University of Southern Queensland

Faculty of Health, Engineering and Sciences

Study of Blast Effects on Structures

A dissertation submitted by

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Abstract

Engineers have a duty to the public to preserve life and protect the community and occupants within structure that we build and use. All practicing engineers are obligated to foster the health, safety and wellbeing of the community and the environment. This involves acting on the basis of adequate knowledge and foreseeable risks that pose a potential hazard towards the built environment. The terrorism threat has evolved rapidly in scale and occurrences in recent history and with that the need to create resilient structures.

This dissertation endeavours to undertake a study of the global blast loading effects on structures and identify techniques for improved structural resilience of critical elements. Blasts can be delivered by explosive events either deliberate, accidental or through indirect action. A historical review of case studies and blast incidents was undertaken to identify susceptible structures to blast and development of a structural model in order to simulate a credible scenario and understand the blast effects and predicting the design loading.

The scope of the dissertation is restricted to the blast pressure disturbance effects interacting with a structure delivered by an external air blast and not considering the secondary effects of a blast incident including thermal and high velocity fragments. Common structural members and materials were used to devise a Finite Element model and simulate against the blast loading cases derived from empirical methods. Since the nature of blast load only lasting for a short time and undergoes constant change Non-Linear Transient Dynamic Analysis approach was well suited to undertaking this type of analysis.

Some of the findings include whipping effects due to inertia as the structure accelerating from its initial position to develop resistance against the applied loading even after the applied load has ceased. The global response of a structure due to blast pressure, is generally a consequence of lateral or out-of-plane loading. Longer pressure phase durations tend to result in bending failures while impulsive loads (short pressure phase duration) lead to shear responses. Resilience techniques including steel UC encased in concrete, RC steel plate wraps and RC shear reinforcement lacing have the potential to improve the robustness of structural elements reducing overall displacements and stress responses.

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Limitations of Use

Certification

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

Andrew Fraser

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Notations

C $\quad Global\ dampening\ matrix$

C_D $\quad Drag\ coefficient$

$C_{r\alpha}$ $\quad Peak\ reflected\ pressure\ coefficient\ at\ angle\ of\ incidence\ \alpha$

e $\quad Specific\ internal\ energy$

$f(t)$ $\quad Global\ element\ (internal\ resisting)\ force\ vector$

i_s $\quad Positve\ incident\ impulse$

i_r $\quad Positve\ reflected\ impulse$

L_w $\quad Wavelength$

M $\quad Global\ mass\ matrix$

P_{so} $\quad Peak\ positve\ incident\ overpressure$

P_s $\quad Incident\ overpressure$

P_o $\quad Ambient\ pressure$

P_r $\quad Reflected\ peak\ positve\ incident\ overpressure$

P_{so}^- $\quad Peak\ negative\ incident\ overpressure$

P_{r^-} $\quad Peak\ negative\ normal\ reflected\ pressure\ (psi)$

$p(t)$ $\quad Applied\ load\ vector (may\ be\ time\ dependant)$

R $\quad Stand\ off\ distance$

t_c $\quad Clearing\ time\ for\ refelcted\ pressures$

t_a $\quad Arrival\ time\ of\ blast\ wave$

t_d $\quad Rise\ time$

t_0^+ $\quad Duration\ of\ positve\ phase$

t_0^- $\quad Duration\ of\ negative\ phase$

t_{of} $\quad Fictitious\ posivite\ phase\ pressure\ duration$

t_{of^-} $\quad Fictitious\ negative\ phase\ pressure\ duration$

t_{rf} $\quad Fictitious\ reflected\ posivite\ phase\ pressure\ duration$

t_{rf^-} $\quad Fictitious\ reflected\ negative\ phase\ pressure\ duration$

U $\quad Shock\ front\ velocity$

u $\quad Particle\ velocity$

u_p $\quad Peak\ particle\ velocity\ (PPV)$

$u(t)$ $\quad Unknown\ nodal\ displacement\ vector$

$\dot{u}(t)$ $\quad First\ order\ time\ derivative\ of\ u(t)\ (velocity)$

$\ddot{u}(t)$ $\quad Second\ order\ time\ derivative\ of\ u(t)\ (acceleration)$

W $\quad Explosive\ charge\ weight\ (lb)$

Z $\quad Scaled\ distance$

α $\quad Angle\ of\ incidence\ of\ the\ pressure\ front$

q_o $\quad Peak\ dynamic\ pressure$

ρ $\quad Density\ ahead\ of\ shock\ wave\ in\ mass\ per\ unit\ of\ volume$

σ $\quad Stress\ exerted\ by\ the\ blast\ shock\ in\ force\ per\ unit\ surface$

ρ_0 $\quad Density\ ahead\ of\ shock\ wave\ in\ mass\ per\ unit\ of\ volume$

U_S $\quad Shock\ wave\ velocity$

Chapter 1 - Introduction

Background

Newly built civil structures need to anticipate and consider all perceived load cases to determine a design stands the test of time and protects inhabitants. This should also include credible special case threats where occurrence may be of low probability with high consequence in order to achieve due diligence. While most structural loading is well understood, blast loading falls into a unique category. The term 'blast' is defined as a destructive wave of highly compressed air spreading outwards from an explosion.

In the past blast design was normally considered for Government, Military structures or Industrial structures where manufacturing, processing and storage of hazardous substances such as explosives and flammable liquids and gases. Today, unfortunately terrorism is a reality, and designing for the safety of the occupants within structures has become more important. Blast design today has gained more attention as a design consideration for the safety of life where due diligence in design is required not only for government or military buildings, but for other high risk buildings including banks, hospitals and transport hubs.

Blast design requires a specialised understanding of the blast threat for both the impact loads from the initial wave front of the blast followed by time-dependent pressures, which occur due to thermal effects behind the wave front and suction pressures where equilibrium pressure return. In addition, reflected pressures and blast confinement must be where pressure loading is concentrated requiring additional factors of safety.

Research Goals

This study is focussed on the understanding of the nature of blast effects, its effect on structures and identifying methods for analysis under blast load conditions. The study is intended to highlight deficiencies in current national loading codes and develop ways for optimising a design in order to provide enhance resilience. The objective of the project is to develop a credible blast loading case based on multiple credible scenarios to be applied to structural model and study the effects of the interaction of the structure effects using Strand7 FEA. Identify methods for optimisation of critical elements and structural resilience. The scope of the research is concerned with primary effects of blast (pressure disturbance) interacting with structure, unconfined surface air blasts, secondary of blast (high velocity frag and thermal effects) not considered.

Need for blast design

The need for blast design can attributed to incidents where terrorist attacks have been involved where most recently a Civil hospital in Pakistan's Quetta in Aug this year (2016) an explosive blast ripped through the gate of the emergency department detonated by eight kilograms of explosives. What's interesting about this incident is an attack took place at the same hospital in 2010 and as such should have been foreseen and mitigated. Another recent incident happened in March where an attack on a Brussels Airport and train station occurred in Belgium involving suicide bombers detonating explosives causing large scale damage to glazing, building systems and deformed structures.

Some of the more notable blast incidents involving deliberate attacks include the bombing of a building precinct in Oslo Norway in July 2011 where a car bomb targeted key government buildings in the explosive blast was detonated by 950kg Ammonium Nitrate/Fuel Oil (ANFO). One of the most published blast cases is the Oklahoma federal building bombing in 1995 where a truck bomb detonated an equivalent 2300kg Trinitrotoluene (TNT) in front of the building which led to the buildings partial collapse. Figure 1-1 is shows the north side of the Alfred P Murray building missing after a truck bomb exploded.

Figure 1-1. Alfred P Murray Federal Building Oklahoma City 1995

Blast design is also important when working with known hazardous substances or industrial equipment. The importance of this can be seen in Sept 2016 where a boiler explosion occurred in a Bangladesh Cigarette packaging factory causing near total collapse of the factory building. In 2015 a chemical storage plant in Tianjin China experienced two large explosions, an investigation concluded in that an overheated container of dry nitrocellulose was the cause of the initial explosion.

A notable incident involving flammable gases occurred in London, on May 16, 1968, where a single match triggered the collapse of an entire corner of a 22-story building. A resident living on the 18th floor, lit her stove, triggering a gas explosion. The blast tore through the wall joint connections causing the load-bearing walls came apart, leaving the four apartments above without any kind of structural support. As a result, an entire corner of the building collapsed, shown in Figure 1-2.

Figure 1-2. Gas explosion in the kitchen on the 18th floor of Ronan Point London 1968.

Whenever pursuing blast design loadings, there is an obvious financial burden associated creating a robust structure to extreme loading cases. The need for blast design comes down to the predictability or foreseeability of blast event and the importance preserving the life of users or damage to the structure.

Chapter 2 - Literature Review

Historical and recent blast incidents

A summary of historical blast incidents collated from multiple information sources mainly news articles and Government databases (FBI, 2005) contained at Appendix D. A range of information was collated based on the types of blast events, cause of explosion and further categorised as accidental or deliberate actions. Where information sources were available the types of structures and damaged sustained was given.

While majority of the blast cases listed involved deliberate acts of violence, blast incidents were not just confined to terrorist attacks. Accidental incident involving blasts occurred where hazardous material or highly combustible industrial process were involved. This highlights the need for blast design consideration in a broader application. This year alone has seen at least 25 recorded terrorist's incidents and 3 industrial accidents involving explosive blasts.

Need for structural blast resilience

Blast related incidents such as in Oklahoma (in 1995), London (in 2005), Bali (in 2002), and New York (in 2001) have shown that the performance related failures of structures subjected to intense dynamic loading. Besides deliberate violent attacks targeted towards civil, commercial or military buildings, accidental explosions such as in industrial facilities have shown a similar vulnerability against intense dynamic loading. In 2010 that more than 11,000 terrorism-related attacks were reported in 2010 alone resulting in 13,000 fatalities and another 30,000 injured. This further illustrates that among the 11,000 incidents, 13% involved explosions, including civilian or commercial structures (US Department of State, 2010).

From the blast incident cases contained in Appendix D, highlighted a common trend in susceptible structures to blast events, these included:

- Embassies and consulates
- Government buildings
- Transport hubs including airports and train stations
- VIP accommodation: high profile hotels
- Public facilities: places of worship, hospitals
- Landmark and notable structures
- Chemical plants including petrochemical processing
- Munitions manufacturing and storage depots
- Manufacturing plants

Blast phenomena

The blast phenomena can be described as a pressure disturbance caused by a sudden release of energy being transmitted through a medium. Blasts can be further characterised by the medium type in which it passes through such as air, underwater or underground blast. This research project in limited to considering surface air blasts and the primary effects of the pressure disturbance interacting with structures. The nature of the blast incident has secondary effects beside exerting a pressure disturbance, it can be accompanied by the high velocity fragments, high temperatures gases and other chemical by-products. These secondary effects are not considered throughout the course of this project.

Following the initiation of an explosive event, causes a blast wave to propagate through air as it spreads out in a spherical wave (as illustrated in Figure 2-1) lead by a shock front. The blast wave travels perpendicular to the flow producing an instantaneous increase in pressure, followed by an exponential decay in blast pressures as the shock wave travels away from the point of explosion. As the blast energy deteriorates by the continual expansion of gas and heat, blast pressures decrease to ambient atmospheric conditions (Dusenberry, 2010), as shown in Figure 2-2. The magnitude of the blast effect diminishes with distance from the centre of the explosion which can be seen in Figure 2-3.

Cowperthwaite, 1965, Boogerd, Verbeek, Stuivinga, & et al, 1995 and Ahrens, 1993 suggest that blast shock wave theory can be used to derive peak pressure incidents based on the conservation laws. This was first hypothesized by Rankine-Hugoniot by applying the laws of conservation of mass, momentum and energy to a steady state shock moving in stationary material.

Conservation of mass (where u_0=0):

$$\frac{\rho_1}{\rho_o} = \frac{U}{U-u_1} \tag{1}$$

Conservation of momentum

$$P_1 - P_o = \rho_o u_1 U \tag{2}$$

Energy equation

$$e_1 - e_o = {}^1\!/_2\,(P_1 + P_o)(v_0 - v_1) \tag{3}$$

Note: the subscripts 0 and 1 denote the states just in front of and just behind the shock front, respectively.

Where:
$P = Pressure$
$\rho = density\ ahead\ of\ shock\ wave\ in\ mass\ per\ unit\ of\ volume$
$u = particle\ velocity$
$U = Shock\ velocity$
$e = specific\ internal\ energy$

From the momentum equation above the shock wave stress can be expressed

$$\sigma = \rho_0 U_s u_p \tag{4}$$

Given:
$\sigma = Stress\ exerted\ by\ the\ blast\ shock\ in\ force\ per\ unit\ surface$
$\rho_0 = density\ ahead\ of\ shock\ wave\ in\ mass\ per\ unit\ of\ volume$
$U_S = shock\ wave\ velocity$
$u_p = peak\ particle\ velocity\ (PPV)$

This equation considers stress wave through a solid material, where PPV is generally used in for rock formations as it is a descriptor for vibration. However, what can be described for solids may also be apply (in principle) to gases and liquids. Cooper P 1997 suggests that we are able to estimate the detonation velocity of an explosive at any particular density. These two parameters, D, the detonation velocity, and p, the density of the unreacted explosive, can be used to estimate the detonation or Chapman-Jouguet (CJ) pressure, P_{CJ}. It can be shown that the CJ pressure is:

$$P_{CJ} = \frac{\rho D^2}{\gamma+1} \tag{5}$$

Where:

P_{CJ} is the C-J, or detonation pressure, given in gigapascals (GPa);

ρ is the density of the unreacted explosive, in g/cm^3 ;

y is the ratio of specific heats of the detonation product gases; and

D is the detonation velocity, in km/s.

Generally, the detonation product gases are molecules such as H_20, CO, CO_2, N_2, etc. The particular composition or molar ratio of the product gases is a function of the composition of the explosive. However, for most explosives, the product composition is fairly similar and for the mixture, at the high temperatures and pressures encountered in detonations, is also similar. In the range of explosive densities from around 1 to 1.8, y is approximately equal to 3. Using this approximation and substituting into the above equation we find:

$$P_{cj} = \rho_o D^2/4 \tag{6}$$

Cooper P. K., 1996, describes that this simplified approximation of the equation to predict C-J pressure provides an accuracy within 7% for majority of explosives.

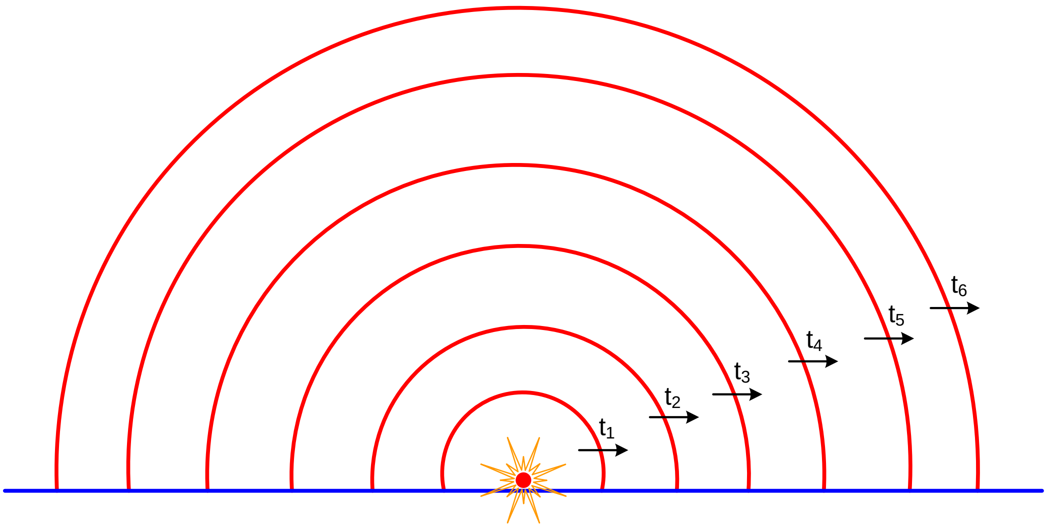

Figure 2-1. *Surface blast*

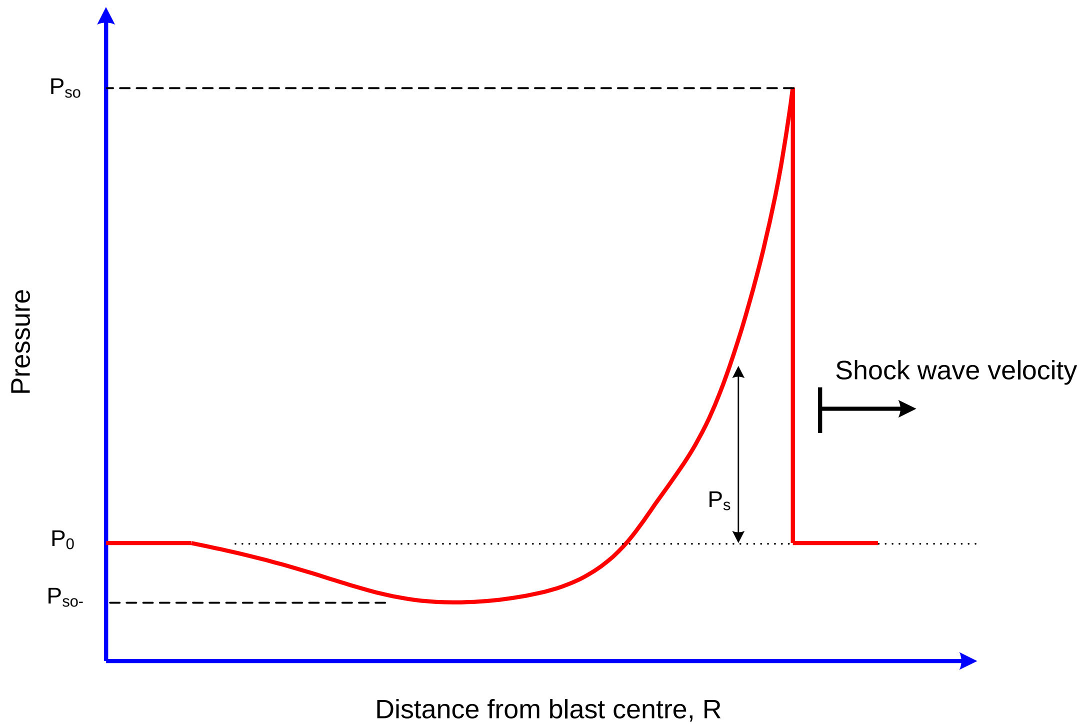

Figure 2-2. Variation of incident overpressure at a distance from blast centre at a given time (Cormine, Mays, & Smith, 2009)

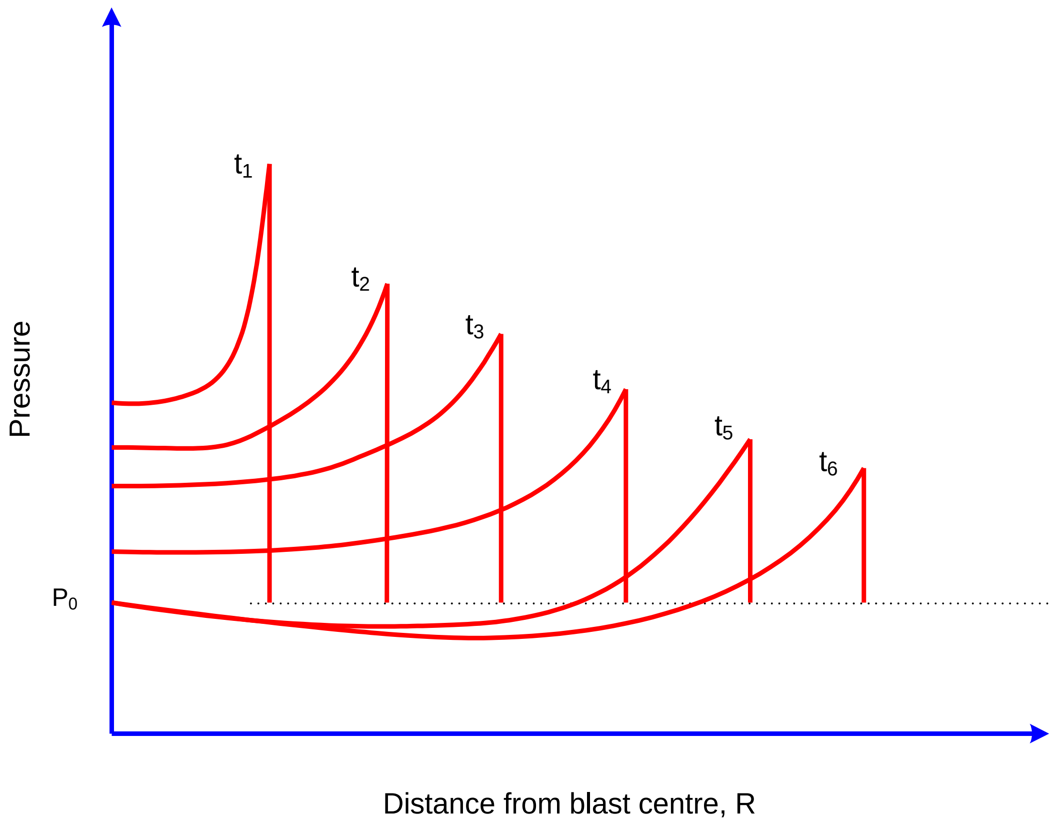

Figure 2-3. Incident overpressure (Ps) variation with the distance (R) from the charge centre (Source: Ngo et al. 2007)

Blast Characteristics

A typical blast waveform in Figure 2-4 is considered, which is caused by free-air detonation of a high explosive event at a stand-off distance (R). The blast wave propagates outward radially with decreasing velocity (Us). The time required for the shock front to propagate a distance from the point of detonation is known as the time of arrival (ta). A structure located a distance R, known as the standoff distance, from the point of detonation will experience an instantaneous increase in pressure from ambient pressure (Pa), to the peak overpressure of the shock front (Pso). As the blast wave continues to spread, the structure will experience an exponential decline in pressure until ambient conditions are reached. This duration of positive blast pressures (t0+), is the positive phase duration. The instantaneous increase in pressure can be followed by a rapid decrease, with the formation of a negative pressure. This negative phase is created by the rapid return of atmospheric conditions rushing to fill the void left by the blast front (Cormine, Mays, & Smith, 2009).

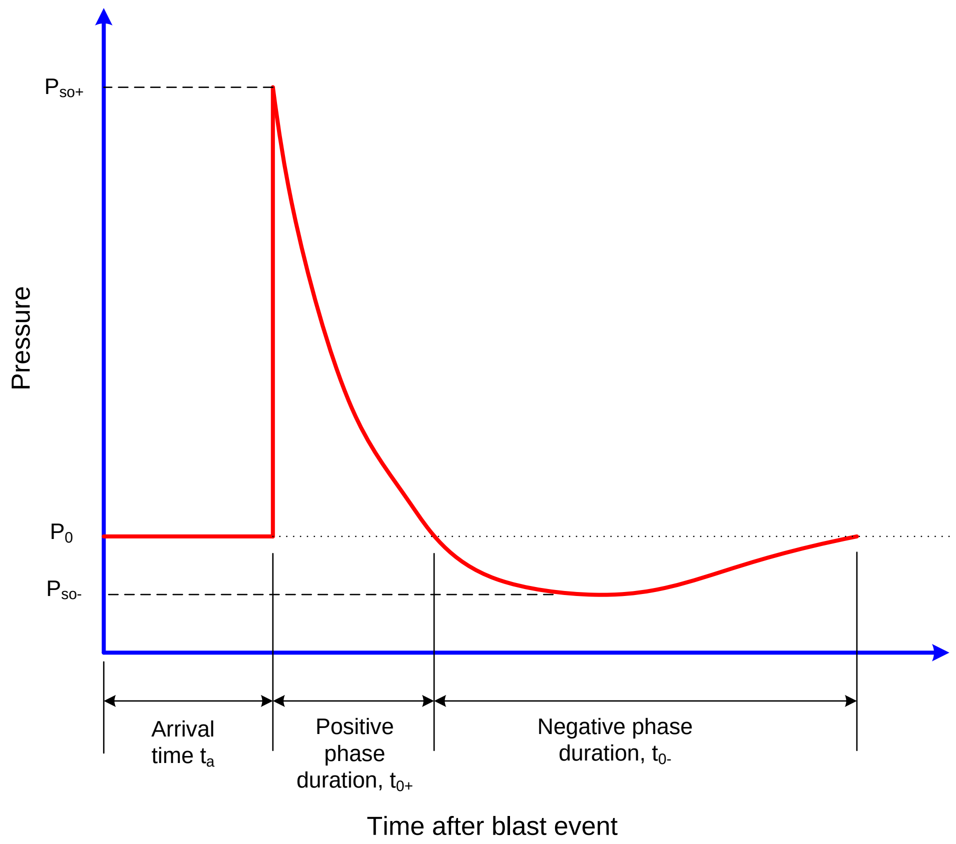

Figure 2-4. Shock wave pressure time history from blast Detonation (adapted from Dusenberry, 2010 Pg 164)

Detonation

Blast pressures, in particular peak incident and time dependant pressures can vary based on the type of response from an explosive event. A detonation is considered the worse response from an explosive event. A detonation is the supersonic combustion of a high explosive. This results in a self-propagating exothermic chemical reaction which transforms the original energetic material into vast quantities of gas. The initial detonation of a high explosive produces pressures of 10 – 30 GPa and temperatures of 3000-4000 °C (Smith & Hetherington, 1994). A typical detonating response to a blast incident is shown in Figure 2-4. The blast effects of solid materials or energetics are well established and understood. This is particularly true for high-explosive materials.

Deflagration

Unlike a detonation, deflagration event occurs when there is a combustion of a low explosive material. However, the rate of combustion is much lower, considered subsonic, and the corresponding shock front is much less powerful than high explosives. Both result in the formation of a blast wave as the ambient air is forcibly compressed by the expanding high pressure gas. This spherical blast wave produces a near instantaneous increase in pressure, followed by exponentially decreasing blast pressures as the wave travels away from the point of detonation. Eventually, as blast energy is dissipated by the continual expansion of gas and heat, blast pressures decrease to ambient atmospheric conditions (Smith & Hetherington, 1994). Figure 2-5 describes a typical blast pressure time curve from deflagration event.

