (TM 5-1300, 1990). All of these fall into the general category of thermal explosions.

Mendis et.al. state that “detonation of a condensed high explosive generates hot gases under pressure up to 300 kilo bar and a temperature of about 3000-4000C°. The hot gas expands forcing out the volume it occupies. As a consequence, a layer of compressed air (blast wave) forms in front of this gas volume containing most of the energy released by the explosion.” Blast wave instantaneously increases to a value of pressure above the ambient atmospheric pressure and the speed of the wave can exceed the speed of sound. In blast literature it is referred to as the **side-on overpressure** or **peak over pressure**, which is an indicator of the intensity of blast, that decays as the shock wave expands outward from the explosion source. After a short time, the pressure behind the front may drop below the ambient pressure as seen in Figure 4. During such a negative phase, a partial vacuum is created and air is sucked in. This is also accompanied by high suction winds that carry the debris for long distances away from the explosion source (Mendis, Gupta, and Ramsay, 2007).

Mendis, Gupta & Ramsay defines basic properties of a material called explosive material as:

1. An explosive “must contain a substance or mixture of substances that remains unchanged under ordinary conditions, but undergoes a fast chemical change upon stimulation.”
2. Explosion resulting reaction “must yield gases whose volume—under normal pressure, but at the high temperature resulting from an explosion—is much greater than that of the original substance.”
3. The change must be exothermic in order to heat the products of the reaction and thus to increase their pressure.”

Figure 2: Blast Wave Propagation.
(from work of Mendis, Gupta, Ramsay and Ngo, 2007)

At this stage it would be beneficial to be aware of following concepts and be able to simply summarize them for the discussion that will be developed. Therefore some important concepts that would be helpful for further arguments with explosion event and blast phenomenon which are based on the book by Smith and Hetherington (1994) are as follows:

Chemical Explosive: A compound or mixture which, upon the application of heat or shock, decomposes or rearranges with extreme rapidity, yielding much gas and heat. Many substances not ordinarily classed as explosives may do one, or even two, of these things. For example, a mixture of nitrogen and oxygen can be made to react with great rapidity and yield the gaseous product nitric oxide; yet the mixture is not an explosive since it does not evolve heat, but rather absorbs heat. For a chemical to be an explosive, it must exhibit all of the following:

1. **Formation of Gases:** Gases may be evolved from substances in a variety of ways. When the wood or coal is pulverized, so that the total surface in contact with the oxygen is increased, and burned in a furnace or forge where more air can be supplied, the burning can be made more rapid and the combustion more complete. When the wood or coal is immersed in liquid oxygen or suspended in air in the form of dust, the burning takes place with explosive violence. In each case, the same action occurs: a burning combustible forms a gas.

2. **Evolution of Heat:** The generation of heat in large quantities accompanies every explosive chemical reaction. This rapid liberation of heat that causes the gaseous products of reaction to expand and generate high pressures. This rapid generation of high pressures of the released gas constitutes the explosion. It should be noted that the liberation of heat with insufficient rapidity will not cause an explosion.

3. **Rapidity of Reaction:** Rapidity of reaction distinguishes the explosive reaction from an ordinary combustion reaction by the great speed with which it takes place. Unless the reaction occurs rapidly, the thermally expanded gases will be dissipated in the medium, and there will be no explosion. Again, consider a wood or coal fire. As

the fire burns, there is the evolution of heat and the formation of gases, but neither is liberated rapidly enough to cause an explosion.

4. **Initiation of Reaction:** A reaction must be capable of being initiated by the application of shock or heat to a small portion of the mass of the explosive material. A material in which the first three factors exist cannot be accepted as an explosive unless the reaction can be made to occur when desired.

2.2 EXPLOSIVE AIR BLAST LOADING

The threat for a conventional bomb is defined by two equally important elements, the bomb size, or charge weight *W*, and the standoff distance *R* between the blast source and the target as seen on Figure 3 (Longinow, 2003). For example, the blast occurred at the basement of World Trade Centre in 1993 has the charge weight of 816.5 kg TNT. The Oklahoma bomb in 1995 has a charge weight of 1814 kg at a stand off of 4.5m. As terrorist attacks may range from the small letter bomb to the gigantic truck bomb as experienced in Oklahoma City, the mechanics of a conventional explosion and their effects on a target must be addressed. The observed characteristics of air blast waves are found to be affected by the physical properties of the explosion source. Figure 3 shows a typical blast pressure profile. At the arrival time t_A, following the explosion, pressure at that position suddenly increases to a peak value of overpressure, P_{so}, over the ambient pressure, P_o. The pressure then decays to ambient level at time t_d, then decays further to an under pressure P_{so-} (creating a partial vacuum) before eventually returning to ambient conditions at time $t_d + t_{d-}$. The quantity P_{so} is usually referred to as the peak side-on overpressure, incident peak overpressure or merely peak overpressure (TM 5-1300, 1990). Smith and Hetherington (1994) states that, incident peak over pressures P_{so} are amplified by a reflection factor as the shock wave encounters an object or structure in its path. Except for specific focusing of high intensity shock waves at near 45° incidence, these reflection factors are typically greatest for normal incidence (a surface adjacent and perpendicular to the source) and diminish with the angle of obliquity or angular position relative to the source. “Reflection factors depend on the intensity of the shock wave, and for large explosives at normal incidence these reflection factors **may enhance the incident pressures by as much as an order of magnitude**

(Mendis, Gupta, Ramsay and Ngo)." Throughout the pressure-time profile, two main phases can be observed; portion above ambient is called positive phase of duration t_d, while that below ambient is called negative phase of duration, t_{d-}. The negative phase is of a longer duration and a lower intensity than the positive duration. As the stand-off distance increases, the duration of the positive-phase blast wave increases resulting in a lower-amplitude, longer-duration shock pulse. Charges positioned extremely close to a target structure impose a highly impulsive, high intensity pressure load over a localized region of the structure; charges positioned further away produce a lower-intensity, longer-duration uniform pressure distribution over the entire structure. Eventually, the entire structure is surrounded in the shock wave, with reflection and diffraction effects creating focusing and shadow zones in a complex pattern around the structure. Negative phase is the phase known to cause the weakened structure is subjected to impact by debris that may cause additional damage to property and life (Smith and Hetherington, 1994).

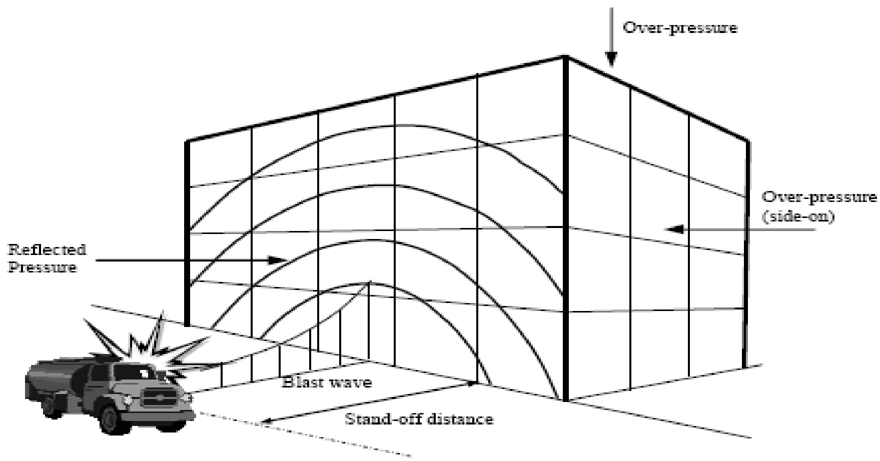


Figure 3: Blast loading on a building.
(From work of Mendis, Gupta, Ramsay and Ngo, 2007)

If the exterior building walls are capable of resisting the blast load, the shock front penetrates through window and door openings, subjecting the floors, ceilings, walls, contents, and people to sudden pressures and fragments from shattered windows, doors, etc. Building components not capable of resisting the blast wave will fracture and be further fragmented and moved by the dynamic effect of the blast pressure that immediately follows the shock front. Building contents and people will be displaced in the direction of blast wave propagation. In this manner the blast will propagate through the building.

2.2.1 Blast Wave Scaling Laws

Smith and Hetherington (1994) mention that all blast parameters are primarily dependent on the amount of energy released by a detonation in the form of a blast wave and the distance from the explosion. According to commonly accepted standard TM 5-1300,1990, a universal normalized description of the blast effects can be given by scaling distance relative to $(E/P_o)^{1/3}$ and scaling pressure relative to P_o, where E is the energy release (kJ) and P_o the ambient pressure (typically 1 atmosphere or 101.3 kN/m2). For convenience, however, it is general practice to express the basic explosive input or charge weight W as an equivalent mass of TNT. This is due to the fact that, blast science is first evolved with inventing of TNT. Therefore, blast effects of TNT have been very well studied. All other explosives are compared to TNT. Even nuclear explosions are rated in terms of TNT equivalents. (Longinow, 2003)

Results are then given as a function of the dimensional distance parameter (scaled distance) $Z = R/W_{1/3}$, where R is the actual effective distance from the explosion. W is generally expressed in kilograms. Scaling laws provide parametric correlations between a particular explosion and a standard charge of the same substance.

2.2.2 Prediction of Blast Pressure

Blast wave parameters for conventional high explosive materials have been the focus of a number of studies during the 1950's and 1960's following the World War II by scientists such as Baker and Brode. Based on Mendis et.al. evolution of such equations is summarized as follows. Estimations of peak overpressure due to spherical blast based on scaled distance $Z = R/W_{1/3}$ was introduced by Brode (1955) as:

$$P_{s0} = \frac{6.7}{Z^3} + 1 \text{ bar } (Pso \rangle 10 \text{ bar}) \tag{2}$$

$$P_{s0} = \frac{0.975}{Z} + \frac{1.455}{Z^2} + \frac{5.85}{Z^3} - 0.019 \text{ bar } (0.1 \text{ bar} \langle Pso \langle 10 \text{ bar}) \tag{3}$$

On 1961 Newmark and Hansen introduced a relationship to calculate the maximum blast overpressure, P_{so}, in bars, for a high explosive charge detonates at the ground surface as:

$$P_{s0} = 6784 \frac{W}{R^3} + 93 \left(\frac{W}{R^3} \right)^{\frac{1}{2}} \tag{4}$$

Another expression of the peak overpressure in kPa is introduced by Mills on 1987, in which *W* is expressed as the equivalent charge weight in kilo-grams of TNT, and *Z* is the scaled distance:

$$P_{s0} = \frac{1772}{Z^3} + \frac{114}{Z^2} + \frac{108}{Z} \tag{5}$$

As stated by Smith and Hetherington (1994), as the blast wave propagates through the atmosphere, the air behind the shock front is moving outward at lower velocity. The velocity of the air particles, and hence the wind pressure, depends on the peak overpressure of the blast wave. This later velocity of the air is associated with the dynamic pressure, q (t). The maximum value, q_s, is given by TM 5-1300, 1990 as:

$$q_s = 5 p_{s0}^2 / 2(p_{s0} + 7 p_0) \tag{6}$$

If the blast wave encounters an obstacle perpendicular to the direction of propagation, reflection increases the overpressure to a maximum reflected pressure P_r *as*:

$$P_r = 2 P_{s0} \left\{ \frac{7 P_0 + 4 P_{s0}}{7 P_{s0} + P_{so}} \right\} \tag{7}$$

A full discussion and extensive charts for predicting blast pressures and blast durations are given by TM5-1300 (1990). Some representative numerical values of peak reflected overpressure are given in Table 1. (Mendis, Gupta, Ramsay, 2007)

Table 1: Peak reflected overpressures *Pr* (in MPa) with different *W*-*R* combinations. (from work of Mendis et.al., 2007)

R \ *W*	100 kg TNT	500 kg TNT	1000 kg TNT	2000 kg TNT
1m	165.8	354.5	464.5	602.9
2.5m	34.2	89.4	130.8	188.4
5m	6.65	24.8	39.5	60.19
10m	0.85	4.25	8.15	14.7
15m	0.27	1.25	2.53	5.01
20m	0.14	0.54	1.06	2.13
25m	0.09	0.29	0.55	1.08
30m	0.06	0.19	0.33	0.63

Mendis et. al. idealizes the reflected overpressure for design purposes as an equivalent triangular pulse of maximum peak pressure P_r and time duration t_d, yielding the reflected impulse.

$$i_r = \frac{1}{2} \mathrm{P}_r t_d \tag{8}$$

Duration t_d is related directly to the time taken for the overpressure to be dissipated. Overpressure arising from wave reflection dissipates as the perturbation propagates to the edges of the obstacle at a velocity related to the speed of sound (U_s) in the compressed and heated air behind the wave front. Denoting the maximum distance from an edge as *S* (for example, the lesser of the height or half the width of a conventional building), the additional pressure due to reflection is considered to reduce from $P_r - P_{so}$ to zero in time $3S/U_s$. Conservatively, U_s can be taken as the normal speed of sound, which is about 340 m/s, and the additional impulse to the structure evaluated on the assumption of a linear decay.

After the blast wave has passed the rear corner of a prismatic obstacle, the pressure similarly propagates on to the rear face; linear build up over duration $5S/U_s$ has been suggested by TM 5-1300, 1990. **For skeletal structures the effective duration of the net overpressure load is thus small, and the drag loading based on the dynamic pressure is then likely to be dominant**. Conventional wind loading pressure coefficients may be used, with the conservative assumption of instantaneous build up when the wave passes the plane of the relevant face of the building, the loads on the front and rear faces being numerically cumulative for the overall load effect on the structure. Among many formulations proposed for the rate of decay of the dynamic pressure loading; a parabolic decay (i.e. corresponding to a linear decay of equivalent wind velocity) over a time equal to the total duration of positive overpressure is a practical and widely accepted approximation suggested by TM 5-1300, 1990.

Finally following equation relates the distance from the point of a ground-level explosion to peak overpressure.

$$X = M^{1/3} \exp[3.5031 - 0.7241\ln(P) + 0.0398 (\ln(P))^2] \quad (9)$$

here, X = Distance in feet to a given overpressure P

M = TNT equivalent mass, lbs

P = overpressure, psi

This equation by Lees, F.,1980 is valid for an explosion at ground level at 20°C ignoring any redirection of the overpressure by structures and terrain. If the explosion occurred up in the air (unconfined in all directions), the distance X would be reduced by a factor of 1.26.

2.2.3 Shock wave basics

As introduced in the previous sections rapid expansion of hot gases resulting from the detonation of an explosive charge gives rise to a compression wave called a **shock wave**, which propagates through the air. The front of the shock wave can be considered infinitely steep, for all practical purposes. This is explained as the time required for compression of the undisturbed air just ahead of the wave to full pressure just behind the wave is essentially zero.

Propagation pattern of the shock wave is generally dependent on the explosive source. If the explosive source is spherical, the resulting shock wave will be spherical. Since its surface is continually increasing, the energy per unit area continually decreases. Consequently, as the shock wave travels outward from the charge, the pressure in the front of the wave, named the **peak pressure**, steadily decreases. At great distances from the charge, the peak pressure is infinitesimal, and the wave can be treated as a sound wave (Longinow, 2003).

Behind the shock wave front, the pressure in the wave decreases from its initial peak value. At some distance from the charge, the pressure behind the shock front falls to a value below that of the atmosphere and then rises again to a steady value equal to that of the atmosphere. The part of the shock wave in which the pressure is greater than that of the atmosphere is called the **positive phase** and, immediately following it, the part in which the pressure is less than that of the atmosphere is called the **negative or suction phase**. (Figure 4) Pressure and impulse effect resulting from a blast for center of the target structure are as given by figure 5 for pressure effect and by figure 6 for impulse effect. These figures demonstrate vanishing time and pattern of pressure and impulse loading on a structure.

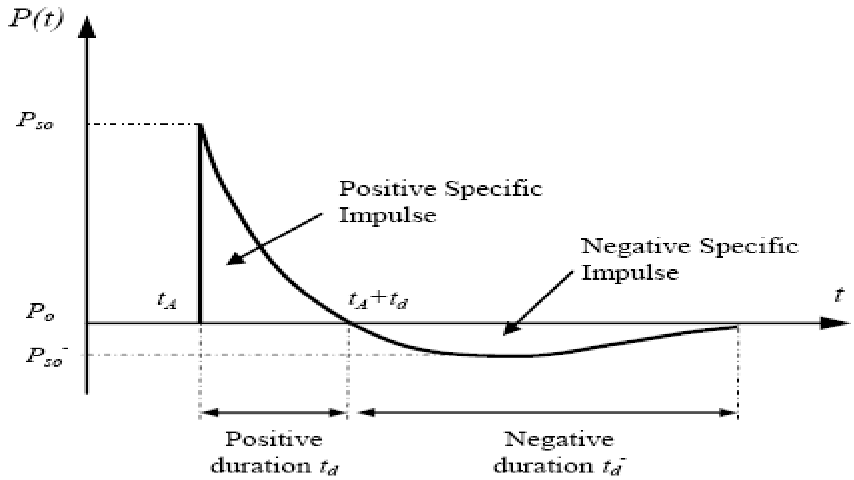


Figure 4: Blast wave pressure – Time History.

(from work of Krauthammer, 2003)

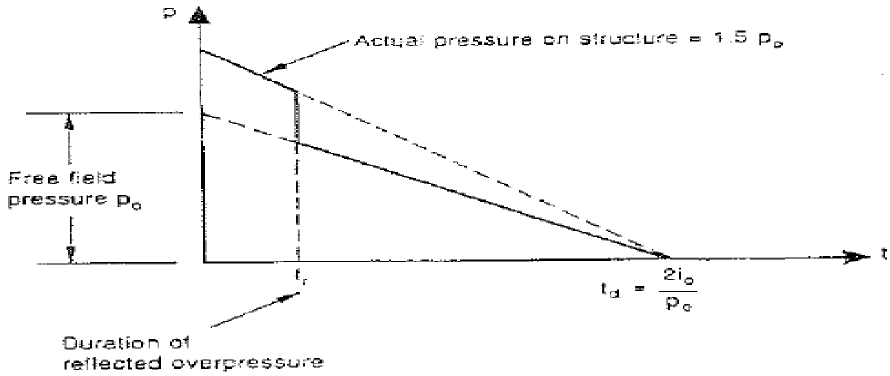


Figure 5: Transient Pressure at the center of the target.

(from work of Smith and Hetherington, 1994)

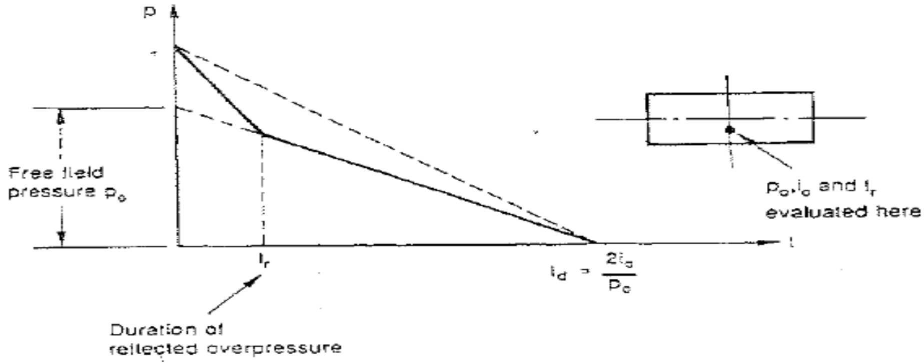


Figure 6: Transient Pulse at the center of the target.

(from work of Smith and Hetherington, 1994)

2.2.4 Seismic and Blast Effects on Structures

Before any word describing the difference between blast and seismic loads it can be said that, blast loading is very different than earthquake loading. The first difference is in the way a given structure is loaded. In the case of an earthquake the structure is subject to ground motions that shake the structure from the ground (base or foundation). In the case of an explosion produced by an air or a surface burst, the structure is loaded by means of a compression wave (shock wave) over some area. Since a portion of the blast energy is coupled into the ground, the structure is also subject to ground motions similar to an earthquake, though **much less intense**.

A second difference is the duration of loading (rate of loading). For earthquakes, the duration of induced motions (shaking) can range from seconds to minutes. Additional loadings are produced by “aftershocks,” which are generally less intense than the initial shaking. For conventional explosives, the duration of a pressure wave is on the order of milliseconds.

For example, in the Oklahoma City event in 1990, the yield of the weapon was approximately 1815 kg or 4,000 lb TNT equivalent. The truck containing the explosive was positioned about 3.05m (10’) from the building. The peak pressure at the face of the buildings was about 13.1 MPa (1,900 psi), and the duration of the positive phase of the pulse was approximately 3 ms. Forensic judgment by Mlakar et. al. about the size of the crater asserted that, a fair portion of the energy coupled into the ground, producing ground shock. However, judging by the damage, clearly air blast was the primary damage mechanism. Further, earthquakes shake an entire building, but produce mostly horizontal loads at floor-slab levels, concentrating in the specially designed, laterally stiffer structural systems. Blast usually does not attack the entire structure uniformly, but produces the most severe loads to the nearest structural elements, both vertical and horizontal, with little regard to their stiffness. Uplift pressure load on floors is also a specific blast effect.

2.3 MATERIAL BEHAVIOR AT HIGH STRAIN-RATE

Blast loads typically produce very high strain rates in the range of $10^2 – 10^4$ s^{-1}. This high loading rate would alter the dynamic mechanical properties of target structures and, accordingly, the expected damage mechanisms for various structural elements. For reinforced concrete structures subjected to blast effects the strength of concrete and steel reinforcing bars can increase significantly due to strain rate effects. Figure 7 shows the approximate ranges of the expected strain rates for different loading conditions. It can be seen that ordinary static strain rate is located in the range: 10^{-6}-10^{-5} s^{-1}, while blast pressures normally yield loads associated with strain rates in the range: $10^2 – 10^4\ s^{-1}$.

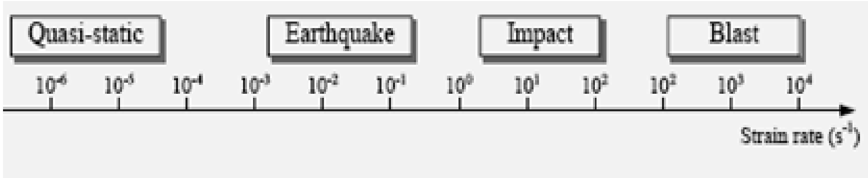


Figure 7: Strain rates associated with different types of loading.
(from work of Mendis, Gupta, Ramsay and Ngo, 2007)

2.3.1 Dynamic Properties of Reinforcing Steel under High-Strain Rates

Due to the isotropic properties of metallic materials, their elastic and inelastic response to dynamic loading can easily be monitored and assessed. Norris et al. (1959) tested steel with two different static yield strength of 330 and 278 MPa under tension at strain rates ranging from 10^{-5} to 0.1 s^{-1}. Strength increase of 9 - 21% and 10 - 23 % were observed for the two steel types, respectively. Mendis et.al summarizes other research works as; "Harding (1967) conducted tensile experiments using the tensile version of Split Hopkinton's Pressure Bar (SHPB) on mild steel using strain rates varying between 10^{-3} s^{-1} and 2000 s^{-1}. It was concluded from this test series that materials of body-centered cubic (BCC) structure (such as mild steel) showed the greatest strain rate sensitivity." Mendis et.al states, It has been found that the lower yield strength of mild steel can almost be doubled; the ultimate tensile strength can be increased by about 50%; and the upper yield strength can be considerably higher. In contrast, the ultimate tensile strain decreases with increasing strain rate. Malvar (1998) also studied strength enhancement of steel reinforcing bars under the effect of high strain rates. This was described in terms of the dynamic increase factor (DIF), which can be evaluated for different steel grades and for yield stresses, f_y, ranging from 290 to 710 MPa as represented by equation:

$DIF = \left(\frac{\dot{\varepsilon}}{10^{-4}} \right)^{\alpha}$ where for calculating yield stress $\alpha = \alpha_{fy}$,

$$\alpha_{fy} = 0.074 - 0.04\left(f_y / 414 \right) \tag{10}$$

for ultimate stress calculation $\alpha = \alpha_{fu}$,

$$\alpha_{fu} = 0.019 - 0.009\left(f_y / 414 \right)$$

2.3.2 Modeling of Strain Rate Effect

The mechanical properties of steel are affected noticeably by the rate at which strain takes place. If the mechanical properties under static loading are considered as a basis, the effects of increasing strain rate can be illustrated in Fig. 8 and is summarized by Liew and Chen based on Yandzio and Gough as follows:

1. The yield point increases substantially to a dynamic yield stress value;
2. The elastic modulus generally does not change in dependence with the loading rate;
3. The ultimate tensile strength increases slightly, however the percentage increase is less than that for the yield stress; and
4. The elongation at rupture either remains unchanged or is slightly reduced due to the increased strain rate.

In the present formulation, the rate-dependent plasticity is based on the model proposed by Perzyna (1968).

$$\sigma_y' = \left[1 + \left(\frac{\varepsilon'^{p1}}{\gamma}\right)^m\right]\sigma_y \tag{11}$$

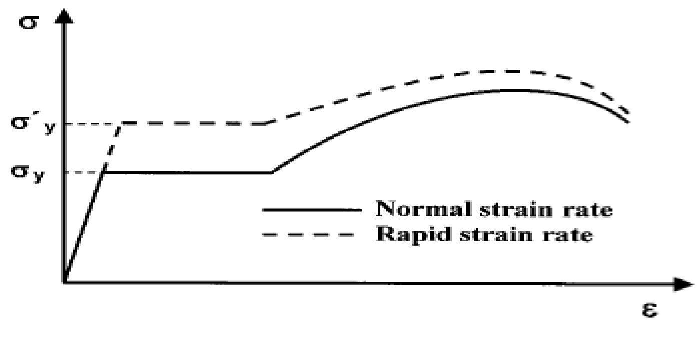


Figure 8: Strain rate effect on structural steel.
(from work of Liew and Chen, 2004)

Where σ_y' yield stress considering strain rate effect;

ε'^{p1} : Equivalent plastic strain rate;

m : strain rate hardening parameter

γ : viscosity parameter; and

σ_y : static yield stress.

