Development of a Software Tool to Investigate the Local & Global Response of Buildings to Blast Loading

By

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Bachelor of Engineering in Structural Engineering
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Submitted to the Department of Civil and Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of

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Abstract

Well-publicized intentional and accidental explosions in the last two decades have exposed the lack of resilience in structures triggering disproportionate failure. This has fuelled change in the civil engineering industry with government agencies leading the way. This research further contributes to the topic of blast resistant design with particular focus placed on the response of building structures subjected to external blasts.

A software tool to assess the response of structure to blast loads is firstly presented. The proposed tool integrates a staged process and can broadly be broken down into three core modules: the blast load condition, the response of the target structural element(s), and frame stability in the event of a support being compromised. By automating this process, the resistance of a building can be investigated under a number of possible blast situations in quick succession.

In addition, the application incorporates a design feature that sizes 2D moment frames for wind and gravity loading, for the sole purpose of studying blasts on different frame strengths and geometries. The latter stages of the report demonstrate the capabilities of the tool by firstly proposing standard input metrics based on industry norm, and following on from this exploring the effects of each input through a parametric analysis. Example input parameters include blast weight, standoff distance, wind speed, number of bays & stories, target column location and element plastic limits.

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1 Introduction

1.1 Background

Structures are designed to cater for conditions specific to their use and location. National Standards guide structural engineers with the development of a structural scheme, within the prescribed geometry, to satisfy the loading conditions specific to that structure. Based on collected data and the ability to approximate future trends through probabilistic analysis, loadings such as wind and seismic activity can be accurately predicted for future time periods. These principals unfortunately cannot be applied so easily to blast loadings.

A blast on a structure can either be pre-meditated or accidental. An example of the former would be most commonly known as a terrorist attack while the latter might arise from the ignition of a gas leakage. Focusing on terrorist attacks, the spectrum of possible explosion situations in addition to the limited collected data is such that it is extremely difficult to narrow a blast design procedure on a structure to a small number of possibilities. One thing is for certain, a terrorist attack will attempt to maximize failure of their target with the priority of causing loss of life.

In 1995, a domestic terrorist attack was carried out on the Alfred P. Murrah Federal building in Oklahoma City. A truck filled with homemade explosives, thought to be the equivalent of 4,000lbs of TNT, was parked in the close vicinity of the federal building and detonated. Aftermath studies by the National Institute of Standards and Technology (NIST) and the Federal Management Emergency Agency (FEMA) concluded that the blast initially wiped out 4% of the total floor area, and this led to a further 38% of the floor area being destroyed through progressive collapse. Figures 1.1 to 1.4 show the damage of the explosion and also display some of the characteristics of the blast. The blast was determined to be approximately 15.6ft from the nearest support of the Federal Building. Due to the lack of redundancy, the building was incapable of containing the initial 4% of damage, and therefore activated progressive collapse.
Studies of the initial structural construction drawings and design guidelines of that time period showed that the Federal Building was satisfactory for the design period, but seriously lacked the redundancy and continuity recommended for structures today.

Another example, which demonstrates the catastrophic results of structures with poor resilience under blast loading, is the partial collapse of Ronan Point apartment tower. In 1968, a gas explosion
occurred on the 18th floor of the building and this caused the collapse of a corner section of the building, killing 17 people. In this instance, the investigation of the original design concluded that the design techniques and construction methods were deeply flawed (ASCE). The superstructure consisted of precast panels and hollow core slabs with the connections relying mainly on friction to transmit forces, therefore seriously lacking capacity to re-distribute loads. An example of one of the main connection details can be seen in Figure 1.6 below, and this portrays the poor continuity of the detail. The floor slab bears onto the vertical panel and the primary restraint holding the panel and slab comes in the form of a clamped threaded rod. The Building Codes at that time were not satisfactory for these types of structures and have since been developed to provide better designed structures.

These unfortunate incidences, along with the events of 9/11 have led to significant research efforts on the abilities of a building to cope with adverse loadings, particularly through the provision of resilience and robustness in the superstructure.

1.2 Blast Loading

During an explosion, a rapid exothermic chemical reaction occurs producing a large amount of energy in a small-confined volume. The explosive material transforms into a hot, dense, high-pressure
gas, which expands outwards in an attempt to reach equilibrium with the surrounding air. This formation is known as a shock wave.

As the shock wave radiates outwards, the pressure decreases with the cube of the distance due to geometric divergence, in addition to energy dissipation with heating the air. Figure 1.3 shows the influence of distance on the resultant pressures changing significantly over a 20ft height from 8,600psi to 500psi. The positive shock wave dispersing out from the source at supersonic velocities is followed with a negative shock phase where the pressure of the explosion dips below the ambient temperature. This pulls air back towards the origin of the explosion in an attempt to reach equilibrium. A typical time history graph of an explosion is illustrated in Figure 1.7 below.

![Figure 1.7 - Typical pressure time history (FEMA 426)](image)

1.3 Preventative Measures

Unlike natural hazards, acts of terrorism are executed with the intention of maximizing their consequences. In the context of blasts, terrorists deliberately locate explosives to exploit their target in an attempt to cause most devastating results. Therefore, it is virtually impossible to eliminate the threat of blast attacks entirely on an urban building. However, through a combination of structural and non-structural measures, the risk of potential catastrophic failure to a target building can be controlled and in most cases prevented. The U.S. Department of Homeland Security (USDHS) and U.S. Department of Defence (DoD) have been particularly active on this topic in recent years, publishing several documents and procedures to assist engineers with mitigating the impact of blast loading on structures, and in turn human life.
The bombing in Oklahoma City in 1995 unfortunately provides a fitting example of how through the absence of preventative measures, both structural and non-structural, can lead to the collapse of a building. Three key elements collectively triggered the collapse of the Federal Building and are described below. These will form the core of this research.

- **Loading Condition** – The two primary contributors of a blast are the weight of TNT $W$ and the distance between the detonated explosive and target building elements, known as the standoff distance $R$.

![Significance of Standoff Distance (FEMA 426)](image)

The standoff distance has more influence on the resultant blast pressure than the weight of the explosive, which is due to rapid energy dissipation over distance. Figure 1.8 demonstrates the significance of this, showing how distancing the threat reduces the cost of protection exponentially. That being said, increasing the standoff distance can be difficult to implement in urban areas with land being a precious commodity.

- **Element response** – The blasts addressed in this research will initially impact the building in a confined manner due to the highly localized pressures released. The strength of the immediate primary building components are consequently the first structural concern.

- **Frame stability** – Following the localized impact of the blast is the issue of global stability. If components are compromised, structure surrounding them must have sufficient capacity to create new load paths and re-distribute forces safely to other support members. Failing this will trigger disproportionate failure and possible collapse of the structure.
1.4 Motivation

The consequence of a terrorist attack on a building is a topic that has received a lot of attention in recent times. Government agencies have published documents to assist engineers with the design of resilient buildings capable of withstanding adverse loadings. One such document issued by the UFC, 'Structures to Resist The Effects of Accidental Explosions' describes methods for quantifying design pressures, impulses and durations of an explosion based on empirical data, while also enclosing "design procedures and construction techniques whereby propagation of explosion or mass detonation can be prevented" (UFC, 2008). While this document presents a comprehensive breakdown of how one would go about assessing structural elements for a blast, it is not widely used in the industry. In many cases, the localized assessment stage is omitted and the assumption of structural support(s) failing is automatically made. Focus is therefore placed on the structures ability to cope with a support removal by redistribution of loads to adjacent supports and safely into foundations.

To date, there is an absence of a common procedure to fully assess the structural integrity of a building subjected to a blast load. This research will therefore seek to develop an automated software application with the ability of simulating a blast event, assessing the immediate components of a structure and further investigating possible progressive collapse of a building. The second part of the research will conduct studies using the constructed application and varying input parameters (e.g. blast magnitudes, building geometries, geographical locations, loading conditions, material limits etc.) to provide a better understanding on blasts and their effects on buildings. Furthermore, investigations on various building topologies will be carried out to see if certain geometries are more susceptible to blasts, and if in fact certain buildings are inadvertently blast resistant due to building code regulations and specific design loading requirements.
2 Structural Response to Blasts

An explosion can impact a structure from both a local and global perspective. As previously noted, if a primary element of a building fails, then the structure surrounding that element must be robust enough to cater for the additional loads, and create new load paths. Failing this will instigate disproportionate failure. Subsequently, engineers with the task of designing a structure for blast events must consider the target-localized area of the blast in combination with overall global stability of the structure.

"This combination of threat-specific and threat-independent design provides the most robust structural systems that are able to survive both the anticipated and the unimaginable catastrophic events" (O'Duesnberry, 2010).

2.1 Building Enclosure

The first component of a building to experience the pressure of an external blast will be the building envelope. Cladding commonly made up of glazing and infill walling has the potential to cause serious injuries to the building occupants. Flying debris entering the building can cause lacerations and blunt trauma injuries, and potentially fatalities. Additionally, if a blast pressure infiltrates the interior of a building, occupants become further at risk, becoming susceptible to eardrum rupture, lung collapse and also being hurled against solid objects.

Nowadays, when a building is designed for blast loading, particular attention is placed on the cladding design. Brittle failure must be avoided, as ductile flexural inelastic systems that can reduce the spread of debris are favoured. Glass, one of the primary materials used as a building enclosure has been advanced to cope better with impulsive pressures. Laminated glass, typically used in the automotive industry, is composed of two or more layers of monolithic glass & inter layered with a polyvinyl sheets. This form improves the ductility of the glazing systems and better holds fragments together after breakage. For these reasons, it is now being used in the building industry.

Glazing support systems have also received attention. Cable support systems shown in Figure 2.1 are employed to absorb energy from the blast, and subsequently reduce impact on the superstructure.
The interaction of the building envelope and primary frame is highly complex when considering impulsive pressures. The first question that needs to be asked is whether the cladding remains intact, receives considerable damage or is totally removed. Assuming the cladding can resist the blast, how will it respond with the primary frame? The answer depends on the natural properties of the cladding and support system. For the case of the cable support systems, the natural period will be orders of magnitude larger than the natural period of the primary structural components supporting it, and will prolong the duration of the load transfer between the building envelope and building frame. Therefore, in a simplified description, the primary member can initially deal with the blast acting on the member surface and following this cater for the loads transferred from the pressure acting on the façade. This assumes the cladding remains intact which is questionable.

In the context of this research, it has been assumed that the cladding has been compromised and transmits no loads to the primary frame. The simulated blasts are highly impulsive and substantial in magnitude, generating reflected pressures of up to 2000psi. With the cladding removed, the magnitude of the pressure acting on the primary support elements will depend only on the catchment area (see Figure 2.2) of the element exposed to the blast waves.
2.2 Structural Elements

Unlike seismic ground motions where the entire structure is excited, blast loading is more concentrated on the structure in the vicinity of the explosion. Perimeter structural components should therefore be robust enough to resist specific external blast events, noting that the structural performance will be that of inelastic, plastic behaviour rather than elastic. Otherwise the results would not produce feasible section sizes. In addition, similar to seismic design, the structure should have a “strong-column, weak-beam” structural arrangement to prevent the formation of plastic mechanisms on the columns and possible global instability concerns.
Boundary conditions are an important consideration when assessing columns as isolated elements. To simulate an accurate model, close consideration of the interaction between the element and the building frame is vital. This research will focus on fixed and pinned boundary conditions, which is an acceptable assumption due to the large difference in natural periods between individual components and those of the holistic structure. Possible column boundary conditions are shown in Figure 2.4 and will be discussed in more detail in Chapter 3.

![Figure 2.4 - Possible column boundary conditions](image)

2.3 Frame Stability

Maintaining the global stability of a building is the fundamental objective when considering blast loading. Unlike general loading conditions (e.g. wind and gravity loads), where a frame is designed to remain elastic, a blast resistant building, due to the magnitude and uncertainty surrounding the loading, must be analyzed to behave in-elastically. Subsequently, the assessment of a structure under blast loading requires heavy computational analysis to best capture the non-linear dynamic response.

As noted above, recent historic events have led to increased efforts to develop a common standard to assist engineers with implementing a resilient structural design capable of withstanding extreme loading conditions. In 2009, the DoD published UFC 4-023-03 'Design of Buildings to Resist Progressive Collapse' to better educate engineers on resilient and blast resistant design. The report offers guidelines on different design methods that can be adapted to re-distribute loads in the event of a support being removed. Two such methods include the *Tie Force Method* and *Alternative Path Method*. The former incorporates vertical, horizontal
and peripheral ties to redistribute loads axially while the latter relies on the ductility and continuity of the structure to bridge over removed structural components.

Figure 2.5 - Increased beam span due to column removal

The *Alternate Path Method*, which will be the primary focus of this research, exerts greater demand on the material of a building frame under extreme conditions, which in turn allows for greater performance levels of the members. The process can either be applied using linear static, non-linear static or non-linear dynamic analysis. With a non-linear dynamic analysis, non-linear properties of the structure are considered as part of a time domain analysis, and the assessment covers the complete loading process, from stress free state through to non-linear behaviour under extreme loading leading to collapse. This precision best calculates the dynamic nature of the structure and the damage-state behaviour of the structural elements.
3 Application Development

The proposed tool can be categorized into four primary components:

1. Frame Analysis & Design
2. Blast Loading
3. Element Response
4. Frame Stability

Java has been adopted as the software tool for coding the application. Java, an object-orientated interface, allows the user to create modular programs and reusable code, which is desirable for the requirements of this research.

To achieve the objective set out for this thesis within the given timeframe, it was necessary to simplify certain areas of the analysis while still maintaining the sensitivity of the results to the parameters. Throughout this chapter, the methods and calculations used in each stage will be presented in detail and any assumption will be documented.

3.1 Frame Design

To facilitate a parametric study of different building geometries, the first requirement of the application will be to assess a building topology, determine the design forces in the critical members under the relevant ASCE 7 loading combinations, and furthermore produce a section size adequate to cater for the loads in accordance with the AISC Construction Manual.

3.1.1 Design Loads

A critical parameter for the frame design will be wind loading. The MWFRS (directional Procedure), as outlined in ASCE 7, will be adopted to determine the magnitude of wind pressure acting on the frame. The design pressure, which varies with height, will be applied as equivalent point loads at each story level.
Wind Pressure (Directional Procedure), $p = qGC_p - q_iGC_{pi} (lb/ft^2)$:

- Velocity Pressure (Directional Procedure), $q_z = 0.00256K_zK_t K_d V^2 (lb/ft^2)$
- Risk category for the building = III
- Basic wind speed = User Defined (mph).
- $K_d$ (Directionally Factor) = 0.85.
- $K_t$ (Topographic Factor) = 1.0.
- Exposure B, Surface Roughness B.