Deflagration is not just relevant for explosive compositions, other solids, liquids and gaseous combustible materials exhibit a variation in blast pressure output. An explosive event from these materials can in many cases result in incomplete combustion, and only a portion of the total mass of the explosive (effective charge weight) is involved in the reaction process. Even detonation reactions can leave residual material behind where the remainder of the mass is usually consumed by deflagration resulting in a large amount of the material's chemical energy being dissipated as thermal energy (Department of Defense, 2014).

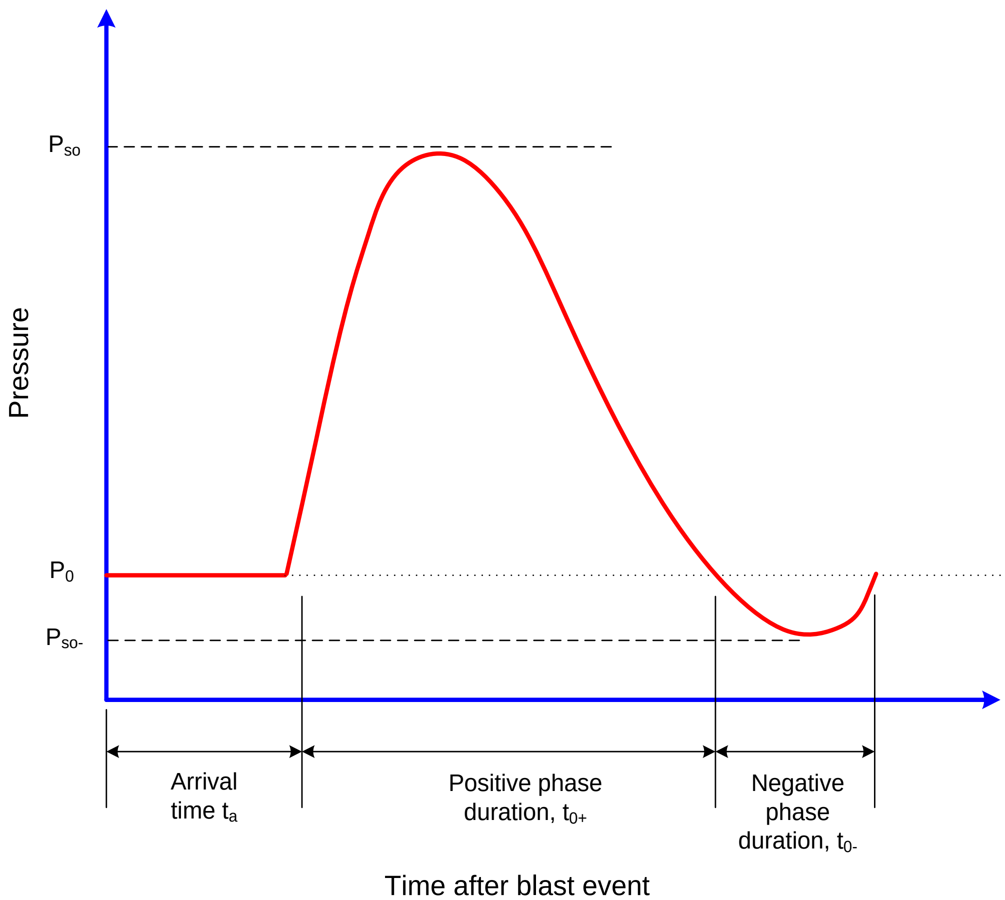

Figure 2-5. Pressure time history from blast deflagration (Dusenberry, 2010)

Reflection of Blast Waves

As a blast waves propagate outward from the point of initiation, these waves can interact with other surfaces which are not necessarily parallel to the blast wave propagation. This can result in a magnification of incident pressures and can experience much higher than the free air pressures. These amplified pressures are known as reflected pressures (Pr). The extent of magnification depends on the context of the layout including standoff distance, size and geometry of the reflecting surfaces and the magnitude of the incident pressures (Cormine, Mays, & Smith, 2009). Reflected pressures can be more critical than incident pressures for design purposes and can vary in magnification from an order of 2 or larger.

Angle of incidence

The angle at which an incident line makes with a perpendicular to the surface of a structure at the is known as the angle of incidence. This angle has a direct effect on the degree of blast reflections and ultimately the resultant blast loading on the structure. In the case of a building structure surfaces generally the side wall, roof and rear wall interactions, the angle of incidence effect may be neglected and studied under incident pressures where surfaces have a high angle of incidence for surfaces that are parallel or behind the blast wave. Surface interactions with a low angle of incidence tend to increase the reflections and therefore cause the magnification of incident pressures leading to higher blast pressure loadings. For smaller structures, the effects of incidence may become negligible as the distance from the blast origin remains relatively constant at all faces of the structure.

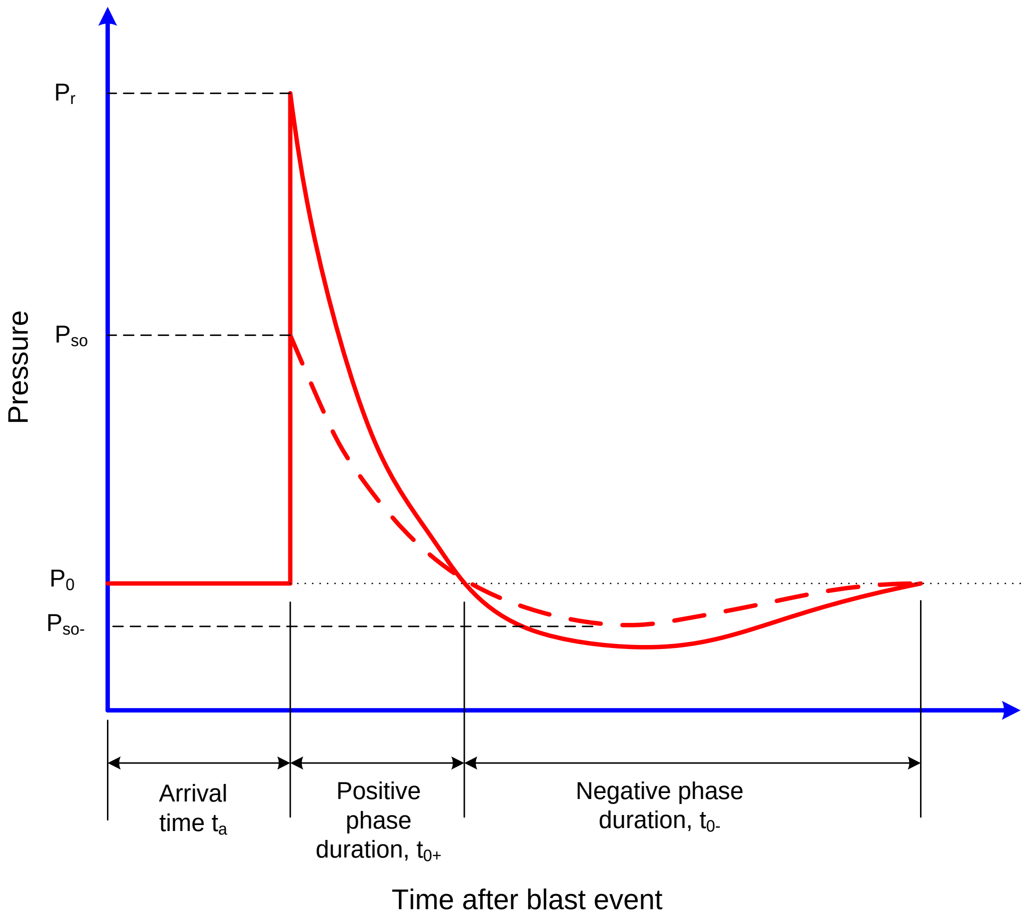

Figure 2-6. Pressure time history of free field blast and reflected blast

(Dusenberry, 2010)

Diffraction

During blast wave interaction with a structure, diffraction could occur at the fringes or edges of the structure. Diffraction causes the blast wave to bend and distort around surface edges, illustrated in Figure 2-7. As a result, diffraction has the potential to reduce the effects of blast pressure on the side of a structure while the front surface will encounter the maximum blast pressure (Cormine, Mays, & Smith, 2009).

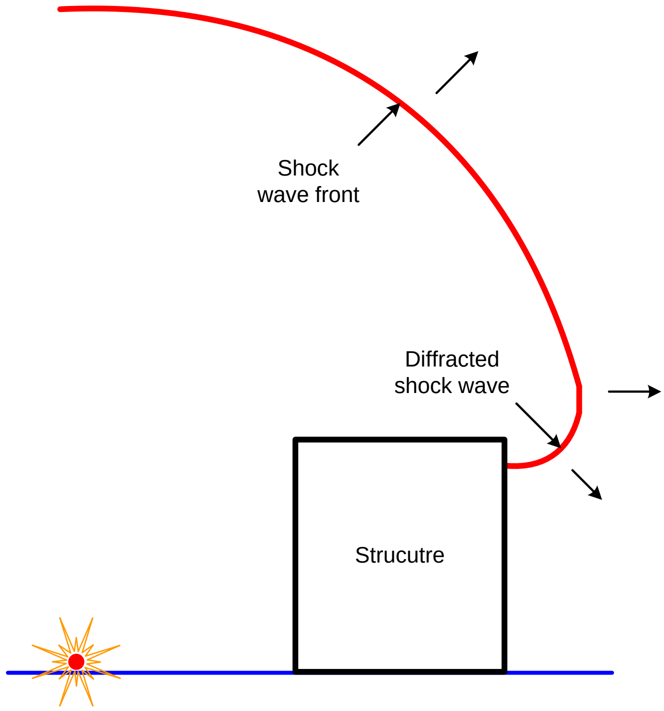

Figure 2-7. Blast wave diffraction

Rarefaction

Refraction is the change in direction of propagation of a blast wave due to a change in density of the medium it interacts with, as illustrated in Figure 2-8. The refracted vector represents the stress wave being transmitted in to the receiving medium.

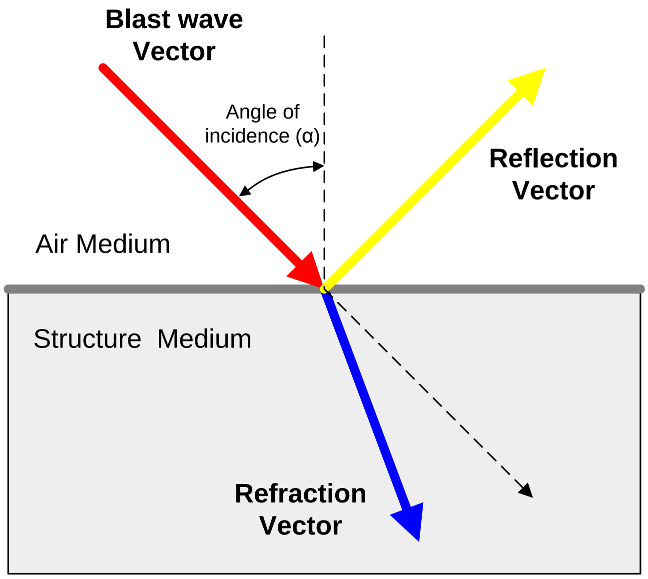

Figure 2-8. Refraction and reflection of blast waves interacting with two mediums (Source: adapted from USQ Study book MIN2001 Drilling and Blasting)

Blast Confinement

When a blast occurs within a structure, the peak pressures will be amplified by their reflections within the structure. In addition, confinement, the effects confinement of the high temperatures and accumulation of gaseous products produced by the chemical process involved in the explosion will exert additional pressures and increase the load duration within the structure. The combined effects of these pressures may eventually destroy the structure unless the structure is designed to sustain the effects of the internal pressures (Department of Defense, 2014).

Predicting blast loads

Blast Loading Design Standards and Guidelines Review

Any design standard or guidelines need to be carefully considered for the applicability to the loading case and the basis for any design assumptions. Typically, major structural design codes have given limited attention to explosive loading, partly due to the scarcity and extreme nature of the loading. There exists a situation, it becomes apparent where the use of the national standards is not sufficient or where part or parts of the standards may be inappropriate due to situations where the loading case is not covered adequately by the Standards, numerical data given does not reflect accurately the actual situation, or the use of the Standard leads to very conservative and uneconomical solutions.

Australians Standards

AS 2187.2—2006 Explosives—Storage and use Part 2: Use of explosives

AS2187.2 sets out limits for vibration and air blast. The study of the blast induced ground vibration effects is outside the scope of this project and therefore not considered. However, the air blast component is relevant although the air blast limits provided in the standard and are governed by discomfort levels as opposed to structural limitations resulting a peak sound pressure level limit of 120db (0.003psi). At this pressure the structural effects are likely to be very minor, therefore it is not considered a factor when analysing structural blast effects.

AS/NZS 1170.0 Supplement 1:2002, Structural design actions—General Principles—Commentary, (Supplement to AS/NZS 1170.0:2002)

While Australian design standards overlook the blast loading effects it gives guidance on methods for obtaining blast loading actions. Australian Structural loading codes do not deal specifically with blast loading scenarios, whoever recommend that special studies be conducted where the methods or information contained in a design codes are outside the scope of the structural design actions. Thus requiring justification for approval under building regulations. The special studies are a means of determining the likely loading effects for use in design that is not covered in the Standard. AS1170 recommend special studies be undertaken usually at the initiative of the structural designer for any structure, subject to satisfying the requirements of the appropriate authority and or client. A special study may be accompanied by field testing as part of the study. The standard provides additional information on actions not specifically covered, the include; movement effects; construction loads; and accidental actions.

Further guidance in AS1170 suggest providing structural robustness for structures by designing and constructing in such a way that prevent damage by events like including fire, explosion, impact or consequences of human errors, to an extent disproportionate to the original cause. The potential damage may be avoided or limited by use of the following:

a) Avoiding, eliminating or reducing the hazards which the structure may sustain. (Security measures)
b) Selecting a structural form that has a low sensitivity to the hazards considered. (Size, shape and location of structural member)
c) Selecting a structural form and design that can survive adequately the accidental removal of an individual element or a limited part of the structure or the occurrence of acceptable localized damage. (Material selection and redundant structural members)
d) Avoiding as far as possible structural systems that may collapse without warning. The design should provide alternate load paths so that the damage is absorbed and sufficient local strength to resist failure of critical members so that major collapse is averted. The materials design Standards usually contain implicit consideration of resistance to local collapse by including such provisions as minimum levels of strength, continuity, and ductility. Connections for example should be designed to be ductile and have a capacity for large deformation and energy absorption under the effect of abnormal conditions.

AS1170 also sets out confirmation test methods that a design is required to pass in order to conform to the standard. The validation methods given relate to calculation methods. They are a specific set of descriptions that separate desired states of the structure from undesired states. For other methods (e.g., prototype testing) special studies are required

a) Ultimate limit states in stability - limiting equilibrium of the structure or parts of the structure. Loss of equilibrium can result in uplift, sliding or overturning.
b) strength (ultimate limit) and for simplicity, a state prior to structural collapse may be considered as an ultimate limit state, e.g., reduced structural system following an accidental action.
c) serviceability limit states include:
    i. Local damage (including cracking), which may reduce the utility of the structure or affect the efficiency or appearance of structural or non-structural elements; repeated loading may affect the local damage (e.g. by fatigue).

ii. Unacceptable deformations that affect the efficient use or appearance of structural or non-structural elements or the functioning of equipment.
iii. Excessive vibrations that cause discomfort to people or affect non-structural elements or the functioning of equipment.

Where Australian Standards lack guidance for the determination of blast loading, a designer may choose to make reference to a reliable source of text related to the loading type, use collected data from case studies or carry out testing to establish factors to be used in design. The building authorities will require that the use of such information be justified therefore level of rigour and expenditure spent on testing should be proportionate to the inherent risk in the design i.e. the risk in not designing to that particular load case. Any information gained using testing methods may be also based on rigours mathematical analysis or based on a longstanding experience from a reputable industry source.

AS1170 states that all perceivable accidental actions shall be considered for the design of the structure including explosions, collisions, fire, unexpected subsidence of subgrade, extreme erosion, unexpected abnormal environmental loads (flood, hail, etc.), consequences of human error and wilful misuse. While it is impractical to design for every accidental actions as they are very low probability events. However, precautions should be taken to limit the effects of local collapses caused by such actions, that is, to prevent progressive collapse. The level of rigour should not be grossly disproportionate the inherent risk in the design.

Foreign Standards and Guides

2014 Unified Facilities Criteria (UFC 3-340) Structures to resist the effects of accidental explosions

The UFC document provides practicable and readily useable information on the application of blast actions loading. The document is based on a comprehensive and extensive range of testing combined with research and development programs. It is the most reliable and up to date for existing and new last design requirements. To date the manual has undergone numerous revisions since its original issue as a result of extensive testing and development projects.

The manuals user friendly design techniques are based on results of numerous large and small scale explosive testing and structural effects for various construction materials. However there have been limited testing for extremely large scale explosions. Therefore, the limit of the manuals application to determining design requirements is restricted to those explosive quantities of less than 25000lbs (11400kg). The guidance provided by this manual produces a simplistic yet conservative triangular pressure time history for a given blast scenario. The manual covers range of scenarios includes free air, surface, semi covered and internal blast scenarios.

FEMA 427 (2003) - Primer for design of commercial building to mitigate terrorist attacks

FEMA 427 manual provides general guidance to structural designers for commercial building to mitigate the effects of hazards primarily resulting from terrorist attacks for any new building. The guidance is primarily limited to the conceptual level with a strategic approach to designing security into a building. While the guidance provided focuses on explosive attacks it also addresses design strategies to mitigate the effects of chemical, biological and radiological attacks. Also the information has wider application s to the design on commercial precincts including offices, retail and multistorey residential housing and industrial buildings.

2004 PCI designer's notebook Blast considerations

The Prestressed Concrete Institute notebook is limited to addressing designs for blast resistance against external blast loads rather than internal blast loads. The suggested blast loading determination is to use empirical methods (described by UFC 3-340) and specific blast software packages for greater accuracy. It's primarily focus is on blast threats from vehicle delivered explosive devices. The notebooks application is based on the premise that there are no formal blast performance criteria requirements of civilian structures. Its application is based on the development of anti-terror requirements by US Government Department reserved for Military applications, Embassies and federal buildings. The design a structure that can sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original localized damage. The primary failure mode of concern is the progressive collapse of a building. The notebook also provides limited guidance on construction techniques including glazing, façade material and the importance of connections.

2010 Design of blast resistant buildings in petrochemical facilities

The purpose of design document is to provide a guide to designers and those in the petrochemical industry involved in the design of new blast resistant buildings and in assessing existing buildings for blast resistance. It provides the basic considerations, including principles, procedures and details involved in structural design and evaluation of buildings for blast overpressure effects. This document focuses on "how to" design, or evaluation of buildings for blast resistance once the blast load is defined for a postulated explosion scenario.

Buildings and Infrastructure Protection Series Preventing Structures from Collapsing to Limit Damage to Adjacent Structures and Additional Loss of Life when Explosives Devices Impact Highly Populated Urban Centers BIPS 05/June 2011

The document has broader emphasis on the urban environment rather than specific structures. Several key areas of concern include the study of the blast response of columns under urban blast loading scenarios and the evaluation of new methods to mitigate the potential for large scale structural failure (progressive collapse) and collapse in response to extreme loading conditions associated with explosive attacks. Another main area of focus is the determination of air blast pressure levels in an urban setting and the influence of the presence of buildings on the pressure and impulse levels that result from explosions. The primary concern of the manual raises the importance of quantify accurately the air blast environment resulting from the detonation of an explosives in an urban setting to evaluate the performance of structures in response to these loads.

Threat analysis

One of the first steps in developing a credible blast loading case is anticipating the type of blast threat. In the case of deliberate malicious accidents this, requires a thorough assessment of the security layout and intelligence gathering of potential extremist's groups or individuals. The subsequent design criteria then become highly sensitive information, where knowledge of threat analysis would provide a would be attacker an advantage in identifying weak points or vulnerable targets to expose of inflict maximum damage for minimal effort (Dusenberry, 2010).

As shown (US Department of State, 2010), blast related events mostly originate from violent attacks, targeting civilian or commercial structures. However, accidental events such as explosions in storage facilities or gas explosions can also occur. The severe nature of loading results in catastrophic failures of structural elements and ultimately loss of life either from direct or indirect effects of the explosion. Therefore, it is essential to estimate and predict the effects of explosions and provide designs that protect structures against the potential explosive events.

Credible threats can be characterised by explosive weight (sometimes referred to as equivalent weight of TNT which governs the magnitude of blast) for solid materials or potential energy where accidental incidents are considered. Blast loading is also largely affected by standoff distance useful resources for estimating explosive weights are provided by FEMA based on various delivery methods, illustrated at Figure 2-9.

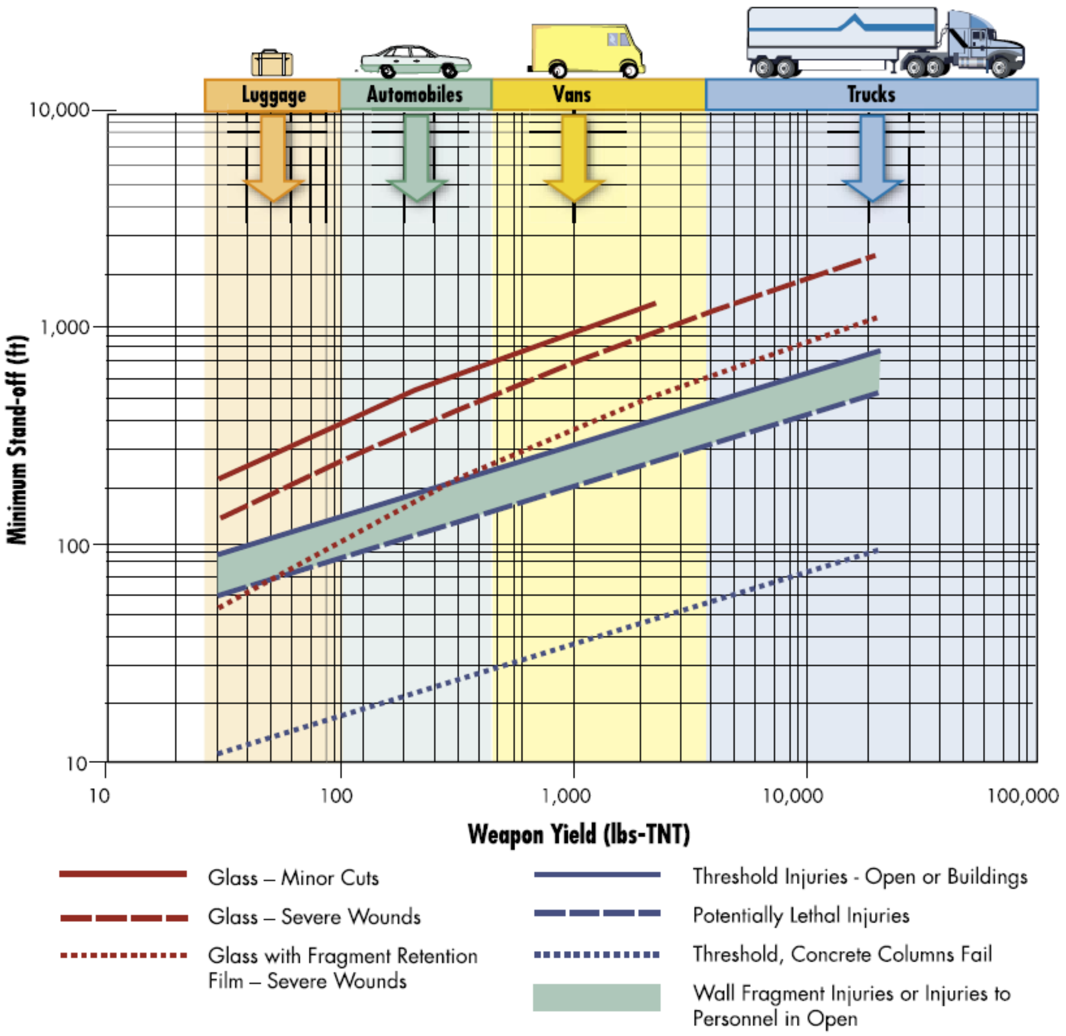

Figure 2-9. Threat assessment
(FEMA, 2003)

Causes of blast events

It is also important to understand the underlying cause of blast events including its delivery system or conditions around which contribute to the blast event. Blast threats can be further categorised for threat analysis as:

- Deliberate
  - Malicious act including:
    - Mail bomb
    - Vehicle borne explosive device
    - Personal borne explosive device
    - Aircraft borne explosive device
    - Weaponry
  - Legitimate act including:
    - Mining operations
    - Excavations
    - Demolitions
- Act of god
  - Lightning strike or earth quake causes sudden release of potential energy source including:
    - High power electrical equipment (arc flash or arc blast)
    - Stored flammable gas or liquid
    - Stored high pressure gas or liquid
- Accidental
  - Munitions manufacturing
  - Flammable gas and liquid processing
  - Condensed phase explosions
  - Chemical reaction with runaway exothermic properties.
  - Rapid physical vapour reactions (two mediums with different temperatures mixtures suddenly to create a rapid phase change generating excess pressure that exceeds containment vessel e.g. molten metal poured to cold mould or water into hot oil)
  - Processing involving combustible dust:
    - Sawmills
    - flour mills
    - sugar refinery
    - metal foundries
  - Flammable gas and liquid storages
  - Explosive ordnance storages
  - High pressure gas or liquid storages
  - heating vessel contents with insufficient pressure relief, or other means
- Indirect
  - Criminal act causes release of large potential energy source
  - Arson causes release of stored energy

Blast Loading Methods

There are many different methods for determining blasts loads. In determining the blast effects on any given structure, there needs to be considerable thought as to the level of accuracy, speed of analysis and risks associated with determining the effects of a blast incident. Some of the methods available include:

- Empirical
  - This method consists of graphs (blast curves) and established equations based on experimental data or first principles of physics describing the behaviour of pressure waves in a fluid like environment.
  - Empirical methods can be readily applied with limited knowledge and experience
  - However, they are only applicable to certain scenarios
- Semi Empirical
  - This is an extension of empirical methods above where experimental data or graphs can be extrapolated or manipulated in a justified manner to develop more accurate predications (Rose, 2001)
  - The method requires a more in depth knowledge as to the applicability and manipulability of the data.
- Analytical (CFD or Hydrocodes)
  - Software codes that models the behaviour of fluid like environments that is highly adaptable to suit varies situations
  - The software requires a high degree of skill and experience to use
  - Requires high performance computing equipment
  - Computing time and accuracy can vary depending on cell sizes chosen and time steps for convergence
- Experimental (field tests)
  - For obvious reasons experimental blast tests are costly and come with considerable risks, particularly full scale tests
  - Field tests require specialist skills with extensive experience and access to purpose designed test sites
  - Blast data results gained from field tests are generally only relevant for individual scenarios.
  - However, field tests could be down sized as scale models to reduce cost and risks

Empirical methods

The empirical methods for determining blast loading consists of published equations, graphs, tables, and figure that allow one to determine the principal loading of a blast wave on a building or a similar structure. The most extensive and widely referenced publication for empirical design is UFC 3-340-02 (Department of Defense, 2014). This manual addresses accidental explosions related to munitions manufacturing, handling, and storage. These empirical methods are based on extensive experimental testing combined with analytical to develop predicative techniques and adopting scaling laws for ease of prediction and application that form the basis calculating various blast scenarios.