It is noted that $\sigma_y^{'}$ is a function of some hardening parameters in general. When $\varepsilon^{'p1}$ tends to zero under very slow rate loading or γ tends to ∞, the solution converges to the static (rate-independent) solution. The suggested values of *m* and γ for mild steel are m =0.2 and γ=40 s^{-1} (Bodner and Symonds 1960; Izzuddin and Fang 1977)

2.4 STRUCTURAL RESPONSE TO BLAST LOADING

Blast phenomenon is a complex issue as presented so far since; it involves much kind of explosives, interacting with the peripheral conditions. Many approaches are developed throughout time to predict expected damage to a structure, some are much analytical and some are more empirical. One of these fast and empirical ways of predicting possible damage to a structure is by means of relating overpressure (incident pressure) to the damage level regardless of the distance to the structure and effect of reflection. By making use of such a method expected damage that is expected to occur for a given overpressure is predicted as in table 2.

Table 2: Explosion Overpressure Damage Estimates. (from work of Longinow, 2003)

Overpressure, psi	Expected Damage
0.04	Very loud noise (143 dB); sonic boom glass failures
0.1	Breakage of small windows under strain
0.15	Typical pressure of glass failure
0.30	10% of windows broken
0.5	Windows shattered, limited minor damage to house structures
0.7	Upper limit for reversible effects on humans
1.0	Partial demolition of houses; corrugated metal panels fail and buckle; skin lacerations from flying glass
2.0	Partial collapse of walls and roofs of houses
2.4	Eardrum rupture of exposed populations
2.5	Threshold for significant human lethality

Table 2: Cont'd.

3.0	Steel frame building distorted and pulled away from foundation
5.0	Wooden utility poles snapped
10	Probable total building collapse. Lungs hemorrhage
20	Total destruction. 99% fatality due to direct blast effects

The Bureau of Alcohol, Tobacco and Firearms (ATF) has published Lethal Air Blast Range and Minimum Evacuation Distance values for vehicles carrying explosives as in a terrorist threat. Table 3 compares these distances with the overpressure formula listed above, assuming that the explosive is TNT or equivalent. A possible explosive used by a terrorist is ANFO, prepared by soaking ammonium nitrate prills in fuel oil (94% ammonium nitrate, 6% fuel oil) and detonated by a high explosive booster or a blasting cap. ANFO has an explosive power (by weight) approaching that of TNT, or even greater if the ANFO is enhanced with aluminum powder.

Table 3: Comparison of Formula Calculations with ATF Distances for Vehicles Carrying Explosives. (From work of Longinow, 2003)

Vehicle	Explosive Capacity, lbs	ATF Lethal Air Blast Range, ft.	Equation calc. At P = 3 psi	ATF Minimum Evacuation. Dist, ft.	Equation calc. At P = 0.12 psi.
Compact Sedan	500	100	125	1500	1464
Full Size Sedan	1000	125	157	1750	1840
Cargo Van	4000	200	250	2750	2928
14-ft Box Van	10000	300	339	3750	3974
Fuel Truck	30000	450	489	6500	5753
Semi-Trailer	60000	600	615	7000	7220

Longinow, 2003 interprets pressure values based on explosive research as; at P = 0.15 psi, glass failure may occur. At 0.3 psi, 10% of the windows in buildings may be broken. The upper limit for reversible effects on humans is at P = 0.7 psi. At P = 2.4 psi, eardrum rupture may occur. P= 2.5 to 10 and higher is in the range of lethality to humans. At P = 3 psi, a steel frame building may become distorted and pulled away from its foundation. At P = 10 psi, there will be probable total building destruction. There are differences of opinion in the literature as to what overpressure should be used for a Protection Action Distance. The 0.12 psi number is suggested based on the ATF information.

But as it is obvious from above discussions blast loading structure interaction is not as simple as listed in above tables and accepting above approaches as main guidance may lead to wrong results. Complexity in analyzing the dynamic response of blast loaded structures involves the effect of high strain rates, the non-linear inelastic material behavior, the uncertainties of blast load calculations and the time dependent deformations. Therefore, to simplify the analysis, a number of assumptions related to the response of structures and the loads has been proposed and widely accepted. To establish the principles of this analysis, the structure is idealized as a single degree of freedom (SDOF) system and the link between the positive duration of the blast load and the natural period of vibration of the structure is established by usual manner as in the dynamic analysis applications. This leads to blast load idealization and simplifies the classification of the blast loading schemes.

2.4.1 Elastic SDOF Systems

As Mendis et.al. states that, the simplest idealization of dynamic action of blast loading problem is by means of the SDOF approach. The actual structure can be replaced by an equivalent system of one concentrated mass and one weightless spring representing the resistance of the structure against deformation. Such an idealization is illustrated in Figure 9. In this approach structural mass, M, is under the effect of an external force, $F(t)$, and the structural resistance, R, is expressed in terms of the vertical displacement, y, and the spring constant, K. The blast load can also be idealized as a triangular pulse having a peak force F_m and positive phase duration t_d (Figure 9). The forcing function is given as based on TM 5-1300,1990

$$F(t) = F_m\left(1 - \frac{t}{t_d}\right) \tag{12}$$

The blast impulse is approximated as the area under the force-time curve, and is given by

$$I = \frac{1}{2}F_m t_d \tag{13}$$

The equation of motion of the undamped elastic SDOF system for a time ranging from 0 to the positive phase duration, t_d, is given by Biggs (1964) as

$$M\ddot{y} + Ky = F_m\left(1 - \frac{t}{t_d}\right) \tag{14}$$

The general solution can be expressed as:

Displacement

$$y(t) = \frac{F_m}{K}(1 - \cos\omega t) + \frac{F_m}{Kt_d}\left(\frac{\sin\omega t}{\omega} - t\right) \tag{15}$$

Velocity

$$\dot{y}(t) = \frac{dy}{dt} = \frac{F_m}{K}\left[\omega\sin\omega t + \frac{1}{t_d}(\cos\omega t - 1)\right] \tag{16}$$

In which ω is the natural circular frequency of vibration of the structure and *T* is the natural period of vibration of the structure which is given by equation.

$$\omega = \frac{2\pi}{T} = \sqrt{\frac{K}{M}} \tag{17}$$

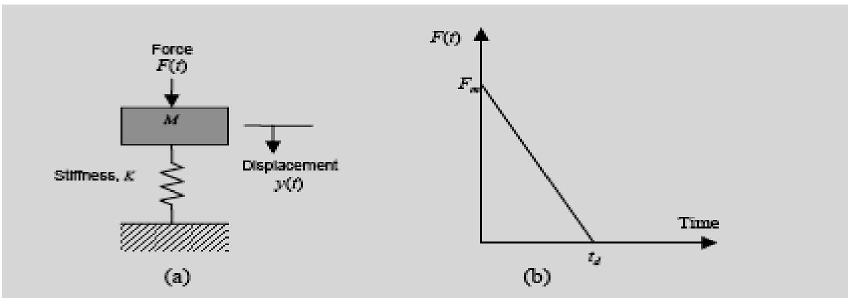


Figure 9: (a) SDOF system and (b) blast loading.

(From Book: Blast and Ballistic Loading of Structures Smith & Hetherington, 1994)

The maximum response is defined by the maximum dynamic deflection y_m which occurs at time t_m. The maximum dynamic deflection y_m can be evaluated by setting dy/dt in Equation 16 equal to zero, i.e. when the structural velocity is zero. The dynamic load factor, DLF, is defined as the ratio of the maximum dynamic deflection y_m to the static deflection y_{st} which would have resulted from the static application of the peak load F_m, which is shown as follows:

$$DLF = \frac{y_{\max}}{y_{st}} = \frac{y_{\max}}{F_m / K} = \psi(\omega t_d) = \Psi\left(\frac{t_d}{T}\right) \tag{18}$$

The structural response to blast loading is significantly influenced by the ratio t_d/T or ωdt (t_d/T =ω dt/2π). Three loading regimes are categorized as follows:

$\omega t_d \langle 0.4$: impulsive loading regime.

$\omega t_d \rangle 40$: quasi-static loading regime.

$0.4 \langle \omega t_d \langle 40$: dynamic loading regime.

2.4.2 Elasto-Plastic SDOF Systems

Structural elements are expected to undergo large inelastic deformation under blast load or high velocity impact. Exact analysis of dynamic response is then only possible step-by-step numerical solution requiring nonlinear dynamic finite element software. However, the degree of uncertainty in both the determination of the loading and the interpretation of acceptability of the resulting deformation is such that solution of an assumed equivalent ideal elastoplastic SDOF system proposed by Biggs is commonly used. Interpretation is based on the required ductility factor μ = y_m/y_e (Figure 10).

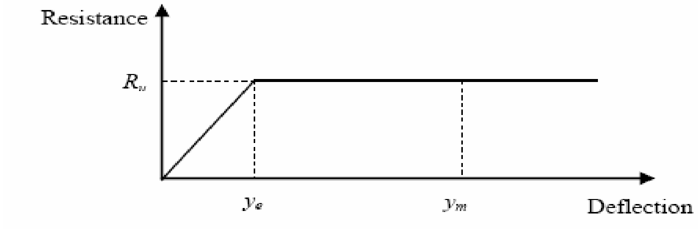


Figure 10: Simplified resistance function of an elastoplastic SDOF system.
(from work of Mendis, Gupta, Ramsay and Ngo, 2007)

For example, a uniform simply supported beam has first mode shape $\varphi(x) = \sin \pi x/L$ and the equivalent mass $M = (1/2)\ mL$, where L is the span of the beam and m is mass per unit length. The equivalent force corresponding to a uniformly distributed load of intensity p is $F = (2/\pi)pL$. The response of the ideal bilinear elastoplastic system can be evaluated in closed form for the triangular load pulse of immediate rise and linear decay, with maximum value F_m and duration t_d. The result for the maximum displacement is generally presented in chart form (TM 5-1300,1990), as a family of curves for selected values of R_u/F_m showing the required ductility μ as a function of t_d/T, in which R_u is the structural resistance of the beam and T is the natural period (Figure 11).

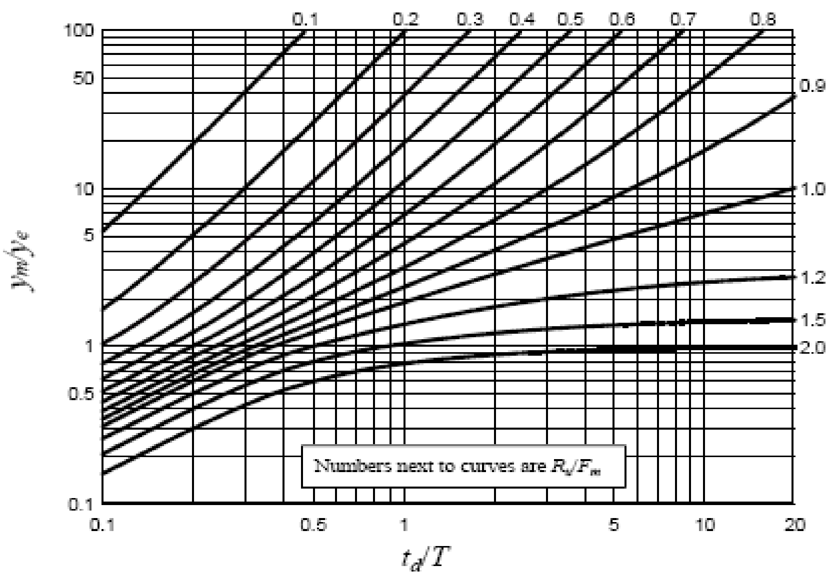


Figure 11: Maximum response of elastoplastic SDF system to a triangular load. (from work of Mendis, Gupta, Ramsay and Ngo, 2007)

2.5 BLAST WAVE-STRUCTURE INTERACTION

Blast loading and structure interaction is actually a very complex phenomenon that requires many issues to be investigated for the ordinary analysis. Blast loads can excite higher structural modes that are usually neglected for other types of hazards. An example of this phenomenon is the vibration of W-section flanges (Rittenhouse et. al.).

The structural behavior of an object or structure exposed to such blast wave may be analyzed by dealing with two main issues. Firstly, blast-loading effects, i.e., forces that are resulted directly from the action of the blast pressure; secondly, the structural response, or the expected damage criteria associated with such loading effects. For this purpose it is possible to consider some equivalent simplified geometry. Accordingly, in analyzing the dynamic response to blast loading, Hetherington and Smith classifies target structures in two types: diffraction type and drag type structures. As these names imply, the former would be affected mainly by diffraction (engulfing) loading and the latter by drag loading. It should be emphasized that actual buildings will respond to both types of loading (Krauthammer, 2003) and the distinction is made primarily to simplify the analysis. The structural response will depend upon the size, shape and weight of the target, how firmly it is attached to the ground, and also on the existence of openings in each face of the structure. Above ground or shallow-buried structures can be subjected to ground shock resulting from the detonation of explosive charges that are on or close to ground surface. The energy imparted to the ground by the explosion is the main source of ground shock. A part of this energy is directly transmitted through the ground as directly-induced ground shock, while part is transmitted through the air as air-induced ground shock. Air-induced ground shock results when the air-blast wave compresses the ground surface and sends a stress pulse into the ground layers. Generally, motion due to air-induced ground shock is maximum at the ground surface and attenuates with depth (TM 5-1300, 1990). The direct shock results from the direct transmission of explosive energy through the ground. For a point of interest on the ground surface, the net experienced ground shock results from a combination of both the air-induced and direct shocks.

2.5.1 Loads from Direct Ground Shock

As a result of the direct transmission of the explosion energy, the ground surface experiences bi-directional vibration motions. Some empirical equations were derived (TM 5-1300, 1990) to predict the direct-induced ground motions in three different ground media; dry soil, saturated soil and rock media. The peak vertical displacement in m/s at the ground surface for rock, D_{rockV} and dry soil, D_{soilV} are given by TM 5-1300 as:

$$D_{Vrock} = \frac{0.25R^{\frac{1}{3}}W^{\frac{1}{3}}}{Z^{\frac{1}{3}}} \tag{19}$$

$$D_{Vsoil} = \frac{0.17R^{\frac{1}{3}}W^{\frac{1}{3}}}{Z^{2.3}} \tag{20}$$

The maximum vertical acceleration, A_v,(m/s^2) for all ground media is given by

$$A_V = \frac{1000}{W^{\frac{1}{8}}Z^2} \tag{21}$$

Above discussion on blast induced ground shock and loading are to give and idea to the reader about magnitude, duration and frequency of this kind of loading. Since earthquake ground shaking is much more powerful than blast loading from ground shock structures designed to sustain earthquake resistance requirements has already withstands ground induced blast loading, meaning direct ground shake has no significant effect on the structure.

2.5.2 Loads from Air-induced Ground Shock

In order that a prediction of a structure's response to ground shock can be made, assumptions have to be made about the transient pressure pulse on the structure. As with blast waves, the pressure experienced by the structure during reflection is greater than the free field pressure. Army Technical Manual TM 5-855-1 suggests that free field pressure values are multiplied by a factor of 1.5 to give reflected overpressures. The time for which the reflected overpressure acts on a particular point P on the structure is determined by the time taken for a tension wave to propagate from a free edge to the point on the structure, thereby relieving the compressive reflected overpressure which is given by:

$$t_r = \frac{1}{c}\left(l_2 + \frac{l_3}{2} - l_1\right) \tag{22}$$

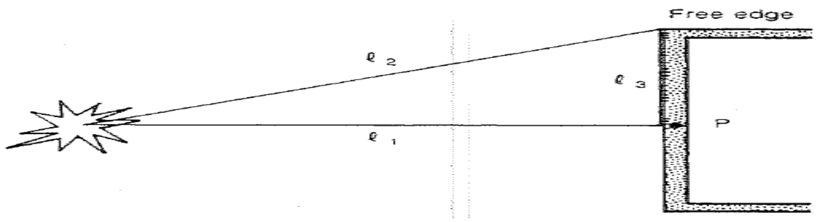


Figure 12: Propagation path lengths for overpressure relief.
(From Book: Blast and Ballistic Loading of Structures Smith & Hetherington, 1994)

Where c is the seismic velocity defined as: $c = \sqrt{\dfrac{E}{\rho}}$

To overcome complications of predicting actual ground motion, one-dimensional wave propagation theory has been employed to quantify the maximum displacement, velocity and acceleration in terms of the already known blast wave parameters (TM 5-1300, 1990). The maximum vertical velocity at the ground surface, V_v, is expressed in terms of the peak incident overpressure, P_{so}, as:

$$V_v = \frac{P_{s0}}{\rho C_p} \tag{23}$$

where ρ and C_p are, respectively, the mass density and the wave seismic velocity in the soil. By integrating the vertical velocity in Equation 22 with time, the maximum vertical displacement at the ground surface, D_v, can be obtained as:

$$D_v = \frac{i_s}{1000 \rho C_p} \tag{24}$$

Accounting for the depth of soil layers, an empirical formula is given by (TM5-1300) to estimate the vertical displacement in meters as:

$$D_v = 0.09 W^{\frac{1}{6}} (H/50)^{0.6} (P_{s0})^{\frac{2}{3}} \tag{25}$$

where W is the explosion yield in 10^9 kg, and H is the depth of the soil layer in meters.

This kind of blast loads are a bit greater than direct ground shock since direct shock is absorbed by soil. As mentioned earthquake ground shaking is much more powerful than blast loading and any building satisfy earthquake resistance requirements already satisfies air induced ground shock loading effect.

2.6 FAILURE MODES OF BLAST-LOADED STRUCTURES

Blast loading effects on structural members may produce both local and global responses associated with different failure modes. The type of structural response depends mainly on the loading rate, the orientation of the target with respect to the direction of the blast wave propagation and boundary conditions. "The general failure modes associated with blast loading can be flexure, direct shear or punching shear. Local responses are characterized by localized bleaching and spalling, and generally result from the close-in effects of explosions, while global responses are typically manifested as flexural failure." (Mendis et.al., 2007)

2.6.1 Global Structural Behavior

According to Mendis et.al. "the essential characteristics of loading and building response for transient loads produced by explosions depend primarily on the relationship between the effective duration of the loading and the fundamental period of the structure on which the loading acts." When the effective duration is very short, for example less than one third of the period, then the impulse due to the transient loading is of major importance, and the response of the structure can be based entirely on a consideration of impulse and momentum. On the other hand, when the duration of the loading is relatively long compared with the fundamental period, then a quasi-static design can be made.

The global response of structural elements is generally a consequence of transverse (out-of-plane) loads with long exposure time (quasi-static loading), and is usually associated with global bending (membrane) and shear responses. Therefore, the global response of above ground reinforced concrete structures subjected to blast loading is referred to as membrane/bending failure. The second global failure mode to be considered is shear failure.(Mendis et. al.) It has been found that under the effect of both static and dynamic loading, four types of shear failure can be identified: diagonal tension, diagonal compression, punching shear, and direct (dynamic) shear. First three shear response mechanisms have relatively minor structural effect in case of blast loading since that require high lateral loads similar to earthquake loading and can be neglected. The fourth type of shear failure which is direct (dynamic) shear failure is primarily associated with transient short duration dynamic loads that result from blast effects, and it depends mainly on the intensity of the pressure waves. The associated shear force is many times higher than the shear force associated with flexural failure modes as is the case in the chapter 5 analysis results heading. The high shear stresses may lead to direct global shear failure and it may occur very early (within a few milliseconds of shock wave arrival to the frontal surface of the structure) which can be prior to any occurrence of significant bending deformations. (Smith and Hetherington, 1994)

2.6.2 Localized Structural Behavior

The close-in effect of explosion may cause localized shear or flexural failure in the closest structural elements. This depends mainly on the distance between the source of the explosion and the target, and the relative strength and ductility of the structural elements. The localized shear failure takes place in the form of localized punching and spalling, which produces low and high-speed fragments. The punching effect is frequently referred to as “bleaching” (Byfield). Bleaching failures are typically accompanied by spalling and scabbing of concrete covers as well as fragments and debris (Figure 13).

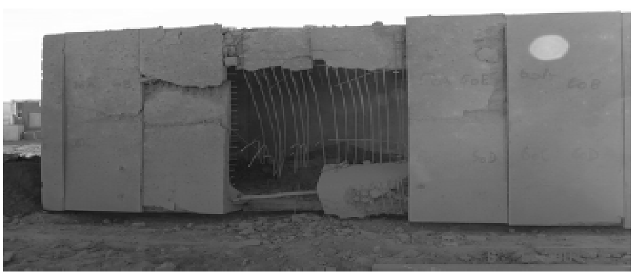


Figure 13: Breaching failure due to a close-in explosion of 6000kg TNT equivalent. (from work of Mendis, Gupta, Ramsay and Ngo, 2007)

2.6.3. Pressure-Impulse (P-I) Diagrams

The pressure-impulse (*P-I*) diagram is an easy way to mathematically relate a specific damage level to a combination of blast pressures and impulses imposed on a particular structural element (Smith and Hetherington, 1994). An example of a P-I diagram is shown in Figure 14 to show levels of damage of a structural member. Region (I) corresponds to severe structural damage and region (II) refers to no or minor damage. There are other *P-I* diagrams that concern with human response to blast in which case there are three categories of blast-induced injury, namely : primary, secondary, and tertiary injury (Smith and Hetherington, 1994).

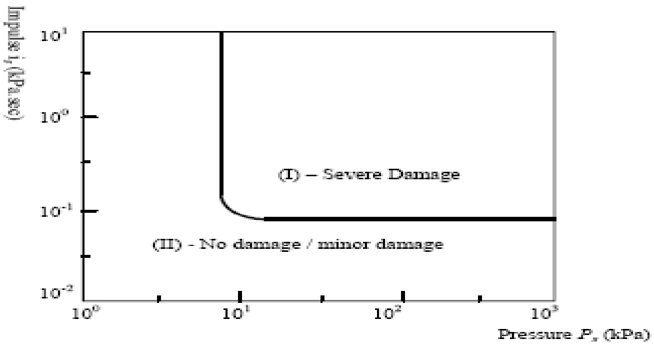


Figure 14: Typical Pressure-impulse (P-I) diagram.
(from work of Mendis, Gupta, Ramsay and Ngo, 2007)

CHAPTER 3

EXPERIENCE LEARNED FROM PAST EVENTS AND STUDIES

3.1 HIGH EXPLOSIVE EFFECTS ON STRUCTURES

As previously mentioned explosives can be categorized as either deflagrating (low) or detonating (high) explosives. Commonly used high explosives include TNT, RDX, and Semtex, all of which have approximately equal yield. Military high explosives produce an instantaneous rise in air pressure, making them particularly effective at fragmenting metal shell casings to produce shrapnel. Terrorists rarely use large quantities of military explosives due to the difficulties of acquisition (with the exception of rebels in Iraq, who have access to large quantities of munitions). Vehicle borne devices often use homemade explosive compounds, such as ammonium nitrate fertilizer based explosives. These homemade compounds detonate and are therefore classified as high explosives. The TNT equivalence is approximately half that of military explosives and the rate at which the shock wave propagates through compounds is slower. (TM 5-1300, 1990) This makes them less efficient for breaking shell casings. Despite this fact, "the violent expansion of hot gases that produce the blast wave is also slower. As pressure time histories from high explosives are often substantially shorter than the natural periods of building components, this slower reaction time can be more effective for imparting enegy into a building's superstructure."(Byfield, 2006)

As mentioned the positive pressure phase of a blast wave is followed by a negative pressure phase, which is a suction state. This suction is of much lower intensity than the positive phase. Despite this the suction on the front face of a building from the pressure phase has been known to cause steelwork connections to fail that would otherwise have survived. (Byfield, 2006)

An important feature of blast waves is that as stated earlier they have ability to reflect off building surfaces (Smith and Hetherington, 1994). This means that they can travel for some distance from sites of explosion. Multiple reflections enhance the destructive capability from an explosion. Blasts in confined spaces (jammed urban districts) can cause extensive structural damage; World Trade Center attack in 1993 is an obvious example to this. (Faschan et.al., 2003) Blasts initiated in open spaces can also produce multiple reflections in reentrant corners of building facades, such as the overhanging floors employed in the Murrah Building in Oklahoma City.

Byfield mentions that a vast amount of data and observations on the performance of buildings subjected to the effects of high explosive bombs were compiled during World War II. The data gathered included 60,000 basic reports on bomb damage, in addition to 5,000 detailed reports on individual damaged structures.

One of the participants in the data gathering process for study of explosive effect on structures was, Lord J. F. Baker (1948) concluded that of the 50 steel framed buildings that he surveyed in detail, "*almost all collapses were the result of inadequate connections between perimeter columns and beams.* Of particular fragility were buildings whose external walls ran in *parallel with the direction of slab span*." In such cases the concrete casing to wall beams was often weakly tied into the floor slabs, leaving the connections between the primary beams and the perimeter columns as the only effective restraint against outward movement of the wall. As mentioned earlier high explosives cause an immediate rise in pressure, which is followed by a negative pressure phase of lower intensity. It was observed that even the relatively low suction pressures from near field events were sufficient to cause widespread failures of these connections, leading to serious floor collapses. Based on his observations Baker recommended that the tying be improved between flooring and wall framing systems. He also recommended strengthening of beam to column connections, which generally failed due to a combination of the prying (force open) action resulting from insufficient ability to accommodate large beam end rotations and tensile loading. Forensic investigations and examination of damaged structures in Hiroshima shortly after the detonation of the atomic bomb, together with subsequent research, demonstrated that the membrane action of flooring systems imparts enormous strength to structures subjected to nuclear blasts. (Walley, 1994)

Especially for near field explosions according to Baker "The effect of the explosion on the building totally depends to large extent on the internal planning". In fill walls are materials of internal planning and may provide load distribution path after a blast to prevent progressive collapse (collapse of all or a large part of a structure accelerated by failure or damage of a relatively small part of it). Concrete infills used in steel frames also provide additional resistance in means of mass which is necessary in case of an explosion. Blast loading imposes extreme loads over very short durations. Unlike conventional loads, the mass of member imparts resistance to load in addition to the conventional structural strength. This inertial response can result in unusual effects. Sometimes, this may create a conflicting situation for earthquake resistance of the building since, additional mass means additional lateral force.

Byfield suggests that concrete framed structures can often sustain significant damage to the perimeter frame without progressive failure which is mainly due to the monolithic nature of the frame providing significant redundancy via combination of three dimensional **vierendeel** actions and bracing from panel walling.

Reinforced concrete prefabricated concrete structures exhibits insufficient resistance to blast effect at their connections, where the reinforcement is lapped. This also presents significant zone of weakness when subjected to the reverse uplift loads from blast. An example of such collapse given by Byfield for this issue is the attack on the Dropping Well Bar in Ballykelly, Northern Ireland in 1982, in which 17 people were killed. The detonation of relatively small amount of explosive contained in hand bag caused precast concrete slab units to become dislodged from their supports, which thereafter crushed occupants in the crowded bar. While tragic, this incident highlights the importance of tying all structural components together regardless of overall structural importance. Dislodging and joint failure also is suggested as an important point to be taken measures by all codes to prevent extreme loading damage for steel buildings.