- $K_z = 2.01 \left( \frac{z_i}{z_g} \right)^2$ as $15ft \leq z \leq z_g$
  - $\alpha = 7, z_g = 1200$
  - $z_i = Height \ to \ floor \ i.$
- Enclosed Building, $GC_{pi} = \mp 0.18.$
  - $G$ (Gust Effect Factor) = 0.85.
  - $C_p$ (External Pressure Coefficient) = 0.8, windward wall.

| Table 3-1 - Wind loading calculation |

Gravity loading is also considered with dead and live loads being input parameters in the application and can be adjusted to suit the occupancy and floor make-up of the building.

3.1.2 Element Design

3.1.2.1 Column Sizing

The frame will be idealized as having rigid stories for calculating the design forces in the columns. The shear stiffness method will then be applied to determine the base shear in both the internal and external columns under lateral loads. Table 3.2 shows the breakdown of the steps taken to calculate the moments in the columns.

- Total base shear, $V_{tot} = \sum_{i=1}^{No.\,Floors} P_i \cdot Tributary\,Area.$
- Resolving base shear, $V_{tot} = 2V_e + jV_i$
- Applying the stiffness method to estimate moment in columns:
\[ k_E = \frac{3EI_{CE}}{h^3} \left(1 + \frac{1}{4} \frac{I_{CI}/I_{CE}}{I_{CS}/I_{CE}}\right) = \frac{2EI_{CE}}{h^3} \]

\[ k_I = \frac{3EI_{CE}}{h^3} \left(1 + \frac{1}{2} \frac{I_{CI}/I_{CE}}{I_{CS}/I_{CE}}\right) = \frac{2.18EI_{CI}}{h^3} \]

- As the floors are assumed infinitely rigid, the \( \frac{1}{4} \frac{I_{CI}/I_{CE}}{I_{CS}/I_{CE}} \) portion goes to zero.

- Calculate approximate shear in exterior and interior columns using \( \frac{V_E}{V_I} = \frac{k_E}{k_I} \)
- \( M_{\text{max(int)}} = V_I \times \text{Column Height (kip - ft)} \)
- \( M_{\text{max(ext)}} = V_E \times \text{Column Height (kip - ft)} \)

### Notation

<table>
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<th>Wind pressure at floor ( i ) (ksi)</th>
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<td>Number of internal columns</td>
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<tr>
<td>( E )</td>
<td>Young Modulus of Steel</td>
</tr>
<tr>
<td>( V_E )</td>
<td>External Column Base Shear</td>
</tr>
<tr>
<td>( I_{CE} )</td>
<td>External Column Moment of Inertia</td>
</tr>
<tr>
<td>( V_I )</td>
<td>Internal Column Base Shear</td>
</tr>
<tr>
<td>( I_{CI} )</td>
<td>Internal Column Moment of Inertia</td>
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<tr>
<td>( M_{\text{max}} )</td>
<td>Maximum Column Moment</td>
</tr>
<tr>
<td>( h )</td>
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<tr>
<td>( k_E )</td>
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</tr>
<tr>
<td>( L )</td>
<td>Beam Length</td>
</tr>
</tbody>
</table>

\( I_{xx} \) of Interior Column, \( I_{CI} = 1.5 \times I_{xx} \) of Exterior Column, \( I_{CE} \)

\( I_{CI} = 1.5 \times I_{CE} \)

### Table 3.2 – Calculation of column forces

With the bending moment and shear and the axial forces obtained, the columns can then be designed to the requirements of the AISC manual and in accordance with the strength design-loading combinations provided in ASCE 7.

**Buckling Check:**

- *Design Load Case* – \( 1.2DL + 1.6LL \)
- *Column Design Strength for flexural buckling limit state*, \( P_c = \phi_c P_n \), where \( P_n = A_g F_{cr} \)
  - \( \phi_c = 0.9 \) (Resistance factor for compression members)
\[ \lambda_c = \frac{KL}{r_n} \sqrt{\frac{f_y}{E}} \]

- For \( \frac{KL}{r_y} \leq 4.71 \sqrt{\frac{f_y}{E}} \):
  \[ F_{cr} = (0.658 \lambda_c^2) f_y \]
- For \( \frac{KL}{r_y} > 4.71 \sqrt{\frac{f_y}{E}} \):
  \[ F_{cr} = \left( \frac{0.377}{\lambda_c^2} \right) f_y \]

<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_c )</td>
<td>Design compressive strength</td>
</tr>
<tr>
<td>( P_n )</td>
<td>Nominal axial strength</td>
</tr>
<tr>
<td>( K )</td>
<td>Effective length factor</td>
</tr>
<tr>
<td>( A_G )</td>
<td>Gross column section area</td>
</tr>
<tr>
<td>( E )</td>
<td>Young’s modulus of steel</td>
</tr>
<tr>
<td>( F_{cr} )</td>
<td>Buckling stress for the section</td>
</tr>
<tr>
<td>( f_y )</td>
<td>Minimum steel yield stress</td>
</tr>
</tbody>
</table>

**Notation**

**Combined Bending & Compression:**

**Design Load Case – DL + 0.6W**

- \( \frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} \leq 1.0 \) for \( \frac{P_r}{P_c} \geq 0.2 \)
- \( \frac{P_r}{Z_{pc}} + \frac{M_{rx}}{M_{cx}} \leq 1.0 \) for \( \frac{P_r}{P_c} < 0.2 \)
- \( M_{cx} = 0.9 f_y z_x \)

<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_r )</td>
<td>Reduced axial compressive strength</td>
</tr>
<tr>
<td>( M_{cx} )</td>
<td>Available flexural strength (kip-ft)</td>
</tr>
<tr>
<td>( M_{rx} )</td>
<td>Required flexural strength</td>
</tr>
<tr>
<td>( z_x )</td>
<td>Column plastic section modulus</td>
</tr>
</tbody>
</table>

**Table 3-3 - Column design procedure**

The effective length factor for the column sizing will be set as an input parameter, and can be adjusted appropriately based on the user’s best engineering judgement.

**3.1.2.2 Beam Sizing**

In the event of a column being compromised, the adjacent beams will need to work together as one member to bridge over the removed support and re-distribute loads to adjacent supports (see Figure 2.5).
Table 3.4 describes the procedure used to design an appropriate beam size based on the defined building geometry.

**Flexural Check:**
- *Design Load Case* \(-1.2DL + 1.6LL\)
- \(\frac{M_{rx}}{M_{cx}} \leq 1.0\)

**Combined Bending & Compression:**
- *Design Load Case* \(-1.2D + 1.0W + 1.0L\)
- \(\frac{P_r}{P_c} + \frac{n M_{rx}}{M_{cx}} \leq 1.0\) for \(\frac{P_r}{P_c} \geq 0.2\)
- \(\frac{P_r}{2P_c} + \frac{M_{rx}}{M_{cx}} \leq 1.0\) for \(\frac{P_r}{P_c} < 0.2\)
  - \(K\) – Factor = User Defined.

**Serviceability:**
- *Beam Deflection* \(< \frac{\text{span}}{250}\) for combination DL + LL and \(< \frac{4}{5}\) in
- \(\delta_{\text{midspan}} = \frac{wL^4}{384EI}\)
  - \(w = \text{distributed load/ft}\)
- \(I_{\text{required}} = \frac{wL^4}{384EI\left(\frac{\text{span}}{250}\right)}\)

*Table 3-4 - Beam design procedure*

Similar to the column design, the beam effective length factor will be an input parameter.

### 3.2 Blast Loading – Empirical Method

The empirical method consists of published equations, graphs, tables, and figures to allow one to determine the principal loading of a blast wave on a building or a similar structure (O'Duesnberry, 2010). This system is favourable due to its simplicity. A blast weight and distance from the target are the primary inputs. From these, a common measure known as the scaled distance can be determined.
Scaled Distance, $Z = \frac{R}{W^{3/2}}$

$R = \text{Standoff Distance (ft)}$

$W = \text{Weight of TNT (lbs)}$

Table 3-5 - Scaled distance equation

From the above equation, it can be seen that $Z$ is proportional to the standoff distance $R$ and inversely proportional to the cube root of the weight of the explosives $W$, clearly identifying the standoff distance as a critical influence on the resultant magnitude of an explosion. Standoff distances are presented in Table 3.6 with varying $Z$ and $W$. As can be seen, the closer the explosive is to the target, the lower the corresponding $Z$ value.

<table>
<thead>
<tr>
<th>Scaled Distance 'Z'</th>
<th>Weight of TNT 'W' (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td>2.00</td>
<td>9</td>
</tr>
<tr>
<td>2.25</td>
<td>10</td>
</tr>
<tr>
<td>2.50</td>
<td>12</td>
</tr>
<tr>
<td>2.75</td>
<td>13</td>
</tr>
<tr>
<td>3.00</td>
<td>14</td>
</tr>
<tr>
<td>3.50</td>
<td>16</td>
</tr>
<tr>
<td>4.00</td>
<td>19</td>
</tr>
<tr>
<td>4.5</td>
<td>21</td>
</tr>
<tr>
<td>5.00</td>
<td>23</td>
</tr>
<tr>
<td>5.50</td>
<td>26</td>
</tr>
</tbody>
</table>

Table 3-6 - Standoff distance $R$ with varying weight of explosive $W$ and Scaled Distance $Z$

The basic blast properties are firstly determined using Figure 3.3. It is then necessary to examine the interaction of the blast wave with the target building. Depending on the angle of incidence with the building, a certain amount of reflection will ensue and the initial properties must be altered accordingly.

Typically, when a blast wave reflects off an object with zero angle of incidence, the wave returns towards the source of the blast. This reflection amplifies the blast pressure and produces a larger pressure force on the target. In some cases, the reflected pressure can be up to eight times larger than the peak incident pressure $P_{so}$. 
To determine the reflected wave pressure and scaled impulse, Figure 3.4 and 3.5 are used. From this the equivalent fictitious blast duration can be determined.

\[
Fictitious \text{ Blast Duration} : t_E = \frac{2i_r}{P_R} \quad i_r - \text{Reflected Impulse (psi - ms)}
\]

\[
P_R - \text{Reflected Pressure (psi)}
\]

Table 3-7 – Equivalent reflected blast duration

The outcome of the above procedure produces the idealised loading conditions of the blast wave, which linearly decreases from the peak reflected pressure down to zero over the calculated duration.
Figure 3.3 - Shock wave parameters for hemispherical TNT explosion (UFC 3-340-02)

Figure 3.4 - Pressure coefficient versus angle of incidence (UFC 3-340-02)

Figure 3.5 - Reflected scaled impulse versus angle of incidence (UFC 3-340-02)
3.3 Element Response

The next step will be to apply the loading conditions of the blast to the target structural components of the building frame. This will be evaluated by analysing the column as an equivalent dynamic single degree of freedom system (SDOF). The distributed mass of the element is lumped together and strain energy is stored in a weightless spring. It should be noted that some analyses based on this model require a non-linear spring.

\[ Q_e(t) = \text{External Loading} \]
\[ m_E = \text{Equivalent Element Mass} \]
\[ k_E = \text{Equivalent Element Stiffness} \]

![Idealised SDOF system](image)

Figure 3.6 - Idealised SDOF system

It is first necessary to determine the actual structural element properties.

- **Total Element Load:** \( Q = P(B)(L*bf) \)
- **Boundary Conditions: Pin - Pin:**
  - **Actual Stiffness:** \( k = \frac{Q}{u} = \frac{384EI}{5L^3} \)
  - **Yield Displacement:** \( u_y = \frac{5qL^4}{384EI} \)
  - **Yield Displacement:** \( M = \frac{qL^2}{8} = \frac{QL}{8} = \frac{48EI}{5L^2}u \)
- **Boundary Conditions: Pin - Fixed**
  - **Actual Stiffness:** \( k = \frac{Q}{u} = \frac{195EI}{L^3} \)
  - **Yield Displacement:** \( u_y = \frac{qL^4}{195EI} \)
  - **Yield Displacement:** \( M = \frac{qL^2}{8} = \frac{QL}{8} = \frac{195EI}{8L^2}u \)
- **Boundary Conditions: Fixed - Fixed**
  - **Actual Stiffness:** \( k = \frac{Q}{u} = \frac{384EI}{L^3} \)
  - **Yield Displacement:** \( u_y = \frac{qL^4}{384EI} \)
  - **Yield Displacement:** \( M = \frac{qL^2}{12} = \frac{QL}{12} = \frac{32EI}{L^2}u \)

Table 3-8 - Column properties
3.3.1 Elastic Behaviour

To capture the response of an elastic system, it is necessary to modify the actual column properties of the column. The kinetic energy and work done in the equivalent system are evaluated to calculate reduction factors, and these are applied to the column properties. Consequently, varying the type of loading and boundary conditions will impact the elastic performance of the SDOF.

- **Stiffness of Equivalent SDOF**: \( k_e = \frac{Q_e}{u} \)

- **Natural Period of Equivalent SDOF**: \( T_n = 2\pi \sqrt{\frac{m_e}{k_e}} \)

- **Boundary Conditions: Pin – Pin**:
  - *Load Transformation Factor*: \( \alpha_Q = \frac{Q_e}{Q} = 0.64 \)
  - *Mass Transformation Factor*: \( \alpha_m = \frac{m_e}{m} = 0.5 \)

- **Boundary Conditions: Fixed – Fixed**:
  - *Load Transformation Factor*: \( \alpha_Q = \frac{Q_e}{Q} = 0.53 \)
  - *Mass Transformation Factor*: \( \alpha_m = \frac{m_e}{m} = 0.4 \)

**Table 3-9 - Equivalent column elastic properties**

With the characteristics of the equivalent SDOF and the idealized blast duration known, the response of the system can then be found using Figure 3.7. Based on the ratio of the duration of the loading with the natural period of the system, the dynamic load factor (DLF) and load duration is read off the graph.
To check the adequacy of the column under elastic response, the following procedure is performed.

\[
\begin{align*}
    u_{rm} &= DLF \\
    u_y &= u_{rm} - u_e \\
    u_e &= DLF < u_y \text{ (for the system to remain elastic).}
\end{align*}
\]

• \( u_e \) = Static Displacement
• \( u_y \) = Yield Displacement
• \( u_{rm} \) = Peak Elastic Displacement

**Table 3-10 - Elastic response check**

3.3.2 Elasto-Plastic Behaviour

In the event the column is insufficient to remain elastic due to the blast loading, a check must be carried out on the plastic performance of the element. Similar to the elastic method, reduction factors are calculated through assessing the kinetic energy and work done by the SDOF, and applied to the actual column properties.