Blast curves methods have been widely used using the scaled standoff technique by using distance from the centre of the blast explosion to the point of interest and the energy content of the confined/congested flammable mass. From this scaled pressure and impulse values are read from blast charts containing flame speed curves. Vapour cloud blast load prediction can be more complex than loads for high explosive detonations. In these cases, it is necessary to develop the release scenario for the flammable material using simplified methods consisting of graphs (blast curves) of pressure and impulse, or of duration versus scaled standoff is an acceptable practise (Task Committee on Blast Resistant Design, 2010).

The procedure described in UFC 3-340-02 (Department of Defense, 2014) to develop blast loading case and illustrated in Figure 2-10, requires the following Inputs:

TNT equivalent weight W_e given:

$$W_e = W_{exp} \frac{H^d_{exp}}{H^d_{TNT}} \tag{7}$$

Where:

$W_{exp} = Weight\ of\ actual\ explosive$

$H^d_{exp} = Heat\ of\ detonation\ of\ actual\ explosive\ (joules\ per\ unit\ of\ weight)$

$H^d_{TNT} = Heat\ of\ detonation\ of\ TNT\ (joules\ per\ unit\ of\ weight)$

Scaled distance Z given:

$$Z = R/W^{1/3} \tag{8}$$

Where:

$R = Standoff\ distance\ from\ strucutre\ surface$

$W\ = TNT\ equivalent\ weight$

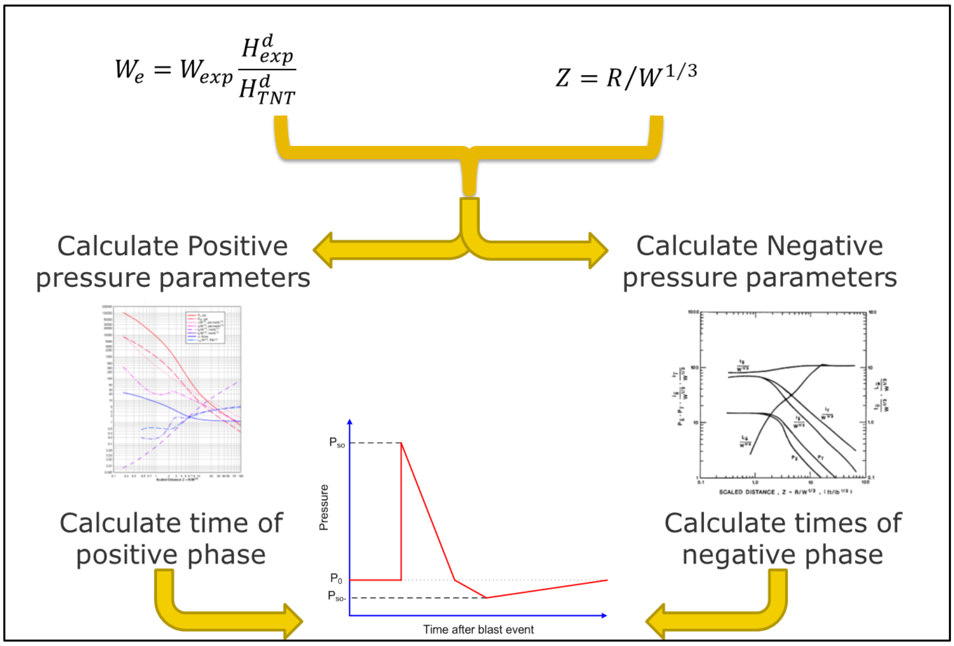

Figure 2-10. UFC-340-02 Procedure for determining blast pressure time history curves

Analytical methods

The difficulty of blast loading provides numerous variables and complexities, which all must be accounted for at the same time. The coupling of this type of dynamic loading and structure response which was intended to be predominantly static loads complicates the analysis when modelling both the linear and non-linear response of a structural members. Therefore, it is important that validation of analytical models is undertaken for structures under blast loading, but requires experimental data, which are not always readily available and the data from which the models are defined in not in abundance. Using existing similar experimental data can be used although any inference of data read across needs to be undertaken carefully.

Hydrocodes

For a more realistic and thorough analysis, numerical methods that are founded on the fundamental first principles of physics are commonly used. These include hydrocodes or computational fluid dynamics (CFD) that provide a means of modelling structures and the atmosphere interactions (Remennikov A. R., 2005). These methods provide a significant benefit in calculating blast loading actions as they are able replicate realistic structural layouts, ranging from individual member to complex and intricate structures, and provide a range of any number of data collection points for blast parameters required in order to provide a thorough understanding of the blast wave interactions with the structure and resulted stresses induced.

Hydrocodes however, are not without some drawbacks. In order to achieve high levels of accuracy for shock simulations and provide accurate to simulations of complex structures, small cell sizes are required and which as a result consume significant computational time and processing power (Remennikov A. , 2003). While many programs merely simulate the blast in air, more accurate models are possible with programs which model the dynamic response of the structure. Such programs include Air3D, ANSYS, AUTODYN, COMSOL and LS-DYNA, although the inclusion of this structural response requires an understanding of the structure and is reinforced through comparison to, or understanding of, experimental methods (Ngo, 2007).

ConWep

The more common computer aided blast modelling for basic geometries is the Conventional Weapons Program or better known as ConWep (Hyde, 1992). ConWep is a program to defence industry, however the United Nations have developed guidelines for adequate ammunition management by establishing the UN SaferGuard Program. This program provides online tools including the Blast Parameter Calculators to predict key blast parameters in determining free-field pressures and loads on structures.

The key information required in order for ConWep to predict the blast loading from the program is the charge weight and standoff range to structure. From this information the ConWep software is able to determine the a blast pressure time history on the basis of a polynomial fit of empirical data from a set of large scale explosions calculated by Kingery and Bulmash (Remennikov A. , 2003). This polynomial provides a good fit to the data set, within 6.4%, however the data set only includes blasts of TNT measured without consideration of environmental conditions (Swisdak, 1994). Environmental conditions, such as inversion layers and wind direction may have influenced these experiments, and such a small number of blasts may also raise concerns regarding variation between lots and packing densities. Notwithstanding this, ConWep continues to be one of the simplest and easiest methods of basic blast prediction. The output however, is limited to providing positive phase blast pressure time histories and neglects any negative phase.

TNT equivalence

For convenience in predicting blast pressures, the energy release of high explosives is commonly measured as a value relative to that of TNT. This TNT equivalence is used to determine a TNT charge weight capable of producing the same explosion energy, blast pressure, or blast impulse as the explosive of interest. The TNT equivalence is different for energy, peak pressure, and impulse, and separate TNT equivalent energy values are reported for many materials. Blast predictions made using a TNT equivalent approach tend to be inaccurate for deflagration events (Dusenberry, 2010).

Locking, 2011 suggests that TNT equivalence of energetic material in common modelling explosives is difficult to ascertain, with one review of the available literature observing a spread from 1.09 to 1.80. This wide spread of possible equivalence factors is due to the different methods which can be used to assess different elements of a blast (Cooper, 1994). The primary areas of comparison are peak pressure and impulse, with methods of measurement broadly grouped into thermochemical and physical methods.

Blast Scaling Laws

Blast parameters including pressures, load duration, impulse, shock wave velocity, arrival times, and are often presented in scaled form. Scaling laws can be applied to explosions, allowing data from one explosion trial to be applied to a geometrically similar cases. As a result, scaling has adaptability in blast predictions, allowing modelling to be used to predict loads for a variety of explosion energy and standoff distance. The most common form of scaling is called “cube root scaling” owing to the fact that blast parameters are scaled by the cube root of the explosion energy (Dusenberry, 2010). Once scaled distance have been calculated Figure 2-11 can be used to determine the required blast parameters.

$$Z = R/W^{1/3} \tag{9}$$

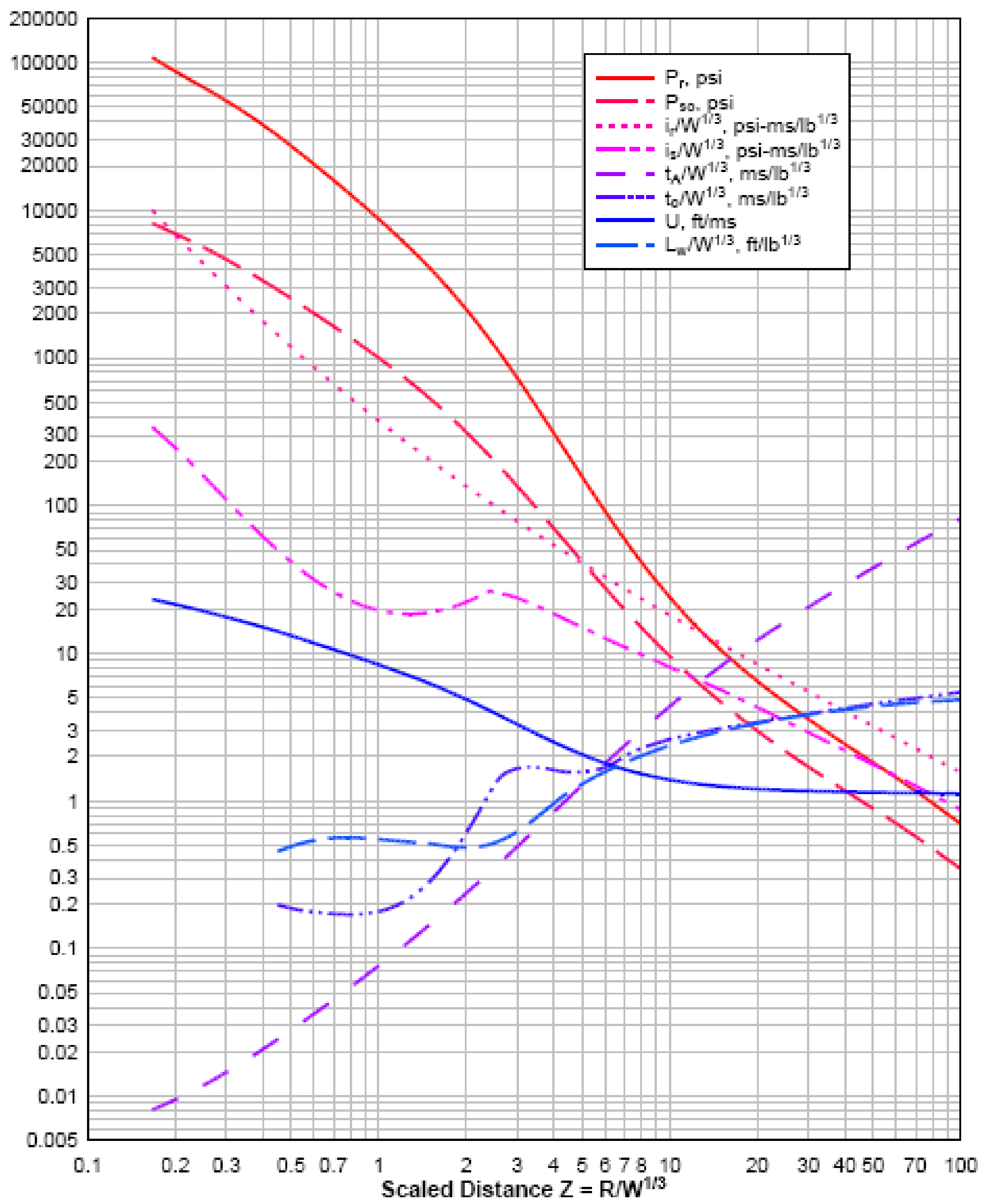

Figure 2-11. Blast Parameters for TNT Surface Bursts
(Department of Defense, 2014)

Comparison Blast prediction methods

In order to compare the various methods employed in blast determination, field tests and numerical results from the International users conference for LS DYNA (Huang, 2010) are illustrated in Figure 2-12 and Figure 2-13, to validate CFD Software and compare similar blast simulation software Air3D. In order to compare empirical blast methods, the same test data parameters were used:

- Blast test parameters
  - W = 27.26g TNT
  - R = 1.5m

- Test structure wall constructed from 10mm steel plate
  - H = 180mm
  - D = 60mm
  - W = 180mm

From these parameters blast wave pressure time history and impulse curves were developed using the empirical methods contained within UFC-340 and ConWep to compare and contrast numerical vs empirical vs experimental results.

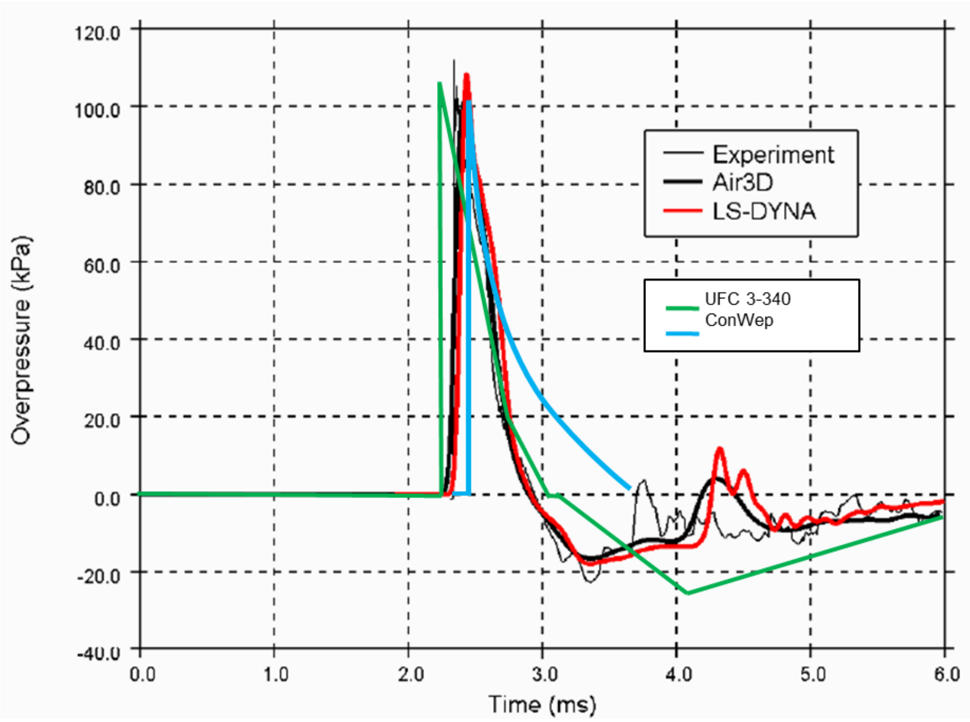

Figure 2-12. Comparison of Blast Pressure Time History Prediction Methods

Table 2-1. Comparison of Numerical vs Empirical vs Experimental Data

Method	Arrival time (ms)	Peak Positive Pressure (kPa)	Peak Negative Pressure (kPa)	Positive Phase duration (ms)	Negative phase duration (ms)
Experimental	2.25	112	-22	0.65	3
LSDYNA	2.24	109	-18	0.64	3.1
diff % with experimental	-0.44	-2.68	-18.18	-1.54	3.23
Air3D	2.22	102.00	-17.00	0.70	3.10
diff % with experimental	-1.33	-8.93	-22.73	7.14	3.23
UFC-340	2.20	107.00	-24.00	0.75	3.20
diff % with experimental	-2.22	-4.46	8.33	13.33	6.25
ConWep	2.45	101.00	N/A	1.20	N/A
diff % with experimental	8.16	-9.82	N/A	45.83	N/A

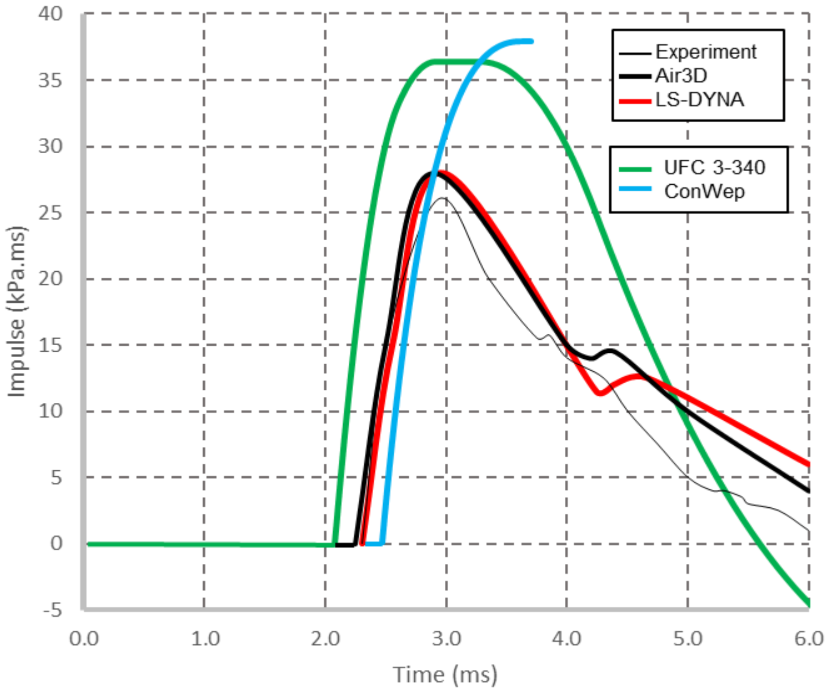

Figure 2-13. Comparison of Blast Impulse Time History Prediction Methods

While the numerical results show a more realistic blast pressure curve fit, the empirical methods show the peak positive pressures and arrival times were comparable (<5% UFC, <10% ConWep peak Pressures and <3% UFC, <9% ConWep for arrival times) however, only the UFC blast curve provided a positive phase blast pressure history. Comparing the impulse time history curves at Figure 2-13, both the UFC and ConWep methods provide overly conservative positive phase predictions with larger impulses. But again the ConWep derived Impulse, due to the fact it neglects negative phase of the blast, stops at the peak of the impulse. The UFC derived impulse appears to provide a conservative pressure loading duration in comparison to the numerical and experimental methods.

Method of studying a structure subjected to blast

The analysis and design of structures subjected to blast loads require a detailed understanding of blast phenomena and the dynamic response of various structural elements. A structural model needs to consider modern day building materials and common structural element forms. Common building materials include steel, reinforced concrete and timber. It is anticipated that reinforced concrete will be used in developing the material model. The material strength limits and degree of high rates of strain (strain hardening) will also need to be carefully considered as they will likely effect the dynamic aspects of the modelling. Common building structural elements including beams, slabs, walls, columns and footings will be considered when developing a structural model with due consideration for relevant application to various situations.

Structural damage acceptability is governed by the tolerable levels of deformation, cracking or strength limits. One possible method of understanding the interaction of blast pressure and the structures is to assess the whole structure altogether or individual components of a structure based on the full scale model. Composite construction can have major advantage for blast-design applications due to the mass effect of composite systems with steel elements and concrete elements. The inelastic action in a composite system generally will limit deflection and local deformations, and partially mitigate rebound effects through the damping effect of concrete cracking (Dusenberry, 2010).

Static vs Dynamic Analysis

Any structure undergoing extremely varying load conditions is a likely candidate for dynamic nonlinear analysis as it is more suited to providing an accurate description of the stress conditions likely to be encountered in the design compared with a static approach. Conversely an elastic static approach is likely to provide excessively conservative design values if only the peak incident pressure is considered without any consideration for the effects of load duration i.e. impulse.

Margins of safety factors against structural failure are attained through the use of acceptable deformation criteria. Structures that are subjected to blast loading is normally allowed to undergo a state of plastic (permanent) deformation in order to absorb the explosive energy. Whereas the response to conventional loads including dead and live load are normally required to remain in the elastic range. Therefore, the more deformation a structure or member cans endure, the more blast energy that can be absorbed.

Dynamic analysis rather than static is able to account for the very short duration (ms) of the loading effects. Also, the inertial effect that is a crucial component to dynamic loading computations greatly improves response accuracy. This occurs due to the time the mass is mobilized, the loading effect becomes greatly diminished, in effect enhancing the response of the structure. In addition, by having some degree of tolerance against damage occurring, it is possible to account for the energy absorption of ductile systems that occurs through plastic deformation. Finally, due to the loading being so rapid, we are able to utilise the enhanced material strength that often occurs with very high strain rates (Dusenberry, 2010).

Single degree of freedom (SDOF)

A SDOF system is a method which motion is defined just by a single independent co-ordinate as function of time. SDOF systems are more often used as a very crude estimate for a much more complex structures. The use of SDOF models has been extensively used for column design under dynamic load conditions although it is typically developed with flexural failure in mind (Cormine, Mays, & Smith, 2009). However, shear failure is the dominant failure mode for columns subjected to close in blast (Dusenberry, 2010). Also, SDOF methods do not account for the effects of tightly spaced stirrups on column response. Any analysis undertaken using a SDOF approach will be used for the preliminary design and a more sophisticated approach, using finite elements, will be required for the detailed design and verification. For SDOF systems, material behaviour can be modelled using idealized elastic, perfectly-plastic stress-deformation functions, based on actual structural support conditions and strain-rate enhanced material properties. The model properties selected to provide the same peak displacement and fundamental period as the actual structural system in flexure.

Strain hardening

As disused in static vs dynamic loading, under short impulsively applied loads, the structures strength of the material is increased. This characteristic is referred to as strain hardening. A structures design limit states will need to account for the increase in strength factor for flexural or tensile response, to account for strain-hardening effects. Construction of steel and composite concrete structures typically has a linear stress-strain relationship up to the yield stress (Dusenberry, 2010), although they can undergo high levels of elongation without an increase in stress, approximately 10 to 15 times that required to reach yield limits. Therefore, stress increases the strain hardening effect up to a range until a total elongation of 20 to 30 percent is reached. This response has an advantage in the design for blast resilience for resisting the effects of a blast (Dusenberry, 2010).

Redundancy and load distribution

The blast loading damage effects for close in explosions has the potential to severely damage vertical structural supports creating a situation where building collapse is possible. A design that satisfies all required ultimate limit strength and serviceability criteria would be inadequate without redundancy of load path in the event of loss of vertical support member. To limit the extent of collapse of adjacent components and design needs to consider highly redundant structural systems. Analysis against progressive collapse of the building should ensure the structure can endure the removal of one primary exterior vertical or horizontal load-carrying element. This may involve removal of; a connection at a critical joint; nearest column from a blast; beam or a portion of a load bearing/shear wall system for interior events in order to assess the response against redundancy test and load redistribution.

Inertia effects

Inertia effects are largely ignored during static analysis. During structural loading action, the structure accelerates from its initial position to develop resistance against the applied loading. The structural resistance increases with an increased deflection, the difference between applied load and the resistance is reduced and the structure will eventually decelerate. Ultimately the structure will come to rest when the developed resistance is matched with the applied load.

Structural response to blast

Following the bombing of the Alfred P. Murrah Building, investigators had shown that FEA models would have accurately predicted the failure of the bearing columns nearest the point of detonation, and the resulting progressive building collapse (Cormine, Mays, & Smith, 2009). These models also showed that the use of structural mitigation strategies including composite wraps or steel jackets, may have increased the columns blast resilience necessary to prevent their ultimate failure, and resulting partial collapse of the building which is known as the Oklahoma bombing incident. (Dusenberry, 2010)

Structural members subjected to blast loading effects, depending on the magnitude of the effects, may produce both local and global responses related with different failure modes. The type of response depends primarily on the rate of loading, the orientation of the target in relation to the blast origin and blast wave propagation and boundary conditions. The failure modes associated with blast loading involve bending, direct shear or punching shear. Local effects categorized by localized delamination of composite materials, breaching and spalling, as a result from the close-in effects of explosions, while global responses are typically revealed as flexural failure.

Analysis tools for modelling Structural responses to blast

The difficulty in modelling structural responses to blast loading and is due to the impermanent short duration the structure will be exposed to such as rapid change in pressure being applied. A suitable analysis must will need to accounts for elastic-plastic and dampening behaviour of the material properties, inertia of structural mass and both local and global failures. Some of the FEA software tools capable of accounting for dynamic loading and structural responses include Strand7, LSDYNA, ANSYS and MIDAS to name a few.