In the case of Alfred P. Murrah Federal office building located at Oklahoma City one of main reasons causing damage is the absence of structural internal partition walls that substantially limits the ability to redistribute loads. In the absence of these alternative load paths modern multistory buildings are classified as being susceptible to column damage. (Byfield, 2006) The blast in the Murrah Building destroyed three columns located on the front face of the building. These columns supported transfer beams that supported intermediate columns. Thus the framing system adopted is partly responsible for widening the zone of the building that collapsed.

In the bombing of housing complex for U.S. military forces in the Khobar Towers in Dahran, in the Eastern Province of Saudi Arabia where equivalent yield of approximately 9000 kg of TNT was detonated, resulting blast propelled the concrete barriers called “Jersey barriers” into the first four floors of the building, which combined with the blast loading succeeded in destroying the lower precast panels of the façade. As the precast units in the remaining three floors above were left unsupported the entire façade of the building collapsed. This residential building was entirely constructed using closely spaced configuration of precast concrete panels, which were well tied together. Byfield states, that “The multiple lines of closely spaced vertical supports created numerous alternative load paths and formed a structure not to progressively collapse. Therefore, this event demonstrates the importance of load bearing internal partitions in redistributing loads.”

The HSBC headquarters in Istanbul also survived massive truck bomb of 1500 kg of ANFO Explosive (Equivalent TNT weight of 1230 kg- TNT equivalent factor is 0.82) without progressive collapse. This may in part be as result of the high strength and ductility design of the building due to the threat from earthquakes. Similarly, in 1993 vehicle bomb detonated 2 m outside the perimeter columns in the basement of World Trade Center one, did not result in progressive collapse. The steelwork was exceptionally strong and reflected the blast, which caused a collapse of the reinforced concrete structure for a distance of some 100 from the detonation without affecting the global stability of the tower (Robertson, 2005)

3.1.1 Brittle Buildings

The basic theoretical establishment of the probabilistic approach assumes that both loading and resistance can be modeled using the log-normal probability distribution functions. If this theory hold true, then the probability of structural component failing is predictable. As the probability of failure is estimated to be so low by statistical analysis, the consequences of failure need not considered. This seemingly safe approach can create brittle structures because no effort is made to ensure that ductile failure modes govern building performance.

The majority of components in structure will have strength in excess of that assumed during design. In fact it has been shown that the steel-concrete composite beams that are widely used everywhere in modern high-rise steel frame buildings can typically resist twice their design loads, when subjected to large sagging deflections (Byfield, 2006). According to Byfield, 2006, this overstrength can create brittle buildings because the weakest link in load path can become the beam column connections. It is also stated that connection designers do not generally consider the high beam end rotations that would occur in severely overloaded beams, Fig. 15. End rotations create prying action that has been mentioned to lead to bolt fracture. Thus, routine designs often create structures with over strength beams connected together by brittle connections.

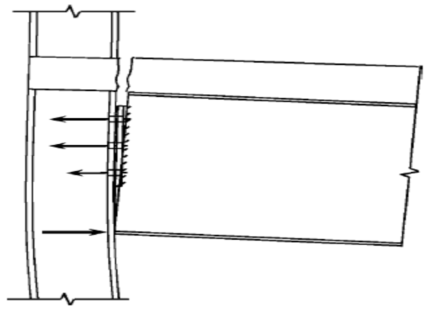


Figure 15: Prying action at steel connections.
(from work of Byfield, 2006)

The important factor in extreme short duration loads is the ability to absorb energy without brittle connection failures. Steelwork beams and columns are particularly good at absorbing energy through plastic deformation.

Solution to the connection failure is inspired from automotive industry. By concentrating on ensuring ductile failures, automotive engineers have been able to significantly reduce the number of road accidents. Crumple zone design has also reduced the weight of vehicles, as designers concentrated on ensuring relatively weak components fail, rather than their connections. Likewise, savings can be made in the volume of steel and concrete used in buildings by moving to similar system (Byfield, 2004). Beams could be designed to resist working loads in the conventional manner. Thereafter, true strength of the beams should be determined, with the connections designed to resist the maximum load transferable from the beam. Further more in situations where terrorist attack is considered threat, strength calculations should be inclusive of impulsive and strain rate effects.

3.1.2 Design of Connections

Careful consideration should also be given to the detailing of connections in order to ensure ductility in addition to ensuring the strength of connections exceeds that of the beams. Connections for steel and concrete framed structures designed to resist seismic loads are likely to have a good ability to resist blast loading. Some examples of details for steel frames subjected to blast are contained in Chapter 5 Part 44, page 96 of the U.S. Departments of the Army, Navy and Air Force code TM 5-1300, 1990. Importantly these details avoid the use of bolts in tension and they concentrate on providing continuity in load paths. Structural grade steels can harden by 50% under the high rates of strain produced during blast. Moreover, it was widely believed that high strains also increased the strength of bolts in tension. The recent analysis of the response of standard structural grade bolts subjected to rapid rates of loading shows this not to be the case. (Munoz-Garcia et al. 2005) The tests revealed that high strain rates cause significant reduction in both tensile strength and ductility, with failure exclusively via thread stripping. Such brittleness under high strain rates is also observed in butt welds. This strain rate weakening combined with strain rate hardening for plate material can be expected to reduce the ductility of joints and lead to brittle failure mechanisms for many popular structural details used in non-seismic regions. Brittleness of bolts can be partially overcome by the use of stainless steel bolts, since Munoz-Garcia et al. have recently shown that stainless steel bolts harden under high rates of strain.

3.2 BEHAVIOR OF STEEL STRUCTURES SUBJECTED TO BLAST LOADING

The local properties of extreme blast loading are unique and different from other major structural hazards in that these loads are, in general, localized; with high magnitude pressures affecting only a relatively small part of the structure. Perhaps the most important local blast loading property that can affect steel structures is the fact that these loads are applied to the structure in a form of pressure. This results in *a direct loading, distributed along the length of the member,* and not *an end loading,* as is the case for gravity or seismic loads.

The temporal properties of the blast loading are that it is highly impulsive; exhibiting high-pressures that last for a very short duration (transient). As such, these loads can excite higher structural modes that are usually neglected for other types of hazards. An example of this phenomenon is the vibration of W-section flanges. (Rittenhouse et. al., 2001)

Most structures are complex in behavior even under static loads, and their response to dynamic loads might include additional complications from combinations of elastic and inelastic vibration modes.

It is reasonable for practical design purposes to adopt approximate methods that permit rapid analysis of complex structures with reasonable accuracy. These methods usually require that both the structure and the loading be idealized to some degree.

Steel Frame Type and Blast Effect

The lateral stability of a moment frame is dependent on the bending stiffness of rigidly connected beams and columns. Adequate diagonal bracing or shear walls at selected locations provide the lateral stability of a braced frame. Elements of lateral stability often are distributed more uniformly in moment frames, in which case each part of the building is more likely to be stable on its own. Therefore, according to McNamara, 2003 moment frames are the better choice for blast-resistant design. In braced frames, the diagonal braces or shear walls can be knocked out by an engulfing blast wave, reducing the effectiveness of the braced frame, unless special features are included to mitigate this potential behavior. Therefore, bracing systems, which could be severely impacted by local blast effects are less robust than uniform moment frames and would be discouraged or combined with uniform moment frames. A perimeter moment frame strengthened on the first level above grade is also recommended. (AISC, www.aisc.com)

3.3 EARTHQUAKE RESISTANT DESIGN, BLAST AND PROGRESSIVE COLLAPSE RELATIONSHIPS

Some engineers suggest that current seismic design provisions, both for new buildings and for strengthened existing buildings, can improve resistance to blast loads and progressive collapse. However, there have been few attempts to quantify such improvement. Hayes et. al. conducted a study on the analysis of the possible relationship between seismic detailing and blast and progressive collapse resistance, under finance of the Federal Emergency Management Agency of the Department of Homeland Security (FEMA) at the U.S. Army Engineer Research and Development Center. This study mainly focused on the specific case; analysis of the Alfred P. Murrah Federal Building, which was severely damaged in a 1995 terrorist attack with an equal charge of 1900 kg TNT in a range of approximately 4.5 meters. First and the main step of seismic strengthening applied in this study were to evaluate the structure for seismic vulnerabilities as if it were located in a **seismically active region**. Three strengthening schemes were then designed for the vulnerabilities found during the evaluation: a pier-spandrel system and a new special concrete moment frame, both for the long and vulnerable side of the building which is the street side, and a set of internal shear walls. In addition to these strengthening schemes, the original ordinary concrete moment frame on the street face of the building was redetailed to bring it into compliance with current building code provisions, without including a lateral load analysis. The three strengthening schemes and redetailed frame were then analyzed for their responses to the same explosion that occurred in 1995. Hayes et. al. states that blast and corresponding progressive collapse analyses proves that the pier-spandrel and special moment frame schemes, as well as the redetailed original system, reduced the degree of direct blast-induced damage and subsequent progressive collapse, compared with the behavior of the original building. However, Internal shear walls, were not as effective in reducing the blast and progressive collapse damage they conclude. A key finding of the study was that strengthening the perimeter elements using current seismic detailing techniques improved the survivability of the building, while strengthening elements internal to the building envelope was not nearly as effective in reducing damage.

Through out study of Hayes et. al. Earthquake structural strengthening schemes are mainly focused on the most seismic region which is the Zone 4 (Corresponding to Earthquake Zone I of Turkish Code which is the most active zone) earthquake resistance according to FEMA provisions. It is stated that the conclusions cannot be directly extrapolated to seismic strengthening for lesser seismic demands (e.g., intermediate moment frames). The need for strength and toughness enhancement was due to the stringent detailing requirements needed to meet the high seismic demands at the assumed site (Zone I, according to TEC). It is also stated that results cannot be directly transferred to other structural systems (e.g., wood buildings, steel moment frames, etc.). Authors of this study warns about general use of earthquake strengthening measures and encourages further analysis while stating that strengthening on the exterior elements and the consideration of preventing the onset of progressive collapse are **likely to be generally applicable**.

Improvements in blast and progressive collapse resistance can result from some well-placed seismic strengthening measures. But one should understand that it does not imply that seismic design details in and of themselves can replace specific measures to mitigate blast and progressive collapse vulnerabilities is also stated in this study. The study suggests that the proper application of current-practice seismic detailing for high-seismicity regions can reduce vulnerability to blast and progressive collapse. Knowledge of this benefit may convince an existing building owner in a high seismic area to take what might otherwise be viewed as only an incremental step in seismic strengthening to consider the added protection against blast and progressive collapse as further justification for performing the strengthening measures. Knowing the difference of resistance provided by different moment resisting frames against blast and progressive collapse one can make double selection.

CHAPTER 4

PROGRESSIVE COLLAPSE: BASICS AND ANALYSIS PROCEDURE

4.1 PROGRESSIVE COLLAPSE

On 16 May 1968, in Newham, east London, a gas explosion that knocked out load-bearing precast concrete panels near the corner of the on the 18th floor of the 22-story Ronan Point apartment tower has occurred. The loss of support at the 18th floor caused the floors above to collapse leading to a chain reaction of collapses all the way to the ground. The ultimate result can be seen in Figure 16: the corner bay of the building has collapsed from top to bottom. Mrs. Ivy Hodge the renter of the house survived but four others died.

While the failure of the Ronan Point structure was not one of the larger building disasters, it was particularly shocking in that the magnitude of the collapse was completely out of proportion to the triggering event. This type of one failure triggering to-another failure (accelerating or precipitating) was labeled “progressive collapse” and afterwards, British standards and regulatory agencies enforced some measures to change the practice of building design to prevent the recurrence of such tragedies.

Figure 16: Ronan Point building after 16 May 1968 collapse.
(from work of Nair, 2003)

4.1.1 Progressive Collapse and Disproportionate Collapse

Nair, 2003 gives the definition of progressive collapse as “collapse of all or a large part of a structure precipitated by failure or damage of a relatively small part of it.” The General Services Administration (GSA, 2003) offers a somewhat more specific description of the phenomenon: “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.”

It has also been suggested that the degree of “progressiveness” in a collapse be defined as the ratio of total collapsed area or volume to the area or volume damaged or destroyed directly by the triggering event. In the case of the Ronan Point collapse, this ratio was of the order of 20 (Nair, 2003).

By any definition, the Ronan Point disaster would qualify as a progressive collapse. In addition to being progressive, the Ronan Point collapse was called "*disproportionate*." A corner of a 22-story building collapsed over its entire height as a result of a fairly modest explosion that did not take the life of a person within a few feet of it. The scale of the collapse was clearly disproportionate to the cause. While the Ronan Point collapse was clearly both progressive and disproportionate, it is instructive to examine other collapses in the same light.

4.1.1.1 Murrah Federal Office Building

The Murrah Federal Office Building in Oklahoma City was destroyed by a bomb on 19 April 1995. The bomb, in a truck at the base of the building, destroyed three columns. Loss of support from these columns led to failure of a transfer girder which supports intermediate columns and thus floor areas supported by those columns. The result was the general collapse event seen in Figure 17.

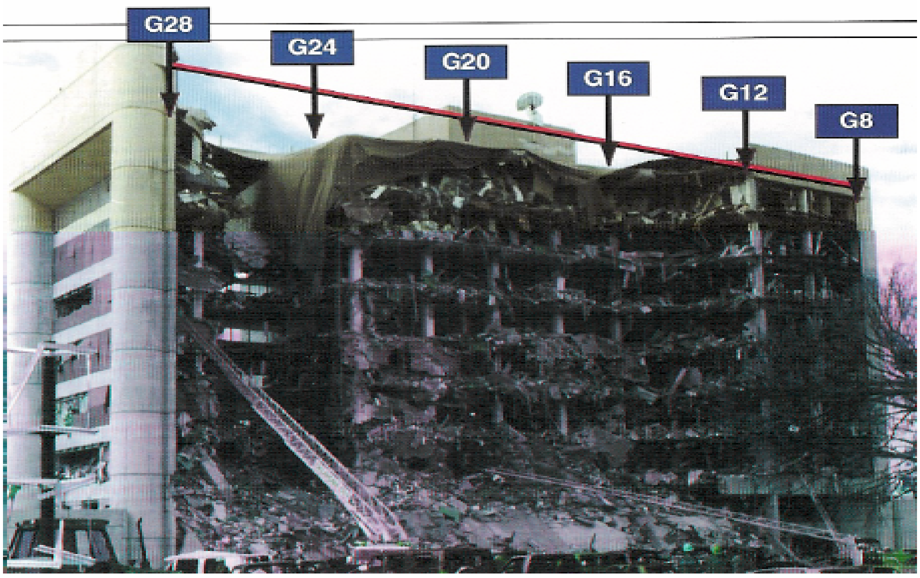


Figure 17: Murrah Federal Office Building after 19 April 1995 attack. (from work of Nair, 2003)

The Murrah Building disaster clearly was a progressive collapse since collapse involved a clear sequence or progression of events: column destruction; transfer girder failure and then collapse of structure above.

But it is the critical question in this case that “Was collapse disproportional?” remains controversial. The answer is not nearly as clear as in the case of the Ronan Point collapse. The Murrah collapse was large. But according to Nair, 2003 the cause of the collapse was very large too, that is large enough to cause damage over an area of several city blocks.

Ultimately, we must judge the Murrah Building collapse “possibly disproportional” only in the sense that we know now that with some fairly modest changes in the structural design such as earthquake detailing for the most active zone, the damage from the bomb might have been significantly reduced. (Hayes et.al, 2005)

4.1.1.2 World Trade Center 1 and 2

Each of the twin towers of World Trade Center 1 and 2 collapsed on 11 September 2001 following this sequence of events: A Boeing 767 jetliner crashed into the tower at high speed; the crash caused structural damage at and near the point of impact and also set off an intense fire within the building (Figure 18); the structure near the impact zone lost its ability to support the load above it as a result of some combination of impact damage and fire damage; the structure above collapsed, having lost its support; the weight and impact of the collapsing upper part of the tower caused a progression of failures extending downward all the way to the ground.

Figure 18: World Trade Center 1 and 2 on 11 September 2001.
(from work of Nair, 2003)

Nair,2003 asserts that it was a “progressive collapse”, but not a “disproportionate collapse.” It was a very large collapse caused by a very large impact and fire. And unlike the case with the Murrah Building, simple changes in the structural design that might have greatly reduced the scale of the collapse have not yet been suggested or identified.

4.1.1.3 Observations on “Progressive” and “Disproportionate” Collapse

Prevention of progressive collapse is generally acknowledged to be an imperative in structural engineering today. But in fact, virtually all collapses could be regarded as “progressive” in one way or another, and according to Nair, 2003 a building’s susceptibility to progressive collapse should be of particular concern only if the collapse is also disproportionate. Therefore, the engineering focus on this issue should be not the prevention of progressive collapse but the prevention of disproportionate collapse.

4.1.2 Methods Of Preventing Disproportionate Collapse

There are, in general, three alternative approaches commonly accepted and referenced by public advisors to designing structures to reduce their susceptibility to disproportionate collapse. These are:

- Redundancy or alternate load paths
- Local resistance
- Interconnection or continuity

4.1.2.1 Redundancy or Alternate Load Paths

As the name implies in this approach, the structure is designed such that if any component fails, alternate paths are available for the load, therefore collapse does not occur. In its most common application, design for redundancy requires that a building structure be able to tolerate loss of any element, usually a column or a shear wall/bracing without collapse.

The flawed side of the redundancy approach is that it does not account for differences in vulnerability or in other means how much redundancy is required (Nair, 2003). Clearly, one-column redundancy when each column is a W8x35 does not provide the same level of safety as when each column is a 3000 kg/m capacity built-up section. Indeed, an explosion that could take out the 3000 kg/m column would likely destroy several of the W8x35 columns, making one-column redundancy inadequate to prevent collapse in that case. And Nair states that "yet, codes and standards that dictate redundancy do not distinguish between the two situations; they treat every column as equally likely to be destroyed which leads to misinterpretations."

In fact, since it is generally much easier to design for redundancy of a small and lightly loaded column, redundancy requirements may have the unfortunate consequence of encouraging designs with many small (and vulnerable) columns rather than fewer larger columns. For safety against deliberate attacks this may be a handicap.

4.1.2.2 Local Resistance

In this approach, resistance to progressive/disproportionate collapse is rehabilitated by providing critical components that might be subject to attack with additional resistance to such attacks. As Nair, 2003 mentions, this requires some knowledge of the nature of potential attacks. And it is very difficult to describe in a simple and objective way.

4.1.2.3 Interconnection or Continuity

This is, strictly speaking, not a third approach separate from redundancy and local resistance, but a means of improving either redundancy or local resistance (or both) according to Nair, 2003. Studies of many recent building collapses have shown that the failure could have been avoided or at least reduced in scale, at fairly small additional cost, if structural components had been interconnected more effectively. This is the basis of the "structural integrity" requirements.

4.2 CODES AND STANDARDS

Since the progressive collapse of the Ronan Point apartment tower in 1968, British agencies established imperative requirements related with collapse prevention due to explosions. Successively many codes and standards have attempted to address the issue of this type of collapse. A small sampling of current and recent provisions related to progressive collapse will provide an indication of the alternative approaches being considered based on the research by Nair, 2003.

4.2.1 ASCE 7-02

Nair summarizes that The American Society of Civil Engineers *Minimum Design Loads for Buildings and Other Structures* ASCE, 2002 has a section on "general structural integrity" states that: "Buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage. It proposes achievement of this goal through an arrangement of the structural elements that provides stability to the entire structural system by transferring loads from any locally damaged region to adjacent regions capable of resisting those loads without collapse. Main emphasis is made over providing sufficient continuity, redundancy, or ductility, or a combination, in the members of the structure states Nair, 2003."

The focus in the ASCE standard is mainly made on redundancy and alternate load paths over all other means of avoiding susceptibility to disproportionate collapse. But it is stated that the weak side of the code is that the degree of redundancy is not specified, and the requirements are entirely threat-independent.

4.2.2 ACI 318-02

The American Concrete Institute *Building Code Requirements for Structural Concrete* (ACI, 2002) include extensive "Requirements for structural integrity" in the chapter on reinforcing steel details. Though the Commentary states that it "is the intent of this section to improve redundancy" there is no explicit mention of redundancy or alternate load paths in the Code. The Code provisions include a general statement that "In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure" and many specific prescriptive requirements for continuity of reinforcing steel and interconnection of components. There are additional requirements for the tying together of precast structural components. None of the ACI provisions are threat-specific in any way.

4.2.3 GSA PBS Facilities Standards 2003

The 2003 edition of the GSA's Facilities Standards for the Public Buildings Service retained the "Progressive Collapse" heading from the 2000 edition, but replaced all of the words reproduced above with this short statement: "Refer to Chapter 8: Security Design." The structural provisions in Chapter 8 apply only to buildings deemed to be at risk of blast attack. For such buildings, the chapter provides general performance guidelines and references to various technical manuals for study of blast effects.

4.2.4 GSA Progressive Collapse Guidelines 2003

The GSA *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects* (GSA, 2003) begins with a process for determining whether a building is exempt from progressive collapse considerations. Exemption is based on the type and size of the structure (for instance, any building of over ten stories is nonexempt) and is unrelated to the level of threat. Typical non-exempt buildings in steel or concrete have to be shown by analysis to be able to tolerate removal of one column or one 30 -ft length of bearing wall without collapse. Considerable detail is provided regarding the features of the analysis and the acceptance criteria. In some ways, these guidelines appear to be a throw-back to the GSA's PBS Facilities Standards of 2000 in that their central provision is a requirement for one-member redundancy, unrelated to the degree of vulnerability of the member or the level of threat to the structure.

4.3 LINEAR STATIC PROGRESSIVE COLLAPSE ANALYSIS PRINCIPLES

The loadings produced by blast events are typically much higher than the design loadings for which an ordinary structure is designed. As noted, these overpressures are usually well beyond the capacity of the structure. "*Local failures of structural elements in the region of the explosion is likely. Since the risk or threat level is highly variable and local capacities are easily exceeded, more detailed analysis is unnecessary* and it is commonly assumed the element impacted will fail" (GSA, 2003). The effect of the blast is then studied by removing the impacted element (or elements) from the structure and then analyzing the modified structure. As defined earlier progressive collapse is the disproportionate collapse of a structure due to a failure of a much smaller element. Since progressive collapse can encompass a much larger portion of the structure (or the entire structure) with many different collapse possibilities, a specific assessment approach is not possible. It is best to look at the specific guidelines such as General Services Administration (GSA), Department of Defence or ASCE guidelines.

Main idea of all these guidelines is simply , " after removal of the vulnerable element, the remaining structure should not collapse. The structure must have another load path to prevent collapse. Analysis basics and procedure discussed in this section is based on General Services Administration Progressive Collapse Guidelines, 2003. (PCADG of GSA)

The process that will be presented in the following sections consists of an analysis/redesign approach. This method is intended to enhance the probability that if localized damage occurs as the result of an abnormal loading event, the structure will not progressively collapse or be damaged to an extent disproportionate to the original cause of the damage. As every process, analysis for progressive collapse potential of any structure can be summarized as seen on the flowchart, shown in Figure 19.

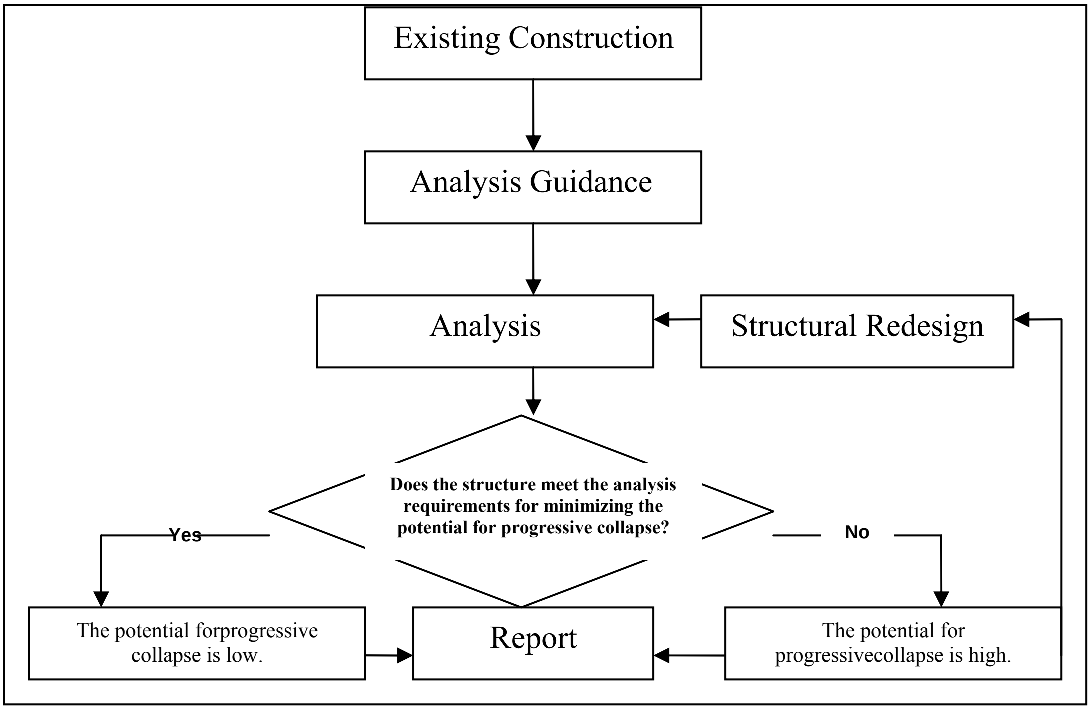


Figure 19: Process for reducing the potential for progressive collapse in new construction. (based on PCADG of GSA, 2003)

Since structural redesign step is not within the scope of this study because we aim to analyze existing structures designed according to Turkish Earthquake Code, 2007 procedure related with this step is not discussed in this study.