• *Natural Period of Equivalent SDOF*: \( T'_n = 2\pi \sqrt{\frac{m_e}{K_e}} \)

• *Boundary Conditions: Pin - Pin:*
  - Load Transformation Factor: \( a_Q = \frac{Q_e}{Q} = 0.53 \)
Mass Transformation Factor: \( a_m = \frac{m_e}{m} = 0.41 \)

Boundary Conditions: Fixed - Fixed:
- Load Transformation Factor: \( a_Q = \frac{Q_e}{Q} = 0.5 \)
- Mass Transformation Factor: \( a_m = \frac{m_e}{m} = 0.33 \)

\( \frac{X_m}{X_E} = \text{Ductility Ratio} = \mu \)
- \( X_m = \text{Maximum Transient Deflection} \)
- \( X_E = \text{Equivalent Elastic Deflection} = \frac{R_u}{K_E} \)
  - \( R_u \) - Ultimate resistance of the element.
  - \( K_E \) - Equivalent stiffness of elastoc system.

Table 3-11 - Column Equivalent Plastic Properties and Requirements

Figures 3.8 & 3.9 are used to estimate the behaviour of the column under a plastic, inelastic response. The ratio of the total deflection to the elastic deflection (Ductility Ratio) can be used to quantify the damage in the column.

![Figure 3.8 - Maximum deflection of Elasto-Plastic SDOF (UFC 3-340-02)](image1)

![Figure 3.9 - Maximum response time of Elasto-Plastic SDOF (UFC 3-340-02)](image2)
3.3.3 Column Re-Design

Upon studying the response of the column for elastic and plastic behaviour, it may be required to upgrade the section size of the frame to resist the subjected blast loading. One of the design steps of the application will therefore be to increase the column section to achieve the plastic limits specified by the user.

Code fragments from the application, shown in Figures 3.10 and 3.11, indicate how the process is implemented. A factor of safety, initially set at a value of 1 for the blast response procedure, is increased in small increments until such time that the ductility ratio falls within the prescribed limits. This increased factor of safety is applied to the initial design loads and results in a larger section size after a number of increments. This upgraded section is then checked for the blast loading and the process is repeated until an adequate section for the blast pressure is determined.

As mentioned above, the limiting values used for the inelastic behaviour of the column (ductility ratio and the angle of rotation at the joint) are inputs required from the user. There are no set limiting
values utilized in the industry for plastic response, however recommendations are made. The UFC suggests the following limiting deformation criteria.

<table>
<thead>
<tr>
<th>Element</th>
<th>Highest level of Protection (Category No.)</th>
<th>Additional Specifications</th>
<th>Deformation Type* (°)</th>
<th>Maximum Deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams, plates, sandwich panels or grids</td>
<td>1</td>
<td>*</td>
<td>2°</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>*</td>
<td>12°</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Frame structures</td>
<td>1</td>
<td>*</td>
<td>0°</td>
<td>4°</td>
</tr>
<tr>
<td>Cold-formed steel floor and wall panels</td>
<td>1</td>
<td>Without tension-membrane action</td>
<td>*</td>
<td>1.25°</td>
</tr>
<tr>
<td>With tension-membrane action</td>
<td>*</td>
<td>4°</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Open-web joists</td>
<td>1</td>
<td>Joints controlled by maximum end reaction</td>
<td>*</td>
<td>2°</td>
</tr>
<tr>
<td>Plate</td>
<td>1</td>
<td>*</td>
<td>2°</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>*</td>
<td>12°</td>
<td>20</td>
<td></td>
</tr>
</tbody>
</table>

* φ = maximum member end rotation (degrees) measured from the chord joining the member ends

| φ = relative sway deflection between stories
| P = Axial Load Applied
| P_e = Euler Buckling Load

Figure 3.12 - Deformation Criteria (UFC 3-340-02)

Research carried out by A. Nassr and presented in ‘Strength and Stability of steel beam columns under blast loads’ uses finite element software to analyse finely meshed columns under blast loads to better understand the response. Nassr also studied the interaction of a permanently deformed column and the compression forces it experiences, and his conclusions can be seen in Table 3.12.

"For a maximum ductility ratio of 3.0, the \( \frac{P}{P_e} < 32\% \)

"For a maximum ductility ratio of 2.5, the \( \frac{P}{P_e} < 40\% \)

"For a maximum ductility ratio of 1.6, the \( \frac{P}{P_e} < 60\% \)

Table 3-12 - Recommended plastic limits (Nassr, 2012)

Due to the uncertainty surrounding the allowable bounds for columns under plastic behaviour, the plastic limits are not fixed in the application and are required inputs. Figure 3.13 below shows a typical output for a re-designed column, based on a ductility limit of 2.5 and maximum joint rotation of 2°.
3.4 Frame Stability

A global appraisal is required if a target column fails. Three methods will be used to assess the global state of the structure: a linear static analysis, linear dynamic analysis and non-linear dynamic analysis.

3.4.1 Linear Static Analysis

When a column fails in a structure, the beams of the adjacent bays must work to transfer the loads to adjacent supports (see Figure 2.5). In the case of this research, if the column is removed, the primary beams must effectively span twice their distance to the adjacent undamaged columns in the bay.

A combined linear check for the increased span will initially be carried out based on the procedure outlined below. It should be noted that it is necessary to apply an amplification factor to better simulate the response of a system when carrying out a linear static analysis for a dynamic loading. The General Services Administration (GSA) and the Department of Defense (DoD) have recommended the value of 2 for the Dynamic Amplification Factor (DAF), however this software tool will keep the DAF as an input parameter so it can be adjusted at the preference of the user.

\[
\frac{P}{P_u} + \frac{G_{mx}M_x}{(1-P/P_{ex})M_{mx}} + \frac{G_{my}M_y}{(1-P/P_{ey})M_{my}} \leq 1.0
\]

\[
\frac{P}{P_p} + \frac{M_x}{1.18M_{mx}} + \frac{M_y}{1.18M_{my}} \leq 1.0 \text{ for } \frac{P}{P_p} \geq 0.15
\]

\[
\frac{M_x}{M_{px}} + \frac{M_y}{M_{py}} \leq 1.0 \text{ for } \frac{P}{P_p} < 0.15
\]

Dynamic Design Stress \(\mu < 10\), \(f_{ds} = f_{dy} = c \times a \times f_y\)

- \(\mu = \text{Ductility Ratio}\)
- \(f_{dy} = \text{Dynamic yield stress.}\)
- \(c = \text{dynamic increase factor on yield stress (A36, } c = 1.21 \text{ for Ultimate Stress)}\)
- $a =$ average strength increase factor (for $f_y \leq 50\text{ksi}$, $a = 1.1$)
- $f_y =$ static yield stress

- **Ultimate dynamic moment**, $M_{pu} = f_{ds}Z$
  - Plastic Section Modulus, $Z = A_cm_1 + A_tm_2$ (for double — symmetric section)
    - $A_c =$ area of cross section in compression
    - $m_1 =$ distance from neutral axis to the centroid of the area in compression
    - $A_t =$ area of cross section in tension
    - $m_2 =$ distance from neutral axis to the centroid of the area in tension

- **Design Plastic Moment**, $M_p = \frac{f_{ds}(Z+S)}{2}$
  - $S =$ Elastic section moduli
  - $Z =$ Plastic section moduli

- **Slenderness limit check**, $C_c = \left(\frac{2\pi^2E}{f_{ds}}\right)^{0.5}$
  - $E =$ modulus of elasticity of steel (psi)

- **Ultimate strength of axially loaded compression member**, $P_u = 1.7AF_a$
  - $A =$ gross area of member
  - $F_a = \frac{\left(1 - \frac{(KL/E)^2}{2G^2}\right)}{\frac{S}{3} \left(\frac{KL/E}{G^2}\right) \frac{(KL/E)^2}{c^2}}$, where $\frac{KL}{r} =$ largest effective slenderness ratio
  - $d = \frac{412}{f_y} \left[1 - 1.4 \frac{P}{P_y}\right]$ when $\frac{P}{P_y} \leq 0.27$ and $d = \frac{257}{f_y}$ when $\frac{P}{P_y} > 0.27$
  - $P =$ Applied compressive load
  - $P_{ex} = \frac{23}{12} A \ast F'_{ex}$ and $P_{ey} = \frac{23}{12} A \ast F'_{ey}$
  - $F'_{ex} = \frac{12\pi^2E}{23(Kl_{bx}/r_x)^2}$ and $F'_{ey} = \frac{12\pi^2E}{23(Kl_{by}/r_y)^2}$
    - $l_b =$ Un-braced length in the plane of bending
    - $r_x / r_y =$ respective radii of gyration

- $P_p = A f_{ds}$

**Table 3-13** - Combined Beam Check with Column Removed
3.4.3 Non-Linear Dynamic (NLD) Analysis

SAP2000 software will be adopted for the numerical global investigations. An algorithm in JAVA will be implemented to produce a text file in SAP2000 coding format. This text file, dependent on the users inputs, produces the inputs required to perform a NLD assessment of the frame. Figure 3.14 below demonstrate a portion of the coding used to write the text file, while Figure 3.15 displays an extract from a typical SAP2000 import text file.

Figure 3.14 - (Application fragment) SAP2000 text file

Figure 3.15 - SAP2000 Import text file extract

UFC 4-023-03 guidelines for progressive collapse contain detailed information on the NLD analysis for structures with removed supports. The general process contained in these guidelines and used for this research is as follows:

- Equilibrium of the model is initially found in its pristine state (no column removed).

- Once established, the column is removed and substituted with an equivalent point load equal to the compression experienced by the target support under the progressive collapse loading combination.
- A time history ramp function is set up to remove the equivalent point load over a short duration. This duration should be less than one tenth of the natural period accompanying the vertical mode shape at the location of the removed column.

- The NLD analysis must have adequate runtime to give the model sufficient time to stabilize. If the model can’t stabilize, it is a good indication that progressive collapse will ensue. Examination of the concluding analysed time steps can be conducted to determine the hinge formations, and the probable collapse mode for the structure.

Figure 3.16 - Extracts from typical SAP2000 import model
3.5 Complete Application and Required Inputs

Based on the above steps, an integrated study can be carried out on the response of a 2D moment frame under blast loading. A summary flow chart of the application process can be seen in the below figure.

![Flow Chart](image)

**Figure 3.17 - Software environment for blast analysis and structural response**
<table>
<thead>
<tr>
<th>Sector</th>
<th>Input Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blast</td>
<td>Blast Weight (TNT Equiv.)</td>
</tr>
<tr>
<td></td>
<td>Blast Factor</td>
</tr>
<tr>
<td></td>
<td>Standoff Distance</td>
</tr>
<tr>
<td></td>
<td>Angle of Incidence with Target Column</td>
</tr>
<tr>
<td>Building Geometry</td>
<td>Number of Stories</td>
</tr>
<tr>
<td></td>
<td>Number of Bays</td>
</tr>
<tr>
<td></td>
<td>Floor to Ceiling Height</td>
</tr>
<tr>
<td></td>
<td>Bay Width</td>
</tr>
<tr>
<td></td>
<td>Frame Tributary Width</td>
</tr>
<tr>
<td></td>
<td>Target Column</td>
</tr>
<tr>
<td></td>
<td>Column Boundary Conditions</td>
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<tr>
<td>Loading</td>
<td>Dead Load</td>
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<td></td>
<td>Live Load</td>
</tr>
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<td></td>
<td>Wind Speed</td>
</tr>
<tr>
<td>Progressive Collapse</td>
<td>Dead Load Combination Factor</td>
</tr>
<tr>
<td></td>
<td>Live Load Combination Factor</td>
</tr>
<tr>
<td></td>
<td>Secondary beam spacing</td>
</tr>
<tr>
<td>Factors</td>
<td>Column Increase Factor</td>
</tr>
<tr>
<td></td>
<td>Column K-Factor (Wind)</td>
</tr>
<tr>
<td></td>
<td>Column K-Factor (Gravity)</td>
</tr>
<tr>
<td></td>
<td>Beam Increase Factor</td>
</tr>
<tr>
<td></td>
<td>Beam K-Factor (Wind &amp; Gravity)</td>
</tr>
<tr>
<td></td>
<td>Dynamic Amplification Factor (DAF)</td>
</tr>
<tr>
<td>Material Details</td>
<td>Maximum Ductility Ratio</td>
</tr>
<tr>
<td></td>
<td>Modulus of Elasticity</td>
</tr>
<tr>
<td></td>
<td>Yield Strength</td>
</tr>
</tbody>
</table>

Table 3-14 - Application inputs
3.6 Application Precision

3.6.1 Blast Data

As previously mentioned, the classification of an explosion in the application is constructed on experimental data presented in UFC 3-340-02 as scaled logarithm graphs. For example, Figures 3.18 and 3.19 shows the inputted data for the peak incident pressure $P_{so}$ in increments of 0.1 (see Figure 3.3). Inherently, reading these values visually can carry a certain amount of human error in addition to the values being accurate to the nearest 0.1. For the purpose of this study however, it provides an accurate means of capturing the blast properties of a wide spectrum of explosion possibilities.

3.6.2 Structural Response

Similar to the blast data, the elastic and plastic response of structural elements due to impulsive pressures are read off scaled logarithm graphs from UFC 3-340-02. The same procedure as the input of the blast data is be adopted with these values. In this case however, closer increments of 0.01 were used for lower values of ‘te/Tn’ (see Figure 3.8) as the majority of blasts considered in this research are near field (i.e. small time durations $\approx 2ms$) and subsequently fell within this zone.
Figure 3.20 - (Application fragment) Component plastic response
4 Analysis & Results

The next part of this research is to examine the interaction of various frame geometries and explosions using the developed application. With the incorporated design function, frames can be easily generated and designed under different loading conditions (i.e. wind & gravity), and this can then be used to identify the inputs that have the most notable influence on responding to blasts. A standard case is firstly established and from this input metrics can be adjusted to assess their influence.

4.1 Standard Case

To define the standard case, the limits of the application are firstly studied. With these identified, a generalized average can be found and this is considered in combination with realistic loading conditions and geometries to establish a standard case. The application inputs for this are displayed in Table 4-1 below and the results can be seen in Appendix A.