Chapter 3 - Methodology

Approach

The methodology employed to study the blast effects on structures is aimed at analysing the global effects of a structural building and the local effects of a critical structural element. In order to achieve this the following case studies have been established:

i. 2 types of structural building FE models subjected to 3 external blast loading scenarios (global effects)
ii. 5 column FE models configurations subjected to a single blast loading scenario (local effects)

The types of structures for the buildings chosen for the FE Model are a Reinforced Concrete (RC) frame and a steel frame due to the wide use of the common building materials. The buildings consist of 4 storeys (or 3 storeys plus ground floor) with two bays of equal span in both the x and z direction. The floor slabs are constructed of RC however; the slab reinforcement is not modelled in detail. The focus is to analyse the critical frame members (columns and beams) whose failure may contribute to total or partial collapse and identify stress and deformation patterns to develop trends as to global and local effects.

The 3 blast scenarios selected are based on extremist terrorist's threats of varying charge size and distance from building depending on the nature of the explosive delivery. The credible threats were derived from the similar incidents likened to the Brussels suicide bombers in Belgium 2016, car bombing in Oslo Norway 2011 and the Oklahoma bombing in the US 1995.

The critical structural member analysis is intended to be carried out by applying a single blast load case from the above scenarios to various FE model configurations of structural columns. The analysis is intended highlight local blast effects and potential for optimisation in order to improve blast performance.

Procedure for methodology

The procedure to analyse global blast effects on structures is as follows:

a. Establish the geometry of the building structure

b. Establish 3 separate external surface blast scenarios based on:

    i. High explosive charge weight of equivalent TNT

    ii. Standoff distance from the building

c. Determine the blast loading using empirical methods contained in UFC 3-340 based on charge weight, standoff distance and geometry of structure

d. Establish FE structural model for RC and steel frame building

e. Establish material properties for FE model

f. Apply blast loading cases to FE Model

g. Conduct transient non-linear dynamic analysis Strand7

The procedure to analyse local blast effects on critical structural elements is as follows:

a. Utilise a single external surface blast scenario and blast loading case established above

b. Establish FE Model for the various column configurations

c. Establish material properties for FE model

d. Applying blast loading cases to FE Model

e. Conduct transient non-linear dynamic analysis using Strand7

Global blast effects on structures

Structural building geometry for global effects model

Table 3-1 contains a summary of the building dimension considered in the global effects study. Figure 3-1 and Figure 3-2 contain the elevation and plan layout for the model.

Table 3-1. Building Structural Model Dimensions

Building Structure Dimensions		
Overall Height	12	m
Floor Height	3	m
Length of Front, Side and Rear Walls	20	m
Column Spacing	10	m

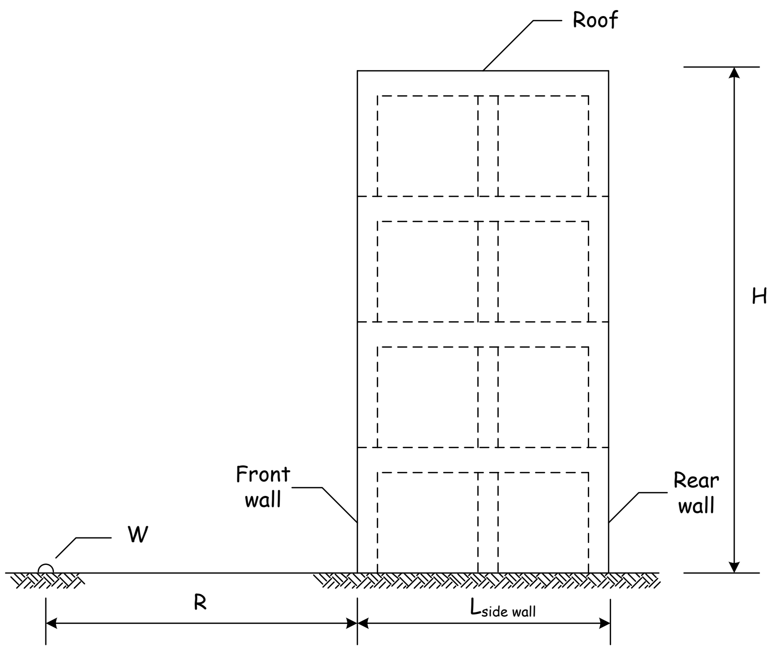

Figure 3-1. Model Structure Subjected to Blast Action (elevation)

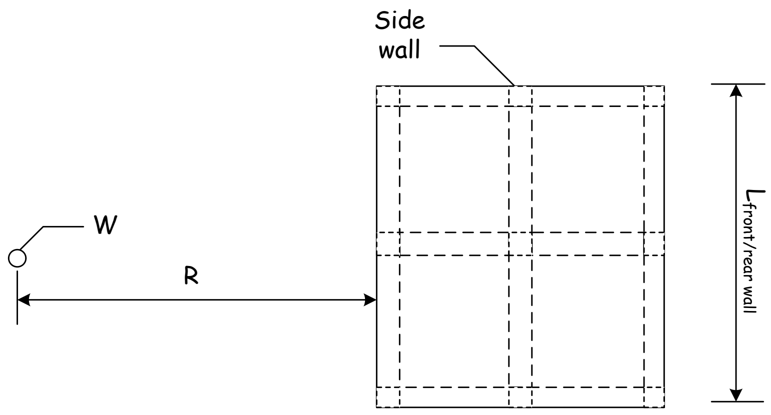

Figure 3-2. Model Structure Subjected to Blast Action (plan)

Blast scenarios for structural building case study

Taking reference from Figure 2-9 three blast threats have been chosen:

Scenario 1 - Explosive device carried by personal delivered by commercial luggage

Scenario 2 - Car bomb planted near building main entrance

Scenario 3 - Van bomb planted near building main entrance

Equivalent charge weights have been estimated based on the above threats which are governed by the equation:

$$W_e = W_{exp} \frac{H^d_{exp}}{H^d_{TNT}} \quad (10)$$

In order to simplify the problem explosive charge weights were all given in TNT therefore there is no need to factor for TNT equivalence.

Respective explosive charge weights (W) associated with scenarios:

Scenario 1 - Personnel borne - 100lbs (45.36kg)

Scenario 2 - Car bomb - 700lbs (317.5kg)

Scenario 3 - Van bomb - 4000lbs (1814.4kg)

Relative distance from blast source to target R:

Scenario 1 - Personal borne – 35ft (10.7m)

Scenario 2 - Car bomb – 90ft (27.4m)

Scenario 3 - Van bomb – 90ft (27.4m)

Note that R is affected by relative height at which the blast impacts the target. This will also affect the angle of incidence and consequently the reflected pressures values. Therefore, as the blast travel upwards, R becomes greater and the angle of incidence increases.

Where R_G = ground distance to target

$$R = \sqrt{{R_G}^2 + H^2} \quad (11)$$

Angle of incidence can be determined from

$$\alpha = \tan^{-1}(H/R_G) \quad (12)$$

UFC 3-340 suggests applying a minimum 20% safety factor to the charge weight (1.2*W):

Scenario 1 - Personal borne – 100 lbs (45.4kg)

Scenario 2 - Car bomb – 600 lbs (272.2kg)

Scenario 3 - Van bomb – 4000lbs (1814.4kg)

Determine scaled distance $Z = R/W^{1/3}$. For the base of the front wall of structure:

Scenario 1 - Personal borne Z = 4.93ft/lb$^{1/3}$ (2.81m/kg$^{1/3}$)

Scenario 2 - Car bomb Z = 9.44ft/lb$^{1/3}$ (3.78m/kg$^{1/3}$)

Scenario 3 - Van bomb Z = 5.34ft/lb$^{1/3}$ (2.12m/kg$^{1/3}$)

Prediction of surface blast loading using UFC 3-340 on structural model

The UFC 3-340 is to be used manual is US based, therefore all units contained within are imperial. The intent is to obtain the blast predications including blast pressures, based on imperial units for charge weight and dimensions (i.e. psi, lbs and ft) for the purposes of simplicity in deriving blast values. Once the blast loading cases have been established the values will be factored to SI units when applied to FE model for analysis.

UFC 3-340 procedural steps for blast pressure time curve determination

1. Determine the following critical blast parameters for free-field blast wave from Figure 2-11 for corresponding scaled ground distance *Z*:
   (a) *Peak incident pressure,* P_{so}
   (b) *Shock front velocity, U*
   (c) Scaled unit positive incident impulse $i_s/W^{1/3}$
   (d) Scaled positive phrase duration $t_o/W^{1/3}$
   (e) Scaled arrival time $ta/W^{1/3}$
   (f) Multiply scaled values by $W^{1/3}$ to obtain absolute values *is*, *to* and *ta*

<u>Positive loading front wall</u>

2. Determine front wall reflected and incident pressure and impulse:
   Reflected Pressure Coefficient Cr from Figure 3-3 for Pso α=0deg
   (a) $P_r = C_{r\alpha} \times P_{so}$
   (b) $i_r/W^{1/3}$ for $\mathrm{P_{so}}$ and $'\alpha'$ from Figure 3-4
   (c) $i_r = i_r/W^{1/3}\ (W^{1/3})$
   (d) $i_s = i_s/W^{1/3}\ (W^{1/3})$

3. Determine velocity of sound Cr in reflected overpressure region for Pso from Figure 3-5

4. Determine clearing time tc
   (a) $t_c = 4S/(1+R)C_R$
   (b) $S = height\ of\ front\ wall\ or\ one\ half\ its\ width, whichever\ is\ smaller$
   (c) $R = S/G$
   (d) $G = height\ of\ front\ wall\ or\ one\ half\ its\ widthwhichever\ is\ greater$

5. Calculate positive phase duration
$$t_{of} = \frac{2i_s}{P_{so}}$$

6. Determine peak dynamic pressure qo from Figure 3-6 for Pso

7. Calculate peak pressure acting on the front wall after the clearing time
   $P_{so} + C_D q_o$ from CD = 1 for front walls

8. Calculate duration of the reflected pressure
$$t_{rf} = \frac{2i_{r\alpha}}{P_{r\alpha}}$$

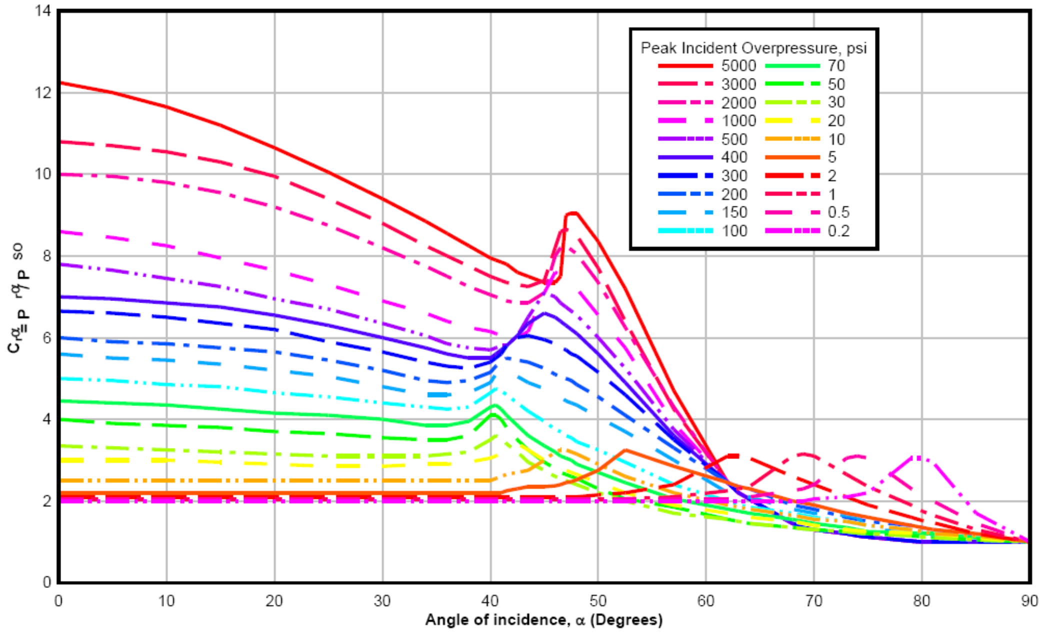

Figure 3-3. Reflected Pressure Coefficient versus Angle of Incidence

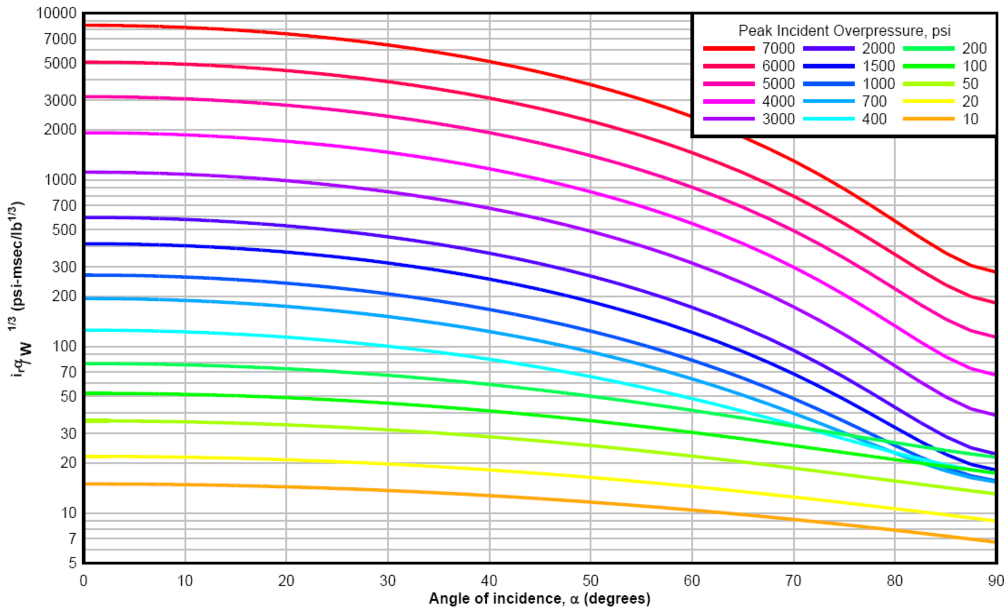

Figure 3-4. Reflected Scaled Impulse versus Angle of Incidence

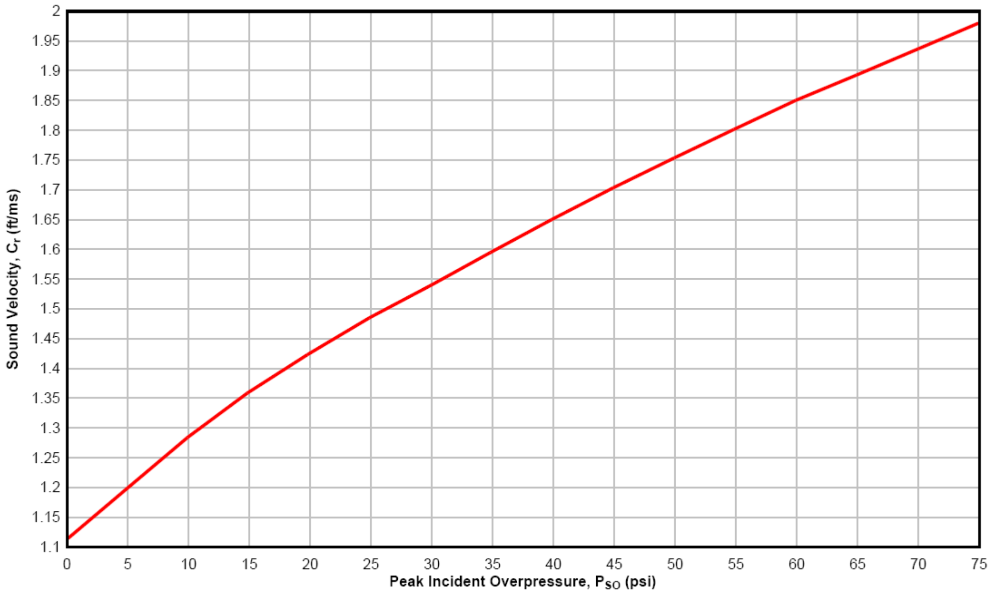

Figure 3-5. Velocity of Sound in Reflected Overpressure Region vs Peak Incident Overpressure

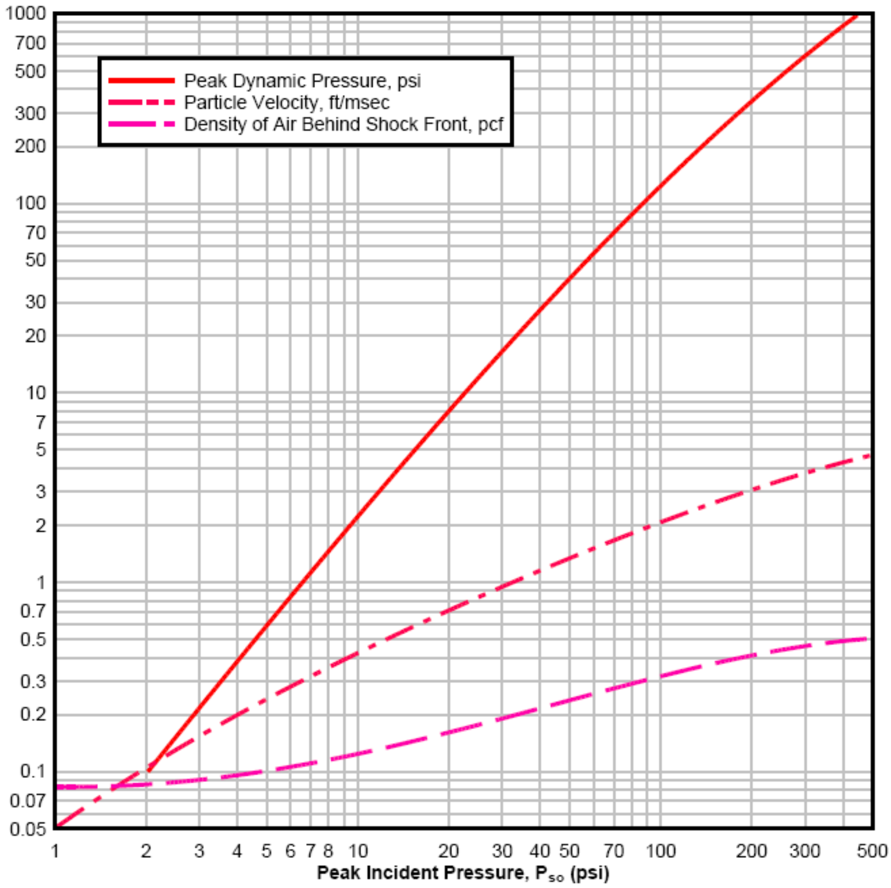

Figure 3-6. Peak Incident Pressure vs Peak Dynamic Pressure, Density of Air Behind the Shock Front, and Particle Velocity

<u>Negative loading front wall</u>

9. Determine scaled distance Z for Prα and $i_r/W^{1/3}$from Figure 2-11.
10. Using Z values determine peak pressure and impulse in negative phase from Figure 3-12.
    (a) P_{r^-}
    (b) $i_{r^-}/W^{1/3}$
    (c) $i_{r^-} = i_{r^-}/W^{1/3}(W^{1/3})$

11. Calculate negative phase duration trf-

$$t_{rf}^- = \frac{2i_{r\alpha}^-}{P_{r\alpha}}$$

12. Calculate negative phase rise time
    (a) $0.25 \times t_{rf}^-$
    (b) $t_o + 0.25t_{rf}^-$

13. Construct front wall pressure time curve

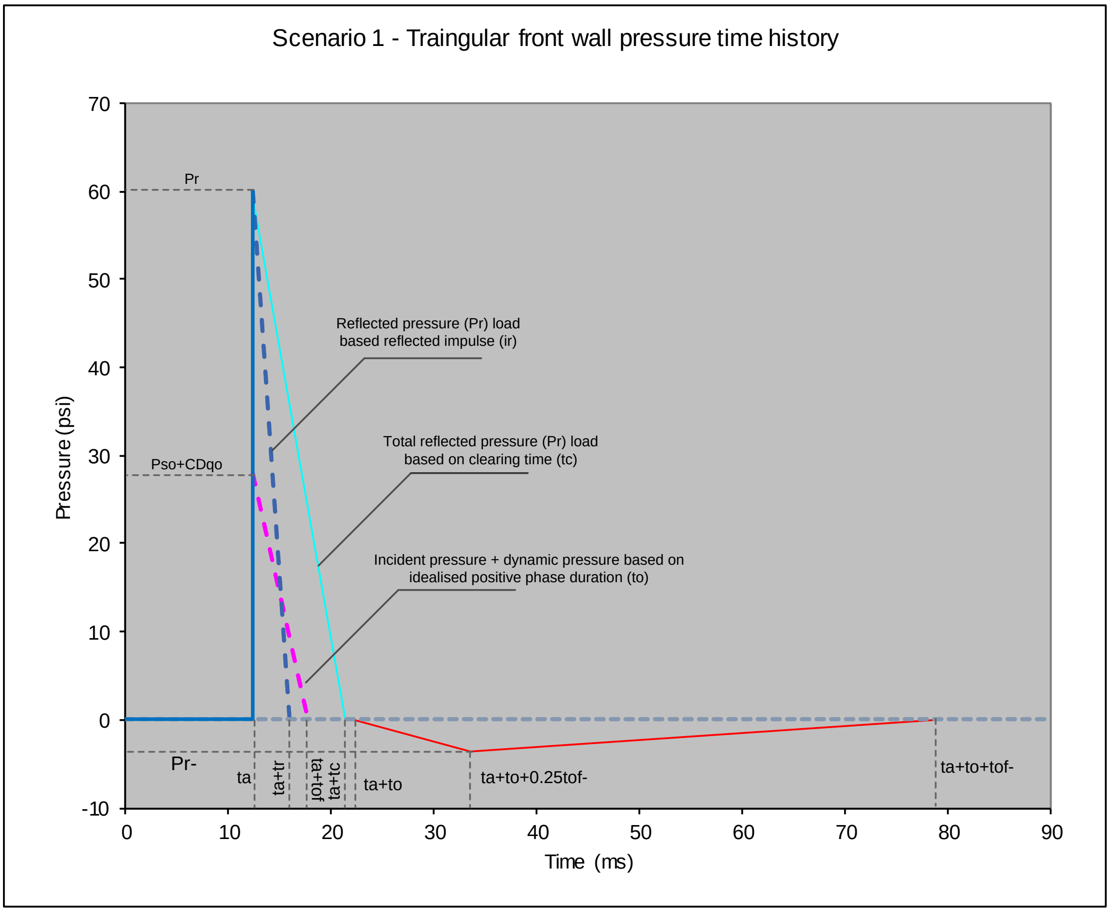

Figure 3-7. Blast pressure time history at front wall

In the case of the front wall blast pressure time history contained in Figure 3-7, there are 3 triangular pressure time histories contained in the positive phase that need to be checked, these include the reflected pressure curve based on the reflected impulse, reflected pressure curve based on the required time to clear the front wall and the incident pressure plus dynamic pressure based on the idealised positive phase duration. The most extreme contour of these interacting positive phase plots become the final blast curve. In the case of all the scenarios the reflected pressure curve based on the required time to clear the front wall becomes the governing pressure time history and is used to construct the blast loading histories as seen in Table 3-2 to Table 3-5 and Figure 3-8 to Figure 3-11.