Linear elastic, static analysis approach may be used to assess the potential for progressive collapse in all new and upgraded construction. Other analysis approaches may also be used, such as A Nonlinear Procedure implying the use of either static or dynamic finite element analysis methods that capture both material and geometric nonlinearity. Empirically determined damage criteria must be utilized to predict the potential collapse of a structural element. One such set of damage criteria that may be utilized in conjunction with a nonlinear analysis approach is included in Table 4. providing the maximum allowable ductility and/or rotation limits for many structural component and construction types to limit the possibility of collapse. The values listed are for typical elements in conventional construction (i.e., construction that has not been hardened to resist abnormal loading)

Because of the inherent challenges, complexities and costs involved, Nonlinear Procedures have been used less frequently for progressive collapse analyses than have Linear Procedures. In addition, infrequent usage of Nonlinear Procedures was, until only recently, reinforced by limitations in computer hardware and analysis software. However, advancements in computer hardware and general-purpose analysis software packages over the past few years have now made it possible to employ sophisticated structural assessment techniques on large and complex structures, including dynamic time history nonlinear response of high-rise structures containing thousands of members and connections covering a wide range of inelastic constitutive relations for the purpose of practical design applications. Structural engineers, with proper experience and knowledge in structural dynamics, can now construct a global model of the whole structure to capture both material and geometric non-linearity, and to perform the required dynamic time-history non-linear analyses of the entire structure.

Table 4: Acceptance criteria for nonlinear analysis. (from PCADG of GSA, 2003)

COMPONENT	DUCTILITY (μ)	ROTATION Degrees (θ)	ROTATION %Radians (θ)	NOTES
Reinforced Concrete Beam		6	10.5	
R/C One-way Slabs w/o tension membrane		6	10.5	
R/C One way Slabs w/ tension membrane		12	21	
R/C Two-way slabs w/o tension membrane		6	10.5	
R/C Two-way Slabs w/ tension membrane		12	21	
R/C Columns (tension controls)		6	10.5	
R/C Columns (compression controls)	1			
R/C Frames		2	3.5	H/25 max sidesway
Prestressed Beams	2			
Steel Beams	20	12	21	
Metal Stud Walls	7			
Open Web Steel Joist (based on flexural tensile stress in bottom chord)	6			

Table 4: Cont'd.

Metal Deck	20	12	21	
Steel Columns (tension controls)	20	12	21	
Steel Columns (compression controls)	1			
Steel Frames		2	3.5	H/25 Max sidesway
Steel Frame Connections; Fully Restrained • Welded Beam Flange or Cover plated (all types) • Reduced Beam Section		1.5 2	2.5 3.5	See GSA PCADG Appendix D
Steel Frame Connections; Proprietary		2 to 2.5	3.5 to 4.5	See GSA PCADG Appendix D
Steel Frame Connections; Partially Restrained • Limit State governed by rivet shear or flexural yielding of plate, angle or T-section • Limit State governed by high strength bolt shear, tension failure of rivet or bolt, or tension failure of plate, angle or T-section		1.5 1	2.5 1.5	See GSA PCADG Appendix D
One-way Unreinforced Masonry (unarched)	1			
One-way Unreinforced Masonry (compression membrane)	1			
Two-way Unreinforced Masonry (compression membrane)	1			
One-way reinforced Masonry		2	3.5	
Two-way Reinforced Masonry		2	3.5	
Masonry Pilasters (tension controls)		2	3.5	
Masonry Pilasters (compression controls)	1			
Wood Stud Walls	2			
Wood Trusses or Joist	2			

Table 4: Cont'd.

Wood Beams	2			
Wood Exterior Columns (bending)	2			
Wood Interior Columns (buckling)	1			

In case of a Non-Linear Analysis case is carried out the analysis considerations and allowable extents of collapse (The allowable extent of collapse for the instantaneous removal of a primary vertical support member along the exterior and within the interior of a building.), are the two main questions to be answered in the assessment of the potential for progressive collapse.

The following described procedure of GSA, 2003 is stated to use a linear elastic, static approach coupled with the following:

- Criteria for assessing the analysis results
- Alternative analysis cases
- Specific loading criteria to be used in the analysis

4.3.1 Analysis Techniques

GSA, 2003 recommends the use of analysis technique discussed in it's related sections using well-established linear elastic, static analysis techniques. As obvious it is vital to model the structure as close to real as possible for correct analysis and is recommended that 3-dimensional analytic models be used to account for potential 3-dimensional effects and avoid overly conservative solutions. Nevertheless, GSA allows the use of 2-dimensional models provided that the general response and 3-dimensional effects can be adequately idealized.

4.3.2 Procedure

Determination of potential for progressive collapse is suggested by the following procedure. In GSA, 2003

Step 1. The components and connections of both the primary and secondary structural elements shall be analyzed for the case of an instantaneous loss in primary vertical support. The applied downward loading shall be consistent with that will be presented in Section 4.3.3.

Step 2. The result from the analyses performed in Step 1 is evaluated by utilizing the analysis criteria defined in Section 4.3.5.

It is apparent and stated in GSA, 2003 that if the analysis results show that the structural members and/or connections are not in compliance with the analysis criteria presented in Section 4.3.5. the building exhibits a high potential for progressive collapse and the members and/or connections consistent with the procedure outlined in GSA, 2003 shall be rehabilitated.

4.3.3 Analysis Considerations and Loading Criteria

GSA recommends the following analysis considerations in the assessment for progressive collapse for typical structural configurations. Several atypical structural configurations are addressed in GSA, 2003 (structures such that having re-entrant corner, vertical or plan irregularities) but, since the structure in our analysis case is not an atypical structure, as will be discussed, these kind of structures will not be discussed.

4.3.4 Typical Structural Configurations

The analysis scenarios selected for investigation shall be sufficient in number to include all unique structural differences that could affect the outcome of predicting either the low or high potential for progressive collapse. Such unique structural differences shall include, but are not limited to, differences in beam-to-beam connection type (simple vs. moment connection); significant changes in beam span and/or size; and significant changes in column orientation or strength (weak vs. major axis). Additional analysis scenarios may be required for such cases. For facilities that have a relatively simple, uniform, and repetitive layout (for both global and local connection attributes), with no atypical structural configurations, the following analysis scenarios may be used which will also be the case for our analysis model.

4.3.4.1 Framed Structures

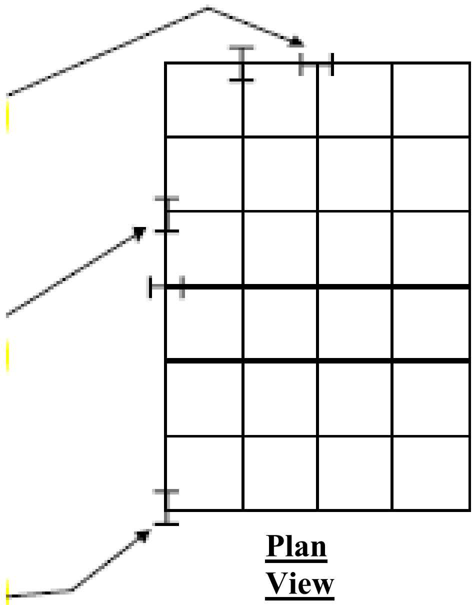
Exterior Considerations

The following exterior analysis cases shall be considered in the procedure outlined in Section 4.3.2.

1 . Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at or near the middle of the short side of the building.

2. Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at or near the middle of the long side of the building.

3. Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at the corner of the building.

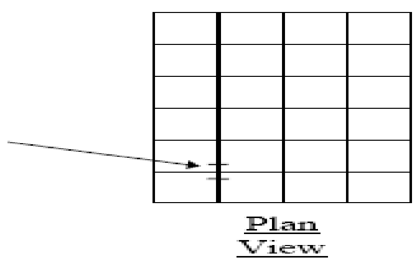


(from PCADG of GSA, 2003)

Interior Considerations

Facilities that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis case(s) in the procedure outlined in Section 4.3.2.

1. Analyze for the instantaneous loss of one column that extends form the floor of the underground parking are or uncontrolled public ground floor area to the next floor (1 story) the column considered should be interior to the perimeter column lines.



(from PCADG of GSA, 2003)

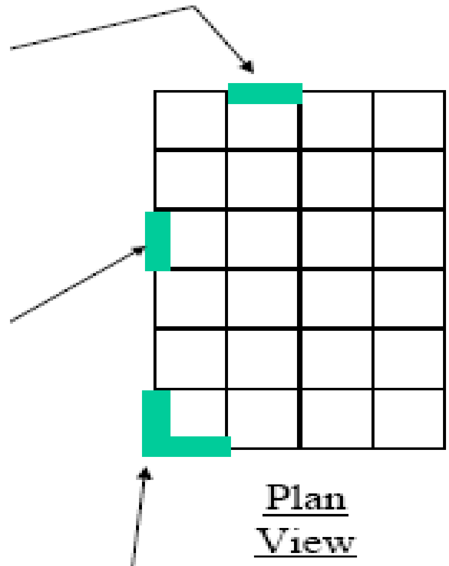
4.3.4.2 Shear/Load Bearing Wall Structures

Exterior Considerations

There may be combination structures that use steel framing combined with load bearing wall sections. In this case, the following exterior analysis cases shall be considered in the procedure outlined in Section 4.3.2.

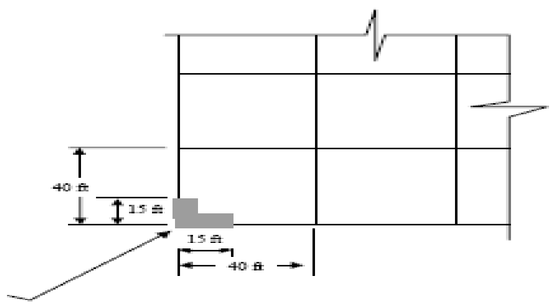
1. Analyze for the instantaneous loss of one structural bay or 30 linear feet of an exterior wall section (whichever is less) for one floor above grade, located at or near the middle of the short side of the building.
2. Analyze for the instantaneous loss of one structural bay or 30 linear feet of an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

Analyze for the instantaneous loss of the entire bearing wall along the perimeter at the corner structural bay or for the loss of 30 linear feet of the wall (15 ft in each major direction) (whichever is less) for one floor above grade*.



(from PCADG of GSA, 2003)

*The loss wall section for the corner consideration must be continuous and include the corner. For example, if the structural bay of a facility is 40 ft by 40 ft, the wall section that would require removal consists of 30 ft of the wall beginning at the corner and extending 15 ft in each major direction.

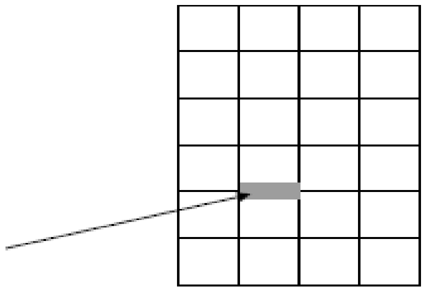


(from PCADG of GSA, 2003)

Interior Considerations

Facilities that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis cases in the procedure outlined in Section 4.3.2.

Analyze for the instantaneous loss of one structural bay or 30 linear feet of an interior wall section (whichever is less) at the floor level of the underground parking area and/or uncontrolled ground floor area. The wall section considered should be interior to the perimeter bearing wall line.



(from PCADG of GSA, 2003)

4.3.5 Analysis Loading

For static analysis purposes the following vertical load shall be applied downward to the structure under investigation:

$$\text{Load} = 2(DL + 0.25LL) \qquad (5.1)$$

where,

DL = dead load

LL = live load

Depending on the facility characteristics and/or the outcome of the initial exemption process, the user may only be required to perform one of the analysis cases: Exterior column/shear wall or interior column/shear wall. For example, if the facility does not contain any uncontrolled parking areas and/or public areas, the user will not be required to perform the analyses for the interior considerations.

4.3.6 Atypical Structural Configurations

All structures are generally unique and are often not *typical* (i.e., buildings often contain distinguishing structural features or details), hence, developing a set of analysis considerations that applies to every facility is impractical. Thus, the user of this guideline must use engineering judgment to determine critical analysis scenarios that should be assessed, in addition to the situations presented in Section 4.3.4. The intent of these provisions should be reflected in these analysis scenarios. Specifically, the scenarios should consider cases where loss of a vertical support (column or wall) could lead to disproportionate damage. Possible structural configurations that may result in an atypical structural arrangement include, but are not limited to, the following configurations:

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

Structural configuration in our case study does not include any of these irregularities and not considered as an atypical structure. Therefore, atypical structural configurations will not be discussed in detail.

4.3.6.1 Analysis Criteria

Structural collapse resulting from the instantaneous removal of a primary vertical support shall be limited. In general, the allowable collapse area for a building is based on the structural bay size.(GSA, 2003) However, to account for structural configurations that have abnormally large structural bay sizes, the collapsed region can also be limited to a reasonably sized area. The allowable extent of collapse for the instantaneous removal of a primary vertical support member along the exterior and within the interior of a building defined in GSA is as follows:

Exterior Considerations

The maximum allowable extents of collapse resulting from the instantaneous removal of an exterior primary vertical support member one floor above grade shall be confined to:

1. The structural bays directly associated with the instantaneously removed vertical member in the floor level directly above the instantaneously removed vertical member or,

2. 1,800 ft^2 (167 m^2) at the floor level directly above the instantaneously removed vertical member whichever is the smaller area. (Figure 20.a).

Interior Considerations

The allowable extents of collapse resulting from the instantaneous removal of an interior primary vertical support member in an **uncontrolled** ground floor area and/or an underground parking area for one floor level shall be confined to:

1. the structural bays directly associated with the instantaneously removed vertical member

or

2. 3,600 ft^2 (335 m^2) at the floor level directly above the instantaneously removed vertical member whichever is the smaller area (Figure 20.b).

Above statements are valid If there is uncontrolled ground floor area and/or an underground parking area present in the facility.

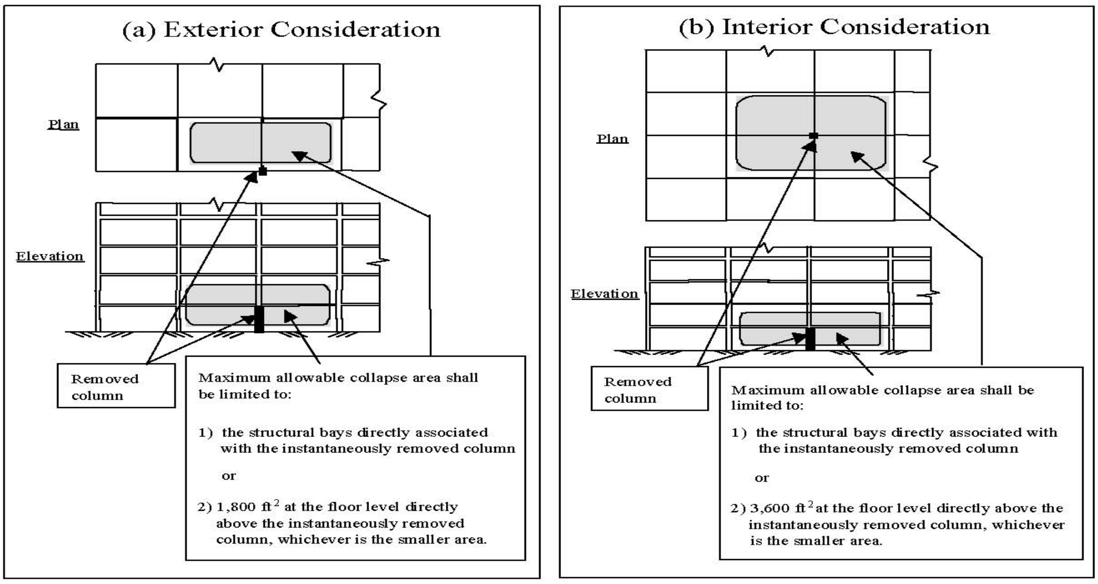


Figure 20:. : An example of maximum allowable collapse areas for a structure that uses columns for the primary vertical support system. (from PCADG of GSA, 2003)

4.3.7 Acceptance Criteria

An examination of the linear elastic analysis results shall be performed to identify the magnitudes and distribution of potential demands on both the primary and secondary structural elements for quantifying potential collapse areas.

Upon removing the selected column from the structure, an assessment should be made as to which beams, girders, columns, joints or connections, have exceeded their respective maximum allowable demands. The magnitude and distribution of demands will be indicated by **D**emand-**C**apacity **R**atios (*DCR*). Member ends exceeding their respective DCR values will then be released and their end moments are redistributed. These values and approaches are based, in part, on the methodology presented in:

- FEMA 274, 1997.
- FEMA 356, 2000.
- Interim Antiterrorism/Force Protection Construction Standards, Guidance on Structural Requirements (Draft), 2001.
- Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. GSA, 2000.

Acceptance criteria for primary and secondary structural components shall be determined as

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

where,

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

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

Using the DCR criteria for the linear elastic approach, structural elements and connections with DCR values exceeding those given in Table 5 are considered to be severely damaged or collapsed. For atypical structural configurations, a value of (3/4)*DCR should be used (factor of 3/4 for uncertainties). Under no conditions is a DCR less than 1.0 required.

The approach used in estimating the magnitude and distribution of the potential inelastic demands and displacements used in these GSA progressive collapse guidelines (GSA, PCADG) is similar to the *'m-factor'* approaches currently employed in FEMA 273 and 356 for linear elastic analysis methods.

To be able to properly select DCR values for the appropriate connection Appendix D. of GSA, PCADG should be consulted.

The step-by-step procedure for conducting the linear elastic, static analysis is as follows.

Step 1. Remove a vertical support from the location being considered and conduct a linear-static analysis of the structure as indicated in Section 4.3.2. Load the model with 2(DL + 0.25LL).

Step 2. Determine which members and connections have DCR values that exceed the acceptance criteria provided in Table 5.1. If the DCR for any member end or connection is exceeded based upon **shear force**, the member is to be considered as failed member. In addition, if the flexural DCR values for both ends of a member or its connections, as well as the span itself, are exceeded (creating a three hinged failure mechanism – Figure 21), the member is to be considered a failed member. Failed members should be removed from the model, and all dead and live loads associated with failed members should be redistributed to other members in adjacent bays.

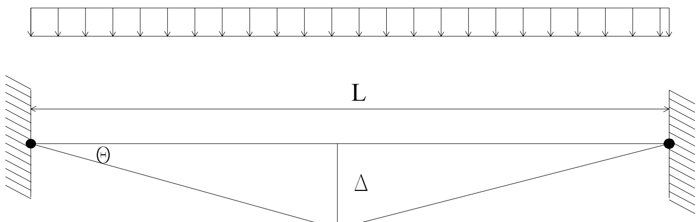


Figure 21: Formation of Failure Mechanism.

Step 3. For a member or connection whose Q_{UD}/Q_{CE} ratio exceeds the applicable flexural DCR values, place a hinge at the member end or connection to release the moment. This hinge should be located at the center of flexural yielding for the member or connection. Use of rigid end offsets and/or stub members from the connecting member is advised to model the hinge in the correct location. For sake of simplicity GSA, PCADG recommends for yielding at the end of a member the center of flexural yielding should not be taken to be more than ½ the depth of the member from the face of the intersecting member, which is usually a column (Figure 22). This value is in accordance with FEMA provisions.

Step 4. At each inserted hinge, apply equal-but-opposite moments to the stub/offset and member end to each side of the hinge. The magnitude of the moments should equal the expected flexural strength of the moment or connection, and the direction of the moments should be consistent with direction of the moments in the analysis performed in Step 1.

Step 5. Re-run the analysis and repeat Steps 1 through 4. Continue this process until no DCR values are exceeded. **If moments have been redistributed throughout the entire building and DCR values are still exceeded in areas outside of the allowable collapse region, the structure will be considered to have a high potential for progressive collapse.**

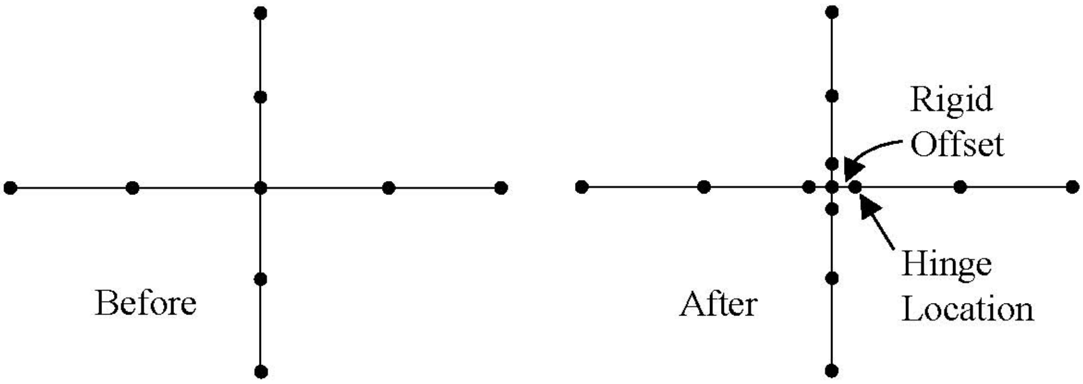


Figure 22: Rigid offset placement.
(from PCADG of GSA, 2003)

Table 5: Acceptance criteria for linear procedures— steel frame components. (from PCADG of GSA, 2003)

Component/Action			Values for Linear Procedures DCR
Beams – flexure			
	a.	$\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	3
	b.	$\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	2
	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 lowest resulting value shall be used.	
Columns – flexure			
For 0 < P/P_{CL} < 0.5			
	a.	$\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{300}{\sqrt{F_{ye}}}$	2
	b.	$\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{460}{\sqrt{F_{ye}}}$	1,25
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 lowest resulting value shall be used.	

Table 5: Cont'd

Component/Action			Values for Linear Procedures
			DCR
Columns – flexure			
For $P/P_{CL} > 0.5$			
	a.	$\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{260}{\sqrt{F_{ye}}}$	1
	b.	$\frac{b_f}{t_w} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{400}{\sqrt{F_{ye}}}$	1
Columns Panel Zone – Shear			2
Column Core – Concentrated Forces[2]			1.5
Fully Restrained Moment Connections			
Pre-Northridge (Pre 1995)			
Welded unreinforced flange (WUF)			2
Welded flange plate (WFP)			2
Welded cover plated flanges			2
Bolted flange plate (BFP)			2
Post-Northridge (FEMA 350) Public Domain			
Improved WUF-bolted web			2
Improved WUF-welded web			2
Free flange			2
Welded top and bottom haunches			2
Reduced beam section			2
Post-Northridge (FEMA 350) Proprietary[3]			
Proprietary System			≤3 (See Footnote 3)

Table 5: Cont'd

Component/Action	Values for Linear Procedures *DCR*
Partially Restrained Moment Connection	
Top and bottom clip angle	
a. Shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Tension failure of horizontal leg of angle	1.5
c. Tension failure of rivets or bolts	1.5
d. Flexural Failure of angle	3
Double split tee	
a. Shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Tension failure of rivets or bolts	1.5
c. Tension failure of split tee stem	1.5
d. Flexural Failure of split tee	3
Bolted flange plate	
a. Failure in net section of flange plate or shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Weld failure or tension failure on gross section of plate	1.5
Bolted end plate	
a. Yield of end plate	3
b. Yield of rivets or bolts	2 (rivets); 1.5 (high strength bolts)
c. Failure of weld	1.5
Composite top and clip angle bottom	

1. where bf = Width of the compression flange

F_{ye} = Expected yield strength

h = Distance from inside of compression flange to inside of tension flange

t_w = Web thickness

P_{CL} = Lower bound compression strength of the column

P = Axial force in member taken as Q_{uf}

t_f = Flange thickness

d = Beam depth

d_{bg} = Depth of the bolt group

2. Column core concentrated force capacity shall be determined from AISC (1993) LRFD Specifications equations K1-1, K1-2, K1-4 and K1-8.

3. A DCR of 2 will be used for all untested proprietary fully restrained moment connections. A DCR of 1 will be used for all other untested proprietary connections. Under no circumstances should a DCR value exceeding 3 be used for any proprietary connection.

4. DCR values are for connection to strong axis of column. For connections to weak axis of column Figure 23 treat as atypical (DCR*0.75).

5. No DCR values less than 1.0 are required, even for atypical conditions.

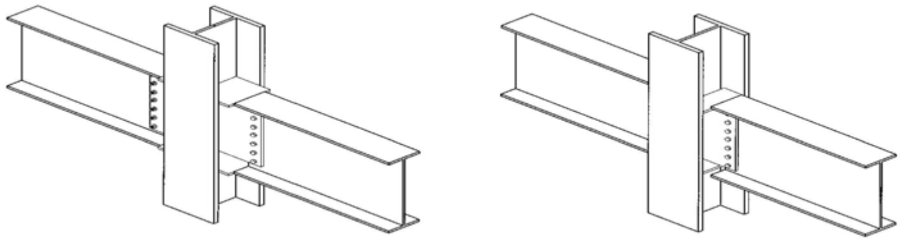


Figure 23: (a) Fully Rigid Connection (b) Typical Shear Only Connection Weak axis connections. (from PCADG of GSA, 2003)

4.3.8 Material Properties

The design material strengths may be increased by a strength-increase factor to determine the expected material strength due to dynamic loading effect. GSA, 2003 states that, “these strength increase factors should be used only in cases where the designer or analyst is confident in the actual state of the facility’s materials.” These values are provided in Table 6 and Table 7.

Table 6: Default lower-bound material strengths — steel frame components. (from PCADG of GSA, 2003)

Properties based on ASTM and AISC Structural Steel Specification Stresses				
Date	**Specification**	**Remarks**	**Tensile Strength, MPa (ksi)**	**Yield Strength, MPa (ksi)**
1900	ASTM, A9	Rivet Steel	344.74 (50)	206.84 (30)
	Buildings	Medium Steel	413.69 (60)	137.90 (20)
1901-1908	ASTM, A9	Rivet Steel	344.74 (50)	172.37 (25)
	Buildings	Medium Steel	413.69 (60)	206.84 (30)
1909-1923	ASTM, A9	Structural Steel	379.21 (55)	193.05 (28)
	Buildings	Rivet Steel	317.16 (46)	158.58 (23)
1924-1931	ASTM, A7	Structural Steel	379.21 (55)	206.84 (30)
	Buildings	Rivet Steel	317.16 (46)	172.37 (25)
	ASTM, A9	Structural Steel	379.21 (55)	206.84 (30)
		Rivet Steel	317.16 (46)	172.37 (25)
1932	ASTM, A140-32T issued as a tentative revision to ASTM, A9 (Buildings)	Plates, Shapes, Bars	413.69 (60)	227.53 (33)
		Eyebar flats unannealed	461.95 (67)	248.21 (36)
1933	ASTM, A140-32T discontinued and ASTM, A9 (Buildings) revised Oct.30, 1933	Structural Steel	379.21 (55)	206.84 (30)
	ASTM, A9 tentatively revised to ASTM, A9-33T (Buildings) revised Oct.30, 1933	Structural Steel	358.53 (52)	193.05 (28)
	ASTM, A140-32T adopted as a standard	Rivet Steel	358.53 (52)	193.05 (28)
1934	ASTM, A9	Structural Steel	413.69 (60)	227.53 (33)
	ASTM, A141	Rivet Steel	358.53 (52)	193.05 (28)

Table 6: Cont'd.