![Figure 4.1 - Standard analysis case](image)

Table 4-1: Application Inputs for Standard Case

<table>
<thead>
<tr>
<th>Metric</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bay Width</td>
<td>25ft</td>
</tr>
<tr>
<td>Design Wind Speed</td>
<td>150Mph</td>
</tr>
<tr>
<td>Blast Weight</td>
<td>2000lbs</td>
</tr>
<tr>
<td>Standoff Distance</td>
<td>25ft</td>
</tr>
<tr>
<td>Angle of Incidence</td>
<td>0</td>
</tr>
<tr>
<td>Story Height</td>
<td>13.5ft</td>
</tr>
</tbody>
</table>
### Table 4-1 - Inputs used for standard case

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blast Weight</td>
<td>2000</td>
<td>lbs</td>
</tr>
<tr>
<td>Blast Factor</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Standoff Distance</td>
<td>25</td>
<td>ft</td>
</tr>
<tr>
<td>Angle of Incidence with Target Column</td>
<td>0</td>
<td>Degrees</td>
</tr>
<tr>
<td>Number of Stories</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Number of Bays</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Floor to Ceiling Height</td>
<td>13.5</td>
<td>ft</td>
</tr>
<tr>
<td>Bay Width</td>
<td>25</td>
<td>ft</td>
</tr>
<tr>
<td>Frame Tributary Width</td>
<td>12.5</td>
<td>ft</td>
</tr>
<tr>
<td>Target Column</td>
<td>2</td>
<td>Interior</td>
</tr>
<tr>
<td>Column Boundary Conditions</td>
<td>Pin - Pin</td>
<td></td>
</tr>
<tr>
<td>Dead Load</td>
<td>142</td>
<td>lbs/ft²</td>
</tr>
<tr>
<td>Live Load</td>
<td>90</td>
<td>lbs/ft²</td>
</tr>
<tr>
<td>Wind Speed</td>
<td>150</td>
<td>Mph</td>
</tr>
<tr>
<td>Dead Load Combination Factor (PC)</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>Live Load Combination Factor (PC)</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>Secondary beam spacing</td>
<td>5</td>
<td>ft</td>
</tr>
<tr>
<td>Column Increase Factor</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Beam Increase Factor</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Dynamic Amplification Factor (DAF)</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Maximum Ductility Ratio</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>29000</td>
<td>kips/in²</td>
</tr>
<tr>
<td>Yield Strength</td>
<td>50</td>
<td>kips/in²</td>
</tr>
</tbody>
</table>

**4.2 Loading Condition**

There are three important inputs when considering the effect of a blast on a structure, two of which are the weight of explosive and standoff distance, discussed in detail in Chapter 2. The third condition, which impacts the reflected pressure acting on the structure, is the angle of incidence. When a shock wave from an unobstructed blast hits a target element or building with zero angle of incidence, full reflection occurs and therefore results in the largest design pressure. As the angle increases from zero, the influence of reflected waves on the initial shock waves reduces and therefore the corresponding design pressures also reduce.
4.2.1 Weight of Explosives

The results of varying the weight of explosive can be seen in Figure 4.2 below. As expected, the scaled distance $Z$ reduces non-linearly with an increase in explosive weight $W$. It is also evident that the weight of explosive has more of an influence on the resultant $Z$ for lower weights ($< 1,500\text{lbs}$) than for higher weights.

The corresponding pressures are also plotted in Figure 4.2 and clearly portray the significance of reflection on the initial over pressure. The ratio of the peak reflected pressure to peak over pressure was found to increase with increasing weight (e.g. $\approx 6$ for $W = 1,500\text{lbs}$ and $\approx 8$ for $W = 3,000\text{lbs}$).

Figure 4.3 demonstrate the local response of the structure, indicating the column sections required to achieve the plastic limits set out in the standard case. The internal column size can be seen to be adequate for explosive weights of up to $1,500\text{lbs}$ while the external column fails the plastic limits for explosive weights greater than $700\text{lbs}$.
4.2.2 Standoff Distance

As the standoff distance \( R \) is directly proportional to the scaled distance \( Z \), the result of varying \( R \) produces a linear trend of \( Z \) values, which can be seen in Figure 4.4. Additionally, changing \( R \) can be seen to have a significant effect on the corresponding pressure, which is expected due to its dominant influence on \( Z \). Furthermore, increasing \( R \) from 20ft to 25ft results in a reduction in reflected pressure by 50%, clearly displaying the importance of distancing an explosion from a target.
The effect on the structure with varying R is notable. Again, the external column of the standard case performs poorly for most cases while the internal column passes the plastic limits until the standoff distance reduces below 28ft. The section size required to withstand the blast at 20ft is W14x257, which is approximately 3.5 times the weight of the original external column size.

Figure 4.5 - Local structure assessment with varying standoff distance

4.2.3 Angle of Incidence

There is no change to the initial blast properties with varying the angle of incidence (i.e. peak over pressure $P_{so}$). This is because the angle of incidence is only considered when assessing the extent of reflection off the obstruction. Therefore the only change, which is illustrated in Figure 4.6 below, is the peak reflected pressure. This pressure can be seen to decrease with increasing the angle of incidence from zero to 40 degrees where it experiences a sudden jump to 45 degrees and then begins to drop again. This is the expected trend and follows the same trend as the angle of incidence coefficients presented in Figure 3.4.
4.3 Element Analysis – Column Boundary Conditions

The first part of the local assessment is to examine the response of the target column with varying boundary conditions. The three support arrangements examined for these studies are illustrated in Figure 4.7 below. The analysis method for the target column is described in detail in the Methodology chapter.

![Figure 4.6 - Blast properties with varying angle of incidence](image)

Figure 4.6 - Blast properties with varying angle of incidence

![Figure 4.7 - Column boundary conditions](image)

Figure 4.7 - Column boundary conditions

![Figure 4.8 - Ultimate resistance equations (UFC, 2008)](image)

Figure 4.8 - Ultimate resistance equations (UFC, 2008)
4.3.1 Blast Weight & Boundary Conditions

Figure 4.9 shows the elastic response of the internal column of the standard frame (W14x109) with varying explosive weight and column support restraints. Changing the former from 1000lbs to 2900lbs increases the magnitude of the blast properties while changing the latter alters column properties, and in turn their response to the blast pressure.

The bars in the figure demonstrate the response of the element to the blast pressure in terms of displacement, while the horizontal projections represent the yield displacement limit for the different column boundary conditions. The columns subjected to the simulated blast events are seen to exceed the elastic yield limits in all column boundary arrangements. Expectantly, the elastic response of the column contrasts notably with a change in boundary conditions, which is due to the change in properties. As you shown below, changing boundary conditions alters the period of the element and this subsequently alters the input parameter for the elastic response chart. Therefore, reducing the period of the element (i.e. increasing stiffness) results in element more receptive to blast pressure.

- Input Parameter (see Figure 3.6) = $\frac{t_e}{T_N}$
- \( T_N = \sqrt{\frac{m}{k}} \)

- Column stiffness under uniformly distributed blast pressures

  - Pin-Pin: \( k = \frac{77EI}{L^3} \)
  - Pin-Fix: \( k = \frac{185EI}{L^3} \)
  - Fix-Fix: \( k = \frac{84EI}{L^3} \)

Figure 4.10 displays the plastic feedback of the SDOF models. Again, due to their column properties, the moment released column experiences the largest displacements with the fixed supports showing the smallest results. The contours in the figure represent the ductility ratio (ratio of total deflection to the deflection at elastic limit) for each column arrangement. The process is outlined below.

- Input Parameters (see Figure 3.8) = \( \frac{r_U}{P_R} \) & \( \frac{t_E}{T_N} \)

- Output, Ductility Ratio = \( \frac{X_m}{X_E} \)

  - \( X_E = \frac{r_U}{K_E} \)

---

**Notation**

<table>
<thead>
<tr>
<th>( r_U )</th>
<th>Ultimate flexural resistance</th>
<th>( X_m )</th>
<th>Maximum Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_R )</td>
<td>Reflected blast pressure</td>
<td>( X_E )</td>
<td>Equivalent Elastic Deflection</td>
</tr>
<tr>
<td>( t_e )</td>
<td>Equivalent Blast Duration</td>
<td>( k, m )</td>
<td>Column stiffness and mass</td>
</tr>
<tr>
<td>( T_N )</td>
<td>Column Natural Period</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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Interestingly, the calculated ductility ratios with the three different boundary arrangements are relatively close to each other considering their maximum displacements are quite different. This is due to the calculation of the equivalent elastic deflection (as shown above), which varies with changing support conditions.

4.4 Element Analysis - Building Geometry

4.4.1 Beam Tributary Width

In this instance, the tributary width of the frame is varied from 10ft to 32ft. Increasing this metric results in a larger wind catchment area for the frame and also a larger tributary width for the floor loads. Subsequently, as the tributary width increases, the frame gets stronger and therefore is more capable to resist blasts. Figure 4.11 shows the results of varying the tributary width of the standard case based on released column boundary arrangements, and furthermore displays the performance of the internal column against the standard blast case in the form of the ductility ratio.
For small tributary widths, the frame performs poorly for the localized assessment. The internal columns experience large deformations under plastic behaviour showing signs of failure. Additionally, the plastic response further deteriorates when the external column is considered. The tributary width required for the internal columns to pass the allowable limits for the blast is 18ft while the external column can be seen to fail for all but the largest tributary width design case.

4.4.2 Beam Length

Varying the beam span (or bay width) changes the beam section required but also influences the amount of floor load being distributed into the columns. Therefore, similar to with varying the tributary width, the frame elements get stronger with increasing the beam span. Figure 4.12 show this trend but in this case all the designed column sections are not adequate to satisfy the prescribed blast. Even though sections are shown to increase with an increasing beam span, they only do so in small increments when compared to the above results. This is due to the dominant influence of the 150Mph design wind speed in the design of the 2D frame.
4.4.3 Column Height

The result of altering the column height has a significant impact on the response of the structure to the blast. Increasing the column height necessitates larger frame members to cater for both service and ultimate conditions. In addition to this, in the context of a blast, the increase in column height reduces the stiffness of the element \((k \propto 1/L^3)\) and therefore produces a larger natural period. Figure 4.13 illustrates how the section size required to sustain the blast load increases considerably with an increase in column height. For example, the section size required to pass the plastic criteria for a column height of 13.5ft is W14x157 while the section size required for the same loading with a column height of 16ft is W14x257.
4.4.4 Stories, Bays & Design Wind Pressure

A number of building geometry inputs were altered in this section to study their interaction with each other but also their resistant capabilities against blast pressures. Tall frames more susceptible to wind were focused on to attain a wide variety of column sizes while the floor height and beam size remained constant throughout.

The Figures below demonstrate the results of the study. Wind speed is varied from relatively low values up to very large coastal wind speeds. Expectantly, the column sizes gradually increase with increasing design wind speed, confirming the governing design load condition to be wind and not gravity for the column size. Additionally, the number of bays per frame is also varied from 4 to 8.

Three blast-loading conditions are illustrated to cross-examine the frame members. For increasing story heights, the corresponding column sizes increase in strength and therefore become more blast resistant. Internal columns for all frames can be seen to work well for blasts (A) & (B). Internal columns for blast (C) however are seen to be generally inadequate and only begin to pass for the 24 & 26 story frames designed for higher wind pressures (60 to 80psf). Again, similar to previous studies, the external columns behave poorly in comparison to the internal columns, with all failing blast (C).
**Legend:**
- $X_{Axis} \rightarrow$ WindSpeed (Mph), RHS $Y_{Axis} \rightarrow$ Section Weight, W14 x _
- LHS $Y_{Axis} \rightarrow$ Average Design Wind Pressure (psf)
- Bar Chart - Column Size Required for Frame
  - From left to Right - 4 Bays Internal & External, 6 Bays Internal & External, 8 Bays Internal & External.

- **Light Orange** = Blast Event (A), $W = 2000$lbs TNT, $R = 30ft$
- **Medium Orange** = Blast Event (B), $W = 2000$lbs TNT, $R = 25ft$
- **Dark Orange** = Blast Event (C), $W = 2000$lbs TNT, $R = 20ft$
- **Purple** = Average design wind pressure (psf)
4.4.5 Stories, Design Wind Pressure & Column Height

In this analysis, the number of bays is fixed at 6 while the column height is varied from 12ft up to 16ft in 1ft increments. The wind speed also varies from 100Mph to 200Mph. As indicated above, altering the column heights has a significant influence on the blast response of the structure.

Internal and external columns designed for 12ft story heights are seen to be primarily within plastic limits in all cases for the simulated explosion, while all columns fail for story heights of 15ft & 16ft for the 16-story frame. Additionally, as predicted, the more stories the frame has the better it performs under blast loading. This can be seen for the 24 and 26 story frames where all internal columns are satisfactory for the prescribed limits in the standard case.
Legend:
- $X_{\text{Axis}} \rightarrow$ WindSpeed (Mph), RHS $Y_{\text{Axis}} \rightarrow$ Section Weight, W14 x __
- LHS $Y_{\text{Axis}} \rightarrow$ Wind Pressure (psf)
- Bar Chart – Column Size Required for 6 Bay Frame:
  From left to Right – 100Mph Int & Ext, 150Mph Int & Ext, 175Mph Int & Ext, 200Mph Int & Ext.

- Light Orange to Dark Orange – Average Design Wind Pressure, 100Mph, 125Mph, 175Mph & 200Mph.
- Purple =
  Section size req’d for 2000lb blast at 25ft
4.5 Frame Analysis

One specific frame profile is considered for the global assessment. A 20-story, 6-bay frame has been adopted with removal of one end column. The main properties of the frame can be seen in Figure 4.14.

The objective of this analysis is to assess the generic frame under different wind conditions and column heights. Altering these metrics results in different column sizes and subsequently modifies the frame properties. The beam size is varied with changing column heights but is fixed in all other cases. Therefore, there are two beam sections considered, W21x68 and W21x83 for frames with a typical story height of 13ft and 15ft respectively. The other parameter considered in the study is the live load factor under progressive collapse analysis. The three load combinations are as follows:

- 1.1DL + 0.25LL
- 1.1DL + 0.5LL
• 1.1DL + 0.75LL

12 different frame arrangements have been examined under a linear dynamic and non-linear dynamic analysis in SAP2000. The governing criteria and section sizes for each frame can be seen in Table A.2 in Appendix A.

Figure 4.15 - Location of maximum moment

Figure 4.16 shows the results of the linear dynamic analysis. The maximum moment occurring in the beam directly above the removed column has been considered. The performance against the plastic capacity of the beam has been calculated and results shown. It should be noted the steel strength used to calculate the plastic capacity is based on yield strength and not the increased dynamic yield strength.

Figure 4.16 - Ratio of maximum linear dynamic moment to beam plastic capacity
As expected, the frame cases analysed under larger live load factors experience higher stresses. In addition, the frames designed for a story height of 15ft, even though exceed the plastic capacity, can be seen to perform a better than the frames designed with a typical story height of 13ft. This is mainly due to the increase in beam size. In relation to the design wind speed, the frames designed in higher wind speed regions produce a stronger frame, which in turn perform better under the progressive collapse analysis.

The results of the non-linear dynamic analysis are displayed in Figures 4.18 to 4.23 below. The properties of the hinges are configured based on the Federal Emergency Management Agency (FEMA) specifications and can be seen illustrated in Figure 4.17 below. Even though the specific properties of the plastic hinge curve differ based on the type of structural component, Figure 4.17 can be used as a measure of the non-linear behaviour of the frames. The coloured indicators characterize the limit states outlined by ASCE and FEMA. The light blue mark 'Life Safety' on the inelastic curve is considered the limit. Surpassing this calls upon the residual strength of the member, which is approximately 20% of the yield strength.