Table 3-2. Scenario 1 to 3 summary of Blast pressure time history for front wall

	Front wall results									
Scenario	Pr	Pr-	Pso+Cdqo	ta	tr	tc	tof	to	tof-	0.25tof-
1	60.00	3.5	28	12.33	3.53	9.01	5.43	9.86	45.10	11.27
2	30.00	2.5	15	33.02	11.32	22.94	13.37	23.59	113.22	28.31
3	152.00	5	70	21.93	6.66	18.35	13.49	25.30	202.42	50.61

Table 3-3. Scenario 1 Blast pressure time history at front wall

Scenario 1	Front Wall		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	12.33	0	Pressure front arrival
ta	12.33	60.00	Pressure front of reflected pressure, Pr
ta + tc	21.33	0	End of positive Phase
ta+to	22.20	0	Start of Negative Phase
ta+to+0.25tof-	33.47	-3.5	Peak negative reflected pressure, Pr-
ta+to+tof-	67.29	0	End of blast loading

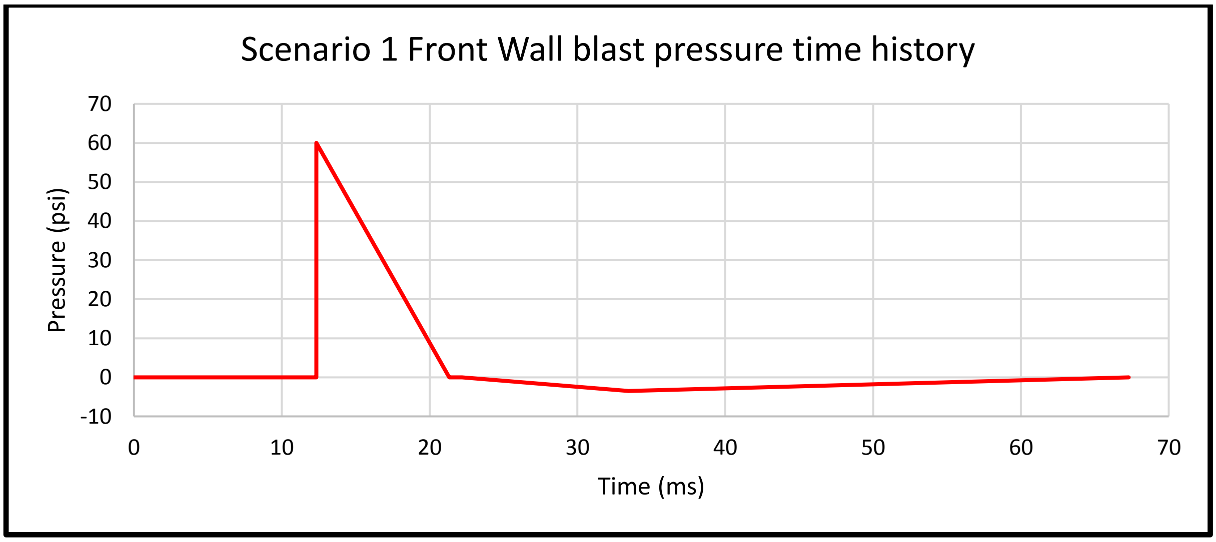

Figure 3-8. Scenario 1 Blast pressure time history at front wall

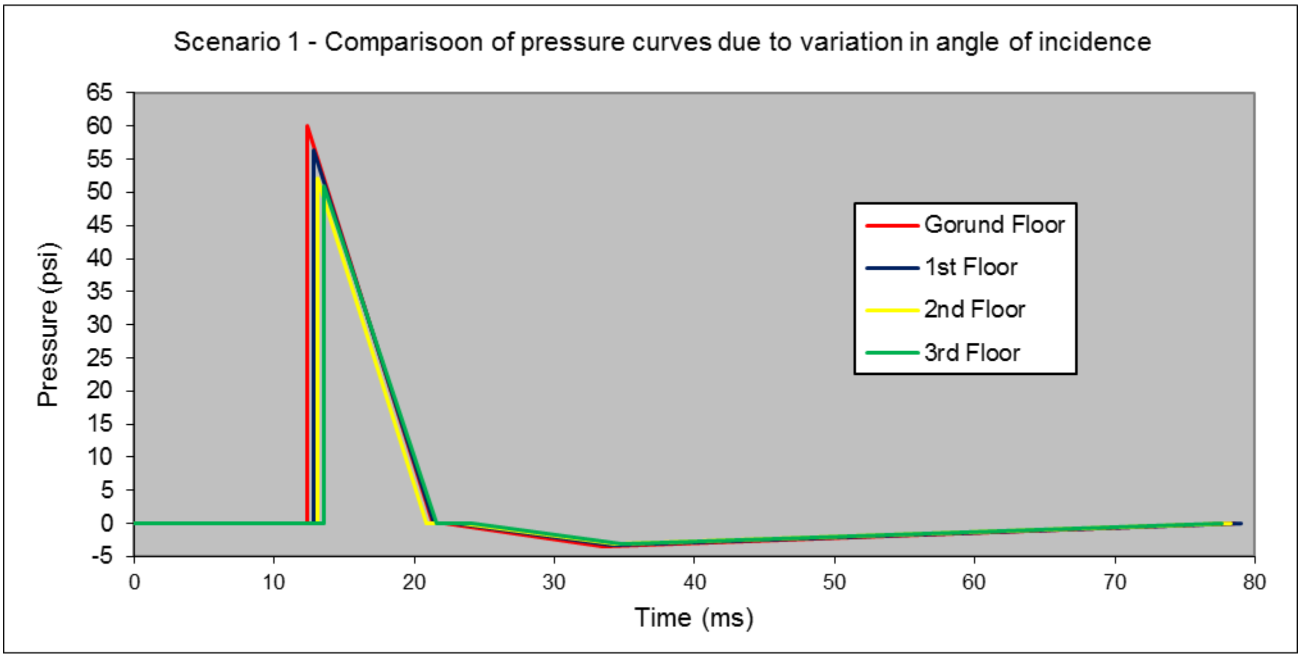

Figure 3-9. Scenario 1 Comparison of Pressure Time Histories Due to the Variation in Incidence Angle

Due to the variation in angle of incidence as the blast wave propagates upwards toward the higher levels of the structure, it reduces the pressures loading on the structure. This is due to the fact the standoff distance becomes greater and the angle of incidence increases. A comparison of the various pressure time histories for the front wall at each floor level is shown in Figure 3-9. As expected the Figure shows an increase in arrival time and decrease of incident pressures and phase durations as the blast travels upwards against the structure. The variation in peak pressures and phase durations fall within a 10% range for the chosen structural model. Due to the small scale of the model structure and the small variation in blast pressure time loadings at each floor levels. A single blast pressure time history with be used based on worst case for each scenario at the ground floor being applied to each structure surface front, side and rear walls.

Table 3-4. Scenario 2 Blast pressure time history at front wall

Scenario 2	Front Wall		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	33.02	0	Pressure front arrival
ta	33.02	30.00	Pressure front of reflected pressure, Pr
ta + tc	55.97	0	End of positive Phase
ta+to	56.61	0	Start of Negative Phase
ta+to+0.25tof-	84.92	-2.5	Peak negative reflected pressure, Pr-
ta+to+tof-	169.83	0	End of blast loading

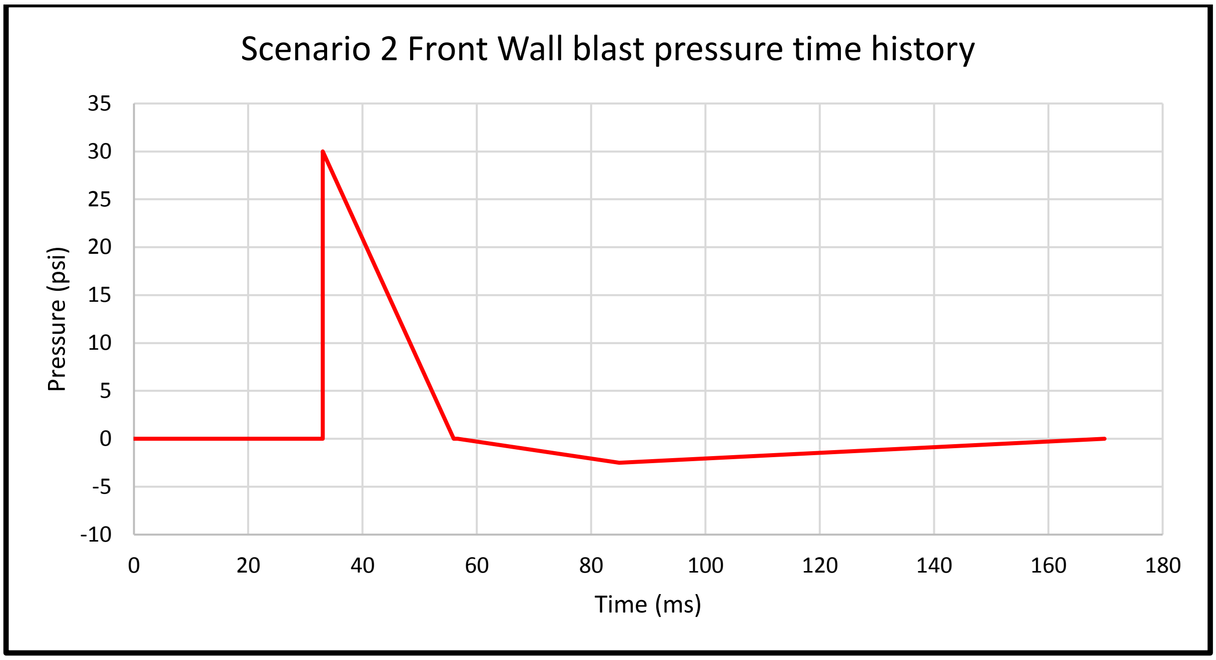

Figure 3-10. Scenario 2 Blast pressure time history at front wall

Table 3-5. Scenario 3 Blast pressure time history at front wall

Scenario 3	Front Wall		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	21.93	0	Pressure front arrival
ta	21.93	152.00	Pressure front of reflected pressure, Pr
ta + tc	40.28	0	End of positive Phase
ta+to	47.23	0	Start of Negative Phase
ta+to+0.25tof-	97.84	-5	Peak negative reflected pressure, Pr-
ta+to+tof-	249.66	0	End of blast loading

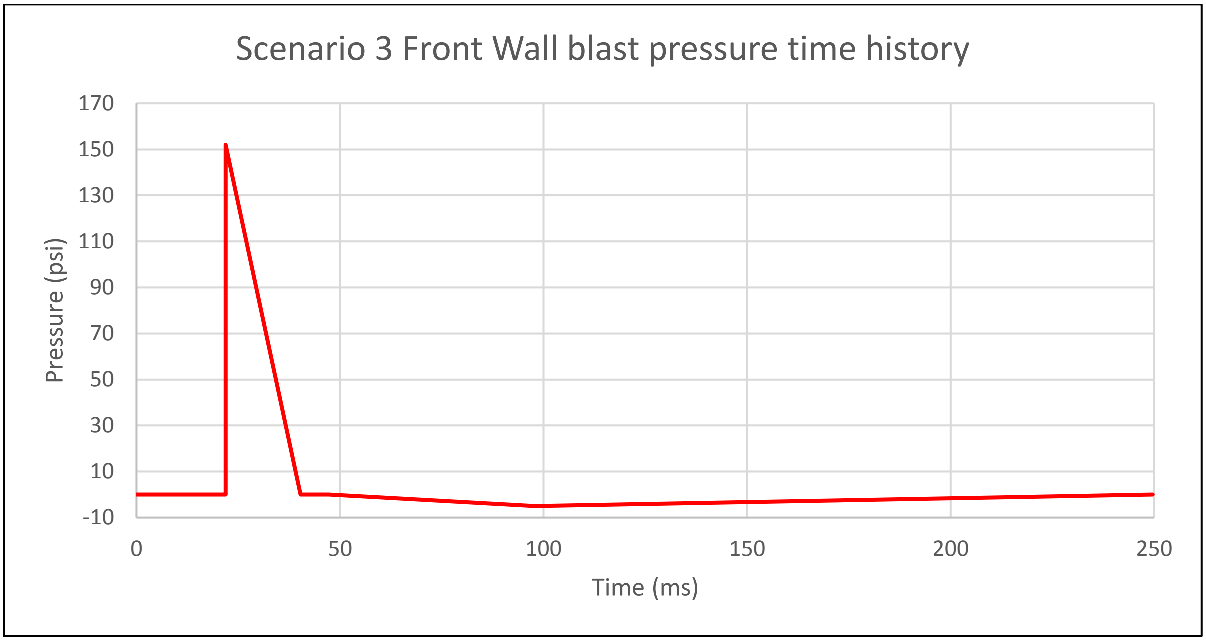

Figure 3-11. Scenario 3 Blast pressure time history at front wall

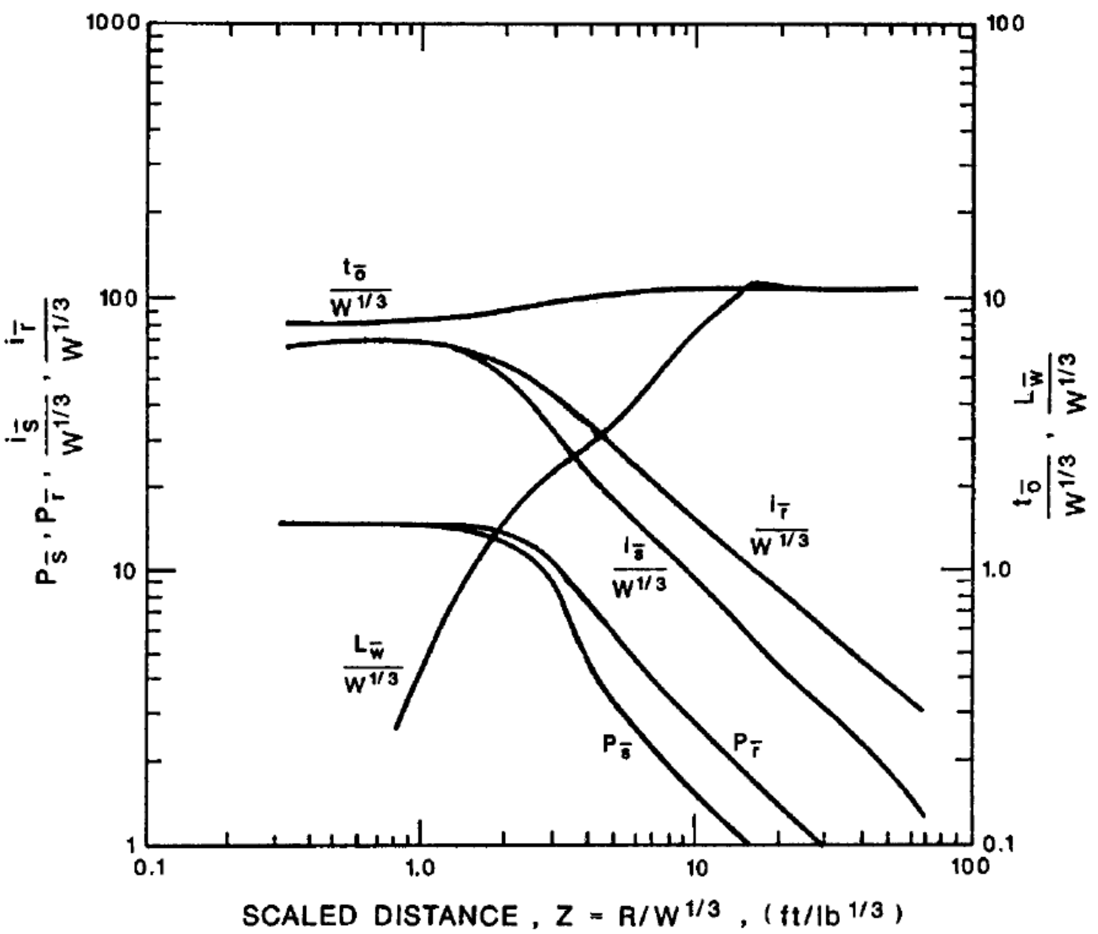

Figure 3-12. Negative Phase Shock Wave Parameters for a Spherical TNT Explosion in Free Air at Sea Level

Side Wall Loading Positive Phase

14. Calculate Lw/L ratio
    (a) $L = half\ length\ of\ side\ wall$
    (b) $L_{wf}/W^{1/3}$ from Z mid distance R along side wall using Figure 2-11
    (c) $L_{wf} = L_{wf}/W^{1/3}(W^{1/3})$
    (d) L_{wf}/L

15. Determine corresponding rise time td, positive phase duration tof and equivalent positive phase load factor
    (a) C_E from Figure 3-13
    (b) $t_d/W^{1/3}$ Figure 3-14
    (c) $t_{of}/W^{1/3}$ Figure 3-15

16. Calculate $C_E P_{sof}$, td and tof from step 15
    (a) $t_d = t_d/W^{1/3}(W^{1/3})$
    (b) $t_{of} = t_{of}/W^{1/3}(W^{1/3})$

17. Determine qo for C_EPsof from Figure 3-6

18. Calculate peak positive pressure PR is the sum of contribution of the equivalent uniform pressure and drag pressure
    (a) C_D =-0.4 for side walls
    (b) $P_R = C_E P_{sof} + C_D qo$

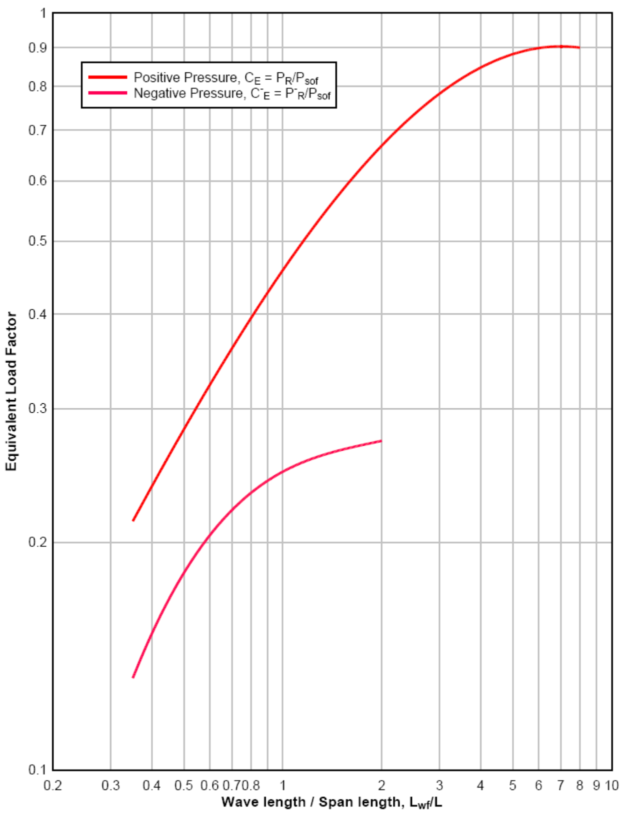

Figure 3-13. Peak Equivalent Uniform Roof Pressures

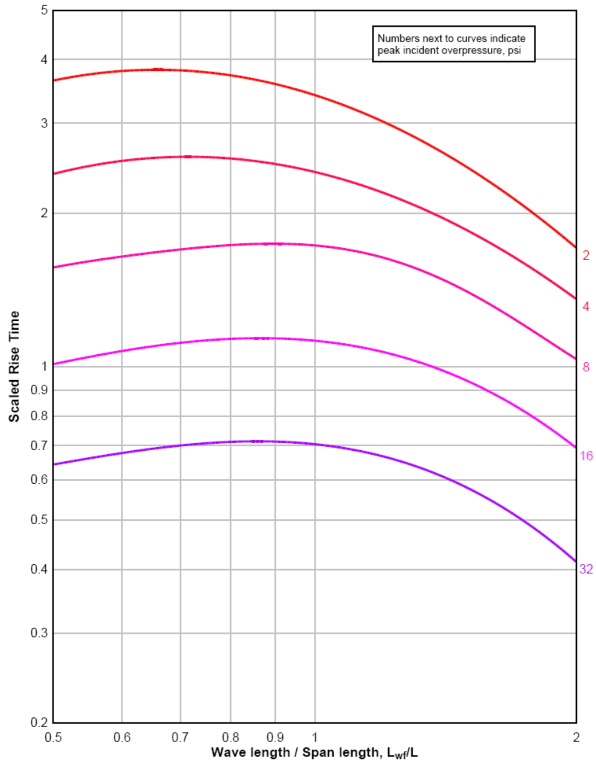

Figure 3-14. Scaled Rise Time of Equivalent Uniform Positive Roof Pressures

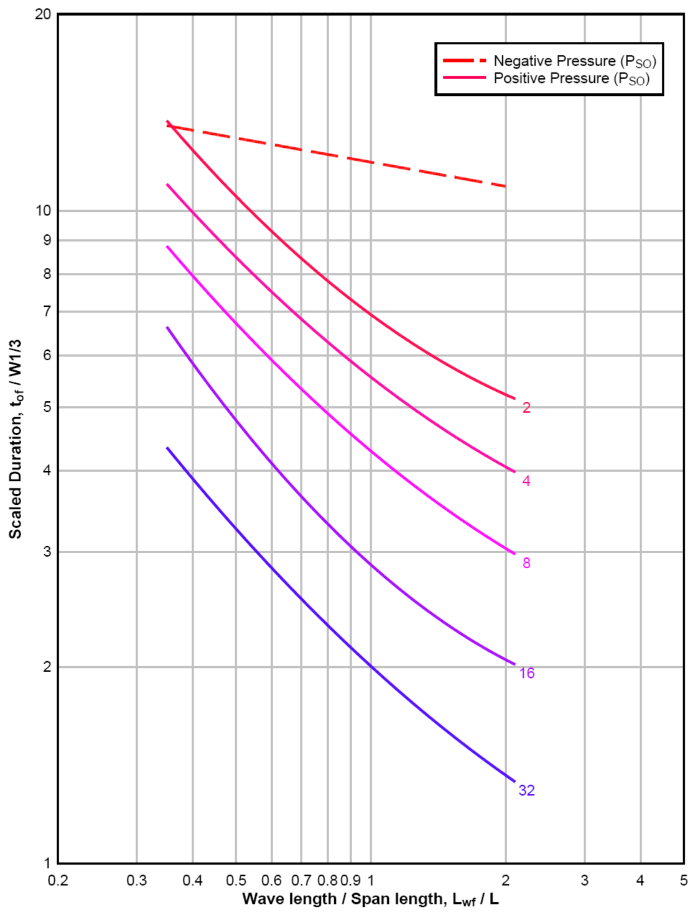

Figure 3-15. Scaled Duration of Equivalent Uniform Roof Pressures

<u>Side Wall Loading Negative Phase</u>

19. Determine corresponding equivalent negative phase load factor C_E-and scaled negative phase duration tof -/$W^{1/3}$ from Lw/L
    (a) C_E- from Figure 3-13
    (b) $t_{of}^{-}/W^{1/3}$ *from* Figure 3-15

20. Calculate Pr- and tof-
    (a) *Pr-=CE- x Psof*
    (b) *Tof-*$=t_{of}^{-}/W^{1/3}(W^{1/3})$

21. Calculate negative phase rise time
    (a) *0.25tof-*
    (b) *To+0.25tof*
    (c) *To+tof*

22. Construct side wall pressure time curve

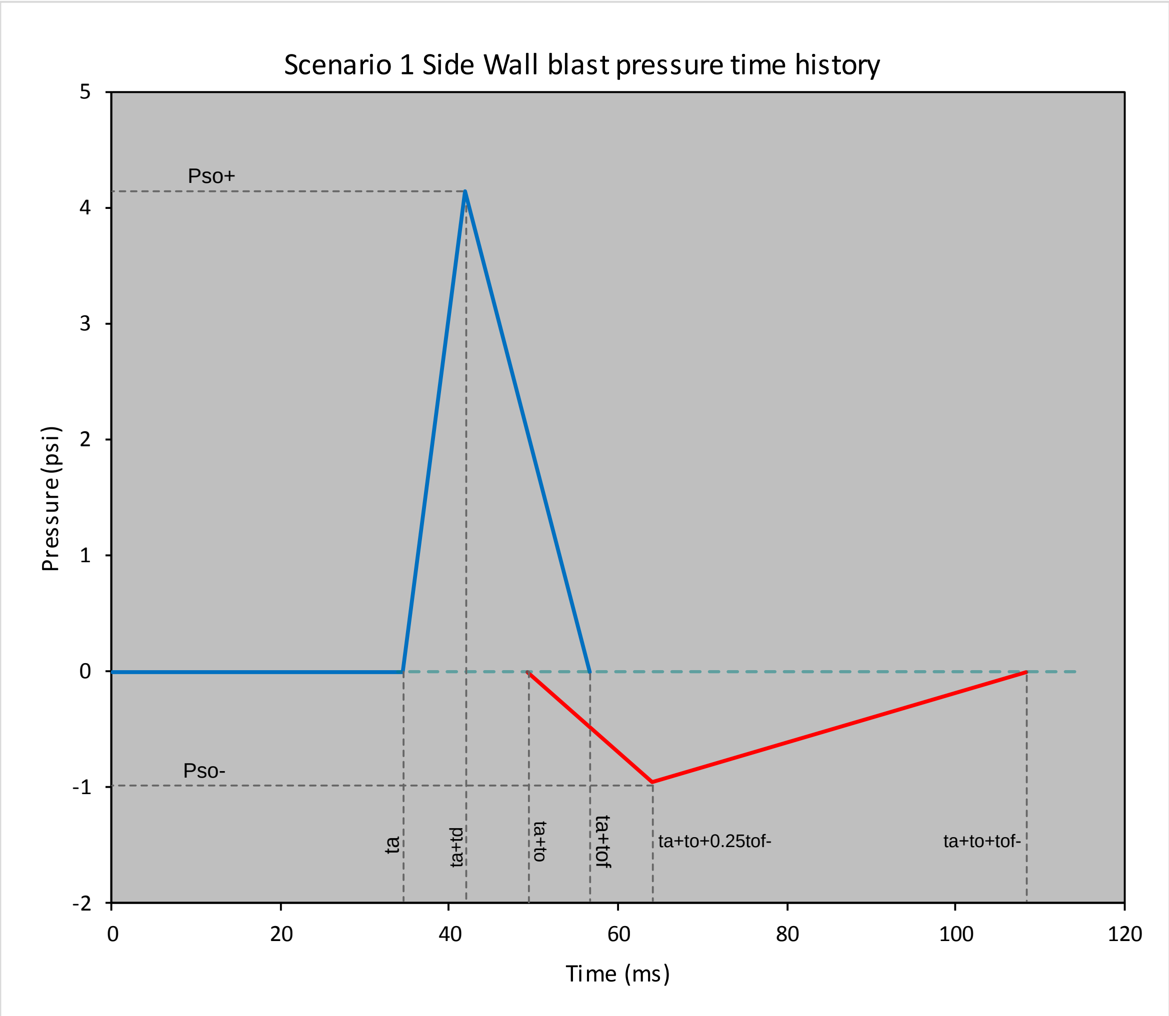

Figure 3-16. Scenario 1 Blast Pressure Time History at Side Wall

The triangular pressure time history in Figure 3-16 form applies to the remaining faces of the structure including the rear wall and roof due the absence of reflected pressure as the remaining surfaces ae parallel or behind the blast wave.

Table 3-6. Scenario 1 to 3 summary of Blast pressure time history for side walls

	Side wall results							
Scenarios	CE*Psof + Cdqo	Pr-= CE-*Psof	ta	td	tof	to	tof-	0.25tof-
1	4.14	0.96	34.53	7.40	22.20	14.80	59.19	14.80
2	5.56	1.84	47.18	8.49	23.59	24.53	37.74	9.44
3	5.80	5.00	42.17	15.18	42.17	33.74	219.29	54.82

Table 3-7. Scenario 1 Blast pressure time history at side wall

Scenario 1	Side Walls		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	34.53	0	Start of positive phase
ta+td	41.93	4.14	Peak positive incident pressure
ta+tof	56.72	0	End of postive Phase
ta+to	49.32	0	Start of Negative Phase
ta+to+0.25tof-	64.12	-0.96	Peak negative incident pressure
ta+to+tof-	108.51	0	End of blast loading

<u>Combined pressure loadings</u>

From Figure 3-16 there exists overlapping pressure time histories between the start of negative phase (ta +to) and end of positive phase (ta + tof). Essentially this presents a positive and negative phases working against each other. In order to construct a useable pressure time curve, it is required that the sums of the pressures between these phases be applied to result in a single given pressure attribute at any given time. Two linear equations of the positive and negative pressure lines are needed to solve:

$$P(ta + to) = \left(\frac{P_{so}}{(t_a+t_{of})-(t_a+t_d)}\right) \times \left((t_a + t_{of}) - (t_a + t_o)\right) \tag{13}$$

$$P(ta + tof) = \left(\frac{P_{so}-}{(t_a+t_o+0.25t_{of-})-(t_a+t_o)}\right) \times \left[\left((t_a + t_o + 0.25t_{of-}) - (t_a + t_o)\right) - \left((t_a + t_o + 0.25t_{of-}) - (t_a + t_{of})\right)\right] \tag{14}$$

From this point any condition where there exists a combined pressure loading case between phases, the above equations will be used to create new pressure readings.