1961 - 1990	ASTM, A36/A36M-00	Structural Steel		
	Group 1		427.48 (62)	303.37 (44)
	Group 2		406.79 (59)	282.69 (41)
	Group 3		413.69 (60)	268.90 (39)
	Group 4		427.48 (62)	255.11 (37)
	Group 5		482.63 (70)	282.69 (41)
1961 on	ASTM, A572, Grade 50	Structural Steel		
	Group 1		448.16 (65)	344.74 (50)
	Group 2		455.05 (66)	344.74 (50)
	Group 3		468.84 (68)	351.63 (51)
	Group 4		496.42 (72)	344.74 (50)
	Group 5		530.90 (77)	344.74 (50)
1990 on	A36/36M-00 & Dual Grade	Structural Steel		
	Group 1		455.05 (66)	337.84 (49)
	Group 2		461.95 (67)	344.74 (50)
	Group 3		482.63 (70)	358.53 (52)
	Group 4		482.63 (70)	337.84 (49)

[1] Lower-bound values for material prior to 1960 are based on minimum specified values. Lower-bound values for material after 1960 are near minus one standard deviation values from statistical data.

[2] The indicated values are representative of material extracted from the flanges of wide flange shapes.

Table 7: Factors to translate lower-bound properties to expected-strength steel properties. (from PCADG of GSA, 2003)

Property	**Year**	**Specification**	**Factor**
Tensile Strength	Prior to 1961		1.10
Yield Strength	Prior to 1961		1.10
Tensile Strength	1961 - 1990	ASTM A36/A36M-00l	1.10
	1961 - present	ASTM A572/A572M-89, Group 1	1.10
		ASTM A572/A572M-89, Group 2	1.10
		ASTM A572/A572M-89, Group 3	1.05
		ASTM A572/A572M-89, Group 4	1.05
		ASTM A572/A572M-89, Group 5	1.05
	1990 - present	ASTM A36/A36M-00l & Dual Grade Group 1	1.05
		ASTM A36/A36M-00l & Dual Grade Group 2	1.05
		ASTM A36/A36M-00l & Dual Grade Group 3	1.05
		ASTM A36/A36M-00l & Dual Grade Group 4	1.05
Yield Strength	1961 - 1990	ASTM A36/A36M-00l	1.10
	1961 - present	ASTM A572/A572M-89, Group 1	1.10
		ASTM A572/A572M-89, Group 2	1.10
		ASTM A572/A572M-89, Group 3	1.05
		ASTM A572/A572M-89, Group 4	1.10
		ASTM A572/A572M-89, Group 5	1.05

	1990 - p resent	ASTM A36/A36M-00l Plates	1.10
		ASTM A36/A36M-00l Dual Grade, Group 1	1.05
		ASTM A36/A36M-00l Dual Grade, Group 2	1.10
		ASTM A36/A36M-00l Dual Grade, Group 3	1.05
		ASTM A36/A36M-00l Dual Grade, Group 4	1.05
Tensile Strength	All	Not Listed[1]	1.10
Yield Strength	All	Not Listed[1]	1.10
1. For materials not conforming to one of the listed specifications.			

Table 7: **Cont'd.** (from PCADG of GSA, 2003)

4.3.9 Modeling Considerations

General

As is the case for all analysis the analytic model(s) used in assessing the potential for progressive collapse should be modeled as accurately as possible to the anticipated or existing conditions. This includes all material properties, design details, etc. In addition, the analyst shall realistically approximate the type of boundary conditions in the light of above explained considerations (e.g., fixed, simple, etc.), and should be aware of any limitations or anomalies of the software package(s) being used to perform the analysis.

Vertical Element Removal

The vertical element (i.e., the column, bearing wall, etc.) that is removed should be removed instantaneously. While the speed at which an element is removed has no impact on a static analysis, the speed at which an element is removed in a dynamic analysis may have a significant impact on the response of the structure. Also the vertical element removal shall consist of the removal of the vertical element only. This removal should not obstruct into the connection/joint or horizontal elements that are attached to the vertical element at the floor levels. Since the analysis method applied in chapter 5 for the case study does not have the ability to incorporate the speed of element removal in to the analysis, it is not an important issue for the case study. It is accepted and applied as the element is suddenly removed for the sake of consistency. An example sketch illustrating the correct and incorrect way to remove a column is shown in Figure 24.

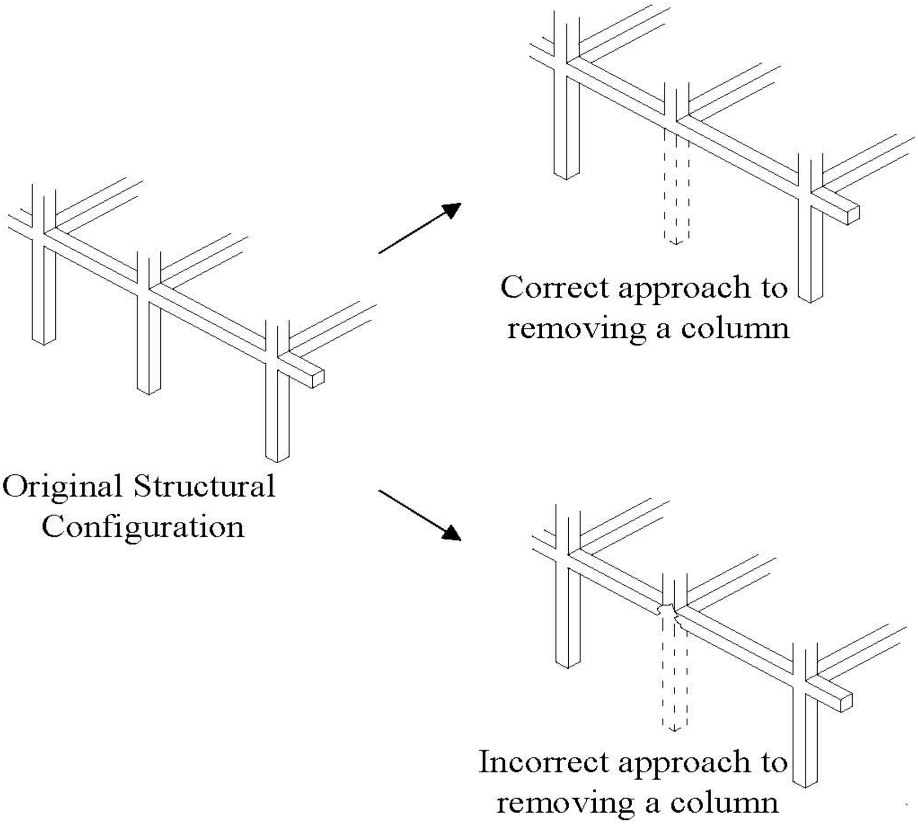


Figure 24: Sketch of the correct and incorrect approach for removing a column. (from PCADG of GSA, 2003)

4.4 NONLINEAR STATIC PROGRESSIVE COLLAPSE ANALYSIS PRINCIPLES

In the light of discussions about blast loading, structural characteristics and material behavior under blast load, the analysis method used in this study for purpose of nonlinear analysis will be introduced in this section. This is one of the recently developed and practical methods. The Analysis Method is based on a single degree nonlinear system, consisting of a nonlinear spring and a concentrate mass idea and is created first to illustrate the procedure of progressive collapse. Analysis procedure is based on the method developed by Gilsanz and Wenjun for Design Engineers of Gilsanz Murray Steficek, Co.

Through out the introduction of the analysis procedure first, in Part I the detailed description of the analysis philosophy is discussed. In Part II a nonlinear static analysis procedure for existing buildings is discussed. Gilsanz and Guo states the basic concept of the procedure as energy balance, i.e., the structure must absorb the potential energy generated due to the removal of one element.

4.4.1 Part I

Idealization of Progressive Collapse

Gilsanz and Guo describe their procedure as is similar to a single degree freedom system as shown in Figure 25. The states of the nonlinear spring are illustrated in Figure 26. Point A, B, C, D and E in Figure 25 and Figure 26 denote same state. Table 8 is the list of system variables.

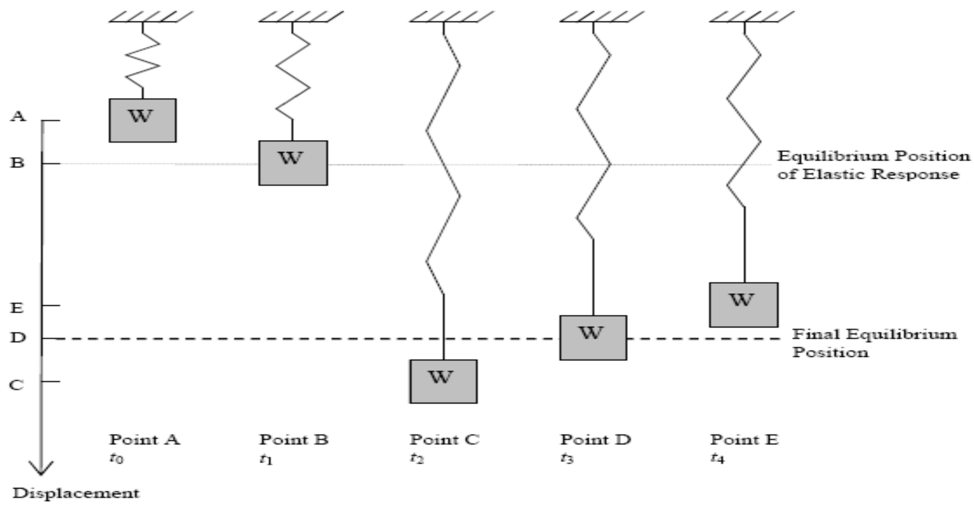


Figure 25: Illustration of Progressive Collapse Procedure.
(from work of Gilsanz and Guo, 2003)

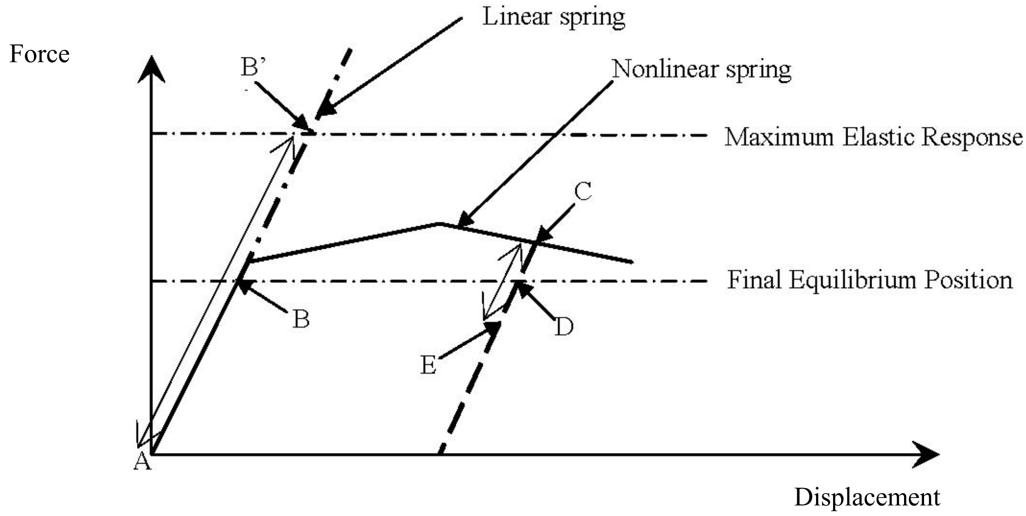


Figure 26: Force vs. Displacement Diagram of Spring.

(from work of Gilsanz and Guo, 2003)

Table 8: System Variables. (from work of Gilsanz and Guo, 2003)

Point	Force	Potential Energy	Kinetic Energy	Energy absorbed by Spring
A	Down	-W*A1	0	0
B	Zero	-W*B1	+	+
C	Up	-W*C1	0	W*C
D	Zero	-W*D1	+	+
E	Down	-W*E1	0	W*E

: A, B, C, D, and E denote the displacement coordinate at those points.

Energy dissipated in the structure due to damping is minimum compared with the energy absorbed due to plastic deformation. Thus, Gilsanz and Gou do not consider damping in the following description of the progressive collapse procedure.

At point A, when the column/shear wall is removed, the system has the maximum potential energy. Since the force in the spring is zero at this time, the system is falling down due to the weight of the system, W.

From point A to B, the downward velocity increases and reaches its maximum at point B. After point B, the downward velocity decreases because the force in the spring is greater than the weight of the system, W. If the yield capacity is greater than 2W, the response of the system is linear static as the straight line AB' shown in Figure 26.

At point C, the falling system has zero velocity and all the potential energy is absorbed by the spring. Point C can be obtained by above energy balance condition. After point C, the system starts rebound because force in the spring is greater than the weight of the system, W.

At point D, the system has maximum upward velocity. From point D to point E, the upward velocity decreases and becomes zero at point E. If the unloading curve of the spring is straight, it can be seen that distance CD equal to DE. Point D will be the final state.

Implications of this idealization are listed by Gilsanz and Wenjun as follows:

For the system not to fail, the strength of the spring at point C must be greater than the weight of the system.

If the weight of the system is greater than the maximum strength capacity of the spring, the system will fail.

If the weight of the system is smaller than half of the yield strength of the spring, the system has only elastic response and will not collapse.

The magnitude of the vibration between point C and point E is generally small compared with the elastic response and generally there is no load reversal. Hence the system will not fail as it oscillates around point D.

4.4.2 Part II.

Nonlinear Static Analysis Procedure

Following is a description of the nonlinear static analysis procedure method proposed by Gilsanz and Gou:

1 Put a load proportional to the reaction of the removed column and increase it gradually to get the pushover curve of the structure.

2 If the reaction is less than half of the yield strength of the pushover curve, the structure has low potential for progressive collapse.

3 If the reaction is greater than the maximum strength of the pushover curve, the structure has high potential for progressive collapse.

4 If conditions of 2 and 3 are not satisfied, generate the capacity curve and compare it with the load curve. This step is explained in Part III.
The above procedure can be used as a preliminary evaluation procedure to verify if conditions of step 2 or 3 are satisfied.

Gilsanz and Guo states basic concept of the analysis as energy balance, i.e., the structure must absorb the potential energy generated due to the removal of one column. "*The capacity curve is generated by dividing the energy absorbed by the structure, area below the pushover curve, by the displacement. The capacity curve is then compared with the load curve,* which is a straight line parallel to X axis with the magnitude equal to the weight supported by the removed column."

4.4.3 Part III

Explanation of Analysis Step 4

Plastic moment hinges and axial hinges are assigned to beam ends. Moment hinge properties are taken from FEMA 356 as shown in Figure 27, Figure 28 is the axial hinge property diagram.

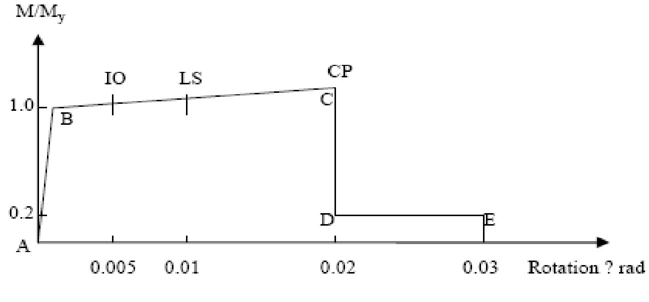


Figure 27: Moment Hinge Properties.(from work of Gilsanz and Guo, 2003)

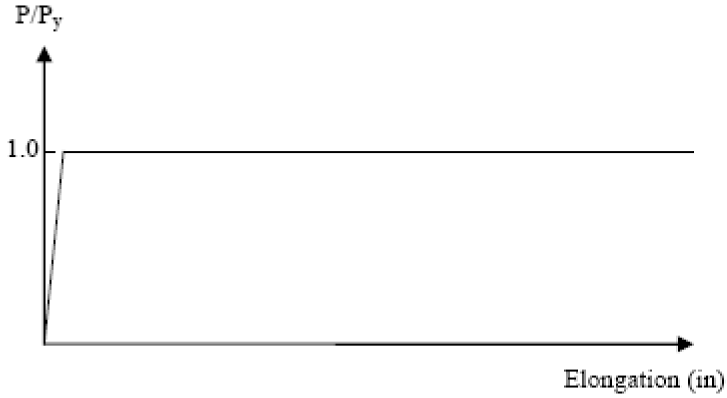


Figure 28: Axial Hinge Properties.
(from work of Gilsanz and Guo, 2003)

Figure 29 shows the loading condition to get the pushover curve. The load P is equal to the reaction of the column removed. The displacement control analysis computes at each displacement step the amount of load required to create the displacement.

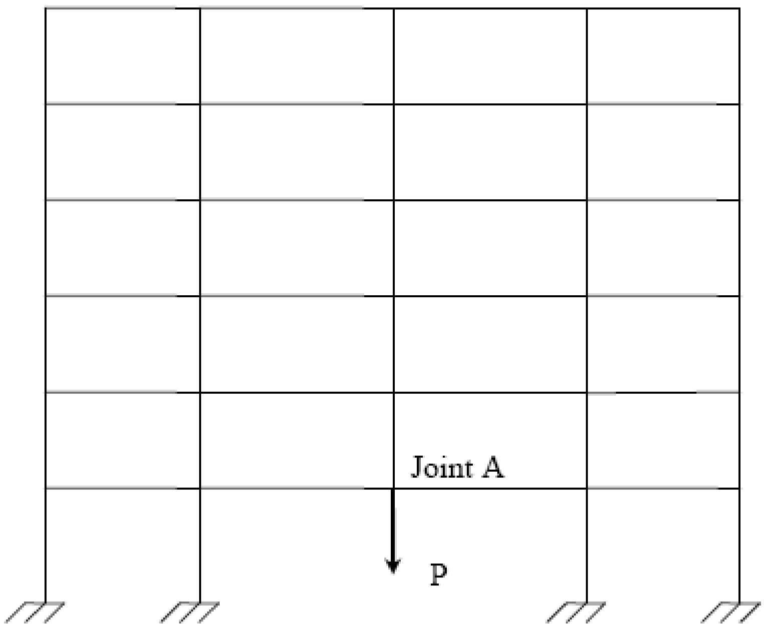


Figure 29: Loading for Pushover Analysis Procedure.
(from work of Gilsanz and Guo, 2003)

Figure 30 on the next page is the pushover curve. Point A, B, C, D, and E on the pushover curve indicates different stages of structure behavior. Before point A, the structure behaves elastically with point A corresponding to the yielding of the structure. After yielding, the beams strength hardened from point A to B. At point B, the hinges fail and there is an abrupt drop. Curve CD indicates that the structure begins to pick up load due to strain hardening. At point D, structure yields due to tension and the slope of the pushover curve becomes smaller. Since it is assumed that elastoplastic deformation model has infinite deformation capacity, the structure can continue to sustain load without failure.

The area below the pushover curve is the energy that the structure can absorb. If we divide the energy below the pushover curve by the corresponding displacement, we can get the capacity curve of the structure. For example, point E' on the capacity curve is obtained by dividing area below OABCDE by the displacement at E. The pushover curve and capacity curve are characteristics of the structure under given load condition.

The load curve is straight in this case, which is equal to the reaction of the removed column. From Figure 30, it can be seen that the capacity curve is lower than the load curve before point F', which means that the structure can not absorb the potential energy before reaching the displacement corresponding to point F'. It is obvious that the structure will collapse if it deflects as much as point F', even the energy can be balanced at point F. Thus, the conclusion is that the 2-D frame shown in Figure 29 has a high potential for progressive collapse.

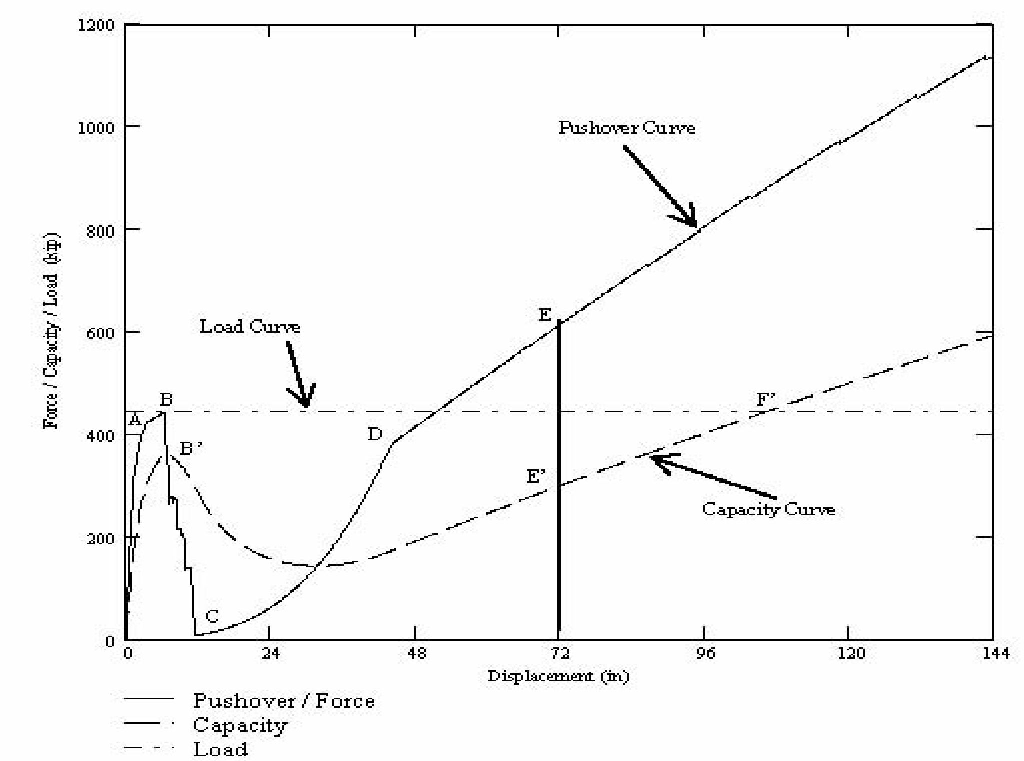


Figure 30: Pushover Curve, Capacity Curve, and Load Curve.
(from work of Gilsanz and Guo, 2003)

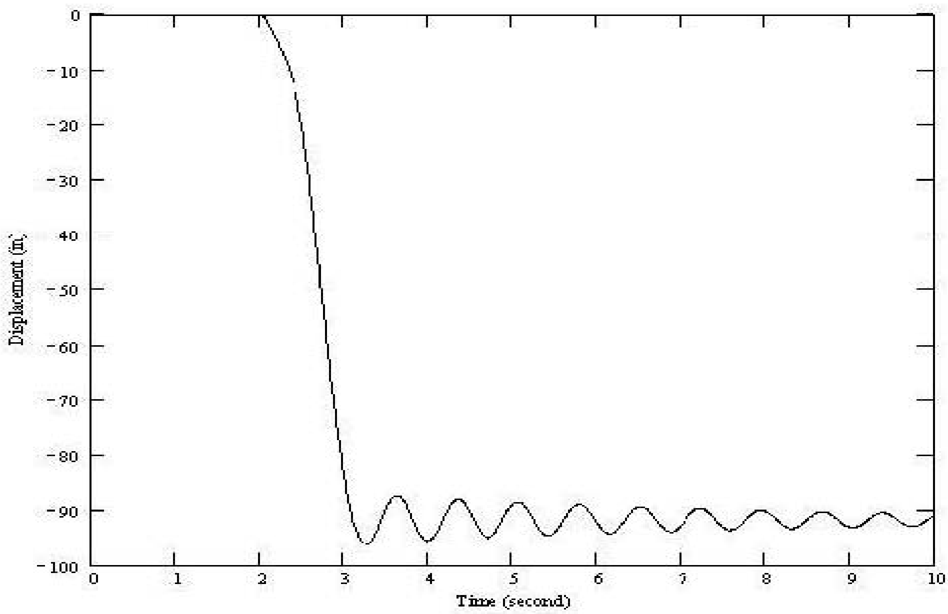


Figure 31: Vertical Displacements vs. Time Diagram.
(from work of Gilsanz and Guo, 2003)

4.5 MITIGATION STRATEGIES

4.5.1 CONSIDERATIONS FOR UPGRADING EXISTING BUILDINGS

Effectively protecting an existing facility by blast strengthening is a relatively difficult task. Realistically, the built environment has a number of inherent weaknesses when considering the possible effects of an extreme event. It is rare that the facility that has systems designed for improved performance in an extreme event. Structures are typically constructed without specific consideration of redundancy or robustness in an extreme event. While risk analysis and vulnerability assessment are essential first steps in any security project, these steps take on a special importance for an existing facility. Due to the particular difficulties of effectively hardening an existing building, it is important that the risk analysis and vulnerability assessment result in a clear understanding of the potential vulnerabilities and of the scale of construction work that may be required to mitigate or prevent damage from the identified threats.

Since the costs of hardening an entire existing facility are often so high, it is common choosing to focus the efforts on specific locations or functions within a facility where risks are highest, where a decision is made to harden some part of an existing facility or a specific structural system or element, the design approach is influenced by a series of factors, some of which are include the following:

• Information about existing conditions;
• Structural elements commonly hidden or obstructed by existing architectural or building services systems that are difficult or costly to remove;
• The level of ductility of the existing construction may limit its strength.

In steel structures, common deficiencies include susceptibility to local buckling of outstanding flanges, and lack of connection ductility. Strengthening of a limited number of structural elements is usually practical, and, as with other types of renovations, it is commonly accepted that it is relatively easy to work with steel construction.

Following discusses the ways to harden an existing structure in means of general concepts. In the end some practical methods recently developed for steel construction, especially for joints are introduced.

4.5.1.1 Local Strengthening to Prevent Failure Initiation

Structural elements and connections in an existing structure can be strenghted to reduce the risk of initiating or spreading failure due to abnormal loading. The intent is to increase the load capacity and ductility of certain critical structural elements or connections so that they can survive the effects of specific or generalized threats.

It is often practical to impart specific resistance for less aggressive threats. Moderate-speed vehicle impacts can be resisted with cost efficient structural upgrades. National Institute of Standards and Technology states in the document named “Best Practices for Reducing the Potential for Progressive Collapse in Buildings (BPRPPCB-NIST, 2006) states that bombs with relatively low energy-to-range ratios can be addressed reasonably with local strengthening.