![Figure 4.17 - Typical plastic hinge curve for flexural elements](image)
The frames designed for a larger story height can be seen to perform better with an edge column removed due to the increase in section sizes. Expectantly, the progressive collapse live load combination has a significant impact on the outcome of the results. This is evident in Figure 4.18 as this is the only frame to show signs of instability with the 2nd floor beam indicating high levels of plastic behaviour due to the gravity loading it experiences during the dynamic analysis.
5 Summary & Conclusions

The primary objective of this research was to establish and implement a procedure that could study the local and global response of a building subjected to blast pressure. A tool has been developed capable of simulating a blast, examining target elements of a building and furthermore assessing stability concerns that may arise with the sudden removal of a support. A design function has also been integrated with the ability of conceptually sizing frames in different wind regions.

A parametric study was then conducted by varying the input parameters of the application. A standard set of input metrics were initially defined and from this inputs were both individually and collectively varied to assess their effects on the blast properties and structural response.

Loading conditions and building geometry were found to have a significant effect on the local response of a structure to blast loading. Internal columns for building frames with 20 stories or more performed considerably well under large blast pressures (≈ 2000psi) while edge columns largely failed the defined plastic limits. Additionally, buildings located in higher speeds (> 140Mph) better-sustained ground explosions due to inherently having stronger structural components.

The global assessment followed similar trends to that of the local response study. A 20-story frame was adopted for the analysis, and design wind speeds, column heights and live load combinations were varied. The dynamic moment and hinge formations were compared and showed how the three variables notably altered the response of the frame.

5.1 Future Research

Currently, the application allows a user to study the effects of a blast on a 2D moment-resisting frame. With further development, the application modules could be refined to better capture the response of a structure to a subjected explosion. Possible alterations to increase the capabilities and accuracy of the application can be seen in Figure 5.1. It should be noted that some of these proposed additions would be demanding computationally and would increase analysis runtime significantly. Sacrificing analysis runtime for more accurate results might not be desired objective considering one of the main advantages associated with this application is its ability to iterate through a variety of different blast situations and structural components in short amounts of time. Further research is required to determine an optimal correlation.
2. Building Frame

Current Application
- 2D moment frame design for wind & gravity loads

Further Developments
- Import designed building FE model and extract structural data required for blast analysis

3. Blast Loading Condition

- External Explosions. Blast properties based on empirical data
- Include more blast situations including internal & upper story level explosions

4. Component Response

- Analysis of columns as a SDOF incorporating elastic and plastic response factors
- Finite numerical component model of columns with better distributed blast pressure
- Study of blast interaction and cladding enclosure. Finite numerical model of critical connections

5. Frame Stability Analysis

- Non-linear dynamic analysis of 2D frame
- Dynamic analysis of more accurate building model (3D frame, composite action with concrete slabs etc.)

Figure 5.1 - Current and possible application modifications
6 References


Appendix A – Standard Case Results

**Java Output:**

**PROJECT TITLE:** Standard Case

**DESIGN WIND PRESSURE:**
Average wind pressure = 38.5595627053097 psf

**FRAME DESIGN:**
Beam Size = W21x57
Internal Column Size = 109
External Column Size = 74

**SAP2000 FILE PRINTED**

**BLAST ANALYSIS:**
Blast Analysis - Scaled Distance 'Z' = 1.9843015892661306
Blast Analysis - Over Pressure 'Pso' = 300.0 psi
Blast Analysis - Peak Reflected Pressure 'Pr(alpha)' = 1221.8138162573853 psi
Blast Analysis - Impulse 'Ir(alpha)' = 1224.87606273068 ms
Blast Analysis - Equivalent Duration 'tr(alpha)' = 1.224876006273068 ms

**PIN-PIN BOUNDARY CONDITIONS**

**COLUMN LOADING & CAPACITIES:**
Target Column Analysis - Applied Column Loading 'Q*' = 4718.57400000302 kipf
Target Column Analysis - Actual Column Stiffness 'k' = 7774.768099034024 kipf.ft^-1
Target Column Analysis - Total Actual Mass 'm' = 0.045984375 kipf.s^2.ft^-1
Target Column Analysis - Static Displacement 'u' = 7.282903782952877 in
Target Column Analysis - Yield Displacement 'uy' = 0.6575900305895439 in

**COLUMN - ELASTIC ASSESSMENT**
Elastic Equivalent - Target Column Analysis - Stiffness of Equivalent SDOF 'ke' = 4975.8515833817755 kipf.ft^-1
Elastic Equivalent - Target Column Analysis - Mass of Equivalent SDOF 'me' = 0.0229921875 kipf.s^2.ft^-1
Elastic Equivalent - Target Column Analysis - Natural Period of Equivalent SDOF 'Tn' = 0.0101937993234757 s
Target Column Analysis - T/Tn used for UFC 3-340 graphs to determine whether column remains elastic = 0.1026031765862475
Elastic Parameter - Peak Response Parameter 'um' = 2.002798540312041 in
| Elastic Check | - 204.56643914088664 |

**COLUMN - PLASTIC ASSESSMENT**
Plastic Equivalent - Target Column Analysis - Stiffness of Equivalent SDOF (k'e) = 3887.384049517012 kipf.ft^-1
Plastic Equivalent - Target Column Analysis - Mass of Equivalent SDOF (m'e) = 0.01908351625000002 kipf.s^2.ft^-1
Plastic Equivalent - Target Column Analysis - Natural Period of Equivalent SDOF (T'n) = 0.01392137441108914 s
Plastic Parameter - Ultimate Resistance = 672.7111111111111112 kipf.ft^-1
Plastic Parameter - T/Tn = 0.0879856386764139
Plastic Parameter - Ultimate Resistance ru / Peak Reflected Pressure = 0.1425666125213065
Plastic Parameter - Max Ductility Ratio 'Xm/Xe' = 3.6
Peak Displacement = 3.7378976179527276 in
End Rotation of Column (Degrees) = 2.6421320738581087 degrees
| Plastic Check | - = EXCEEDS Plastic limits, Re-Design or Structure Assessment is required. Max Ductility Ratio Xm/Xe = 3.6 Allowable Ductility Ratio Specified by User = 2.5 |

**PINNED-FIXED BOUNDARY CONDITIONS**

**COLUMN LOADING & CAPACITIES:**
Target Column Analysis - Applied Column Loading 'Q*' = 4718.57400000302 kipf
Target Column Analysis - Actual Column Stiffness 'k' = 18728.28253022519 kipf.ft^-1
Target Column Analysis - Total Actual Mass 'm' = 0.045984375 kipf.s^2.ft^-1
Target Column Analysis - Static Displacement 'us' = 3.023389246112329 in
Target Column Analysis - Yield Displacement 'uy' = 0.27298872621230796 in

COLUMN - ELASTIC ASSESSMENT
Elastic Equivalent - Target Column Analysis - Stiffness of Equivalent SDOF 'k'e = 11986.100819344123 kipf.ft^-1
Elastic Equivalent - Target Column Analysis - Mass of Equivalent SDOF 'm'e = 0.0229921875 kipf.s^2.ft^-1
Elastic Equivalent - Target Column Analysis - Natural Period of Equivalent SDOF 'T'n = 0.007691766052616154 s
Target Column Analysis - T/T'n used for UFC 3-340 graphs to determine whether column remains elastic = 0.15924509376574947
Elastic Parameter - Peak Response Parameter 'um' = 1.414514254863038 in

COLUMN - PLASTIC ASSESSMENT
Plastic Equivalent - Target Column Analysis - Stiffness of Equivalent SDOF (k'e) = 9364.141265112596 kipf.ft^-1
Plastic Equivalent - Target Column Analysis - Mass of Equivalent SDOF (m'e) = 0.019083515625000002 kipf.s^2.ft^-1
Plastic Equivalent - Target Column Analysis - Natural Period of Equivalent SDOF (T'n) = 0.00896964143727058 s
Plastic Parameter - Ultimate Resistance = 1009.066666666667 kipf ft^-1
Plastic Parameter - T/T'n = 0.1365579677670808
Plastic Parameter - Ultimate Resistance ru / Peak Reflected Pressure = 0.21384991878195977
Plastic Parameter - Max Ductility Ratio 'Xm/Xe' = 3.05
Peak Displacement = 1.9719822114172225
End Rotation of Column (Degrees) = 1.394615870762222 degrees
| Plastic Check | 418.1584875277766 |

FIXED-FIXED BOUNDARY CONDITIONS

COLUMN LOADING & CAPACITIES:
Target Column Analysis - Applied Column Loading 'Q*' = 4718.57400000302 kipf
Target Column Analysis - Actual Column Stiffness 'k' = 38873.84049517012 kipf.ft^-1
Target Column Analysis - Total Actual Mass 'm' = 0.045984375 kipf.s^2.ft^-1
Target Column Analysis - Static Displacement 'us' = 1.4565807565905753 in
Target Column Analysis - Yield Displacement 'uy' = 0.197277091768632 in

COLUMN - ELASTIC ASSESSMENT
Elastic Equivalent - Target Column Analysis - Stiffness of Equivalent SDOF 'k'e = 24879.25791690888 kipf.ft^-1
Elastic Equivalent - Target Column Analysis - Mass of Equivalent SDOF 'm'e = 0.0229921875 kipf.s^2.ft^-1
Elastic Equivalent - Target Column Analysis - Natural Period of Equivalent SDOF (T'n) = 0.005338832783081981 s
Target Column Analysis - T/T'n used for UFC 3-340 graphs to determine whether column remains elastic = 0.135924509376574947
Elastic Parameter - Peak Response Parameter 'um' = 0.9831920106986384 in

COLUMN - PLASTIC ASSESSMENT
Plastic Equivalent - Target Column Analysis - Stiffness of Equivalent SDOF (k'e) = 19436.92024758506 kipf.ft^-1
Plastic Equivalent - Target Column Analysis - Mass of Equivalent SDOF (m'e) = 0.019083515625000002 kipf.s^2.ft^-1
Plastic Equivalent - Target Column Analysis - Natural Period of Equivalent SDOF (T'n) = 0.006225802426934592 s
Plastic Parameter - Ultimate Resistance = 1345.4222222222224 kipf.ft^-1
Plastic Parameter - T/T'n = 0.1967418691241961
Plastic Parameter - Ultimate Resistance ru / Peak Reflected Pressure = 0.285133225042613
Plastic Parameter - Max Ductility Ratio 'Xm/Xe' = 2.55
Peak Displacement = 1.0590649000866061 in
End Rotation of Column (Degrees) = 0.7490924884746943 degrees
| Plastic Check | 2.55 |

COLUMN RE-DESIGN Pin - Pin:
Blast Column Size = 132
Column Increase Factor = 1.06
Updated Ductility Ratio = 2.5
Rotation 1.8130088520064207
Peak Displacement 2.563936675406708

**COLUMN RE-DESIGN PIN - FIX:**
Blast Column Size = 132
Column Increase Factor = 1.06
Updated Ductility Ratio = 1.68
Rotation 0.7588737585516203
Peak Displacement 1.0728952398086757

**COLUMN RE-DESIGN FIX - FIX:**
Blast Column Size = 145
Column Increase Factor = 1.08
Updated Ductility Ratio = 1.65
Rotation 0.475983236346799
Peak Displacement 0.6729209239313043

**BEAM CHECK - COLUMN REMOVED:**
Max Shear with Column Removed = 781.8125 Kips
Max Bending Moment with Column Removed = 1628.7760416666667 Kip-ft
Section size is sufficient to prevent local buckling and enable plastic moment to be achieved
Section size is sufficient to prevent local buckling and enable plastic moment to be achieved
Section fails for Shear, \( V_p < V_{max} \). \( V_p = 312.9214275000001 \) kipf
Ultimate Axial Compressive Load 1068.8709800933068 Kips
Factor for determining \( M_{mx} \) with \( M_{px} \) 0.47765244047146393
\( M_{mx} < M_{px} \), Passed Check
Buckling Capacity 51.53931957168661 Ksi
Euler Buckling Load about X-Axis 1649.6880075259478 Kips
Ultimate Capacity for Dynamic Axial Load 1184.86500000000002 Kips
Combined Check One 3.8130663334940786
Combined Check Two 2.1355045861535853
Section FAILS for Combined Bending & Axial Plastic Check
<table>
<thead>
<tr>
<th>Design Wind Speed, Column Height, Live Load Combination Factor</th>
<th>Beam Size</th>
<th>External Column Size</th>
<th>Internal Column Size</th>
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<tr>
<td>140,13,0.75</td>
<td>W21x68</td>
<td>W14x109</td>
<td>W14x179</td>
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<td>W21x68</td>
<td>W14x120</td>
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<td>210,13,0.5</td>
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<td>W21x68</td>
<td>W14x132</td>
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<td>210,13,0.25</td>
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<td>W14x233</td>
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<td>W21x83</td>
<td>W14x120</td>
<td>W14x193</td>
</tr>
<tr>
<td>165,15,0.75</td>
<td>W21x83</td>
<td>W14x145</td>
<td>W14x211</td>
</tr>
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<td>W21x83</td>
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<td>W14x257</td>
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</tbody>
</table>

Table A.1 - Global Assessment: Design Sections used for SAP Analysis
Appendix B – Sample Calculation

INPUT:

Building Geometry:
- No. of Stories = 5.
- No. of Bays = 6.
- Floor Height = 14ft.
- Bay Width = 20ft.
- Tributary Width of 2D Frame = 20ft
- Joist Spacing = 5ft.

Loading:
- Dead Loads = 120psf.
- Live Loads = 90psf.
- Wind Speed = 150mph (Boston).

Blast Details:
- Weight of TNT = 2000lbs.
- Standoff Distance from Structure = 30ft.
- Angle of Incidence with Structure = 30 degrees.
- Blast Factor = 1.0.
- Target Column = 3rd Interior, First Floor to Second Floor.

Progressive Collapse Combination:
- 1.1DL + 0.25LL.

Material Details:
- Young’s Modulus = 29,000ksi.
- Steel Yield Strength = 50ksi.