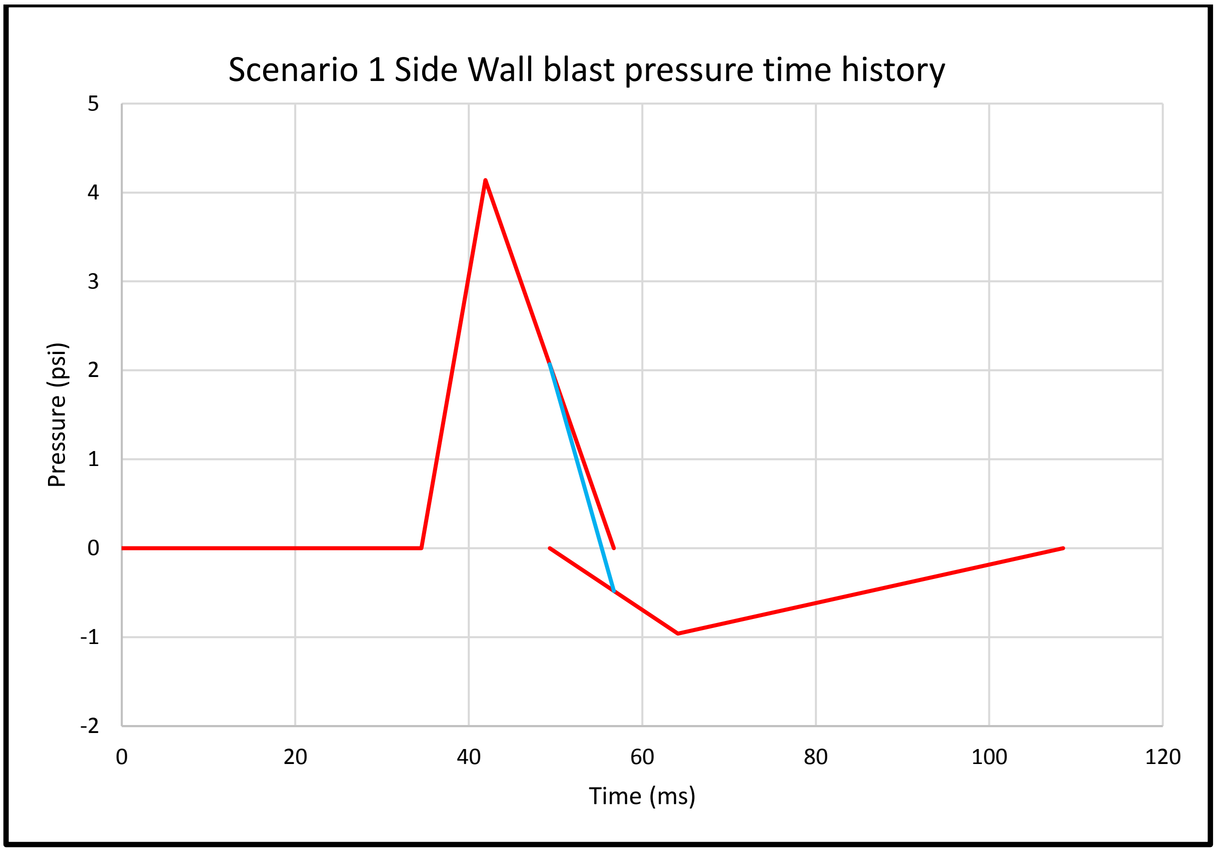

Figure 3-17. Scenario 1 Blast pressure time history at front wall

Table 3-8. Scenario 1 Blast pressure time history at side wall

Scenario 1	Side Walls		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	34.53	0	Start of positive phase
ta+td	41.93	4.14	Peak positive incident pressure
ta+tof	56.72	0	End of postive Phase
ta+to	49.32	0	Start of Negative Phase
ta+to+0.25tof-	64.12	-0.96	Peak negative incident pressure
ta+to+tof-	108.51	0	End of blast loading
	Combined pressure time history between phases		
ta+to	49.32	2.07	Equation (13)
ta+tof	56.72	-0.48	Equation (14)

Table 3-9. Scenario 2 Blast pressure time history at side wall

Scenario 2	Side Walls		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	47.18	0	Start of positive phase
ta+td	55.67	5.56	Peak positive incident pressure
ta+tof	70.76	0	End of postive Phase
ta+to	71.71	0	Start of Negative Phase
ta+to+0.25tof-	81.14	-1.84	Peak negative incident pressure
ta+to+tof-	109.45	0	End of blast loading

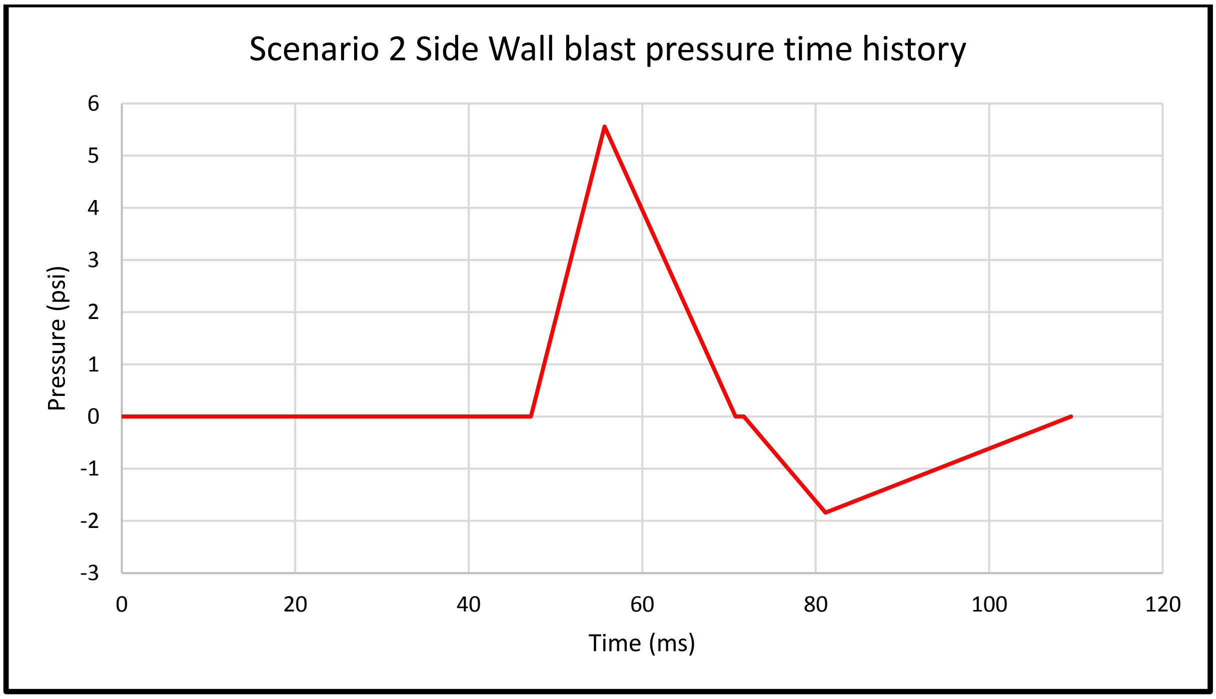

Figure 3-18. Scenario 2 Blast pressure time history at side wall

Table 3-10. Scenario 3 Blast pressure time history at side wall

Scenario 3	Side Walls		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	42.17	0	Start of positive phase
ta+td	57.35	5.80	Peak positive incident pressure
ta+tof	84.34	0	End of postive Phase
ta+to	75.91	0	Start of Negative Phase
ta+to+0.25tof-	130.73	-2.24	Peak negative incident pressure
ta+to+tof-	295.20	0	End of blast loading
	Combined pressure time history between phases		
ta+to	75.91	1.813	Equation (13)
ta+tof	84.34	-0.34	Equation (14)

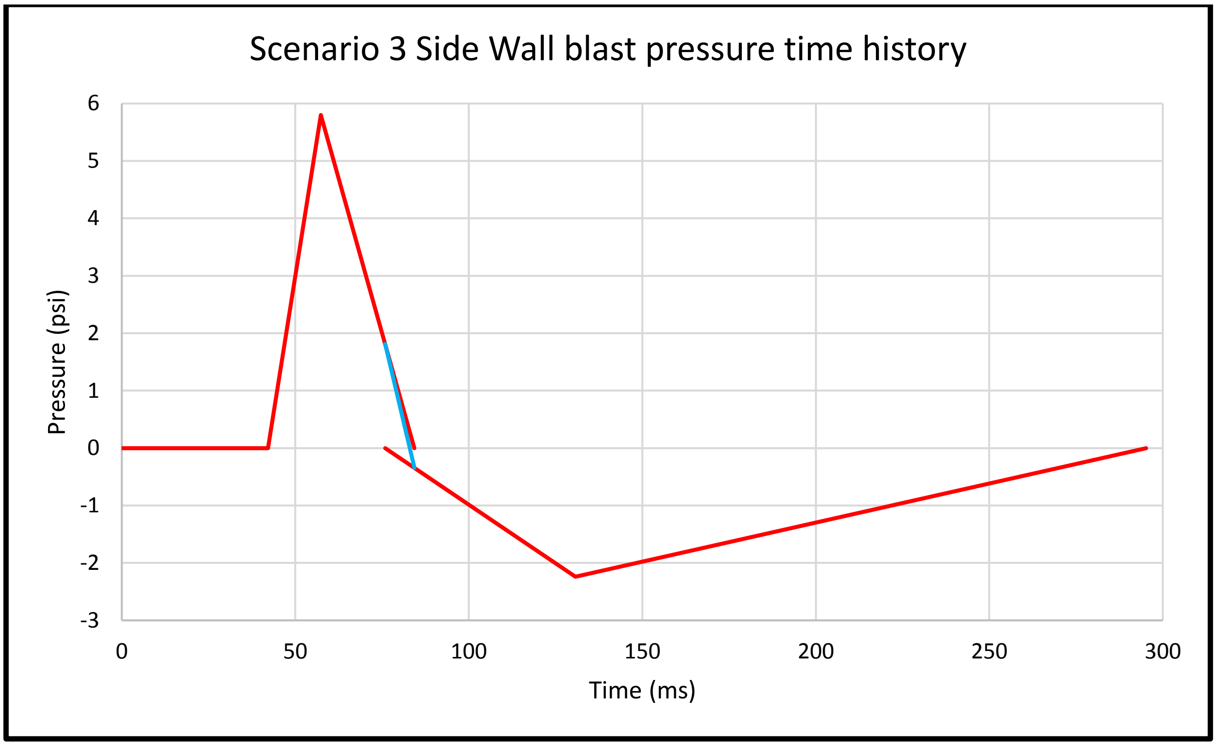

Figure 3-19. Scenario 3 Blast pressure time history at side wall

Roof Loading positive phase

23. Calculate Lwf/L ratio
    (a) L=length of roof
    (b) $L_{wf}/W^{1/3}$ from Z front edge R to wall using Figure 2-11
    (c) $L_{wf} = L_{wf}/W^{1/3}(W^{1/3})$
    (d) L_{wf}/L

24. Determine corresponding equivalent positive phase load factor CE, scaled rise time td/$W^{1/3}$ and scaled positive phase duration tof/$W^{1/3}$
    (a) C_E Figure 3-13
    (b) $t_d/W^{1/3}$ Figure 3-14
    (c) $t_{of}/W^{1/3}$ Figure 3-15

25. Calculate $C_E P_{sof}$, td and tof from step 24
    (a) $t_d = t_d/W^{1/3}(W^{1/3})$
    (b) $t_{of} = t_{of}/W^{1/3}(W^{1/3})$

26. Determine qo for C_EPsof from Figure 3-6

27. Calculate peak pressure
    (a) C_D =-0.4 for roof
    (b) $P_r = C_E Psof + C_D qof$

Roof Loading negative phase

28. Determine corresponding equivalent negative phase load factor C_E- and scaled negative phase duration tof -/W$^{1/3}$ from Lw/L
    (a) C_E- Figure 3-13 from Lw/L
    (b) $t_{of}^{-}/W^{1/3}$ Figure 3-15

29. Calculate Pr- and tof-
    (a) *Pr-=CE- x Psof*
    (b) *Tof-*$=t_{of}^{-}/W^{1/3}(W^{1/3})$

30. Calculate negative phase rise time
    (a) *0.25tof-*
    (b) *to*
    (c) *To+0.25tof*
    (d) *To+tof-*

31. Construct roof pressure time curve

Table 3-11. Scenario 1 to 3 summary of Blast pressure time history for roof

	Roof results							
Scenario	CE*Psof + Cdqo	Pr-= CE-*Psof	ta	td	tof	to	tof-	0.25tof-
1	10.58	0.69	36.99	9.37	19.73	9.86	69.05	17.26
2	1.52	1.09	56.61	16.04	33.02	23.59	132.09	17.26
3	4.20	2.80	33.74	13.49	50.61	25.30	253.03	63.26

Table 3-12. Scenario 1 Blast pressure time history on Roof

Scenario 1	Roof		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	36.99	0	Start of positive phase
ta+td	46.36	10.58	Peak positive incident pressure
ta+tof	56.72	0	End of postive Phase
ta+to	46.86	0	Start of Negative Phase
ta+to+0.25tof-	64.12	-0.69	Peak negative incident pressure
ta+to+tof-	115.91	0	End of blast loading
	Combined pressure time history between phases		
ta+to	46.86	10.08	Equation (13)
ta+tof	56.72	-0.39	Equation (14)

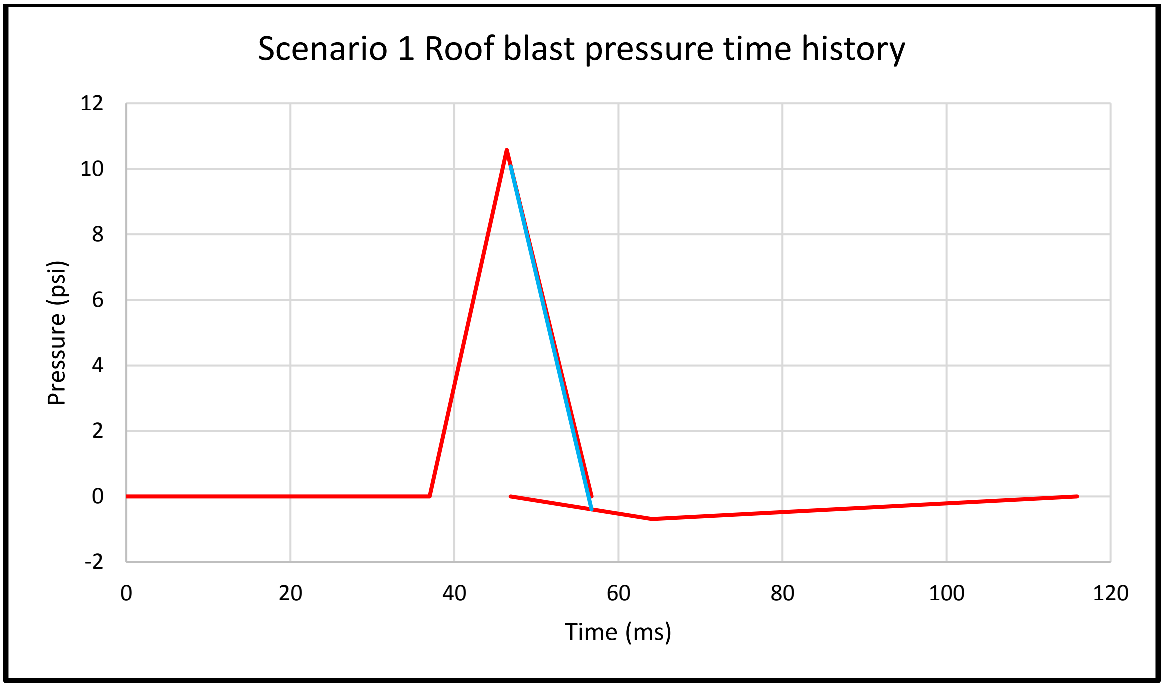

Figure 3-20. Scenario 1 Blast pressure time history for Roof

Table 3-13. Scenario 2 Blast pressure time history on Roof

Scenario 2	Roof		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	56.61	0	Start of positive phase
ta+td	72.65	1.52	Peak positive incident pressure
ta+tof	89.63	0	End of postive Phase
ta+to	80.20	0	Start of Negative Phase
ta+to+0.25tof-	97.46	-1.09	Peak negative incident pressure
ta+to+tof-	212.29	0	End of blast loading
	Combined pressure time history between phases		
ta+to	80.20	0.84	Equation (13)
ta+tof	89.63	-0.60	Equation (14)

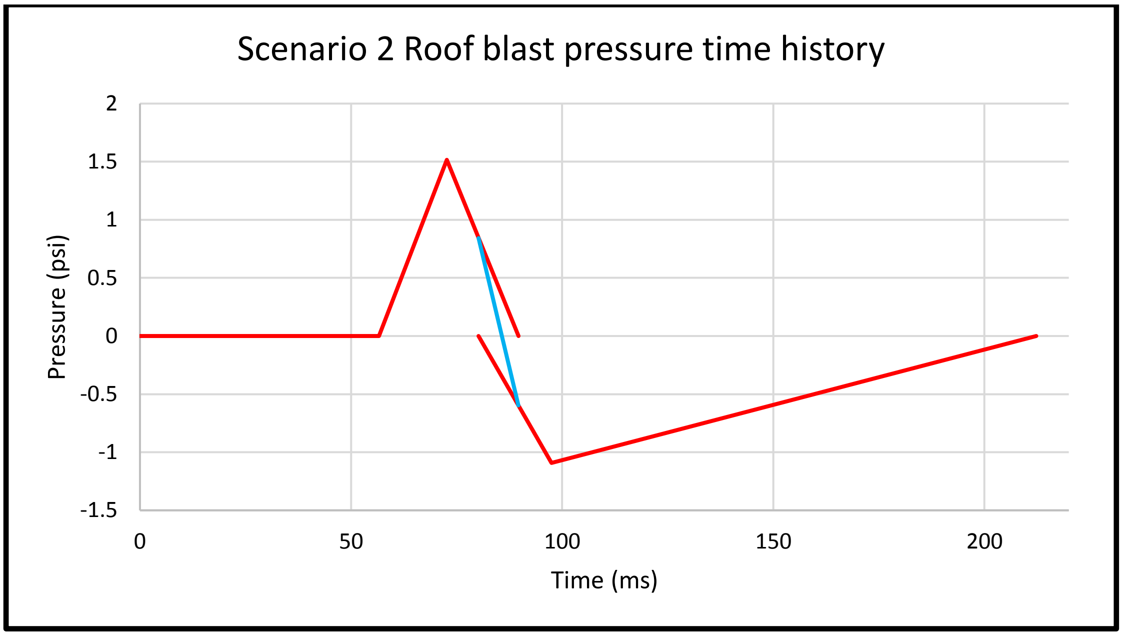

Figure 3-21. Scenario 2 Blast pressure time history for Roof

Table 3-14. Scenario 3 Blast pressure time history on Roof

Scenario 3	Roof		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	33.74	0	Start of positive phase
ta+td	47.23	4.20	Peak positive incident pressure
ta+tof	84.34	0	End of postive Phase
ta+to	59.04	0	Start of Negative Phase
ta+to+0.25tof-	122.30	-2.80	Peak negative incident pressure
ta+to+tof-	312.07	0	End of blast loading
	Combined pressure time history between phases		
ta+to	59.04	2.86	Equation (13)
ta+tof	84.34	-1.12	Equation (14)

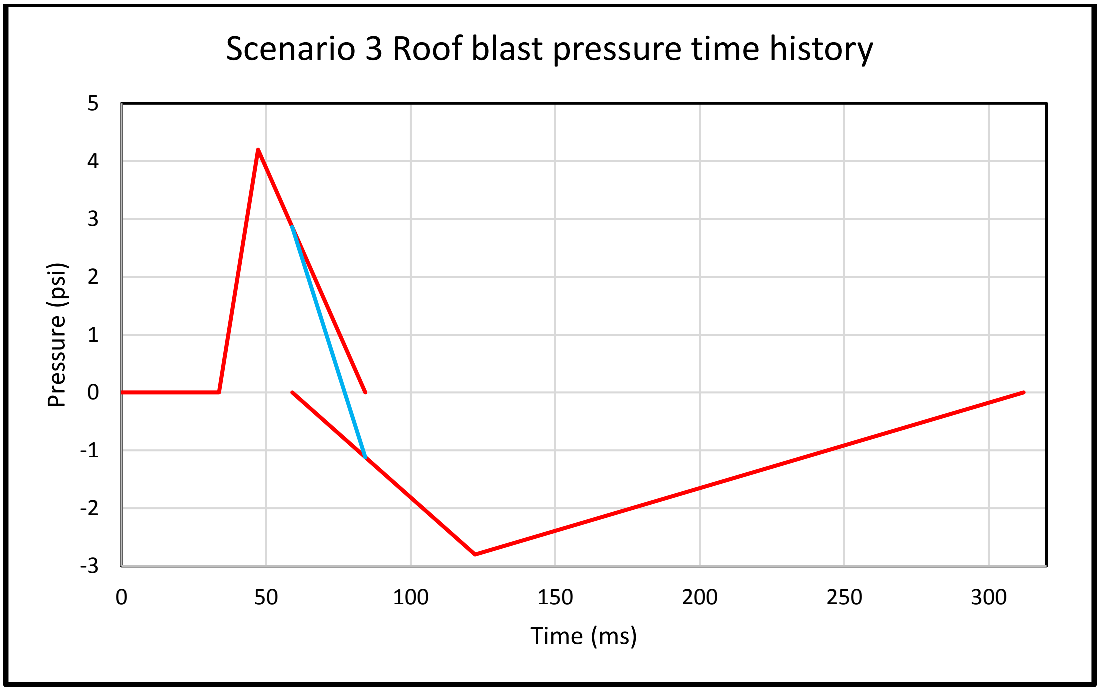

Figure 3-22. Scenario 3 Blast pressure time history for Roof

Rear wall Loading positive phase

32. Calculate Lwf/L ratio
    (a) L=height of structure
    (b) $L_{wf}/W^{1/3}$ for Z at distance R to rear wall using Figure 2-11
    (c) $L_{wf} = L_{wf}/W^{1/3}(W^{1/3})$
    (d) L_{wf}/L

33. Determine corresponding equivalent positive phase load factor CE, scaled rise time td/$W^{1/3}$ and scaled positive phase duration tof/$W^{1/3}$ for Lwf/L
    (a) C_E Figure 3-13
    (b) $t_d/W^{1/3}$ Figure 3-14
    (c) $t_{of}/W^{1/3}$ Figure 3-15

34. Calculate $C_E P_{sob}$, td and tof from step 33
    (a) $C_E P_{sob}$
    (b) $t_d = t_d/W^{1/3}(W^{1/3})$
    (c) $t_{of} = t_{of}/W^{1/3}(W^{1/3})$

35. Determine qo for C_EPsof from Figure 3-6

36. Calculate peak pressure
    (a) $C_D = -0.4$ *for rear walls*
    (b) $P_R = C_E Psof + C_D qof$

Rear wall Loading negative phase

37. Determine corresponding equivalent negative phase load factor CE- scaled negative phase duration tof -/$W^{1/3}$ from Lw/L
    (a) C_E- Figure 3-13
    (b) $t_{of}^{-}/W^{1/3}$ Figure 3-15

38. Calculate Pr- and tof-
    (a) $Pr\text{-}=CE\text{-}\ x\ Psof$
    (b) $tof\text{-}=t_{of}^{-}/W^{1/3}(W^{1/3})$

39. Calculate negative phase rise time
    (a) *0.25tof-*
    (b) *to*
    (c) *to+0.25tof*
    (d) *to+tof-*

40. Construct rear wall pressure time curve

Table 3-15. Scenario 1 to 3 summary of Blast pressure time history for rear wall

	Rear wall results							
Scenario	CE*Psof + Cdqo	Pr-= CE-*Psof	ta	td	tof	to	tof-	0.25tof-
1	1.70	-0.81	64.12	10.85	27.13	16.28	60.42	15.11
2	2.28	-0.84	113.22	16.04	40.10	33.02	113.22	28.31
3	6.60	-2.24	84.34	18.56	50.61	101.21	202.42	50.61

Table 3-16. Scenario 1 Blast pressure time history of Rear Wall

Scenario 1	Rear wall		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	64.12	0	Start of positive phase
ta+td	74.97	1.70	Peak positive incident pressure
ta+tof	91.25	0	End of postive Phase
ta+to	80.40	0	Start of Negative Phase
ta+to+0.25tof-	95.50	-0.81	Peak negative incident pressure
ta+to+tof-	140.82	0	End of blast loading
	Combined pressure time history between phases		
ta+to	80.40	1.13	Equation (13)
ta+tof	91.25	-0.58	Equation (14)