The strength and the ability of the structure to dissipate energy (i.e., structures with high ductility) both are essential for the resistance to most threats and for load redistribution as is the case for all rare events as earthquakes. Therefore, any methodology that increases the capacity and ductility of existing critical elements and connections is a good candidate for consideration to upgrade a structure to prevent progressive collapse. For instance, retrofitting techniques used for seismic loads are, in some cases, applicable candidates to upgrade a structure locally to prevent progressive collapse. Corley et al. (1996) recommended that techniques commonly found in earthquake retrofitting such as column jacketing, can be used to increase ductility and load capacity. It should be pointed out that when such retrofit techniques are used for non-seismic events, potential failure modes of structural members should be considered to determine the appropriate locations for strengthening. In case of steel structures it is easier to modify existing sections and structural configuration.

According to NIST, 2006 elements can be upgraded following either of two perspectives: in response to specific threats and in response to non-specific threats. These two perspectives are discussed below.

4.5.1.2 Upgrade Vulnerable Elements for Specific Threats

If specific threats to a building are known, it is possible to upgrade elements against the expected hazards. For instance, the demands caused by a vehicle crash into a bridge or columns in a building can be estimated for presumed vehicle masses and velocities. In these cases, specific demands can be defined to design remediations so that these critical elements can survive vehicle impact.

An external explosion is another example of a specific threat for which elements can be upgraded (i.e., approximate locations of attack and type and amouth of the explosive source is known), one can reasonably determine the energy release and the potential influence on surrounding structural components. These datas in hand it is possible to reasonably analyze a structure for such an event using availble well established computer modelling programs for this purpose.

4.5.1.3 Upgrade Vulnerable Elements for Non-Specific Threats

This is accomplished by identifying and strengthening vulnerable elements and connections considering their role on the integrity of the structure but without specifying specific hazards. It is imperative, in this approach, that the engineer associates the vulnerability of the structure as a whole with the ductility and strength of individual components, disregarding the nature, location, and time of abnormal loading events. Likewise, an engineer might discover that certain structural components have particularly poor inherent resistance to abnormal loads of any reasonable character.

4.5.1.4 Constraints Originating From Existing Structural System

Sometimes critical elements might be unreachable or it is impractical to install the needed upgrades due to space constraints. To the extent that upgrade components must act compositely with or transfer forces to existing components, it will be essential to be able to develop the necessary connections. Uncertainty about the actual construction, deviations from the available documentation, forms of deterioration and variations in strengths of materials are common in building construction. To the extent that these conditions can not be discerned completely, the engineer is faced with a level of uncertainty that sometimes prohibits appropriate assessment of progressive collapse potential in exiting buildings.

In cases such this, it will be necessary to find alternatives that do not rely on strengthening of the existing member (i.e., adding new members to create redundancy).

4.5.1.5 Enhance Redundancy to Confine Local Failures

If a decision is made to modify the building, the solution will probably require the introduction of redundancy to the structure. Typically, this is accomplished by providing additional rotational and tensile capacity in joints or connections or by creating new alternate load paths, or generally both.

Sometimes the general means to establish the necessary continuity are well established. For example, previous investigations (Corley et al. 1996) of major structural collapses have concluded that the spread of damage in those instances could have been comprised if the structures had been detailed following common practice found in earthquake-resistant design. The idea behind this statement is that high ductility or high capacity for energy dissipation plays a fundamental role for a structure to resist both earthquake loading and impact or blast effects. Corley et al. pointed out that more than 50 % of the collapsed area in the Alfred P. Murrah Building in Oklahoma City would have stood if the structure had been designed with special moment frames found in seismic regions as opposed to the ordinary moment frames used in the building based on the findings of research by Hayes et. al.

When it is difficult technically or economically to provide the required localized resistance, or when uncertainties related to the threat, the as-built conditions, or the response are significant, then the applicable alternative is to strengthen structural elements and systems to increase their ductility and capacity to redistribute and support loads once a localized failure has occurred. Enhanced redundancy, potentially developed in response to specific threats, additionally provides general robustness that offers protection for other, unspecific, threats that affect the building.

4.5.1.6 Local strengthening to enhance global response

For steel-framed buildings, the beam-to-column connections may have been generally designed only for shear forces while the lateral loads in the structure are carried by cross bracing in limited locations or by a few moment frames. To increase the energy dissipation and load capacity for these simply-supported beams, NIST, 2006 advices the designer to create moment connections to columns. An upgrade to provide enhanced moment resistance at columns also will improve the tensile capacity of structural steel connections. This could be one component of a significant increase in the level of redundancy in the structure, by allowing beams to act as catenary elements to span over a dmaged area.(NIST, 2006)

If the local upgrade of connections enforces continuity that did not previously exist, then there is the possibility that the retrofitted structure has enhanced bridging action. Hence, decisions leading toward a final design for improved resistance to progressive collapse should consider the potential for cross benefits-both ways – between local strengthening to prevent initial failures and overall strengthening to limit spreading of failures.

4.5.1.7 Addition of alternate load paths

Generally, the addition of an alternate load path means providing capability for the structure above the first level at grade on the exterior to "bridge over" or redistribute loads after the loss a column at a lower level.(NIST, 2006)

Alternate load paths can be created by introducing modifications in structures that have been designed with planar systems. Such modifications force structural systems to engage the resistance of more components when one or more critical elements have been damaged. This ability to spread out the load over existing elements reduces the demand on each element.

4.5.1.8 Means to enhance redundancy

Redundancy requires alternate load paths and elimination mechanisms. The means to provide these features are as varied as the population of framing systems that exist in buildings of interest.

However, in general, redundancy can be provided by creation of two-way action in the framing system, introduction of secondary trusses, relying on Vierendeel action, creation of "strong floors" in buildings, and introduction of means to hang portion structure from above.

1) Two-way action

Existing structural framing systems that can span two ways have greater robustness than structures that are designed and constructed to span just one way. In a two-way frame, as many as eight nearby columns would be available to help share the load of an interior column. Further, for catenary action which will be explained later, ideal design transfers half the force in each direction.

In some instances, basic detailing such as temperature and shrinkage reinforcement in slabs provides for sufficient two-way action. For robust designs, however, the engineer can specifically consider whether such features in an existing building are adequate or whether robustness can be enhanced by a specific design that provides the needed secondary support.

In general, it may be difficult to add two-way-action features to existing buildings. However, in some framing systems elements such as new beams can suffice. An example might be a floor system with open web joists spanning between beams. Joists on column lines can be augmented or replaced with robust beams that provide support for columns, should they be removed by an extreme event.

2) Secondary trusses

When the potential initiating event is the removal of certain specific columns at low levels in a building, it may be feasible to add diagonal elements at upper levels, to turn two or multiple-story column and beam systems into trusses. (NIST, 2006) In this method, the trusses would be engaged if a lower level column were to be removed, with columns above the initial damage becoming tension members.

Important considerations in such systems are the ability to connect the new diagonal members to the existing structure, the strength of adjacent existing elements to carry the new loads, and the ability of columns to act as tension members. Particular concern needs to be given to column splices (e.g., bolted or welded splices in steel members) designed for compression but suddenly subjected to tension forces. Also, NIST, 2006 states that "consideration needs to be given to the potential that addition of secondary trusses will change the distribution of lateral service loads, affecting the performance of the structure for wind and seismic loads."

An advantage of secondary truss systems declared in NIST document is that they often can be designed to resist the applied forces with relatively little deformation, as compared with other alternatives. This could be an advantage for life safety and further could improve the prospects of rehabilitating a building after an extreme event.

3) Vierendeel action

Moment frames intended to support lateral loads can span of damage through Vierendeel action. Beams experience severe double-curvature deformation, and depending on the extent of the initial damage, columns also receive severe flexural loading.

Vierendeel action often is an applicable means to add robustness to some existing buildings because all the basic features already exist, in some measure. Consideration needs to be given to the proximity of the existing moment frames with respect to the locations where initiating events are likely to occur, and to the forces that occur when Vierendeel behavior is activated. However, NIST states that, if beams and columns-and their connection can be reinforced to support the applied loads, this method to add robustness can be relatively insignificant.

In order to develop Vierendeel action for resistance to progressive collapse, it often is necessary to upgrade a large portion of the structure. It is usually insufficient to upgrade only a few floors and achieve the desired result.

4) Strong floors

It is not always necessary to implement upgrades throughout a building. Sometimes a few floors can be identified, often distributed throughout the building, where resistance will be concentrated. Hence, if a system can be developed wherein individual floors are strengthened to support the load of several adjacent floors, then the areas where intrusive repairs are needed will be limited.

An advantage to the strong floor approach is that the floors with added robustness can be distributed throughout the height of the building. This results in enhanced performance of the building for unspecified events.

5) Allow catenary action to develop

The concept involves engagement of tensile forces in members that hang out loosely or that deform into configurations that allow cable action to be engaged. In catenary action, engineers generally expect that elements (e.g., beams and slabs) that are intended to support load in flexure will deform enough and have sufficiently stiff and strong anchorages that they will take on load as tension members. In this case, adjacent structure needs to be able to resist the high horizontal loads that are necessarily associated with the resolution of the forces in the flexural members that must work while deforming to relatively small angles to the horizontal.

4.5.1.9 Patented Moment Frame Connections

In this part it is intended to present information on patented fully-restrained steel frame moment connections that have been privately developed. A discussion of several types of patented connections is included herein. NIST-BPRPPCB, 2006 states that these proprietary connections have been evaluated by recognized enforcement agencies and found to be acceptable for specific projects and/or for general application within the jurisdiction's authority. There are several other patented connections not included in this part. As a general rule, designers wishing to consider specific patented connections for use in their structures should consult both the licensor of the connection and the related authorities to determine the applicability and acceptability of the individual connection type for the specific design application.

SidePlate Connection System

NIST-BPRPPCB, 2006 references patented SidePlate connection system as being used in both new and retrofit construction, which is shown schematically in Figure 32. Main innovation of its connection geometry centers around a physical separation (commonly referred to as a "gap") between the face of the column flange and the end of the beam, by means of parallel full-depth side plates, which inherently eliminates the highly-restrained condition and the high-order tri-axial strain concentrations that are intrinsic to the basic geometry of 'traditional' moment connection systems. Instead, all moment load transfer from the beam to the column reverts back to simple statics, using predictable equivalent force couples and basic engineering principles. (NIST-BPRPPCB, 2006)

The parallel full-depth side plates act as robust continuity elements to sandwich and connect beam-to-beam, across the column, and are designed with adequate strength and stiffness to force all significant plastic behavior of the connection system into the beam, which, in a worst-case "missing column" scenario, insures the formation of plastic hinges at beam ends, outside the beam-to-column joint itself. It is properties are stated by the patent institute that SidePlate steel frame connection technology replicates the torsional and lateral bending stiffness and strength properties of reinforced concrete beams and girders, in the vicinity of the beam-to-column joint, by creating steel box sections with continuous, robust structural steel plates. Additionally it is also used in the common practice of blast resistant design in U.S. since, it improves the dynamic performance properties when subjected to blast loading. In addition, it is stated that the continuous full-depth side plates replicate the continuous top and bottom main reinforcement steel through the column(s), typically provided in modern reinforced concrete structures to insure discrete beam-to-beam continuity across the column. Moreover, according to NIST-BPRPPCB, 2006 reliance on panel zone deformation of the column's web is eliminated by providing three panel zones [i.e., the two side plates plus the column's own web]. The top and bottom beam flange cover plates are used to bridge the difference between flange widths of the beam(s) and the column.

According to NIST-BPRPPCB, 2006 SidePlate connection's tested cyclic rotational capacity exceeds all current Connection Qualification Criteria [AISC (2002) Seismic Provisions Structural Steel Buildings and FEMA 350] for large inter-story drift angle demands from earthquakes.

Information on the web site of Side Plate Inc. states that the SidePlate moment connection was selected by the General Services Administration (GSA) for blast and progressive collapse testing, as part of a first-ever joint GSA Steel Frame Blast and Progressive Collapse Test Program, to investigate the behavior of conventional steel frame construction and its beam-to-column connections when subjected to high-level bomb blast and subsequent progressive collapse conditions.

SidePlate steel frame connection system outperformed the post-Northridge 'traditional' Welded Unreinforced Flange (WUF-B)connection by:

- 2- and 3-times the gravity load carrying capacity
- 2-times the rotational ductility
- 5-times the energy absorption

Additional information on the SidePlate connection including use, modeling characteristics, full scale testing and performance can be obtained directly from www.sideplate.com.

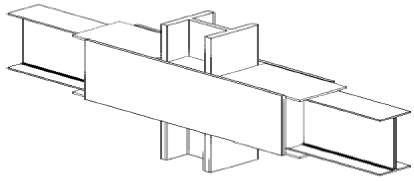


Figure 32: SidePlate moment connection system.

SlottedWeb Connection

The patented SlottedWeb connection is shown schematically in Figure 33. It is similar to the Welded Unreinforced Flange (WUF) moment connection with the addition of slots in the column and/or beam webs to separate the flanges from the web. It is stated at the manufacturer's web page that separating the beam web from the beam flanges reduces the large stress and strain gradients across and through the beam flanges by permitting the flanges to flex out of plane. Moreover, the slots in the beam web adjacent to the beam flanges allow the beam web and flange to buckle independently, thereby eliminating the degrading of the beam strength caused by lateral torsional buckling. The connection has been evaluated and accepted for use as a moment connection in Special Moment Frames (SMF) by the International Conference of Building Officials, ICBO ER-5861.
Additional information on the connection and its performance can be obtained directly from Seismic Structural Design Associates, Inc. web site: www.ssda.net

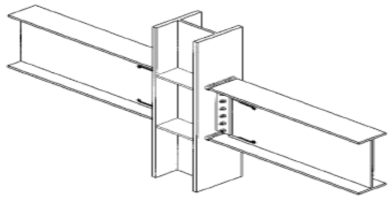


Figure 33: SlottedWeb moment connection.

CHAPTER 5

CASE STUDY

5.1 INTRODUCTION

5.1.1 Properties of Model Steel Building

In order to analyze blast effect on a structure a regular hybrid framed six storey steel building modeled by Ozer 2007 according to the regulations of Turkish Earthquake Code, TS 648 (TSE, 1980) is taken as the sample model for analysis purpose. Table 9 shows the structural steel elements that constitute the model structure. Lateral load resisting frame in X direction is high ductility concentrically braced frame system and in Y direction high ductility steel frame system. (Figure 34-36) Slabs are composite cast in-situ concrete over trapezoidal sectioned aluminum panels and supported by steel beam girder system. Auxiliary beams of 2 m spacing are pin connected to main beam elements. Main beam elements are pin connected to the columns in the direction of column weak axis and rigid connection (connection that transfer moment) in the direction of strong column axis. (Figure 34)

Earthquake characteristics of the building designed as residential or office use are taken as; effective ground acceleration (Earthquake region I) A_o=0.40, Building importance factor I=1, local soil class Z2 (T_A=0.15 s, T_B= 0.40 s). Earthquake reduction coefficient (R), is taken as R_X= 7 in X direction and R_Y= 8 in Y direction.

Total weight of the structure is around 850 tons. Its first and second modal periods are 0.77s and 0.59s at +y and +x directions respectively. Third modal period is around 0.22s and other frequencies are at around 0.1s, before they diminish.

Table 9: Steel Frame Element Types for the model structure.

Frame Element Types of the Model Steel Building	
Element Type	Section Type
Secondary Beams (All Stories)	IPE 270
Main Beams of Axes A-D (All Stories)	IPE 270
Main Beams of Axes 1-5 (1st, 2nd & 3rd Stories)	IPE 400
Main Beams of Axes 1-5 (4th, 5th & 6th Stories)	IPE 360
Columns of 1st, 2nd & 3rd Stories	HE 400 B
Columns of 4th, 5th & 6th Stories	HE 360 B
Steel Bracing Elements	□ 140x140x8

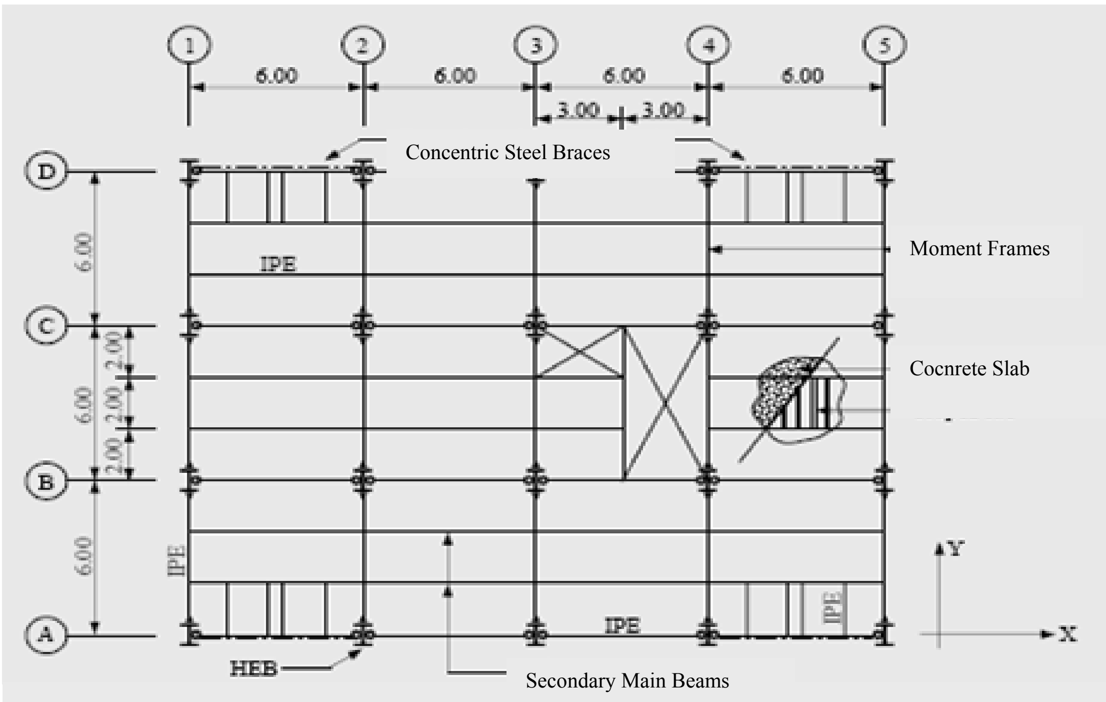


Figure 34: Story plan of six story model steel building.

(From work of Irtem and Turker, 2007)

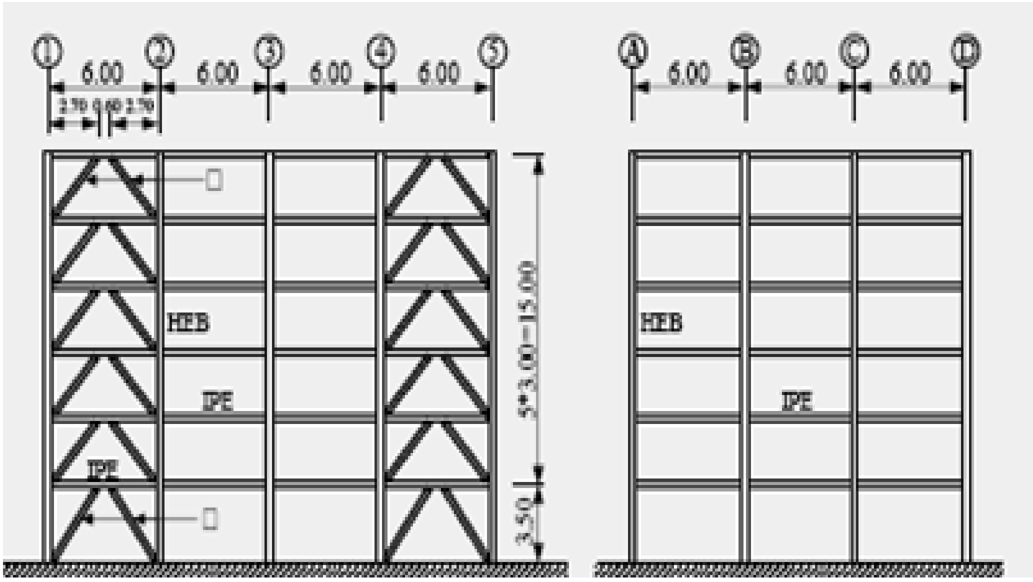


Figure 35: Framing system of the building in perpendicular directions.
(From work of Irtem and Turker, 2007)

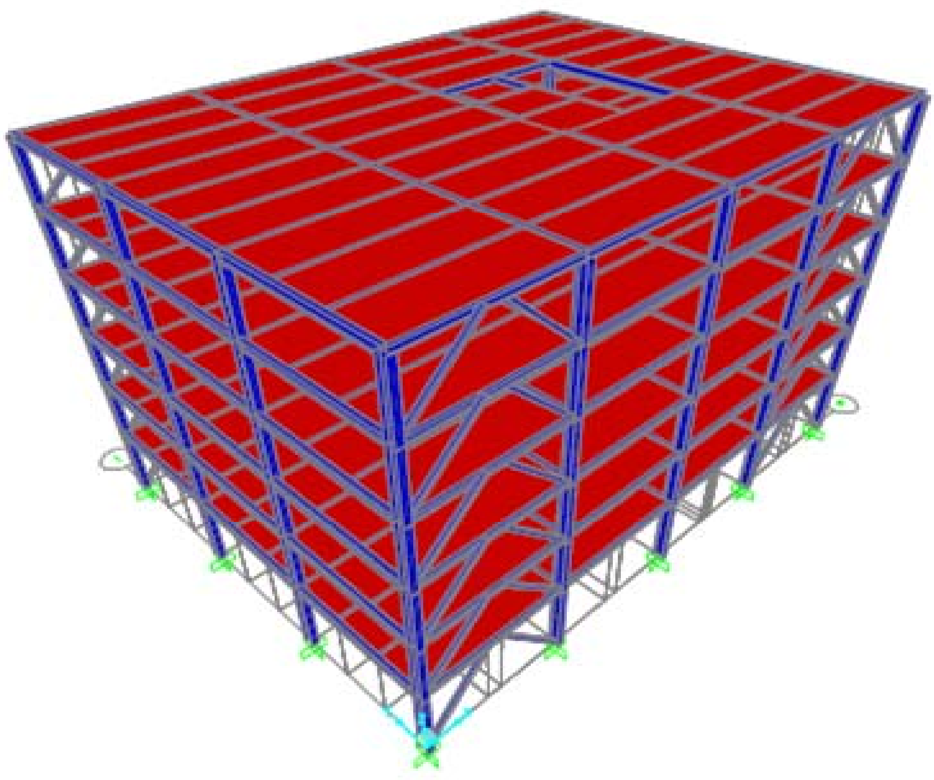


Figure 36: 3 Dimensional Model of the Structure. (SAP 2000).

5.2 ESTIMATION OF BLAST PRESSURE ON MODEL STRUCTURE

A.T.-Blast (Anti-Terrorism Blast) which is a software program developed and distributed by Applied Research Associates, Inc. at no cost, for the purpose of estimating the blast pressure and impulse from a high explosive detonation as a function of standoff distance is used as a tool for estimation of blast pressure on our structure.

Software estimates the blast loads that develop during an open-air explosion. The program allows the user to input minimum and maximum range, explosive charge weight, and angle of incidence. From this information, AT-Blast calculates the following values: Shock Front Velocity (V), Time of Arrival (TOA), Pressure (P), Impulse (I), and duration (td). The results are displayed on screen in a tabular format and may be printed. In addition, the resulting pressure and impulse curves may be displayed graphically.

5.3 ANALYSIS RESULTS

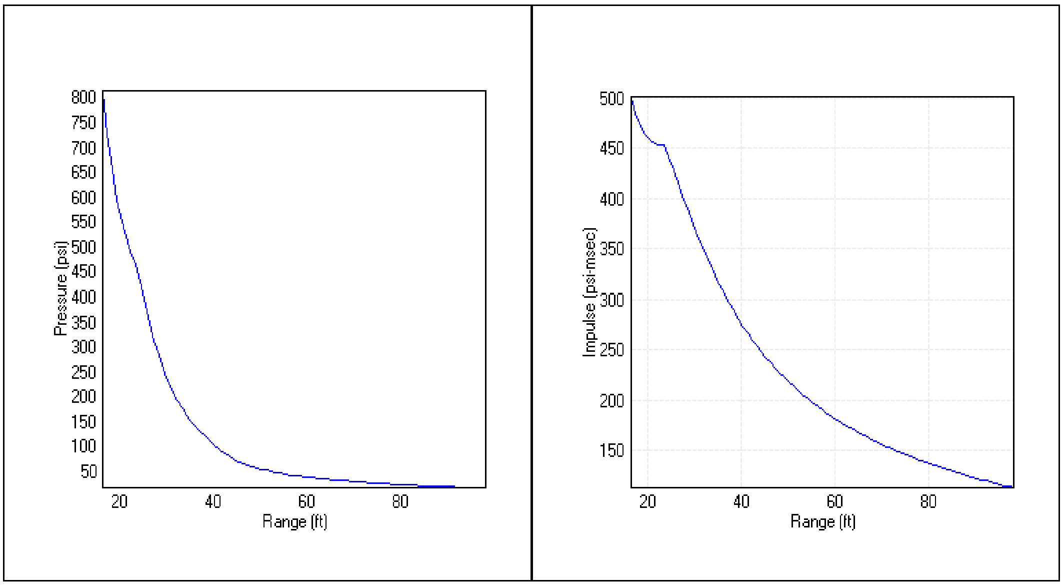
First type of analysis which is a linear static type is based on method of GSA, 2003 described in the fourth chapter and second analysis is based on Nonlinear static pushover analysis proposed by Guo & Gilsanz, 2003. Pushover curve of the structure for lateral blast loading of the structure is also shown for sake of information. Effecting dynamic pressure forces for nonlinear pushover analysis is obtained using AT Blast, which is an analytical blast calculation tool implementing the methods of TM 5-1300,1990. Pressure values are obtained for a case of charge weight of 500 kg Ammonium Nitrate Fertilizer/Fuel Oil (ANFO), because this is a reasonable amount of charge for this kind of residential/commercial building located at Balıkesir when compared with HSBC bombing of 2004. To remember that a charge of 1500 kg of ANFO was used in HSBC bombing for 18 story reinforced concrete building designed against earthquake in Istanbul.