Factors:
- Ductility Limit = 2.5.
- Dynamic Amplification Factor = 2.0.
CALCULATIONS:

Step One – Member Design:

- Wind Impact:
  - Velocity Pressure (Directional Procedure), \( q_z = 0.00256 \times 2.01 \left( \frac{z_i}{1200} \right)^2 \times 1.0 \times 0.85 \times 150^2 \text{ (lb/ft}^2) \)
    - \( 98.4 \times \left( \frac{z_i}{1200} \right)^2 \)
    - \( z_i \) = Height to floor \( i \).
  - Wind Pressure (Directional Procedure), \( p = qGC_p - q_iGC_{pi} \text{ (lb/ft}^2) \)
    - \( p = \left[ 98.4 \times \left( \frac{z_i}{1200} \right)^2 \right] \times 0.85 \times 0.8 - \left[ 98.4 \times \left( \frac{z_i}{1200} \right)^2 \right] \times (-0.18) \)

- Horizontal forces at respective stories:
  - 1st = 6.6Kips.
  - 2nd = 8.1Kips.
  - 3rd = 9.1Kips.
  - 4th = 9.9Kips.
  - 5th = 5.3Kips.

- \( V_{tot} = 2V_E + 5V_I = 39 \text{Kips.} \)
  - \( V_E = \frac{5E}{k_I} = 0.9166 \)
  - \( V_E = 5.23 \text{Kips.} \)
  - \( V_I = 5.7 \text{Kips.} \)

- Column Moment due to wind:
  - \( M_{ext} = 5.23 \times 14 = 73.22 \text{Kip} - \text{Ft.} \)
  - \( M_{int} = 5.7 \times 14 = 79.8 \text{Kip} - \text{Ft.} \)

- Gravity Impact:
  - Ultimate Conditions (1st - 2nd Columns):
    - \( P_{int} = 20 \times 20 \times 5 \times (1.2 \times 120 + 1.6 \times 90) = 576 \text{Kips.} \)
    - \( P_{ext} = 20 \times \frac{20}{2} \times 5 \times (1.2 \times 120 + 1.6 \times 90) = 288 \text{Kips.} \)
  - 1.2 * Dead + Live (1st - 2nd Columns):
    - \( P_{int} = 20 \times 20 \times 5 \times (1.2 \times 120 + 90) = 468 \text{Kips.} \)
    - \( P_{ext} = 20 \times \frac{20}{2} \times 5 \times (1.2 \times 120 + 90) = 234 \text{Kips.} \)
• **Design Forces:**
  - Int, \(1.2DL + 1.6L = 576\text{Kips}\).
  - Int, \(1.2DL + 1.0W + 1.6L = 468\text{Kips}, 79.8\text{kip-ft}\).
  - Ext, \(1.2DL + 1.6L = 288\text{Kips}\).
  - Ext, \(1.2DL + 1.0W + 1.0L = 234\text{Kips}, 73.22\text{kip-ft}\).

• **Column Design:**
  - Java output sizes:
    - Int Column = W14X68. Section used for assessment as target column is interior on 2D frame.
    - Ext Column = W14X48.

• **Process of Java:**
  - Buckling Check:
    - \(\frac{kt}{r} = 47.8 < 4.71 \sqrt{\frac{f_y}{E}}\)
    - \(\lambda_c = \frac{KL}{rN\sqrt{E}}\)
      - 0.632
    - \(F_{cr} = (0.658\lambda_c^2) F_y\)
      - 42.3ksi.
    - \(P_c = \phi_c A_p F_{cr}\)
      - 761Kips > 576Kips ∴ OKAY!
  - Combined Axial & Bending:
    - \(M_{cx} = 0.9 f_y z_x\)
      - 431 Kip – ft.
    - \(P_r/P_c = 0.616\)
      - \(\frac{P_r}{P_c} + \frac{8 M_{rx}}{9 M_{cx}}\)
        - 0.616 + 0.164 = 0.78 < 1.0 ∴ OKAY.

• Column chosen for blast assessment = **W14X68**

• **Beam Forces:**
  - Ultimate Gravity Conditions, \(1.2DL + 1.6L\):
    - \(M_{max} = \frac{wl^2}{12} = \frac{20 \cdot (1.2 \cdot 120 + 1.6 \cdot 90)^2}{12} = 192\text{kip-ft}\)
  - Max Allowable Deflection = \(\frac{\text{span}}{250} = 0.96\text{in}\).
  - Java output size:
    - Beam Size = W12x45.
  - Flexural Check:
    - \(M_{cx} = 0.9 f_y z_x\)
      - 240 Kip – ft.
    - \(\frac{8 M_{rx}}{9 M_{cx}} = 0.71 < 1.0 ∴ OKAY!\)
  - Deflection Check:
    - \(\delta_{\text{midspan}} = \frac{wl^4}{384EI}\)
      - \(w = 20 \cdot (120 + 90) = 4.2\text{kip/ft}\)
      - \(\delta_{\text{midspan}} = 0.3\text{in} < 0.96\text{in} ∴ OKAY!\)

• Beam chosen for blast assessment = **W12X45**

*Step Two – Blast Analysis:*
- **Scaled Distance, Z =** \( \frac{R}{w^3} \)
  - \( 2.381 \)
- **Shock Wave Parameters:**
  - \( P_{so} = 235 \text{psi} \)
  - \( i_s = 328 \text{psi} - \text{ms} \)
  - \( t_o = 12.6 \text{ms} \)
  - \( L_w = 6.3 \text{ft} \)
  - \( q_o = 400 \text{psi} \)
  - \( P_s = P_{so} + q_o \quad \text{735psi} \)
- **Reflected Wave Parameters:**
  - \( C_{ra} = \frac{P_{ra}}{P_{so}} = 5.2 \)
  - \( P_{ra} = 1222 \text{psi} \)
  - \( i_{ra} = 913 \text{psi} - \text{ms} \)
  - \( t_{rf} = \frac{2i_{ra}}{P_{ra}} = 1.5 \text{ms} \)

![Diagram showing pressure and time relationship]

**Step Three – Column Assessment:**

- **Column Properties:**
  - \( Q = P \cdot b_y \cdot Hgt \)
    - \( P_{ra} = P \)
    - \( 2053 \text{kips} \)
  - \( k = \frac{Q}{u} = \frac{384EI}{5L^3} \)
    - \( 4069 \text{kip/ft} \)
  - \( u_y = \frac{9L^2M_y}{384EI} \)
    - \( 0.06 \text{ft} \)
  - \( u_t = \frac{Q}{k} \)
    - \( 0.505 \text{ft} \)
  - \( m = \lambda L \)
    - \( 0.0298 \text{kipf.s}^2.\text{ft}^{-1} \)
- **Column Elastic Properties:**
  - \( k_e = \alpha_Q k \)
    - \( 2604 \text{kip/ft} \)
  - \( m_e = \alpha_m m \)
    - \( 0.0149 \text{kipf.s}^2.\text{ft}^{-1} \)
  - \( T_n = 2\pi \sqrt{\frac{m_e}{k_e}} \)
    - \( 0.015 \text{sec} \)
- Elastic Check:
  o Equivalent SDOF Elastic Chart:
    - \( \frac{t_m}{T} = 2.8 \).
    - \( t_m = 0.042 \text{sec.} \).
    - \( \frac{u_m}{u_s} = 0.38 \).
    - \( u_m = 0.192 \text{ft.} \).
  o \( u_m > u_y \) - Column EXCEEDS Elastic Check. Plastic Check Required.
- Column Plastic Properties:
  o \( k'_e = a'_q k \)
    - \( 2035 \text{ kip/ft} \)
  o \( m'_e = a'_m m \)
    - \( 0.0098 \text{kip/.s}^2 \text{ ft}^{-1} \)
  o \( T'_n = 2\pi \sqrt{\frac{m'_e}{k'_e}} \)
    - \( 0.0138 \text{sec.} \)
  o \( f_{ds} = a \ast c \ast f_y \)
    - \( 70.95 \text{ksi} \)
  o \( M_p = f_{ds} \ast z \)
    - \( 679 \text{ kip-\text{ft}} \)
  o Ultimate Resistance \( \frac{8M_p}{Hgt} \)
    - \( 388 \text{ kip.} \text{ ft}^{-1} \)
    - \( f_y = 0.189 \).
- Plastic Check:
  o Equivalent SDOF Plastic Chart:
    - \( \frac{\lambda_m}{x_e} = 2.3 \).
    - \( \frac{t_m}{T} = 3.5 \).
    - \( t_m = 0.525 \text{sec.} \).
  o Ductility Ratio < Allowable as per user input ∴ Column is WITHIN Plastic Limits.

No Beam Check or SAP2000 analysis required, Assessment Finished.
PROJECT TITLE: 5 - Story Analysis

DESIGN WIND PRESSURE:
Average wind pressure = 31.593497484128637 psf

FRAME DESIGN:
Beam Size = W21x50
Internal Column Size = 68
External Column Size = 45

BLAST ANALYSIS:
Blast Analysis - Scaled Distance 'Z' = 2.381161907119357
Blast Analysis - Over Pressure 'Ps' = 242.8571429 psi
Blast Analysis - Peak Reflected Pressure 'Pr(alpha)' = 1299.2857145149999 psi
Blast Analysis - Impulse 'Ir(alpha)' = 906.9133481498118 psi-ms
Blast Analysis - Equivalent Duration 'tr(alpha)' = 1.3960183476478016 ms

PIN-PIN BOUNDARY CONDITIONS
COLUMN LOADING & CAPACITIES:
Target Column Analysis - Applied Column Loading 'Q*' = 2182.800000386597 kipf
Target Column Analysis - Actual Column Stiffness 'k' = 4059.010314151441 kipf.ft^-1
Target Column Analysis - Total Actual Mass 'm' = 0.02975 kipf.s^2.ft^-1
Target Column Analysis - Static Displacement 'us' = 6.45319868080086 in
Target Column Analysis - Yield Displacement 'uy' = 0.7231349699111663 in

COLUMN - ELASTIC ASSESSMENT
Elastic Equivalent - Target Column Analysis - Stiffness of Equivalent SDOF 'ke' = 2597.7666010569224 kipf.ft^-1
Elastic Equivalent - Target Column Analysis - Mass of Equivalent SDOF 'me' = 0.014875 kipf.s^2.ft^-1
Elastic Equivalent - Target Column Analysis - Natural Period of Equivalent SDOF 'Tn' = 0.013289336743126068 s
Target Column Analysis - T/Tn used for UFC 3-340 graphs to determine whether column remains elastic = 0.10504800763438359
Elastic Parameter - Peak Response Parameter 'urn' = 1.982053879609807 in
    Elastic Check | 174.091830997095492

COLUMN - PLASTIC ASSESSMENT
Plastic Equivalent - Target Column Analysis - Stiffness of Equivalent SDOF (k'e) = 2029.5051570757205 kipf.ft^-1
Plastic Equivalent - Target Column Analysis - Mass of Equivalent SDOF (m'e) = 0.012346250000000001 kipf.s^2.ft^-1
Plastic Equivalent - Target Column Analysis - Natural Period of Equivalent SDOF (T'n) = 0.01549716732276888 s
Plastic Parameter - Ultimate Resistance = 388.53571428571433 kipf.ft^-1
Plastic Parameter - T/T'n = 0.090082162667027
Plastic Parameter - Ultimate Resistance ru / Peak Reflected Pressure = 0.17799876956988694
Plastic Parameter - Max Ductility Ratio 'Xm/Xe' = 2.25
Peak Displacement = 2.58448820618663 in
End Rotation of Column (Degrees) = 1.7623043582397593 degrees

Plastic Check: Column WITHIN Maximum Ductility Ratio. Max Ductility Ratio Xm/Xe = 2.25 Allowable Ductility Ratio Specified by User = 2.5
Appendix C – JAVA Code Extracts

Application Inputs

```java
public static void main(String[] args) throws NumberFormatException,
IOException {

    /* Frame Title */
    String heading = dataStorage.reading();

    /* Frame Geometry */
    int numberOfStories = dataStorage.numbStory();
    double numberOfBays = dataStorage.numbBays();
    double bayWidth = dataStorage.bayWidth();
    double floorHeight = dataStorage.floorHeight();
    double tributaryWidth = dataStorage.tribWidth();

    /* Gravity Loads */
    double deadLoad = dataStorage.deadLoad();
    double liveLoad = dataStorage.liveLoad();

    /* Wind Region */
    double windspeed = dataStorage.windspeed();

    /* Steel Properties */
    double youngsMod = dataStorage.steelE();
    double yieldStrength = dataStorage.steelYield();

    /* Factors */
    double ductilityLimit = dataStorage.ductilityLimit();
    double dynamicAmpFactor = dataStorage.dynamicAmpFactor();
    double kFactor = dataStorage.kFactor();
    double compressionFactor = dataStorage.compressionFactor();
    double columnFactorOfSafety = dataStorage.columnSafetyFactor();
    double beamFactorOfSafety = dataStorage.beamSafetyFactor();
    double windDisplacementFactor = dataStorage.windDisplacementFactor();

    /* Blast Criteria */
    double weightTNT = dataStorage.weightTNT();
    double blastFactor = dataStorage.blastFactor();
    double standoffDist = dataStorage.standoffDist();
    double blastAngle = dataStorage.blastAngle();
    int targetColumn = dataStorage.targetColumn();

    /* Retriving Column Details for Blast Analysis */
```
double columnLx = retrievingColumnSections.Lx(bayWidth, tributaryWidth, deadLoad, liveLoad, numberOfStorys, floorHeight, kFactor, yieldStrength, compressionFactor, numberOfBays, targetColumn, columnFactorOfSafety, windspeed);

double columnSx = retrievingColumnSections.Sx(bayWidth, tributaryWidth, deadLoad, liveLoad, numberOfStorys, floorHeight, kFactor, yieldStrength, compressionFactor, numberOfBays, targetColumn, columnFactorOfSafety, windspeed);

double columnZx = retrievingColumnSections.Zx(bayWidth, tributaryWidth, deadLoad, liveLoad, numberOfStorys, floorHeight, kFactor, yieldStrength, compressionFactor, numberOfBays, targetColumn, columnFactorOfSafety, windspeed);

double columnLinearWeight = retrievingColumnSections.Wgt(bayWidth, tributaryWidth, deadLoad, liveLoad, numberOfStorys, floorHeight, kFactor, yieldStrength, compressionFactor, numberOfBays, targetColumn, columnFactorOfSafety, windspeed);

double columnFlangeWidth = retrievingColumnSections.Bf(bayWidth, tributaryWidth, deadLoad, liveLoad, numberOfStorys, floorHeight, kFactor, yieldStrength, compressionFactor, numberOfBays, targetColumn, columnFactorOfSafety, windspeed);

/**
 * Progressive Collapse Load Factors
 */

double deadLoadFactorPC = dataStorage.dlCombPC;

double liveLoadFactorPC = dataStorage.lCombPC;

double noOfFloorsBeamSupports = dataStorage.noOfFloorsBeamSupports;