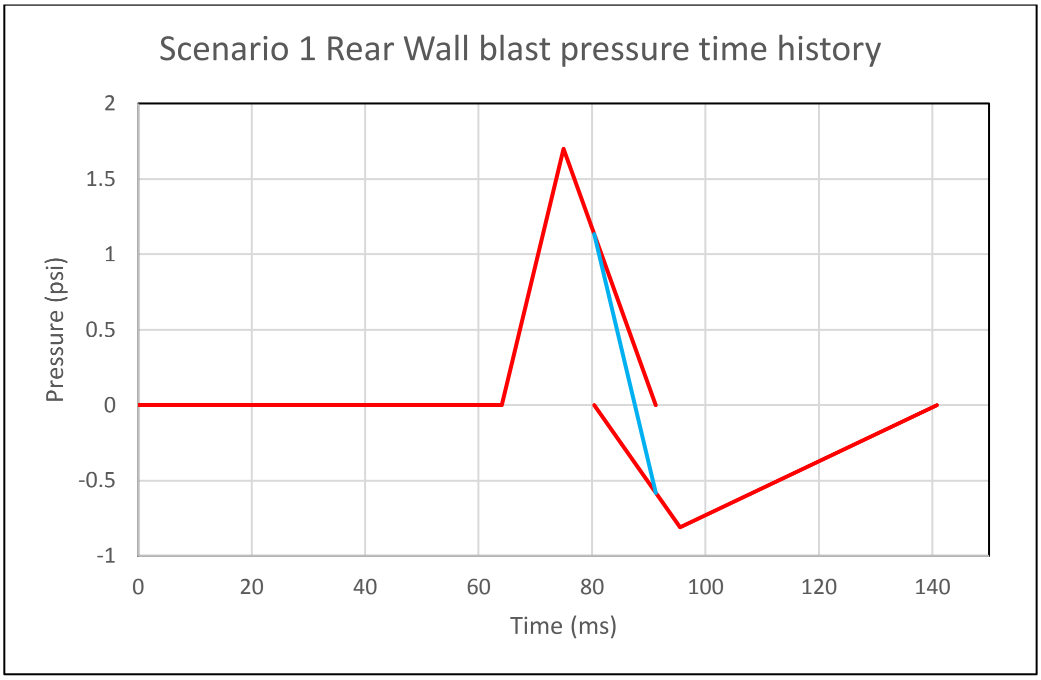

Figure 3-23. Scenario 1 Blast pressure time history of Rear Wall

Table 3-17. Scenario 2 Blast pressure time history of Rear Wall

Scenario 2	Rear wall		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	113.22	0	Start of positive phase
ta+td	129.26	2.28	Peak positive incident pressure
ta+tof	153.32	0	End of postive Phase
ta+to	146.25	0	Start of Negative Phase
ta+to+0.25tof-	174.55	-0.84	Peak negative incident pressure
ta+to+tof-	259.47	0	End of blast loading
	Combined pressure time history between phases		
ta+to	146.25	0.67	Equation (13)
ta+tof	153.32	-0.21	Equation (14)

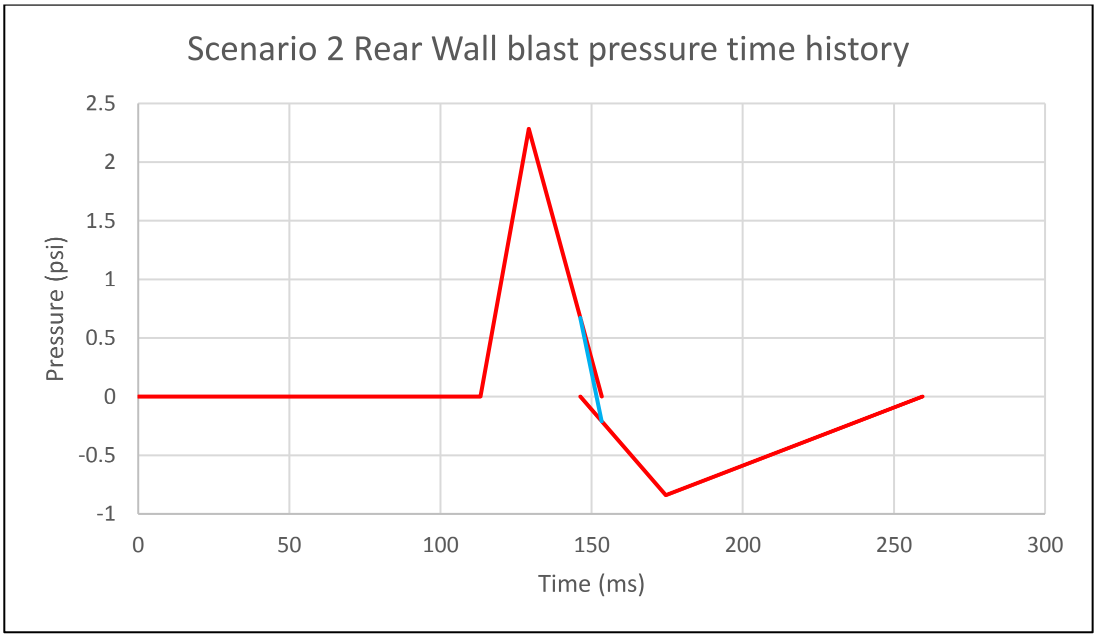

Figure 3-24. Scenario 2 Blast pressure time history of Rear Wall

Table 3-18. Scenario 3 Blast pressure time history of Rear Wall

Scenario 3	Rear wall		
Time description	Time (ms)	Pressure (psi)	Pressure description
	0	0	Initial Condition
ta	84.34	0	Start of positive phase
ta+td	102.90	6.60	Peak positive incident pressure
ta+tof	134.95	0	End of postive Phase
ta+to	185.56	0	Start of Negative Phase
ta+to+0.25tof-	236.16	-2.24	Peak negative incident pressure
ta+to+tof-	387.98	0	End of blast loading

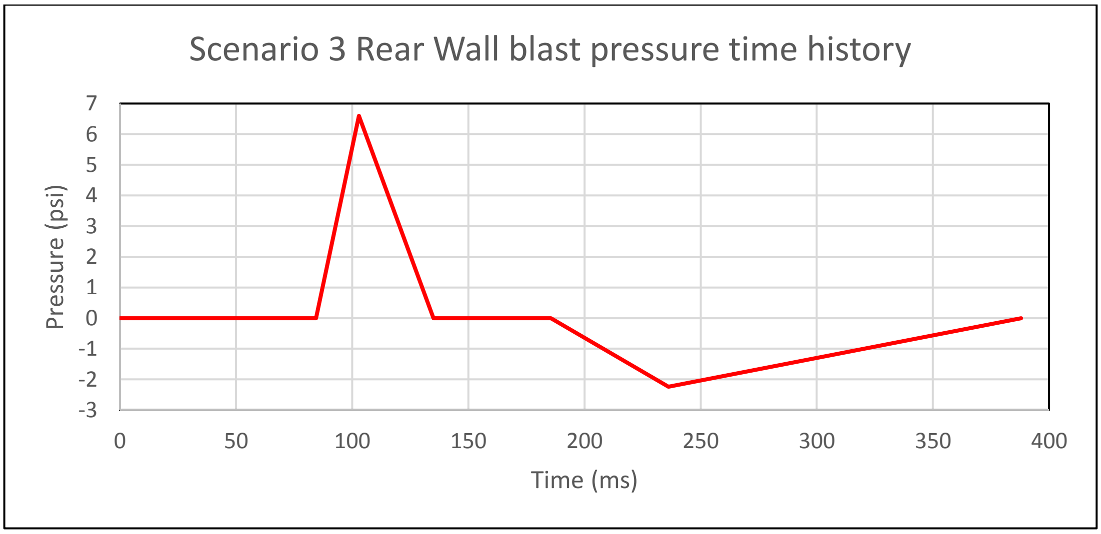

Figure 3-25. Scenario 3 Blast pressure time history of Rear Wall

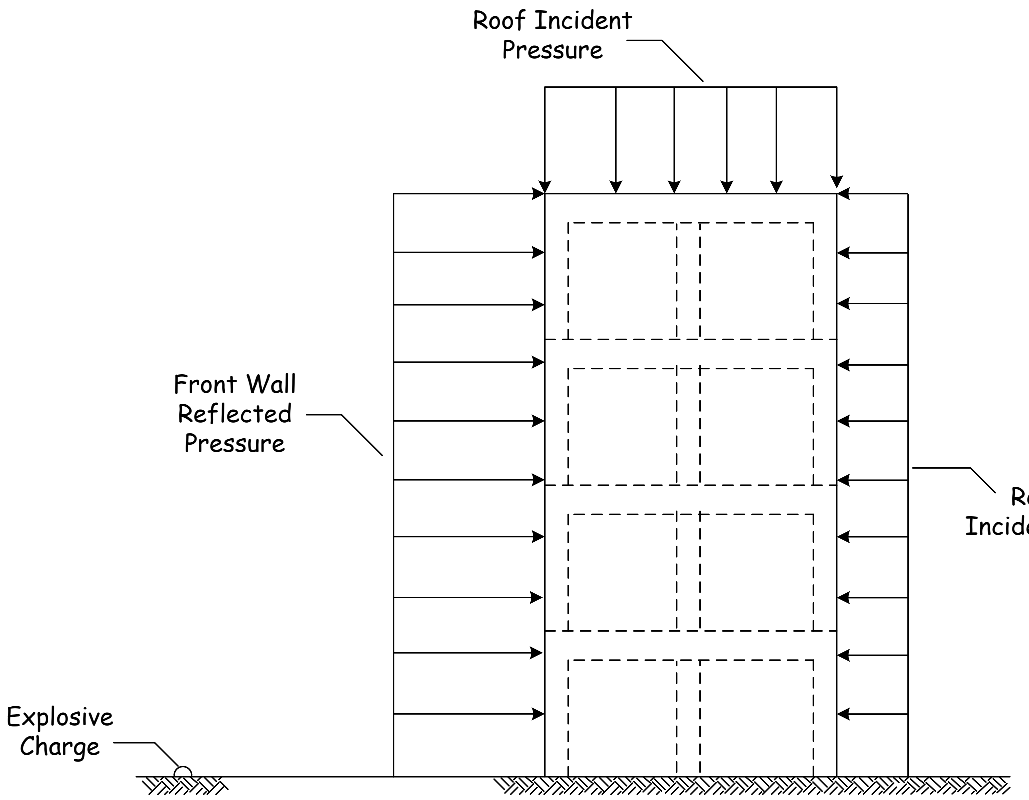

Figure 3-26. Elevation Diagram of Typical Structural Model Peak Blast Loading Interaction

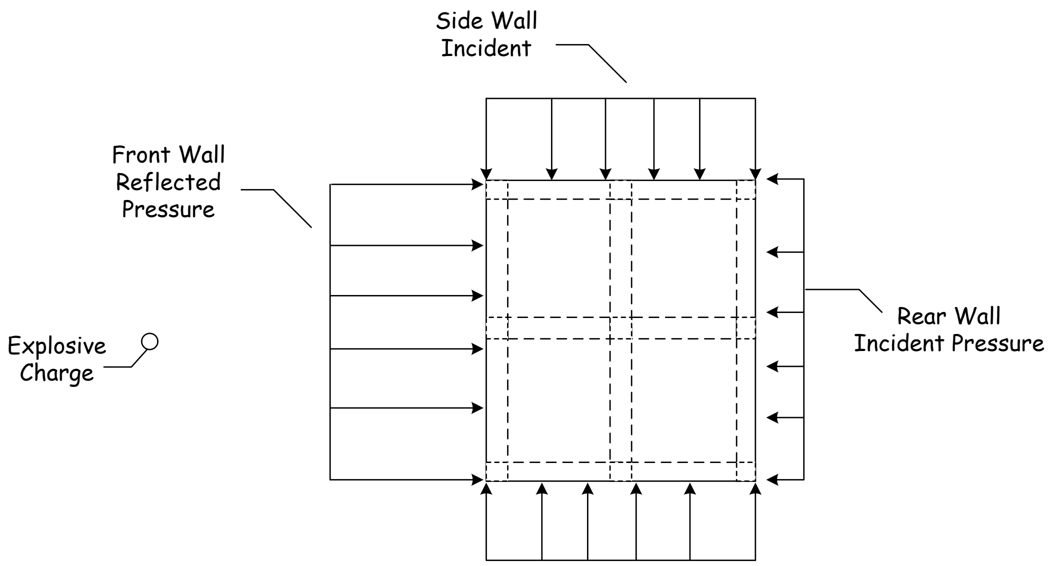

Figure 3-27. Plan Diagram of Typical Structural Model Peak Blast Loading Interaction

FE model structural models

Reinforced Concrete frame building structural elements

The 3D RC frame building model shown in Figure 3-28 depicts the 4 storey (or ground floor plus 3 storeys) separated in two bays in both the x and z directions. The FE model consists of brick and beam elements that make up the columns and beams and plate elements using a RC module to model the floors and roof.

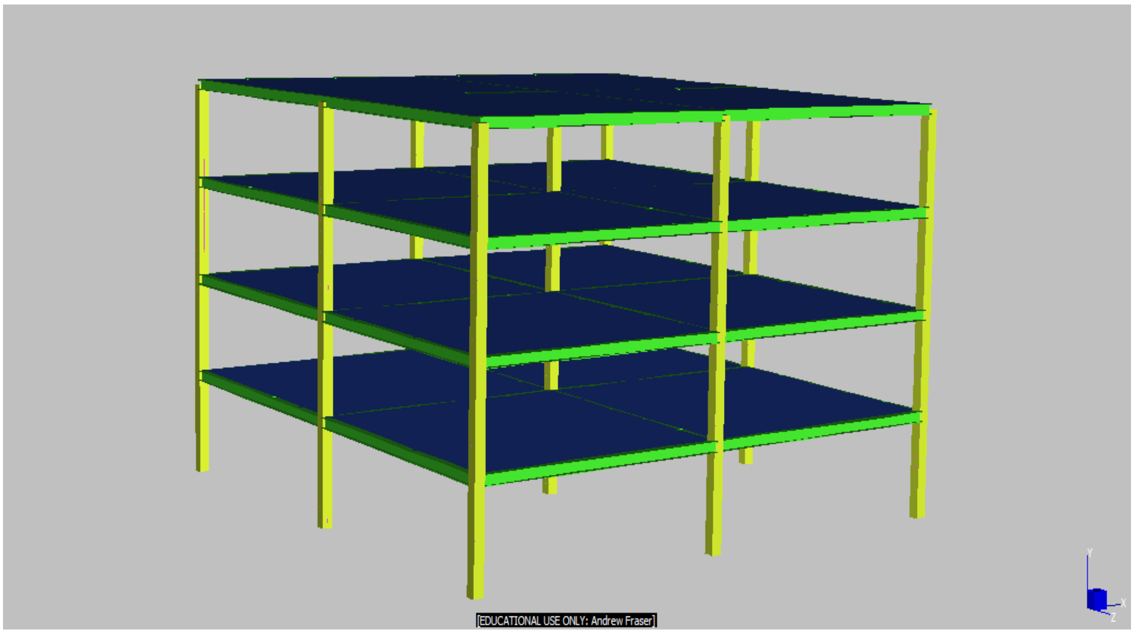

Figure 3-28. Reinforced Concrete Framed Building FE Structural Model

RC Column and beams

The RC columns and beams were modelled using a simple coarse brick and beam method, as shown in Figure 3-29. This method provides a relatively detailed approach where the concrete material is modelled as brick elements and the reinforcement is modelled as beams located at nodes, reinforcement detail is shown in Figure 3-30. Cell or brick size is therefore governed by the reinforcement location. The cell size of bricks at the element external faces were biased to allow the correct placement of reinforcement and in the centre provided a coarse cell core. This technique provides a more detailed model in order to study the global effects whilst still capable of modelling the interaction between concrete and reinforcement elements.

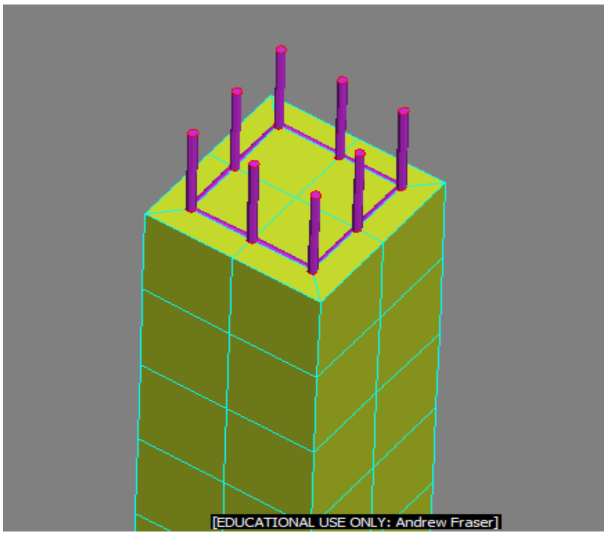

Figure 3-29. RC Column

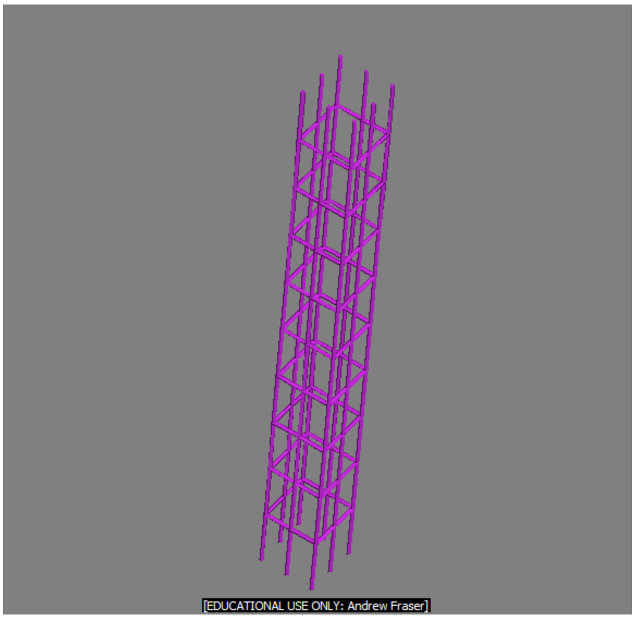

Figure 3-30. RC Column Reinforcement Detailing

RC floor Slab and roof RC plate module

Use of the Plate RC was used to create floors for the structural model. The module uses a smeared approach to analyse reinforced concrete structures of custom geometry and properties using plate elements, seen in Figure 3-31. The plate elements were subdivided proportionately to coincide with the supporting beams and columns to provide a more accurate results.

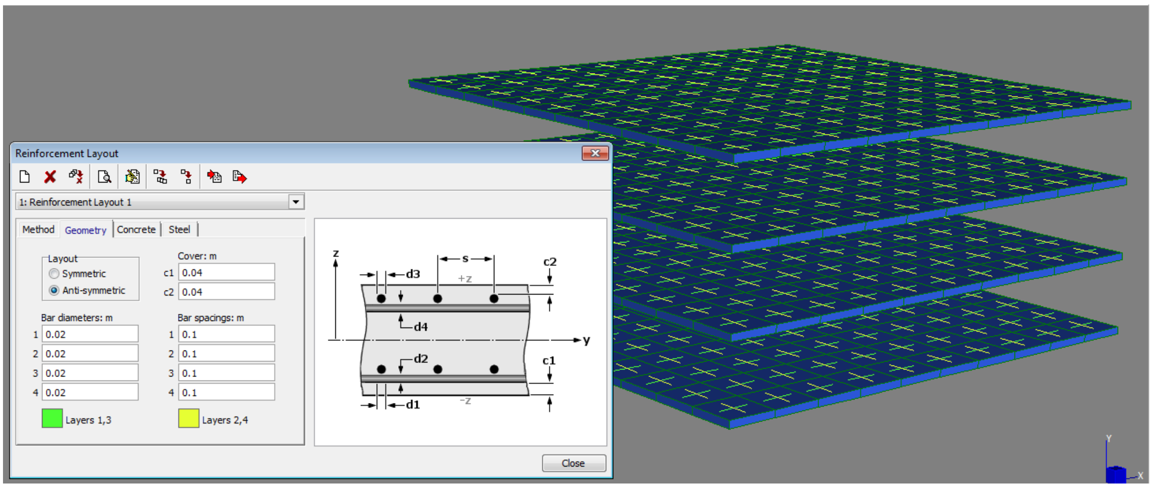

Figure 3-31. RC Floors and Roof

Foundation Restraints

In order to place limitations on the structure the columns at the ground level were contacting x-z plane were restrained at the nodes, as seen in Figure 3-32.

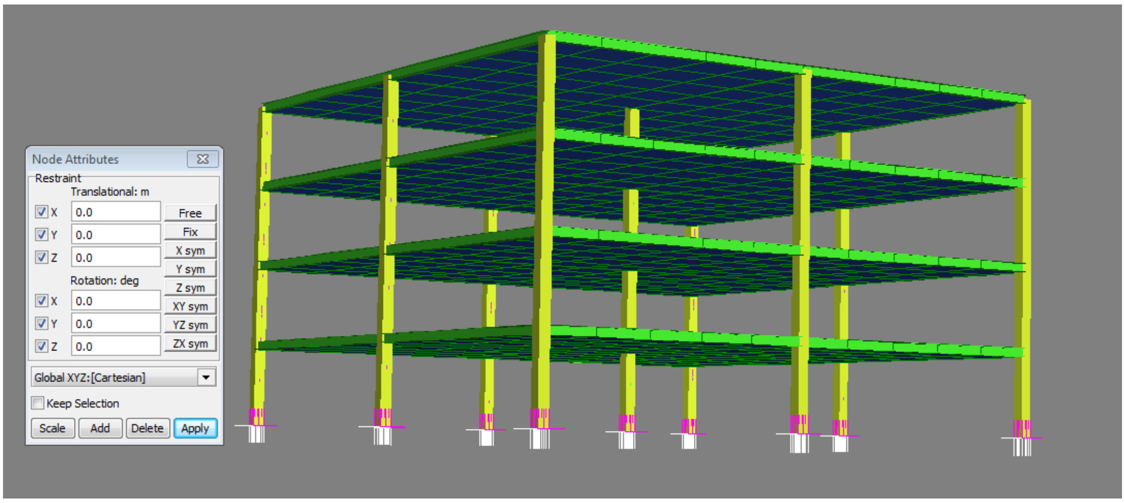

Figure 3-32. Building restraints

Steel frame building structural elements

The 3D RC frame building model shown in Figure 3-33 depicts the same building geometry above however the FE model consists beams with UC and UB steel properties and utilises the same plate elements and RC module to model the floors and roof.

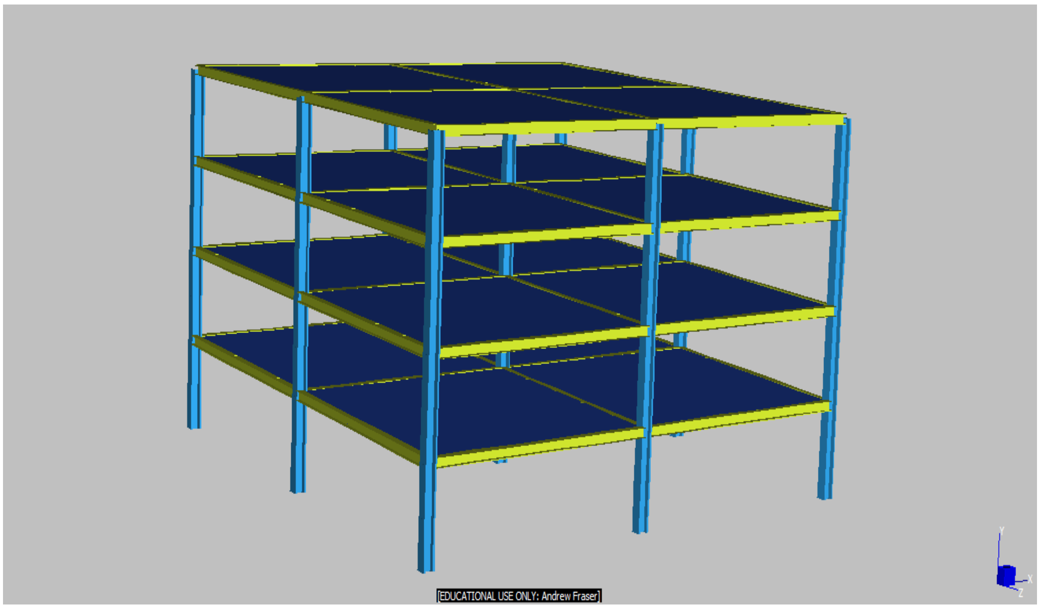

Figure 3-33. Steel Framed Building FE Structural Model

Steel Column and beams

The Steel frame building FE model was identical to the RC frame building with the exception of the column and beams being made up of simple beam elements with UC and UB properties and geometry based on the library module provide within Strand7, as seen at Figure 3-34.

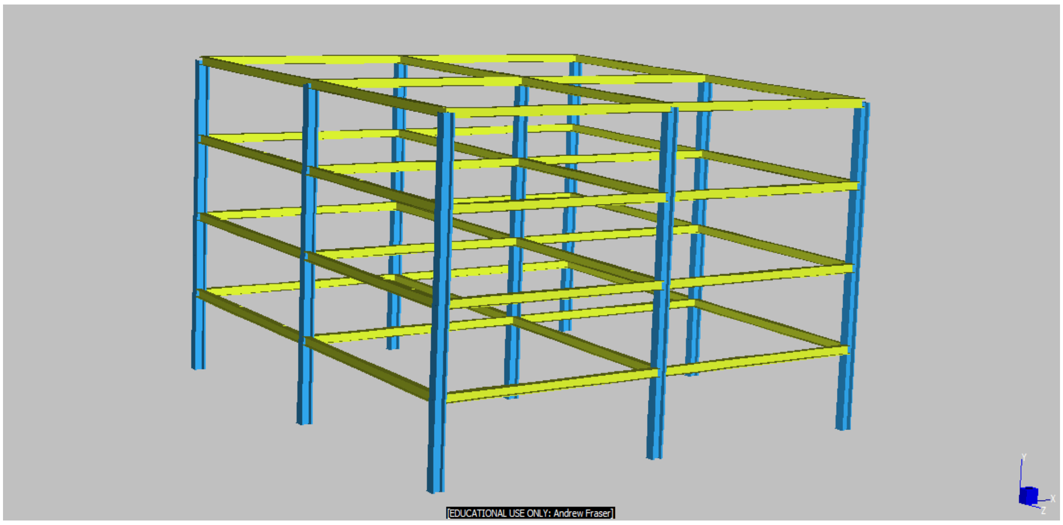

Figure 3-34. Steel frame columns and beams

Material Model

The material properties used in the FE Model are contained in Table 3-19.