Equivalent TNT coefficient for ANFO taken by AT Blast as default is 0.82 which means an explosion of 410 kg of TNT. This is a possible and reasonable amount of explosive to carry with a small truck or VAN type of car. Possibility of such an attack to a building located at Balıkesir is controversial, which raises the question of “why to attack such a building located at a small city, instead of a large and crowded one. But, this study is a first attempt to assess the behavior of a steel building subjected to a terrorist attack that is readily designed to Earthquake code of 2007. Therefore we shall assume that our structure is located at Istanbul which has almost the same seismic conditions.

In many cities our engineering structures are located very close to avenues or streets. In such a terrorist attack one can pass over the side walk, which is a natural barrier on our streets, with a truck and can crush into the structure. Therefore it is reasonable to accept a standoff distance of 5 meters off our structure in this analysis case. Pressure values affecting the frames of our analysis structure are given in the following table. Blast loading values on the model structure is shown in Table 10, pressure and impulse diagrams belonging to charge weight of 500 kg ANFO are as presented in Figure 37. For this case study it is assumed that subject structure is located in an isolated, uncrowded region since no information about the location and distance of the building to other structures is in hand. Therefore pressure values are determined with the assumption of no reflection from the nearby structures. Since information about surface cladding of the building is also not available it is accepted that no rigid surface on the faces of the building exists as a reasonable assumption. (Assumption of a skeletal structure, with all partitioning wall and glass cladding fail to resist at around pressure of 1-2 psi, forming a flexible structure and allowing drag force on the frames to be dominant with idealization of distributed force on the frames, which is reasonable.)

Table 10: Blast loading applied over the frames of the structure.

Range (m)	Velocity (m/msec)	Time of Arrival (msec)	Pressure (kPa)
5	1.82	1.66	3320.38
5.3	1.73	1.83	2989.5
5.61	1.65	2.01	2700.33
5.91	1.58	2.2	2446.54
6.22	1.51	2.41	2222.8
6.52	1.44	2.62	2024.99
6.83	1.38	2.83	1849.52
7.13	1.33	3.06	1693.28



(a) **(b)**

Figure 37: (a) Pressure and (b) impulse diagrams of explosion of 500 kg ANFO.

5.3.1 Analysis Results for GSA Approach

Analysis of the model structure is carried out against loading described in the GSA,2003. In this type of loading dead loads are multiplied by a coefficient of 2 whereas, live load is reduced with a coefficient of 0.5. Blast loading is applied on the structure as it is. Dynamic loading effect of blast is imparted into analysis by increase in the elastic modulus of steel by 1.2 times as suggested in GSA, PCADG.

Calculated values of shear, bending capacities of members of the model structure are as shown in Table 11. Table 12 and Table 13 shows the allowable DCR values of various members of the structure. These values are used as criteria for the demand-capacity comparison under vertical element removal conditions of GSA, PCADG.

No interior analysis cases are set to be run because the structure to be analyzed has no parking space and is all intended to function as residential and commercial offices.

Table 11: Flexural ultimate capacity calculation for analysis per GSA, PCADG.

FLEXURAL FAILURE MOMENT				SHEAR CAPACITY		AXIAL LOAD CAPACITY		
	WEAK	STRONG		WEAK	STRONG	WEAK	STRONG	
IPE 270	6.83	34.12	ton.m	25.66	39.66	247151.90	39.66	ton
IPE 360	13.47	71.84	ton.m	41.47	62.18	248386.22	62.18	ton
IPE 400	16.14	92.14	ton.m	49.54	69.98	265551.78	69.98	ton
HE 360 B	48.50	126.10	ton.m	64.80	194.40	157923.38	194.40	ton
HE 400 B	51.89	151.90	ton.m	77.76	207.36	177196.51	207.36	ton

Table 12: Allowable flexural DCR values for beams per GSA, PCADG.

BEAMS UNDER FLEXURE					
IPE 270 Flexure		**IPE 360 Flexure**		**IPE 400 Flexure**	
bf/2tf<=52/sqrt(Fye)		bf/2tf<=52/sqrt(Fye)		bf/2tf<=52/sqrt(Fye)	
6.62 <=	8.92	6.69 <=	8.92	6.67 <=	8.92
IPE 270 Flexure		IPE 270 Flexure		IPE 270 Flexure	
h/tw<=418/sqrt(Fye)		h/tw<=418/sqrt(Fye)		h/tw<=418/sqrt(Fye)	
40.91 <=	71.69	45.00 <=	71.69	46.51 <=	71.69
DCR=	3	DCR=	3	DCR=	3

Table 13: Flexural DCR value calculation for columns per GSA, PCADG.

COLUMNS-FLEXURE			COLUMNS-FLEXURE		
FOR HE 360 B			FOR HE 400 B		
FOR 0<P/PCL<0.5			FOR 0<P/PCL<0.5		
0<	0.03	<0.5	0<	0.05	<0.5
bf/2tf<=52/sqrt(Fye)			bf/2tf<=52/sqrt(Fye)		
6.67	8.92		0.04	8.92	
IPE 270 Flexure			IPE 270 Flexure		
h/tw<=300/sqrt(Fye)			h/tw<=300/sqrt(Fye)		
28.80	51.45		0.12	51.45	
DCR=	2		DCR=	2	

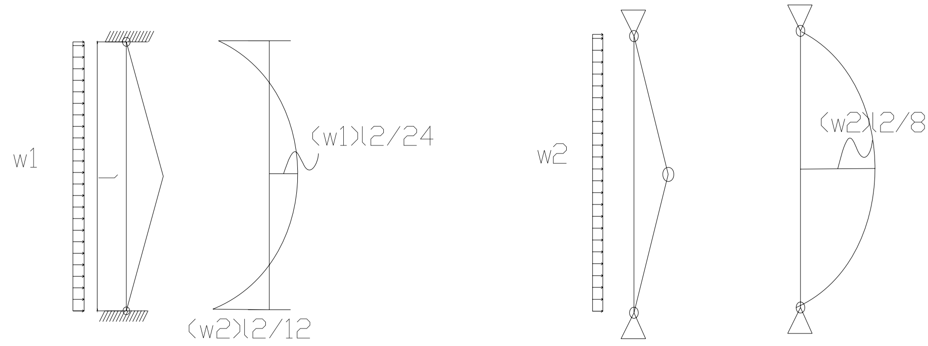


Figure 38: Formation of Three Hinge mechanism under bending moment action.

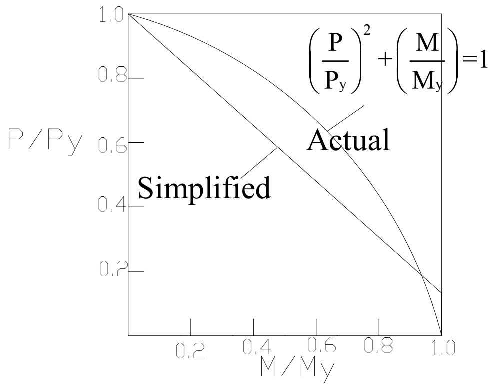
Formation of three hinge mechanism for a vertical column element under bending effect of blast pressure is illustrated in Figure 38. It is clear that, under gradually increasing load, moment diagram increases in amplitude until the moment at midspan reaches the value of plastic moment. Corresponding load W_1 to create plastic moment, $M_p = Z.F_y$ is given by

$$W_1 = M_p.\ 12/\ L^2 \qquad (26)$$

First, two hinges occur in both ends of the element under given loading and support conditions shown in Figure 38 on the left. Further increase in load causes the moment at midspan to increase while the end moments hold at M_p. Thus the midspan moment continues to rise until the midspan moment attains the value $M_{p.}$ It is obvious that these moments can increase no further. Therefore ,the beam now has three hinge mechanisms, so it has reached its load capacity. At this stage an extra moment of magnitude $W_2L^2/8$ is required to create a moment of $M_p/2$ to dissipate remaining midspan moment capacity of the element to form a three hinge mechanism.

$$W_2 = M_p.\ 4/\ L^2 \qquad (27)$$

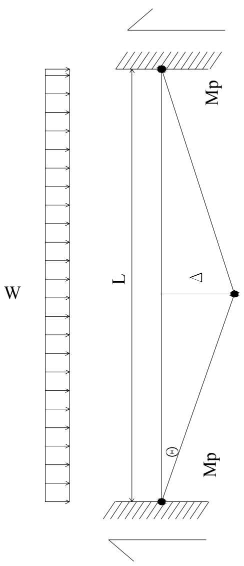
Moment capacity of an axially loaded column is also affected by axial load level. (Gaylord and Gaylord, 1972) Condition of high axial load level on the moment capacity of column elements should be considered by the following formula. Formula is also plotted in on the left hand side of Figure 39.



$$\frac{P}{P_y}+0.85\frac{M}{M_p}=1 \qquad \frac{P}{P_Y}\geq 0.15 \tag{28}$$

$$M=M_p \qquad \frac{P}{P_y}\leq 0.15$$

(From Gaylord and Gaylord, 1972)



$$4\,.\,\theta\,.\,M_p = W\,.\,L\,.\,\frac{L}{2}\,.\,\theta\,.\,\frac{1}{2}$$

$$M_p=\frac{W\,L^2}{16}$$

$$\frac{L}{8}\,.\,V_{max}=\frac{W\,L}{2}\,.\,\frac{L}{8} \tag{29}$$

$$V_{max}=\frac{8\,M_p}{L}$$

$$W=\frac{16\,M_p}{L^2}$$

Figure 39: Formation of shear failure mechanism under distribute pressure effect of explosion.

Among all these relationships between load effect and corresponding capacities it is found after numerical calculations that shear is the governing effect which is illustrated by Figure 39 and formula on the right. Therefore using general shear capacity equation Vall= Vult x A and shear equations given above , equivalent distributed load to the failure of a base story column element, thus removal of the element is computed as 99.2 t/m or failure pressure is 330.67 t/m^2 (or 470.32 psi or 3242.76 kPa) for the smaller of these equations. When pressure values of Table 10 for a charge weight of 500 kg ANFO and standoff distance of 5 m is examined it is seen that members having standoff distance less than 5.1 m will exceed shear capacity and model will be analyzed as failed member removed.

When demand moment, shear forces are compared with using the capacities shown in Table 11, it is observed that no DCR value is exceeded for one element removal in the framing system of the structure along the X direction (Figure 40). Maximum determined DCR values for this case are 0.95 for both beam element just above the removed column and column element one story above and one bay left of the removed column. Maximum determined deformation value for this case is as seen on Figure 41. Displacement of the joint 80, which is the joint where removed column is adjoining with other is 9.23 mm, which is a reasonable small displacement. This value together with acceptance criteria of GSA, PCADG indicates that structure is not susceptible to progressive collapse for this one column loss case.

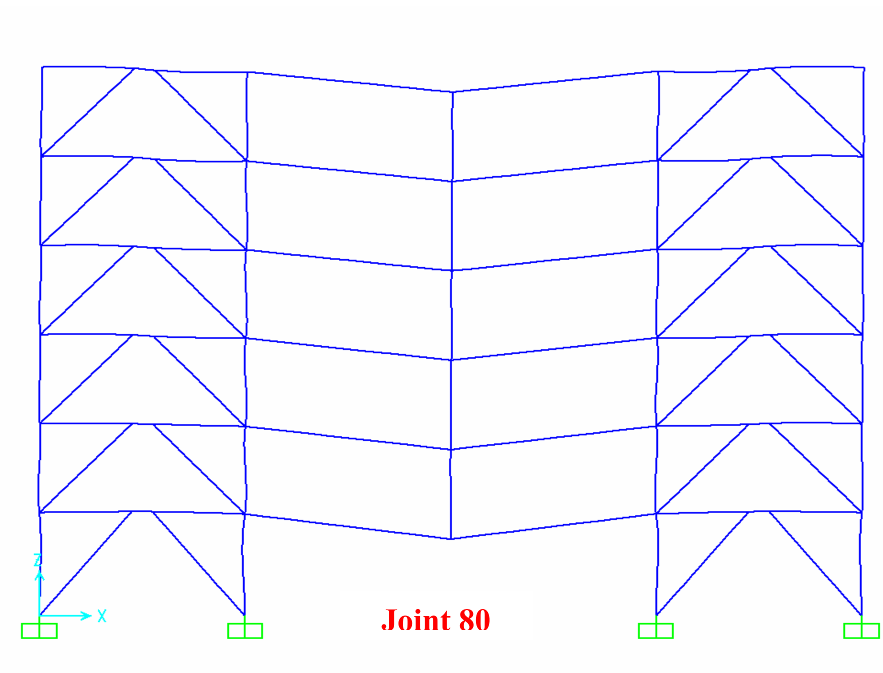


Figure 40: Deformed shape of the framing system after one column removal in long direction per GSA, PCADG.

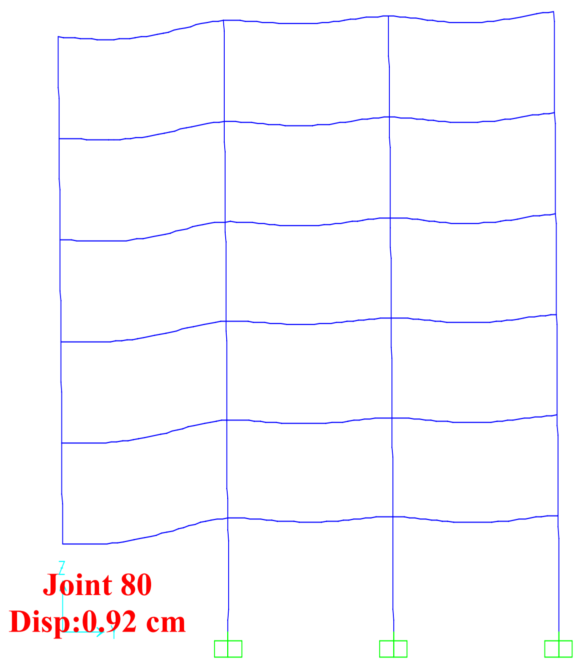


Figure 41: Cross section of the frame Deformed shape and maximum deformation of the framing system after one column removal in long direction per GSA, PCADG.

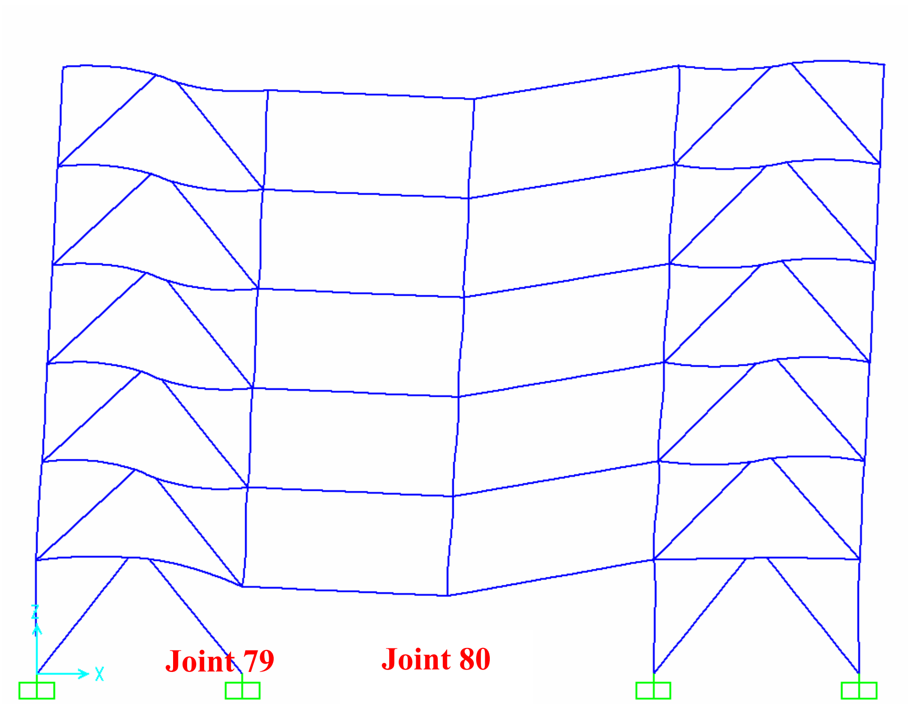


Figure 42: Deformed shape of the framing system after two column removal in long direction.

In addition to one column element removal dictated by GSA, PCADG structure is analyzed for two column removal in the front frame along the long direction of the structure. It is also determined that no DCR value is exceeded for two element removal whose deformed shape is shown in Figure 42. Maximum determined DCR values for this case are 1.60 and 1.82 respectively for the A1 and G1 axis columns just one story above the removed column in the weak and strong axis directions of these elements. Maximum determined deformation values for 4th and 7th axis frames this case are as seen on Figure 43. These deformation values together with acceptance criteria of GSA, PCADG indicates that structure is not susceptible to progressive collapse under two column removal case along the long direction of the structure.

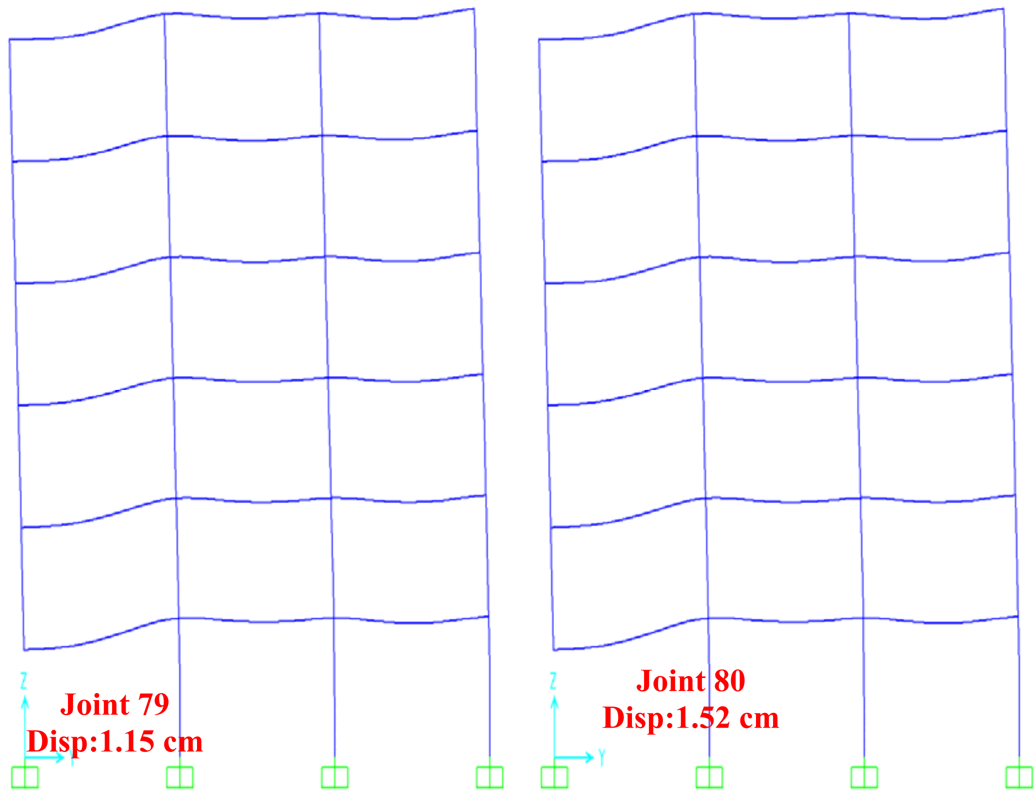


Figure 43: Deformed shape and maximum deformation of the framing system after two column removal in long direction. (cross-section of the frame)

Vertical element removal case for short direction (Y direction) of the structure has to be checked according to GSA, PCADG. First, one of the middle columns has been removed in accordance with GSA. Second, one additional column is removed just to check for its effect on progressive collapse initiation and structures vulnerability to progressive effect. When demand moment, shear forces are compared with using the capacities shown in table 11, it is observed that no DCR value is exceeded for one element removal in the framing system of the structure along the Y direction (Figure 44). Maximum determined DCR values for this case are 1.76 and 1.22 respectively for the 1D-1G axis beam and 1D columns just one story above the removed column. Maximum determined deformation values for this case are as seen on Figure 45. It is apparent from previous conclusions and from this analysis results that the structure is not vulnerable to progressive collapse.

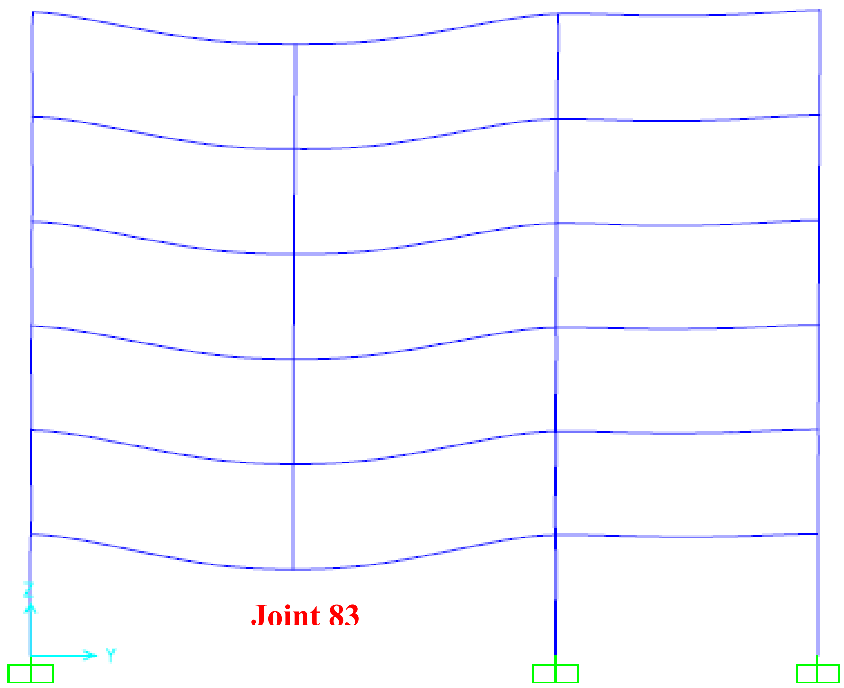


Figure 44: Deformed shape of the framing system after one column removal in short direction.

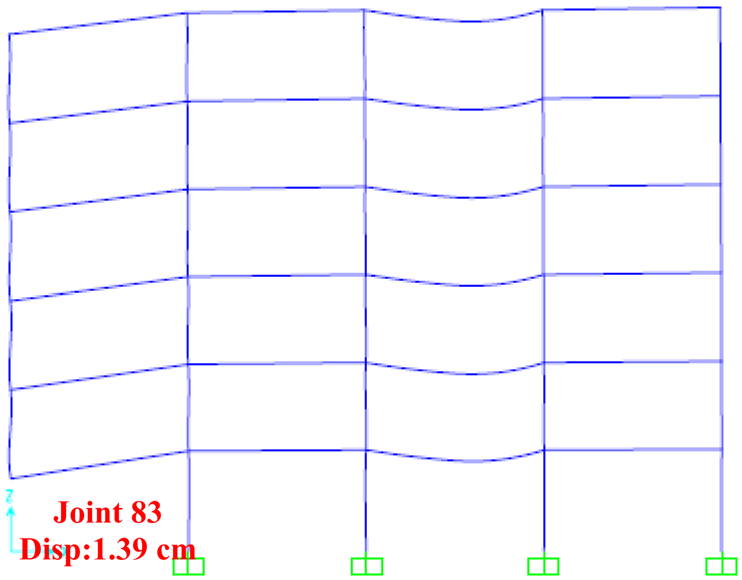


Figure 45: Deformed shape and maximum deformation of the framing system after one column removal in short direction. (cross-section of the frame)

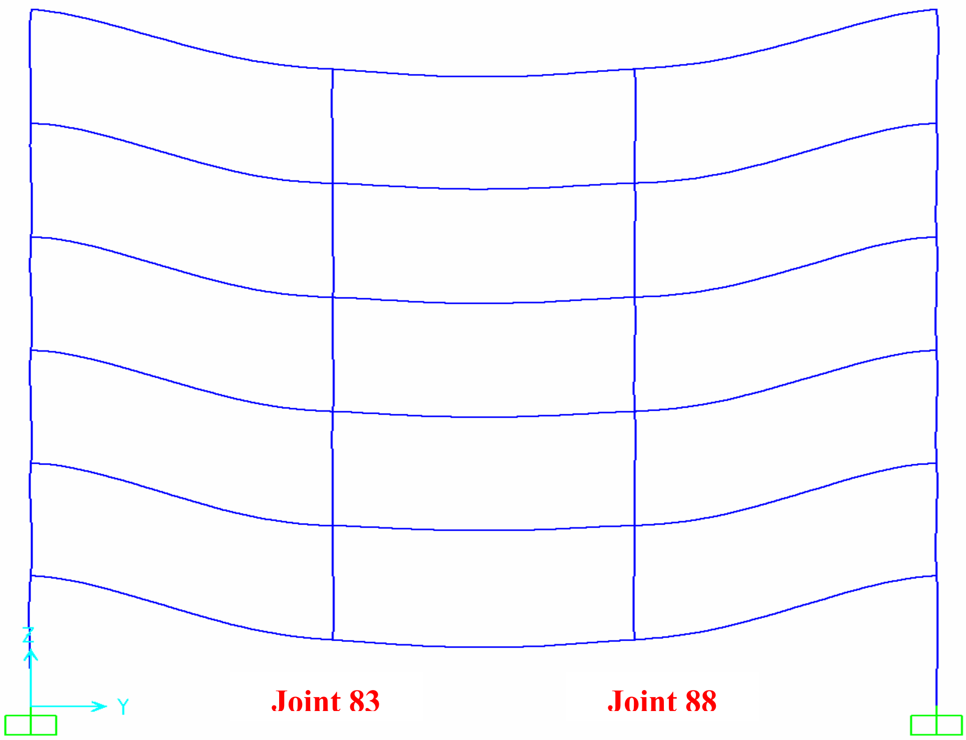


Figure 46: Deformed shape of the framing system after two column removal in short direction.

Additionally one more column removal case in the 18m long direction of the structure is analyzed for checking its effect on progressive collapse initiation and structure's vulnerability to progressive effect. Deformed shape of the relevant frame for two column removal is shown in Figure 46 and its associated maximum deformations are presented in Figure 47. It is determined that four of the members exceed the allowable DCR values. Members labeled 224, 913, 985 and 1057 have exceeded the DCR limit that is 2 for these all column members with section HE400B. Their corresponding maximum determined DCR values are 4.51, 7.83, 9.28, and 4.53 respectively for all these first story columns of 1A-J axis frame just one story above the removed columns. DCR values are all exceeded along the weak directions of these column elements. Maximum determined deformation values for this case are as seen on Figure 47. According to GSA, PCADG moment at these members has released and equal but opposite moment is applied at the end of these members. In order to release the moment rigid end offsets and hinges are assigned to member ends.