/**
 * Beam Details for Column Removed
 */

double beamaxial = windForces.beamAxialForce(floorHeight, numberOfStorys, tributaryWidth, windspeed);

double beamLengthColumnRemoved = bayWidth * 2;

double beamFlangeWidth = retrievingBeamSections.Bf(bayWidth, tributaryWidth, deadLoad, liveLoad, yieldStrength, beamaxial, beamFactorOfSafety, floorHeight, numberOfStorys, numberOfBays, windspeed);

double beamFlangeThickness = retrievingBeamSections.Bt(bayWidth, tributaryWidth, deadLoad, liveLoad, yieldStrength, beamaxial, beamFactorOfSafety, floorHeight, numberOfStorys, numberOfBays, windspeed);

double beamWebThickness = retrievingBeamSections.tw(bayWidth, tributaryWidth, deadLoad, liveLoad, yieldStrength, beamaxial, beamFactorOfSafety, floorHeight, numberOfStorys, numberOfBays, windspeed);

double beamDepth = retrievingBeamSections.d(bayWidth, tributaryWidth, deadLoad, liveLoad, yieldStrength, beamaxial, beamFactorOfSafety, floorHeight, numberOfStorys, numberOfBays, windspeed);

double beamArea = retrievingBeamSections.a(bayWidth, tributaryWidth, deadLoad, liveLoad, yieldStrength, beamaxial, beamFactorOfSafety, floorHeight, numberOfStorys, numberOfBays, windspeed);

double beamZx = retrievingBeamSections.zx(bayWidth, tributaryWidth, deadLoad, liveLoad, yieldStrength, beamaxial, beamFactorOfSafety, floorHeight, numberOfStorys, numberOfBays, windspeed);
double beamRx = retrievingBeamSections.rx(bayWidth, tributaryWidth, 
    deadLoad, liveLoad, yieldStrength, beamaxial, 
    beamFactorOfSafety, floorHeight, numberOfStorys, numberOfBays, 
    windspeed);

double beamRy = retrievingBeamSections.ry(bayWidth, tributaryWidth, 
    deadLoad, liveLoad, yieldStrength, beamaxial, 
    beamFactorOfSafety, floorHeight, numberOfStorys, numberOfBays, 
    windspeed);

double beamLy = dataStorage.beamLyQ;

/** * Building Dimensions */
double buildingWidth = numberOfBays * bayWidth;

double buildingHeight = numberOfStorys * floorHeight;

/** * Scaled Distance Z */
double z = (double) ((standoffDist) / Math.pow(weightTNT * blastFactor, 
    0.33333));

Finding Peak Over Pressure

public class PeakIncidentOverPressurePs0 {

    /** * Data Representation for GRAPH UFC 3 340 Fig 2.15 pg 166. In Increments of 
    * 0.25 */

    private static Map<Double, Double> graf = new HashMap<Double, Double>();

    static {
        graf.put((double) 0.3, (double) 3700);
        graf.put((double) 0.4, (double) 3300);
        graf.put((double) 0.5, (double) 2700);
        graf.put((double) 0.6, (double) 2000);
        graf.put((double) 0.7, (double) 1750);
        graf.put((double) 0.8, (double) 1350);
        graf.put((double) 0.9, (double) 1250);
        graf.put((double) 1, (double) 1000);
        graf.put((double) 1.1, (double) 900);
        graf.put((double) 1.2, (double) 850);
        graf.put((double) 1.3, (double) 800);
        graf.put((double) 1.4, (double) 700);
        graf.put((double) 1.5, (double) 600);
        graf.put((double) 1.6, (double) 500);
        graf.put((double) 1.7, (double) 450);
        graf.put((double) 1.8, (double) 400);
        graf.put((double) 1.9, (double) 350);
        graf.put((double) 2, (double) 300);
        graf.put((double) 2.1, (double) 285.7142857);
        graf.put((double) 2.2, (double) 271.4285714);
        graf.put((double) 2.3, (double) 257.1428571);
        graf.put((double) 2.4, (double) 242.8571429);
        graf.put((double) 2.5, (double) 228.5714286);
        graf.put((double) 2.6, (double) 200);
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<td>6.2</td>
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<td>8.4</td>
<td>14.8</td>
</tr>
<tr>
<td>8.5</td>
<td>14.5</td>
</tr>
</tbody>
</table>
public static double findPeakIncidentOverPressureWith(final double z) {
    // Check that Z is greater than 0
    if (z > 0) {
        float coeff = 10f;
        double nearestTenthZ = Math.round(z * coeff) / 10.0;
        return (graph.get(nearestTenthZ));
    }
    throw new RuntimeException("invalid Z value - Range 0.25 to 40");
}

Finding Reflected Blast Impulse

public class ImpulseIralpha {
    private static final int ZERO = 0;
    private static final int FIVE = 5;
    private static final int TEN = 10;
    private static final int FIFTEEN = 15;
    private static final int TWENTY = 20;
    private static final int THIRTY = 30;
    private static final int THIRTY_FIVE = 35;
    private static final int FORTY = 40;
    private static final int FORTY_FIVE = 45;
    private static final int FIFTY = 50;
    private static final int FIFTY_FIVE = 55;
    private static final int SIXTY = 60;
    private static final int SIXTY_FIVE = 65;
    private static final int SEVENTY = 70;
    private static final int SEVENTY_FIVE = 75;
private static final int EIGHTY = 80;

public static double calculateImpulsesUFCGraph(final double pso, final int angle, final double w) {
    /**
     * Determine which curve to use for calculating 'Pr'
     */
    if (pso < 56.25) {
        return findingImpulsePsoEquals50(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 56.25) && (pso < 68.75)) {
        return findingImpulsePsoEquals625(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 68.75) && (pso < 81.25)) {
        return findingImpulsePsoEquals75(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 81.25) && (pso < 93.75)) {
        return findingImpulsePsoEquals875(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 93.75) && (pso < 112.5)) {
        return findingImpulsePsoEquals100(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 112.5) && (pso < 137.5)) {
        return findingImpulsePsoEquals125(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 137.5) && (pso < 162.5)) {
        return findingImpulsePsoEquals150(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 162.5) && (pso < 187.5)) {
        return findingImpulsePsoEquals175(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 187.5) && (pso < 225)) {
        return findingImpulsePsoEquals200(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 225) && (pso < 275)) {
        return findingImpulsePsoEquals250(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 275) && (pso < 325)) {
        return findingImpulsePsoEquals300(angle) * Math.pow(w, 0.3333);
    } else if ((pso > 325) && (pso < 275)) {
        return findingImpulsePsoEquals350(angle) * Math.pow(w, 0.3333);
    } else if (pso > 275) {
        return findingImpulsePsoEquals400(angle) * Math.pow(w, 0.3333);
    }
    throw new RuntimeException("invalid Pso value");
}

/**
 * Z = 0.3
 *
 * // param angle
 * // param angle
 * // return
 */
private static double findingImpulsePsoEquals50(int angle) {
    switch (angle) {
    case ZERO:
        return 36;
    case FIVE:
        return 36;
    case TEN:
        return 35.5;
    case FIFTEEN:
        return 35;
    }
}
case TWENTY:
    return 34;
case TWENTY_FIVE:
    return 33;
case THIRTY:
    return 31.5;
case THIRTY_FIVE:
    return 30;
case FORTY:
    return 29;
case FORTY_FIVE:
    return 28;
case FIFTY:
    return 26;
case FIFTY_FIVE:
    return 24;
case SIXTY:
    return 22.5;
case SIXTY_FIVE:
    return 20;
case SEVENTY:
    return 18;
case SEVENTY_FIVE:
    return 17;
case EIGHTY:
    return 16;
}
return 0;
}

Wind Assessment

public class windForces {

    public static Double designWindPressure(final double colhgt,
        final double numstorys, final double loadwidth,
        final double windspeed) {

        double designPressure = 0;
        double z = 0;

        while (z <= numstorys) {

            double floorHgt = z * colhgt;

            double floorLoad = 0.0000037614 * Math.pow(windspeed, 2)
                * Math.pow((floorHgt / 1200), 0.2857);

            designPressure = floorLoad + designPressure;

            if (z == numstorys) {
                return (designPressure / numstorys) * 1000;
            }
            z++;
        }

        return 0.0;
    }
}
public static Double windPressure(final double colhgt,
      final double numstorvs, final double loadwidth,
      final double windspeed)
{
    double designPressure = 0;
    double z = 1;
    while (z <= numstorvs) {
        double floorHgt = z * colhgt;
        double floorLoad = 0.0000037614 * Math.pow(windspeed, 2)
            * Math.pow((floorHgt / 1200), 0.2857) * colhgt * loadwidth;
        designPressure = floorLoad + designPressure;
        if (z == numstorvs) {
            return designPressure;
        }
        z++;
    }
    return 0.0;
}

public static Double roofDisplacement(final double colhgt,
      final double numstorvs, final double loadwidth,
      final double windspeed, final double e, final double intlx,
      final double extlx, final double numberOfBavs,
      final double windDispFact)
{
    double storystiffnessFIXED = ((12 * e * extlx) * 2)
        / (Math.pow((colhgt * 12), 3))
        + ((12 * e * intlx) * (numberOfBays - 1) / (Math.pow(
            (colhgt * 12), 3))); //kip/in
    double storystiffnessPIN = ((3 * e * extlx) * 2)
        / (Math.pow((colhgt * 12), 3))
        + ((3 * e * intlx) * (numberOfBays - 1) / (Math.pow(
            (colhgt * 12), 3))); //kip/in
    double storyShear = 0;
    double storyDisplacement = 0;
    double z = 0;
    while (z <= (numstorvs + 1)) {
        z++;
        double floorHgt = z * colhgt;
        double floorLoad = windDispFact * 0.0000037614
            * Math.pow(windspeed, 2)
            * Math.pow((floorHgt / 1200), 0.2857) * colhgt * loadwidth;
        storyShear = floorLoad + storyShear;
storyDisplacement = storyShear / storystiffnessFIXED + storyDisplacement;

if (z == numstorys + 1) {
    return storyDisplacement;
}

return storystiffnessFIXED;

public static Double intMoment(final double colhgt, final double numstorys, final double loadwidth, final double bays, final double windspeed) {
    double baseShear = windForces.windPressure(colhgt, numstorys, loadwidth, windspeed);

    /**
     * Pin Bases
     */
    double shearRatioExtToInt = 0.89;
    double intshear = baseShear * (1 / (2 * shearRatioExtToInt + (bays - 1)));
    return colhgt * intshear;
}

public static Double extMoment(final double colhgt, final double numstorys, final double loadwidth, final double bays, final double windspeed) {
    double baseShear = windForces.windPressure(colhgt, numstorys, loadwidth, windspeed);

    /**
     * Pin Bases
     */
    double shearRatioExtToInt = 0.89;
    double extshear = baseShear * (1 / (2 + (bays - 1) * (1 / shearRatioExtToInt)));
    return colhgt * extshear;
}

Column Design Checks

public static Double maxAxial(final double span, final double trib, final double sdl, final double live, final double numstorys) {
    double ultUdl = trib * ((sdl * 1.2 + live * 1.6) / 1000);
return span * numstorys * ultUdl;
}

public static String columnSizing(final double span, final double trib,
                                    final double sdl, final double live, final double numstorys,
                                    final double colhgt, final double k, final double fy,
                                    final double factCOMP, final double bays,
                                    final double factorOfSafety, final double windspeed) {

double ultAxial = maxAxial(span, trib, sdl, live, numstorys);

double deadAxial = windForces.
 .axialInt(span, trib, sdl, live, numstorys);

double windMoment = windForces.intMoment(colhgt, numstorys, trib, bays,
                                           windspeed);

/**
 * W14x48
 */

double a = 14.1;

double ry = 1.91;

double zx = 78.4;

double slenderness = ((k * colhgt * 12) / (ry));

double landaC = (slenderness / Math.PI) * Math.sqrt((fy / 29000));

if (slenderness <= (4.71 * Math.sqrt(29000 / fy))) {
    double fcr = (Math.pow(0.658, (Math.pow(landaC, 2))) * fy);
    double pn = fcr * a;
    double pc = pn * 0.9;
    double mcx = 0.9 * fy * zx / 12;
    if (deadAxial / pc >= 0.2) {
        double check = deadAxial / pc + (0.8888889)
                              * (windMoment / mcx);
        if ((check * factorOfSafety) < 1.0
            && (ultAxial * factorOfSafety) < pc) {
            return "48";
        }
    }
} else if (deadAxial / pc < 0.2) {
    double check = deadAxial / (2 * pc) + (windMoment / mcx);
    if ((check * factorOfSafety) < 1.0
        && (ultAxial * factorOfSafety) < pc) {
        return "48";
    }
}

} else if (slenderness > (4.71 * Math.sqrt(29000 / fy))) {
    double fcr = (0.877 / Math.pow(landaC, 2)) * fy;
    double pn = fcr * a;
    double pc = pn * 0.9;
    double mcx = 0.9 * fy * zx / 12;
    if (deadAxial / pc >= 0.2) {
        double check = deadAxial / pc + (0.8888889)
                        * (windMoment / mcx);
        if ((check * factorOfSafety) < 1.0
            && (ultAxial * factorOfSafety) < pc) {
            return "48";
        }
    }
} else if (slenderness <= (4.71 * Math.sqrt(29000 / fy))) {
    double fcr = (Math.pow(0.658, (Math.pow(landaC, 2))) * fy);
    double pn = fcr * a;
    double pc = pn * 0.9;
    double mcx = 0.9 * fy * zx / 12;
    if (deadAxial / pc >= 0.2) {
        double check = deadAxial / pc + (0.8888889).
                        * (windMoment / mcx);
        if ((check * factorOfSafety) < 1.0
            && (ultAxial * factorOfSafety) < pc) {
            return "48";
        }
    }
```java
return "48";
}
}

Beam Design Checks

public class beamDesign {

  public static String sectionDesign(final double span, final double trib,
                                      final double sdl, final double live, final double fy,
                                      final double pr, final double beamFoS, final double colhgt,
                                      final double numstorys, final double bays, final double windspeed) {

    double mMax = windForces.intMoment(colhgt, numstorys, trib, bays, windspeed);
    double iReq = beamForces.serviceIReq(span, trib, sdl, live);
    double ultUdl = trib * ((sdl * 1.2 + live * 1.6) / 1000);

    /*
    * W21x50
    */
    double ix3 = 984;
    double ry3 = 1.3;
    double zx3 = 110;
    double sx3 = 94.5;
    double a3 = 14.7;
    double slenderness3 = (0.75 * span * 12) / ry3;

    if (slenderness3 <= (4.71 * Math.sqrt(29000 / fy))) {
        double fe = (Math.pow(Math.PI, 2) * 29000) / (Math.pow(slenderness3, 2));
        double fcr = (Math.pow(0.658, fy / fe)) * fy;
        double pn = 0.9 * fcr * a3;
        double mcx = 0.9 * fy * zx3 / 12;

        if (pr / pn >= 0.2) {
            double check = pr / pn + (0.88889) * (mMax / mcx);
            if ((beamFoS * check) < 1.0 && iReq) {
                return "W21x50";
            }
        }
    } else if (pr / pn < 0.2) {
        double check = pr / (2 * pn) + (mMax / mcx);
        if ((beamFoS * check) < 1.0 && iReq) {
            return "W21x50";
        }
    }

    return "48";
  }
}
```
} else if (slenderness3 > (4.71 * Math.sqrt(29000 / fy))) {
    double fe = (Math.pow(Math.PI, 2) * 29000) / (Math.pow(slenderness3, 2));
    double fcr = 0.877 * fe;
    double pn = 0.9 * fcr * a3;
    double mcx = 0.9 * fy * zx3 / 12;
    if (pr / pn >= 0.2) {
        double check = pr / pn + (0.88889) * (mMax / mcx);
        if ((beamFoS * check) < 1.0 && ix3 > iReq) {
            return "W21x50";
        }
    }
    } else if (pr / pn < 0.2) {
        double check = pr / (2 * pn) + (mMax / mcx);
        if ((beamFoS * check) < 1.0 && ix3 > iReq) {
            return "W21x50";
        }
    }
}

Sourcing Column Properties

public class retrievingColumnSections {

  private static Map<String, Double> graph = new HashMap<String, Double>();

  public static Double findColumn(final double span, final double trib,
        final double sdl, final double live, final double numstorvs,
        final double colhgt, final double k, final double fy,
        final double factCOMP, final double bays,
        final double targetColumn, final double factorOfSafety,
        final double windspeed) {

    graph.put((String) "W14x48", (double) 1);
    graph.put((String) "W14x53", (double) 2);
    graph.put((String) "W14x61", (double) 3);
    graph.put((String) "W14x68", (double) 4);
    graph.put((String) "W14x74", (double) 5);
    graph.put((String) "W14x82", (double) 6);
    graph.put((String) "W14x90", (double) 7);
    graph.put((String) "W14x99", (double) 8);
    graph.put((String) "W14x109", (double) 9);
    graph.put((String) "W14x120", (double) 10);
    graph.put((String) "W14x132", (double) 11);
    graph.put((String) "W14x145", (double) 12);
    graph.put((String) "W14x159", (double) 13);
    graph.put((String) "W14x176", (double) 14);
    graph.put((String) "W14x193", (double) 15);
    graph.put((String) "W14x211", (double) 16);
    graph.put((String) "W14x233", (double) 17);
    graph.put((String) "W14x257", (double) 18);
    graph.put((String) "W14x283", (double) 19);
    graph.put((String) "W14x311", (double) 20);
    graph.put((String) "W14x342", (double) 21);
    graph.put((String) "W14x370", (double) 22);
    graph.put((String) "W14x398", (double) 23);

}
if (targetColumn == 1) {
    String Size = externalColumnForcesAndDesign.columnSizing(span,
    trib, sdl, live, numstorys, colhgt, k, fy, factCOMP, bays,
    factorOfSafety, windspeed);
    return (graph.get(Size));
} if (targetColumn > 1) {
    String Size = internalColumnForcesAndDesign.columnSizing(span,
    trib, sdl, live, numstorys, colhgt, k, fy, factCOMP, bays,
    factorOfSafety, windspeed);
    return (graph.get(Size));
} return 0.0;

public static Double Ixx(final double span, final double trib,
    final double sdl, final double live, final double numstorys,
    final double colhgt, final double k, final double fy,
    final double factCOMP, final double bays,
    final double targetColumn, final double factorOfSafety,
    final double windspeed) {

double sectionNumber = retrievingColumnSections.findColumn(span, trib,
    sdl, live, numstorys, colhgt, k, fy, factCOMP, bays,
    targetColumn, factorOfSafety, windspeed);

    if (sectionNumber == 1) {
        return 484.0;
    } if (sectionNumber == 2) {
        return 541.0;
    } if (sectionNumber == 3) {
        return 640.0;
    } if (sectionNumber == 4) {
        return 722.0;
    } if (sectionNumber == 5) {
        return 795.0;
    } if (sectionNumber == 6) {
        return 881.0;
    } if (sectionNumber == 7) {
        return 999.0;
    } if (sectionNumber == 8) {
        return 1110.0;
    } if (sectionNumber == 9) {
        return 1240.0;
    } if (sectionNumber == 10) {
public static double maxDeflectionPlasticInputsTeDivideTn(final double te, final double tn) {
    if ((te / 1000) / tn < 0.1) {
        return 0.1;
    }
    return (te / 1000) / tn;
}
public static double TeDivideTn(final double te, final double tn) {
    return (te / 1000) / tn;
}

public static double maxDeflectionPlasticInputsRuDivideP(final double ru, final double pr, final double width, final double colhgt) {
    return (ru) / ((pr * width * colhgt * 12) / 1000);
}

private static Map<Double, Double> graph = new HashMap<Double, Double>();
static double retrievingGraphData(final double te, final double tn, final double ru, final double uy, final double pr, final double width, final double colhgt) {
    double graphinput = maxDeflectionPlasticInputsRuDivideP(ru, pr, width, colhgt);
    double tDivideTn = EquivalentSDOFPlasticResponseUm.maxDeflectionPlasticInputsTeDivideTn(te, tn);
    if ((graphinput > 0.099) && (graphinput <= 0.105)) {
        graph.put((double) 0.1, (double) 5.5);
        graph.put((double) 0.11, (double) 6);
        graph.put((double) 0.12, (double) 7);
        graph.put((double) 0.13, (double) 8);
        graph.put((double) 0.14, (double) 9);
        graph.put((double) 0.15, (double) 10);
        graph.put((double) 0.16, (double) 12);
        graph.put((double) 0.17, (double) 14);
        graph.put((double) 0.18, (double) 16);
        graph.put((double) 0.19, (double) 17);
        graph.put((double) 0.2, (double) 19);
    } else if (tDivideTn > 0.05) {
        float coeff = 100f;
        double nearestValue = (Math.round(tDivideTn * coeff) / 100.0);
        return (graph.get(nearestValue));
    } else if (tDivideTn > 0.2) {
        System.exit(0);
    } else {
        return tDivideTn;
    }
}

SAP2000 Text File Fragments
RAMP FUNCTION

```java
pw.println("TABLE: "FUNCTION - TIME HISTORY - RAMP"");
pw.println(" Name=COL_REM Time=0 Value=0 RampTime=0.01 Amplitude=1 MaxTime=5");
pw.println(" Name=COL_REM Time=0.01 Value=1");
pw.println(" Name=COL_REM Time=5 Value=1");
pw.println("\n");
```

CASE DEFINITIONS

```java
pw.println("TABLE: "ANALYSIS CASE DEFINITIONS");
pw.println("Case=MODAL Type=LinModal InitialCond=Zero RunCase=Yes");
pw.println(" Case=DEAD Type=LinStatic InitialCond=Zero RunCase=Yes");
pw.println(" Case=LL Type=LinStatic InitialCond=Zero RunCase=Yes");
pw.println(" Case=SDL Type=LinStatic InitialCond=Zero RunCase=Yes");
pw.println(" Case=WIND Type=LinStatic InitialCond=Zero RunCase=Yes");
pw.println(" Case=COL_EQUIV Type=LinStatic InitialCond=Zero RunCase=Yes");
pw.println(" Case=COL_CANCEL Type=LinStatic InitialCond=Zero RunCase=Yes");
pw.println(" Case=NLS_PC Type=NonStatic InitialCond=Zero RunCase=Yes");
pw.println(" Case=ColEquivCheck Type=LinStatic InitialCond=Zero RunCase=Yes");
pw.println(" Case=LYN_DYN Type=LinDrHis InitialCond=NLSPC RunCase=Yes");
```

LOAD ASSIGNMENTS

```java
pw.println("TABLE: "CASE - STATIC 1 - LOAD ASSIGNMENTS");
pw.println(" Case=LL LoadType="Load case" LoadName=LL LoadSF=1");
pw.println(" Case=SDL LoadType="Load case" LoadName=SDL LoadSF=1");
pw.println(" Case=WIND LoadType="Load case" LoadName=WIND LoadSF=1");
pw.println(" Case=COL_EQUIV LoadType="Load case" LoadName=COL_EQUIV LoadSF=1");
pw.println(" Case=NLS_PC LoadType="Load case" LoadName=NLS_PC LoadSF=" + deadcombo + ");
pw.println(" Case=NLS_PC LoadType="Load case" LoadName=LL LoadSF=" + livecombo + ");
pw.println(" Case=Col_Equiv_Check LoadType="Load case" LoadName=SDL LoadSF=" + deadcombo + ");
pw.println(" Case=Col_Equiv_Check LoadType="Load case" LoadName=LL LoadSF=" + livecombo + ");
pw.println(" Case=Col_Equiv_Check LoadType="Load case" LoadName=COL_EQUIV LoadSF=0");
```

NONLINEAR DATA

```java
pw.println("TABLE: "CASE - STATIC 2 - NONLINEAR LOAD APPLICATION");
pw.println(" Case=NLS_PC LoadApp="Full Load" MonitorDOF=U1");
pw.println("\n");
pw.println("TABLE: "CASE - STATIC 4 - NONLINEAR PARAMETERS");
pw.println(" Case=NLS_PC Unloading="Unload Entire" GeoNonLin=P-Delta ResultsSave="Final State" MaxTotal=200 MaxNull=50 MaxIterCS=10 MaxIterNR=40 ItConvTol=0.0001 UseEvStep=Yes EvLumpTol=0.01 LSPerIter=20 LSTol=0.1");
pw.println(" LSStepFact=1.618 FrameTC=Yes FrameHinge=Yes CableTC=Yes LinkTC=Yes LinkOther=Yes TMaxIter=10 TFTol=0.01 TFAccelFact=1 TFNoStop=No");
```
/* NON LINEAR DYNAMIC DATA */

pw.println("TABLE: \"CASE - DIRECT HISTORY 1 - GENERAL\" ");
pw.println(" Case=NLD_PC OutSteps=500 StepSize=0.01 ");
pw.println(" Case=LYN_DYN OutSteps=500 StepSize=0.01 ");
pw.println(" ");
pw.println("TABLE: \"CASE - DIRECT HISTORY 2 - LOAD ASSIGNMENTS\" ");
pw.println(" Case=NLD_PC LoadType=\"Load case\" LoadName=COL_CANCEL Function=COL_REM LoadSF=1 TimeFactor=1 ArrivalTime=0 ");
pw.println(" Case=LYN_DYN LoadType=\"Load case\" LoadName=COL_CANCEL Function=COL_REM LoadSF=1 TimeFactor=1 ArrivalTime=0 ");
pw.println(" ");
pw.println("TABLE: \"CASE - DIRECT HISTORY 3 - PROPORTIONAL DAMPING\" ");
pw.println(" Case=NLD_PC SpecifyType=Frequency MassCoeff=0.228479465715621 StiffCoeff=5.782x04 Frequency1=1 Damping1=0.02 Frequency2=10 Damping2=0.02 ");
pw.println(" Case=LYN_DYN SpecifyType=Frequency MassCoeff=0.228479465715621 StiffCoeff=5.782x04 Frequency1=1 Damping1=0.02 Frequency2=10 Damping2=0.02 ");
pw.println(" ");
pw.println("TABLE: \"CASE - DIRECT HISTORY 4 - INTEGRATION PARAMETERS\" ");
pw.println(" Case=NLD_PC IntMethod=HilbertHughesTaylor Gamma=0.5 Beta=0.25 Alpha=0 ");
pw.println(" Case=LYN_DYN IntMethod=HilbertHughesTaylor Gamma=0.5 Beta=0.25 Alpha=0 ");
pw.println(" ");
pw.println("TABLE: \"CASE - DIRECT HISTORY 5 - NONLINEAR PARAMETERS\" ");
pw.println(" Case=NLD_PC GeoNonLin=P-Delta DTMax=0 DTMin=0 MaxIterCS=10 MaxIterNR=40 ItConvTol=0.0001 UseEvStep=Yes EvLumpTol=0.01 LSPerIter=20 LSTol=0.1 LSStepFact=1.618 FrameTC=Yes FrameHinge=Yes CableTC=Yes LinkTC=Yes LinkOther=Yes ");

/* JOINT/NODE DATA */

pw.println("TABLE: \"JOINT COORDINATES\" ");

double nodesX = (bays + 1);
double x = numberOfStories;
int joint = 0;

for (int i = 0; i <= (nodesX - 1); i++) {
    for (int j = 0; j <= x; j++) {
        joint++;
        pw.println("Joint=
        + (joint) + " CoordSys=GLOBAL CoordType=Cartesian XorR=
        + (-bayWidth * bays * 0.5 + (i * width)) + " Y=0 Z="
        + (j * colhgt) + " SpecialJt=No GlobalX="
        + (-bayWidth * bays * 0.5 + (i * width)) + " GlobalY="
        + (j * colhgt) + " ");
    }
}

/* ASSIGNMENTS */
pw.println("TABLE: "FRAME OUTPUT STATION ASSIGNMENTS"");

for (int ll = 1; ll < (totFrames + 1); ll++) {

if (ll <= FramesZDir) {

pw.println(" Frame="
+ " StationType=MinNumSta MinNumSta=3 AddAtElmInt=Yes AddAtPtLoad=Yes");
} else {

pw.println(" Frame="
+ " StationType=MaxStaSpag MaxStaSpag=2 AddAtElmInt=Yes AddAtPtLoad=Yes");
}
}

pw.println("TABLE: "FRAME AUTO MESH ASSIGNMENTS"");

for (int cc = 1; cc <= totFrames; cc++) {

pw.println(" Frame="
+ cc
+ " AutoMesh=Yes AtJoints=Yes AtFrames=No NumSegments=0 MaxLength=0 MaxDegrees=0");
}

/* DISTRIBUTED LOADS */

pw.println("TABLE: "FRAME LOADS - DISTRIBUTED"");

for (int dd = (int) (FramesZDir + 1); dd <= (totFrames); dd++) {

pw.println("Frame="
+ dd
+ " LoadCase=LL CoordSys=GLOBAL Type=Force Dir=Gravity DistType=RelDist RelDistA=0 RelDistB=1 AbsDistA=0 AbsDistB=24 FOverLA="
+ live1 * loadwidth + " FOverLB=" + live1 * loadwidth + "");

pw.println(" Frame="
+ dd
+ " LoadCase=SDL CoordSys=GLOBAL Type=Force Dir=Gravity DistType=RelDist RelDistA=0 RelDistB=1 AbsDistA=0 AbsDistB=24 FOverLA="
+ sdl1 * loadwidth + " FOverLB=" + sdl1 * loadwidth + ");
}