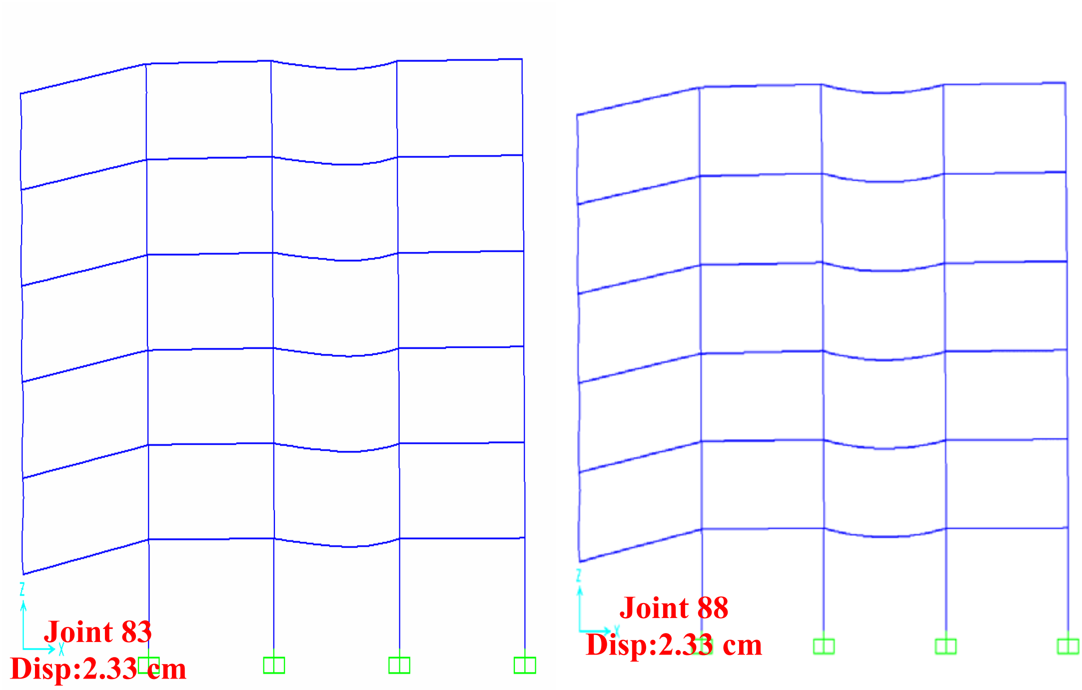


Figure 47: Deformed shape and maximum deformation of the framing system after two column removal in short direction. (cross-section of the frame)

After analyzing under these circumstances it is seen that some of the members exceeds the allowable DCR limits. Therefore it is said that vulnerable area of the structure for this two column removal case is a 6m x 18m area which is related with the bays of the removed columns. This constitutes an area of 108 m2 in total and is within the acceptable limits of GSA, PCADG for this kind of element removal according to section 4.3.6.1.

According to exterior analysis consideration one of shear walls at the corners of the structure, which is the bracing in this case, shall be removed half the way in each direction of the structure. But, since there exists no bracing along the short direction of the building this analysis condition is disregarded.

Further analyses are carried out to determine failure charge weight for the subject structure for a standoff distance of 5 meter. These analysis results are summarized in Table 14. Given number of failed elements are the numbers that are expected to occur at the side of explosion according to analysis procedure. Failure initiation is illustrated in a step by step manner in Figure 48.

Table 14: - Element/Structure behavior under different charge weights. (Vehicle Size)

Vehicle	Explosive Capacity, kg	Effective Air Blast (Member Failure) Range, m.	No of Failed Elements	No of Plastifying Elements	Progressive Collapse Vulnerability
Compact Sedan	225	3.91	None	None	No
Full Size Sedan	500	5.2	1 (1 columns)	None	No
Cargo Van	1815	7.75	2 (2 columns)	6 (2 columns, 4 beams)	No
Mini Truck	2500	11.9	3 (2 columns, 1 beams)	8 (4 columns, 4 beams)	No
14-ft Box Van	4535	13.65	10 (4 columns, 4 beams, 2 Braces)	All	Yes

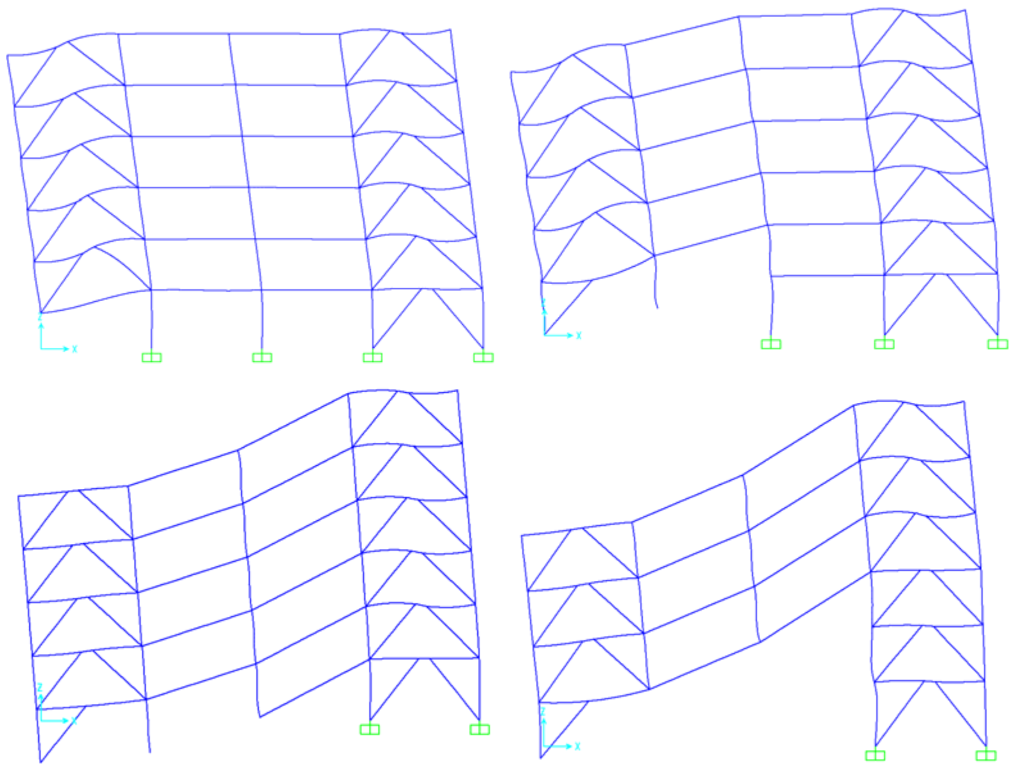


Figure 48: Step by Step Progressive Collapse initiation of the model building.

In conclusion it can be claimed that the structure is not prone to progressive collapse up to a charge weight of 4535 kg ANFO as analyzed per provisions of GSA, PCADG.

5.3.2 Analysis Results for Nonlinear Approach

A displacement controlled pushover analysis is carried out to obtain the results of nonlinear static analysis proposed by Guo & Gilsanz, 2003. Pushover analysis is set to continue from the initial results of dead load analysis. In this type of analysis no dead load increase or live load reduction is applied according to procedure proposed by Guo & Gilsanz, 2003.

For the first case of nonlinear analysis upper connecting joint of the removed +x direction middle column is loaded with a downward load of 59.37 tons, which is equal to reaction of the removed column. Figure 49 shows the result of nonlinear analysis for this case. Special data points of Figure 49 are given in Table 15. Default moment hinges for beam elements are assigned to the adjoining ends of the elements related with the removed element at upper story levels for the collapse mechanism of the structure at that part.

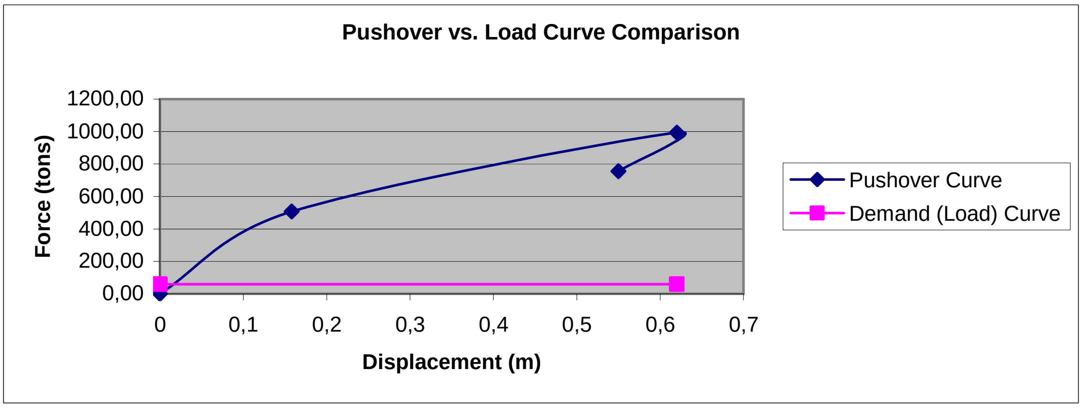


Figure 49: Pushover curve and Load curve comparison for the determination of Progressive Collapse for one column removal.in the long side.

Nonlinear analysis result imply that structure will survive removal of one column at a time, since demand force which is the reaction of the removed column is far more below the pushover capacity curve. Therefore according to Guo & Gilsanz, 2003 it is said that structure has a low potential for progressive collapse which is a compliant situation with the result of linear static analysis described above. It can be observed from nonlinear pushover analysis results that yield displacement of joint 80 which is the top joint of base story middle span column is 16 cm. Displacement obtained from linear analysis is far more below this yield value.

Table 15: Determining Data for Pushover and Load Curves of one column removal in the long side.

Displacement	Force
0	0.00
0.16	508.5 (Uy, Fy)
0.62	993.2
0.55	756
Displacement	Load
0	59.37
0.62	59.37

Figure 50 shows the result of nonlinear analysis for case in which two column elements, at second and third frames, are removed in the direction of 24 meter long side of the building. In this case joint of this later removed column is loaded with a downward load of 84.04 tons, in total structure is loaded with a downward demand load of 143.41 tons, which is equal to sum of the reactions of removed column elements. Special data points of Figure 50 are given in Table 16. Default moment hinges are assigned to the joints of beams related with the removed column elements for collapse mechanism of the structure at upper story joints of the removed column elements.

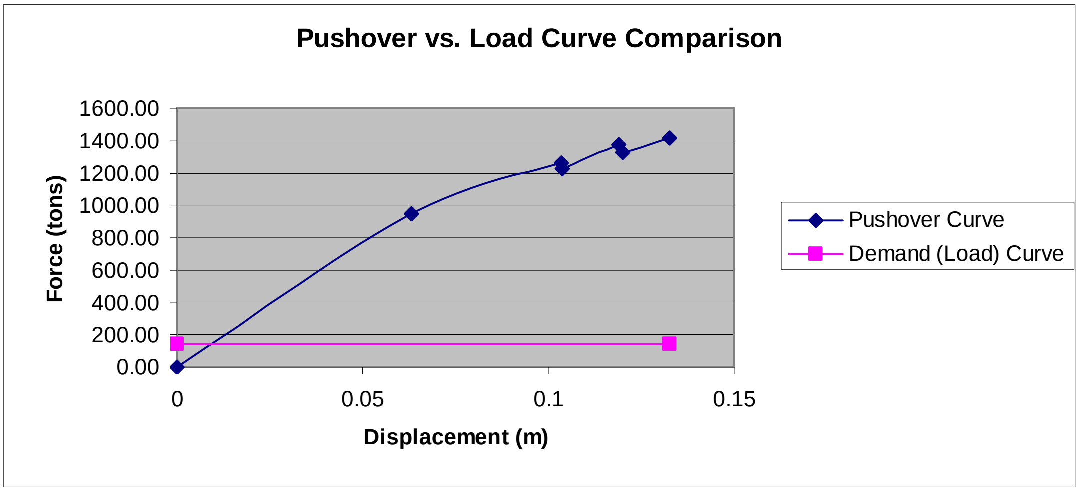


Figure 50: Pushover curve and Load curve comparison for the determination of Progressive Collapse two column removal in the long side of the building.

Nonlinear analysis results for removal of one pair of bracing element in the long side of the building imply that structure will survive removal of bracing and loading is far more below the capacity when compared to pushover curve. Therefore according to Guo & Gilsanz, 2003 it is said that structure has a low potential for progressive collapse It can be observed from nonlinear pushover analysis results that yield displacement of joint 80 which is the top joint of base story middle span column in this case is 1.64 cm. Displacement obtained from linear analysis is far more beyond this yield value.

Table 16: Determining Data for Pushover Curve and Load Curve of one pair of bracing element removal in the long side of the building.

Displacement	Force
0	0.00
0.06300	950.00 (Uy, Fy)
0.1035	1260.00
0.1038	1229.33
0.119	1374.33
0.1201	1326.67
0.1328	1418.67
Displacement	Load
0	143.41
0.1328	143.41

Figure 51 shows the result of nonlinear analysis for case in which one column element, which is loaded with a greater axial load of 58.88 tons is removed in the direction of 18 meter long side of the building. Special data points of Figure 51 are given in Table 17. Default moment hinge in the 3-3 moment direction is assigned to the adjoining ends of the beams connected with the joint of the removed column element. Axial-moment interaction hinges are assigned to column ends.

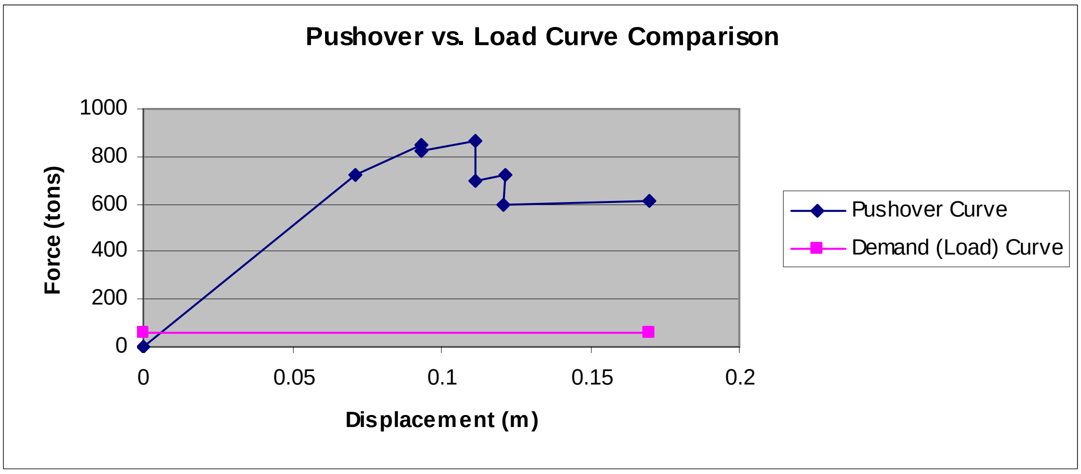


Figure 51: Pushover curve and Load curve comparison for the determination of Progressive Collapse for one column removal in the short side of the building.

Nonlinear analysis results for removal of one column element in the short side of the building imply that structure will survive removal of the element. Load curve is far below capacity when compared with pushover capacity curve. Therefore according to Guo & Gilsanz, 2003 it is said that structure has a low potential for progressive collapse It can be observed from nonlinear pushover analysis results that yield displacement of joint 80 which is the top joint of base story middle span column in this case is 7.10 cm. Displacement obtained from linear analysis is far more beyond this yield value.

Table 17: Determining Data for Pushover Curve and Load Curve of one column element removal in the short side of the building.

Displacement	Force
0	0
0.071	726.5 (Uy, Fy)
0.09310	851.5
0.09310	826.25
0.1116	861.75
0.1116	700
0.1217	726.25
0.1208	600.75
0.17	610
Displacement	Load
0	58.88
0.17	58.88

Nonlinear analysis results for removal of two columns in the short side of the building imply that structure will survive removal of elements. Load curve is below 50% limit when compared with pushover capacity curve. Therefore according to Guo & Gilsanz, 2003 it is said that structure has a low potential for progressive collapse It can be observed from nonlinear pushover analysis results that yield displacement of joint 80 which is the top joint of base story middle span column in this case is 5.95 cm. Displacement obtained from linear analysis is far more beyond this yield value.

Figure 52 shows the result of nonlinear analysis for case in which two column elements, at second and third frames, are removed in the direction of 18 meter long side of the building. In this case joint of this later removed column is loaded with a downward load of 104.13 tons, in total structure is loaded with a downward demand load of 163.01 tons, which is equal to sum of the reactions of removed column elements. Special data points of Figure 52 are given in Table 18. Default moment hinges are assigned to the joints of beams related with the removed column elements for collapse mechanism of the structure at upper story joints of the removed column elements.

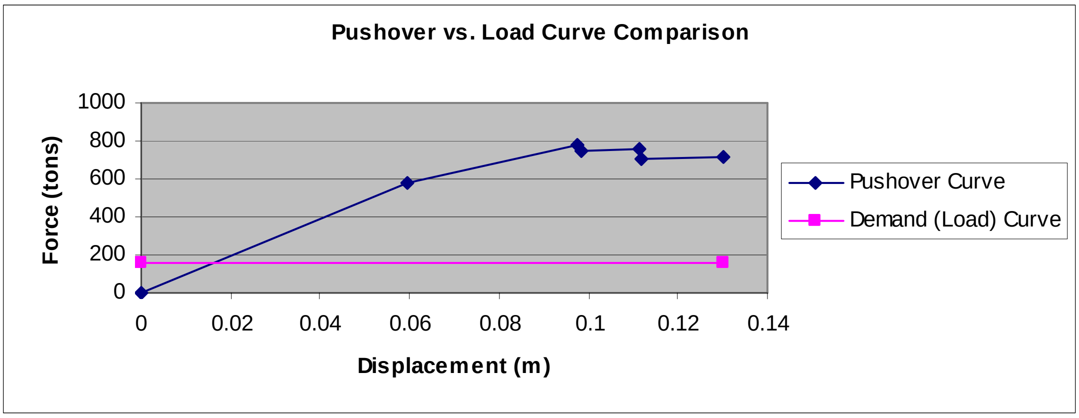


Figure 52: Pushover curve and Load curve comparison for the determination of Progressive Collapse two column removal in the short side of the building.

Table 18: Determining Data for Pushover Curve and Load Curve of two column element removal in the short side of the building.

Displacement	Force
0	0
0.0594	574.8 (Uy, Fy)
0.09750	780.8
0.09830	745.6
0.1112	762
0.1116	704.8
0.13	712
Displacement	Load
0	163.01
0.13	163.01

5.4 DISCUSSION OF RESULTS

Obvious from the analysis results, linear static method of GSA and proposed nonlinear static method of Guo & Gilsanz, 2003 represents a correspondence. They all classify this regular steel braced frame building as being not prone to progressive collapse up to a charge weight of 4535 kg of ANFO under the standoff distance of 5 meters with assumption of no blast wave reflection from nearby structures (assumption is due to lack of information about nearby structures). This much of a charge is the one that can only be transported by a 14 feet box type van vehicle, which is considerably large. Even if some collapse is expected to occur in the short direction of the building according to GSA, PCADG by the effect of a charge of 1815 kg ANFO, extent of this collapse is within the acceptable limits of the GSA, PCADG. Therefore both GSA and nonlinear method of Guo & Gilsanz, 2003 classifies this type of braced steel frame type building as non-susceptible to progressive collapse under this charge, standoff distance and unreflected pressure wave assumption both in short and long direction of the building.

Most accurate results for this kind of dynamic loading on a steel structure can be obtained through a nonlinear time history analysis. But as recognized this kind of analysis is very time consuming and difficult to perform. Even if it is performed, meaning an in-depth analysis of the structure, most of the structural details remain to be unknown to the analyst, since this is an existing structure and most of the details are assumed such as connection details.

As a result it can be stated that analysis method of GSA and nonlinear method of Guo & Gilsanz, 2003 gives consistent and easy to interpret results. Considering the conditions and assumptions made it will be reasonable to use these results to classify this type of regular braced steel frame building as not prone progressive collapse, in other means not prone to disproportionate collapse up to a charge weight of 4535 kg ANFO within a standoff distance of 5 meter under unreflected blast conditions. (failure charge reduces about ¼ of unreflected failure charge under reflected conditions)

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 SUMMARY

Concern about explosive effect on engineering structures evolved after the damage of Second World War. Beginning from 90's with the event of bombing Alfred P. Murrah Federal building located in Oklahoma City this concern deepened and with the attack to World Trade Center twin towers on September 11, 2001 it is peaked. Studies conducted on this issue show that many design code does not consider blast effect to the structures both internal and external. Recent design codes mainly focus on earthquake resistant design and strengthening of the structures. These design methodologies may sometimes satisfy current blast resistant design philosophy, but in some cases code compliant designs may not provide recognizable resistance to blast effect especially for reinforced concrete structures. Therefore designer should carry out earthquake resistant design with the blast resistant design knowledge in mind in order to be able to select the most suitable framing scheme that provide both earthquake and blast resistance. This is only possible if designer deeply understands and interprets blast phenomenon.

In this study, it is intended to introduce blast phenomenon, basic terminology, past studies, blast loading on structures, blast structure interaction, analysis methodologies for blast effect and analysis for blast induced progressive and disproportionate collapse. Final focus is made on the Turkish Earthquake Code Design procedure for steel structures and a case study is carried out to determine whether or not a steel structure designed according to 2007 code requirements comply with blast resistance requirements.

To achieve this goal firstly basic terminology related with materials of explosives and blast phenomenon is introduced. After introduction of basic terminology blast phenomenon, types of explosives and explosions, blast loading and analytical equations of blast, blast structure interaction and dynamic effect of blast and failure modes of blast loaded structures are discussed in chapter two. Chapter three summarizes information gathered from past experiences and observations in titles of building behavior as brittle and ductile buildings and experience on blast behavior of steel structures.

In chapter four a phenomenon related with blast namely progressive collapse is explained in detail through illustrative examples. Following methods of preventing progressive collapse and codes developed to prevent this behavior are discussed in critical points. Afterwards first progressive collapse analysis methodology is explained in detail in chapter four, based on GSA, 2003 provisions to provide basis for further analysis in chapter five. Fourth section of chapter four discusses another analysis methodology based on nonlinear static pushover analysis developed by Guo and Gilsanz, 2003. In the fifth part of chapter four blast and steel frame type, earthquake resistant design and blast resistance relationships are discussed through findings of past studies. Finally mitigation basics, principles and methodologies are discussed for building type steel structures which could provide source of information for future studies.

Chapter five is the analysis of a case study adopted from a readily available design of a steel building designed according to New Turkish Earthquake Code, 2007 in Balıkesir. Properties of the analyzed structure and constructed model of SAP 2000 are illustrated in the first part of the chapter. Then estimation of blast loading using public free software developed for U.S. Army Corps of Engineers, AT Blast is investigated. Analysis results for model building as for GSA, 2003 and nonlinear pushover procedures are given in detail in the third section of chapter five.

6.2 CONCLUSIONS

As discussed earlier analysis results for linear static method of GSA and proposed nonlinear static method of Guo & Gilsanz, 2003 represents a correspondence. They all classify regular steel braced frame type building up to a charge weight of 4535 kg ANFO within a standoff distance of 5 meters with assumption of no blast wave reflection from nearby structures (assumption is due to lack of information about nearby structures) as not prone to progressive collapse. This much of a charge is the one that can only be transported by a 14 feet (4.5 m.) box type van vehicle, which is considerably large. Even if some collapse is expected to occur in the short direction of the building according to GSA, PCADG, extent of this collapse is within the acceptable limits of the standard. Therefore both GSA and nonlinear method of Guo & Gilsanz, 2003 classifies this type of structure as non-susceptible to progressive collapse in both directions. While obtaining these results blast load duration (dynamic loading) and corresponding dynamic displacement (drift) was ignored for determination of failed (removed) elements and conservatively blast load is taken as quasi-static loading.

Prediction of the blast-induced pressure field on a structure and its response involves highly nonlinear behavior. Computational methods for blast-response prediction must therefore be validated by comparing calculations to experiments. Considerable skill is required to evaluate the output of the computer code, both as to its correctness and its appropriateness to the situation modeled. Actually in literature programs listed as accurate as possible are the one making use of computational fluid and solid mechanics. Use of this kind of software gives better results for blast behavior and progressive collapse estimation. But, uncertainty about existing construction may remove the need for sophisticated blast analysis; due to fact that there may be no point in a precise determination of the presumed behavior where no equally precise understanding of the existing structure or its connections is available. By all means this study was an initial attempt to predict blast behavior of model steel structure with the tools at hand.

In the modern and developed countries of the world steel is the most common construction material especially for crowded commercially valuable cities of that country. Turkey is one of the world's fastest developing countries and is a candidate for intensive use of steel as construction material. For high-risks facilities such as government and commercial buildings, design considerations against extreme events (bomb blast, high velocity impact) are very important. It is recommended that guidelines on abnormal load cases and provisions on progressive collapse prevention should be included in the current Building Regulations and Design Standards for our country. Requirements on ductility levels possibly help to improve the building performance under severe load conditions. Therefore it will be a proactive action to impart regulatory provisions into our disaster code against blast or any other extreme loading event to prevent life and property loss, especially for our fragile economy.

As a result it can be stated that analysis method of GSA and nonlinear method of Guo & Gilsanz, 2003 gives consistent and easy to interpret results. Considering the conditions and assumptions made it will be safe to use these results to classify this kind of regular steel framed structure braced at one direction as not prone progressive collapse, in other means not prone to disproportionate collapse up to a charge weight of 4535 kg ANFO within a standoff distance of 5 meter under unreflected blast wave assumption for this kind of initial analysis effort.

6.3 RECOMMENDATIONS FOR FUTURE STUDY

Key issues that remain unresolved concerning progressive collapse mitigation include topics listed below. Reseacher of this and related subjects are further encouraged to investigate these effects for their studies:

- The specific mechanics by which a moment frame transfers from a flexure dominant system to a tensile membrane,

- The reserve axial tension capacity of steel beam-to-column connections (i.e.,“simple” and moment resisting) after reaching significant inelastic rotations,

- The importance and impact of analysis approaches chosen; e.g., is a static linear alternate path analysis predictably conservative or unreliable?

- The overall effectiveness of progressive collapse mitigation provisions for buildings subjected to “real” threats

- Column connection performance including severe beam and column twist, lateral bending, and strain rate effects on weld and base material ductility.

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