Table 3-19. FE Model Material Properties

Material Properties			
Concrete	Density	2400	kg/m^3
	Modulus of Elasticity	30100	MPa
	Compressive Strength f'c	32	MPa
	Designed Max Stress 0.9f'c	28.8	MPa
	Tensile strength f'ct.f = 0.6√f'c	3.4	MPa
	Max allowable compressive strain Ɛc	0.003	
	Peak stress at compressive strain Ɛc	0.0022	
Reinforcement Steel Universal Column Steel Universal Beam	Density	7850	kg/m^3
	Modulus of Elasticity	200	GPa
	Yield Strength fsy	500	MPa
	Uniform Strain Ɛsy	0.05	

Non-Linear Transient Dynamic Analysis for Prediction of structural responses

The structural responses are governed by the material stress capacity of the structure being affected and the resistance against the blast induced stresses. Once the blast wave interacts with the structure blast loading, with its extremely fast rise time and usually short duration, is either dynamic or impulsive, depending on the nature of the loading. The Strand7 FEA package offers a Non-Linear Transient Dynamic Analysis (N-LTDA) well suited to predict structural response to impermanent short load durations (Strand7, Theoretical Manual: Theoretical background to the Strand7 finite element analysis system, 2005). The N-LTDA is governed by the following equation:

$$M\ddot{u}(t) + C\dot{u}(t) + f(t) = p(t) \tag{15}$$

Where:

$M = global\ mass\ matrix$

$C = global\ dampening\ matrix$

$f(t) = global\ element\ (internal\ resisting)\ force\ vector$

$p(t) = applied\ load\ vector(may\ be\ time\ dependant)$

$u(t) = unknown\ nodal\ displacement\ vector$

$\dot{u}(t) = first\ order\ time\ derivative\ of\ u(t)(velocity)$

$\ddot{u}(t) = second\ order\ time\ derivative\ of\ u(t)(acceleration)$

Model configuration for N-LTDA

The FE model approach to NLTDA is described in Figure 3-35.

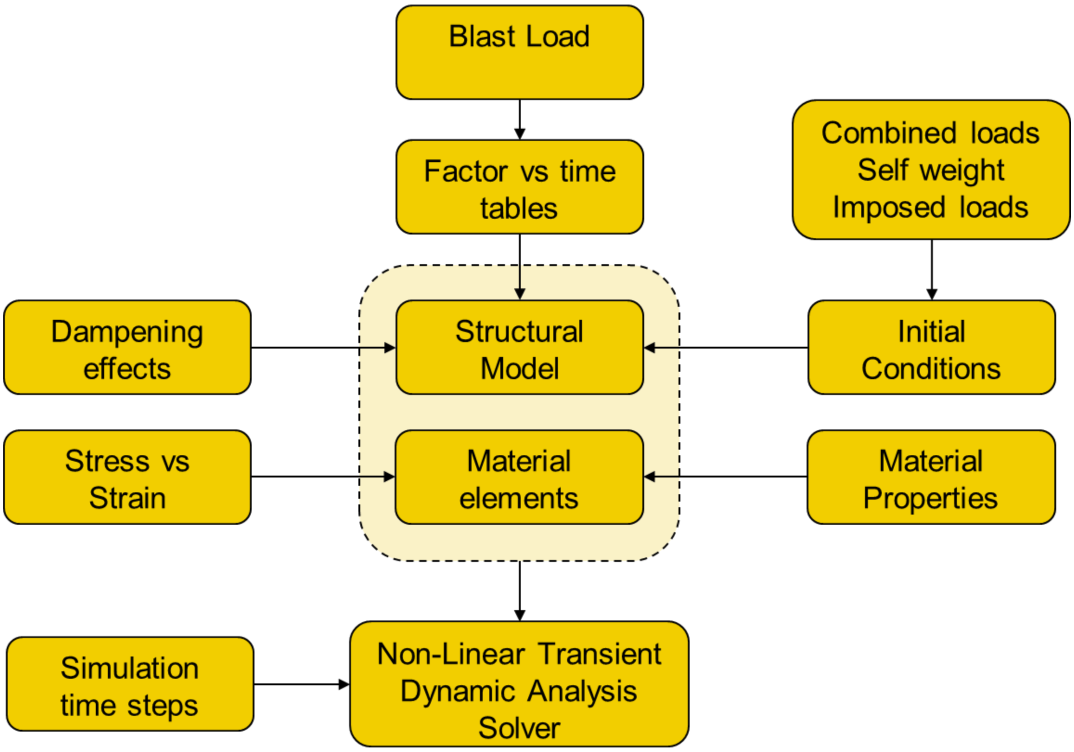

Figure 3-35. Strand7 FEA Non Linear Transient Dynamic Analysis Approach

Blast loads

Blast loading cases previously determined above from empirical methods contained in UFC-3-340 are applied as separate load cases, illustrated at Figure 3-36. Due to the blast pressure being determined in imperial units (psi), the blast loads applied to the FE model are factored to SI units (1psi = 6.9kPa) so the blast pressure time histories can be directly applied without the need to convert pressure units (as seen in Figure 3-37). In order to apply the dynamic loading of the blast pressure to each load case the NLTDA solver load tables need to assign time tables to the load cases shown in Figure 3-38.

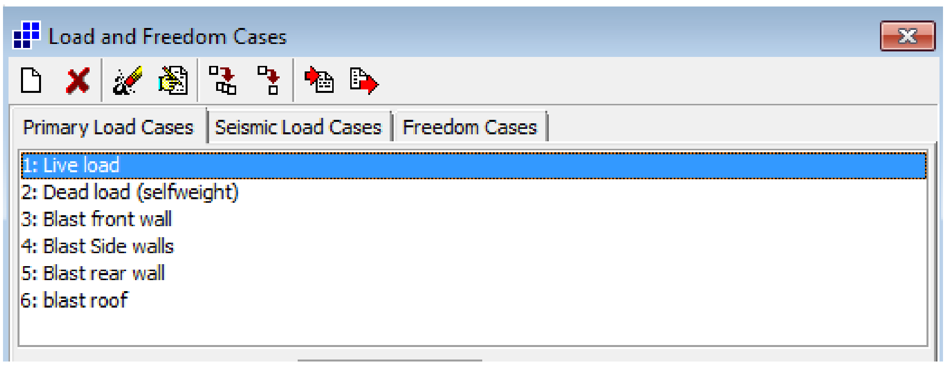

Figure 3-36. Load cases

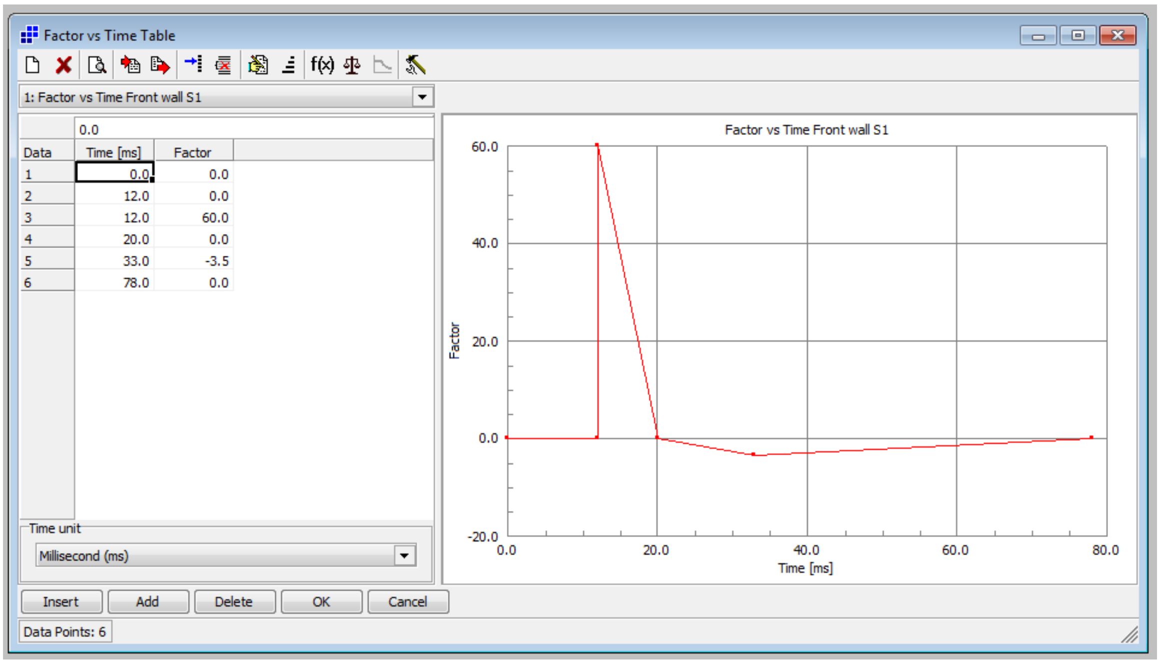

Figure 3-37. Factor vs Time Table for Blast Load Cases

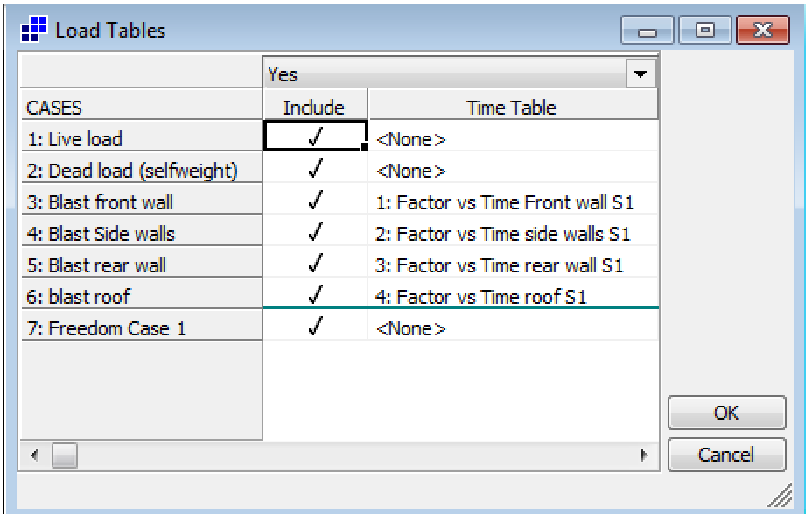

Figure 3-38. NLTDA Load Tables

Initial conditions

In order for the structure to be modelled accurately all permeant loads require a separate static load analysis as this forms the basis for the initial conditions for N-LTDA.

The following permanent loads were considered:

i. Live loads (Q) - applied to floors and roof 7.5kPa (AS/NZS, 2002)
ii. Dead loads (G) - based on structural mass, shown in Figure 3-39.
iii. Permanent load case combination factors - 1.2G + 1.5Q (AS/NZS, 2002), seen in Figure 3-40.

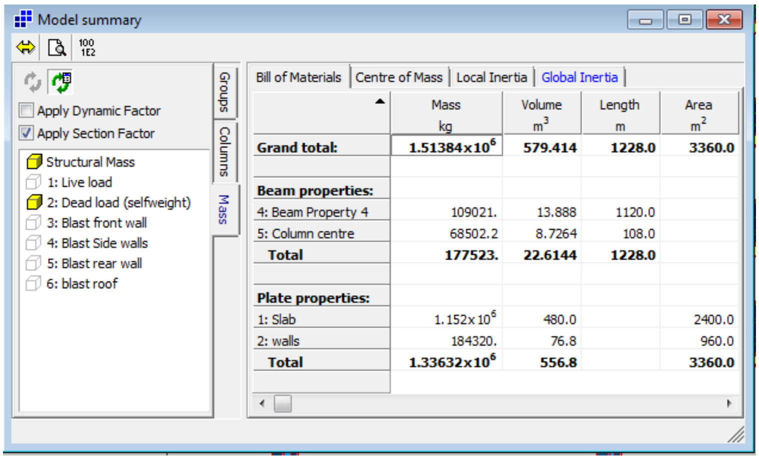

Figure 3-39. Structural mass applied as dead load

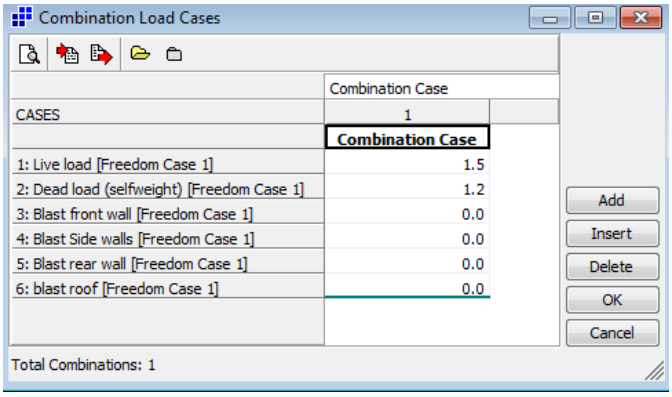

Figure 3-40. Combination Load cases

Damping

For more realistic representation for a 3D model, the effects of damping are required to smooth accelerations and model blast attenuation due to structural damping. Strand7 allows the use of Rayleigh damping which is a simplistic approach involving determining a range of important natural frequencies of the structure.

Stress vs strain

Accurate material properties are required in order to predict the structure behaviour including ultimate limit states and stress vs strain relationship.

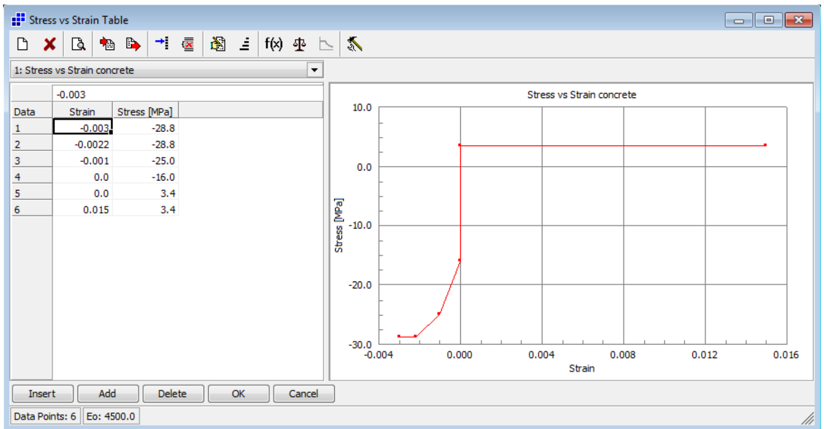

Figure 3-41. Concrete stress vs strain curve

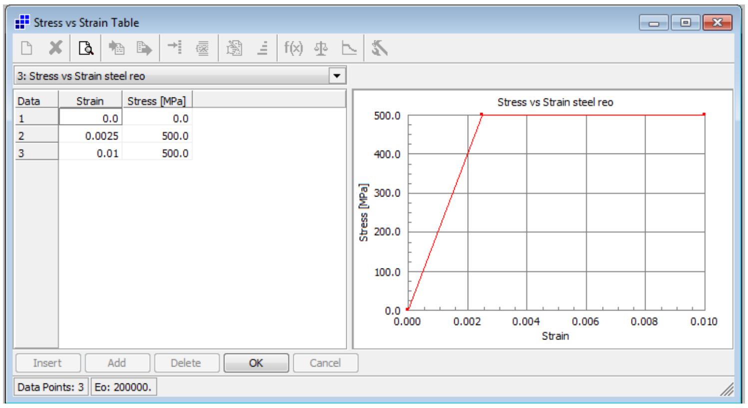

Figure 3-42. Steel stress vs strain curve

Time steps

In order achieve finer results in stress and deflection limits a suitable time step size is required when the pressure loads are changing and faster results when the solution is no longer changing rapidly. Figure 3-43 shows two setups for time steps for the NLTDA solver, the first allows much finer accuracy when the blast wave front impact the structure and the second allows for a much coarser time step where pressure loads are less varied and aids in speeding up solution time.

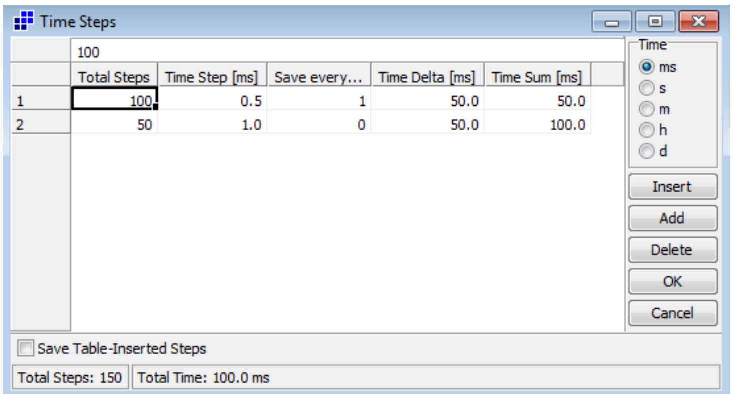

Figure 3-43. NLTDA Solver Time Steps

Local blast effects on structural elements

The FE models chosen to study blast local blast effects and resilience techniques are focused on column structures where failure of these critical elements are likely to lead to partial or total collapse of a building structure as seen in the Oklahoma bombing incident in 1995. Utilising a single external surface blast scenario and loading case established from scenario 1 front wall the FE models are analysed in Strand7 using the same NLTDA solver techniques contained in Figure 3-35 and material properties used in the study the global effects. A FBD of the FE model setup is illustrated in Figure 3-44 showing a column element with fixed end moment subjected to a uniform distribute load.

The FE model column configurations considered are:

i. RC column with standard longitudinal and shear reinforcement detailing,
ii. RC column with standard longitudinal and shear reinforcement detailing and steel plat wrap,
iii. RC column with longitudinal and laced shear reinforcement detailing,
iv. Steel UC, and
v. Steel UC encased in concrete.

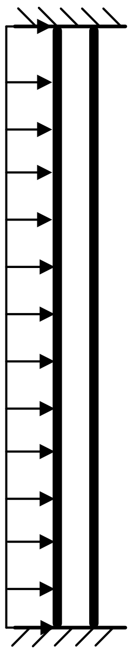

Figure 3-44. Free Body Diagram (FBD) of Column Subjected to Blast Load

RC Column

The FE model for the RC column utilises a more detailed brick and beam element compared to the column model for the building structure as shown in Figure 3-45 and Figure 3-46. This technique provides more detailed by defining the interaction with the reinforcement and concrete. By doing so the stress singularities at the reinforcement ends is minimised as the location and diameter of each slot is modelled by joints to the concrete with rigid links. This approach is a more accurate representation, however, the solution time is the longest and is time consuming to create.

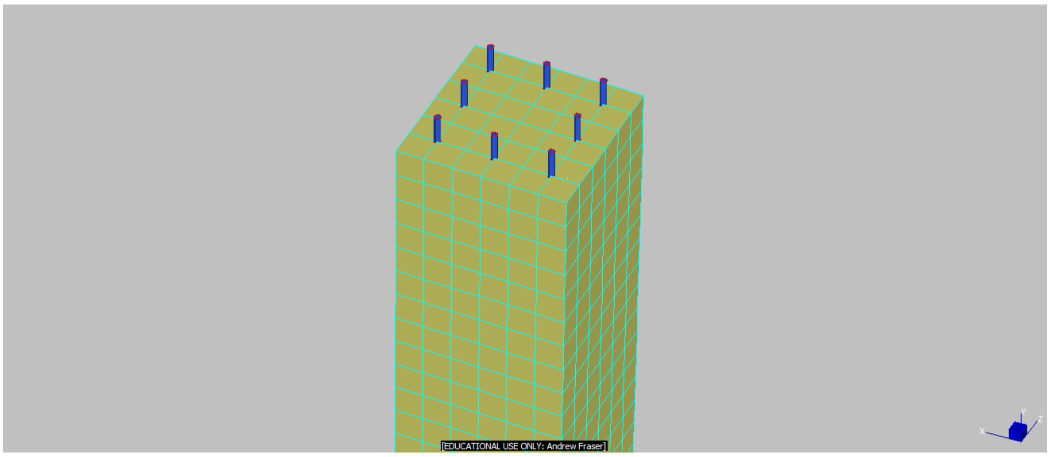

Figure 3-45. Reinforced Concrete Column

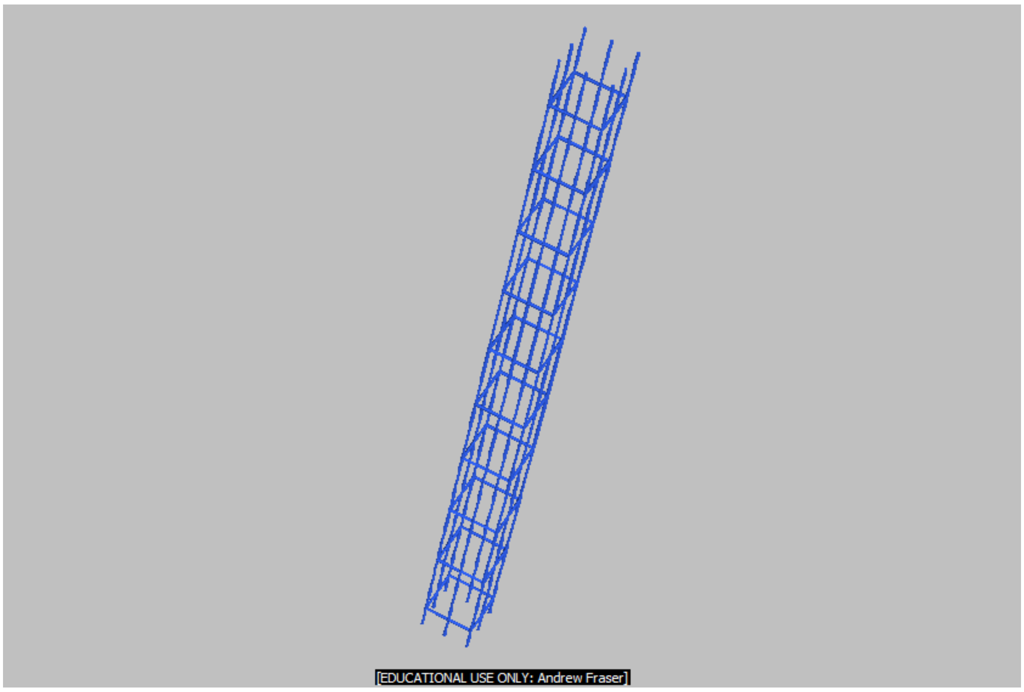

Figure 3-46. Reinforced Concrete Column Reinforcement Detailing

RC column with steel wrap

The FE model at Figure 3-47 is a improved configuration of the RC colum above intended to compare and contrast the effectiveness of the resilinece technique suggest by Cormine, Mays, & Smith, 2009. The FE model is modified with steel plate elements attached to the exterior surface of the brick elements at connecting nodes.

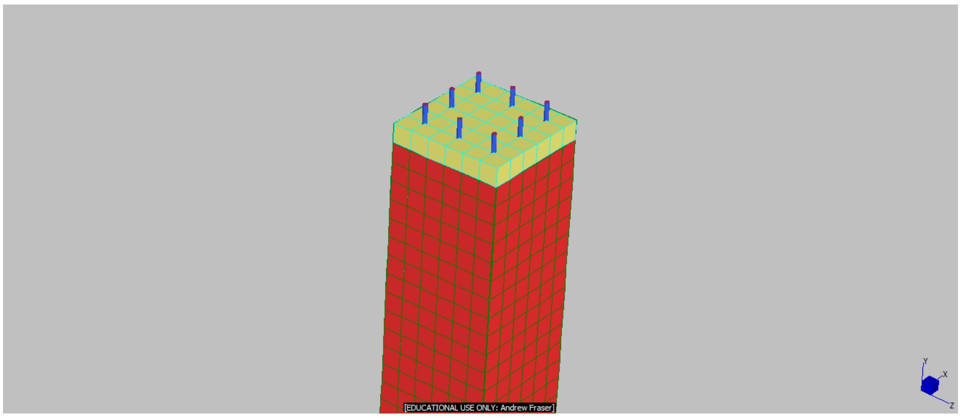

Figure 3-47. Reinforced Concrete Column with Wrapped in Steel Plate

RC column with shear lacing reinforcement

The FE model at Figure 3-48 is a modified version of the RC column with diagonal laced beam elements for the shear reinforcement. Anandavalli, N et al , 2012 suggest optimising the steel shear reinforcement using continuously bent lacing bars attached to tranverse bars along the length of the element have the potential to improve blast performance by enhancing ductility and concrete confinement. Figure 3-49 shows the typical deatailing of shear lacing in a RC structure.

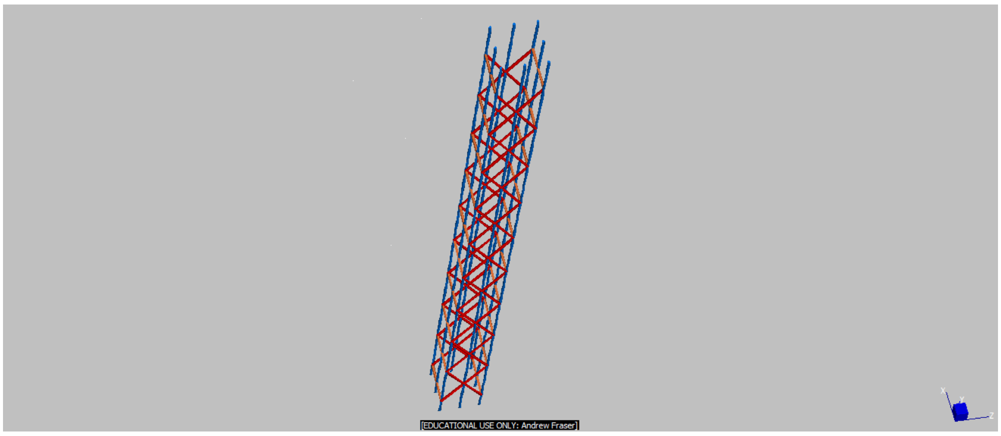

Figure 3-48. Reinforcement Lacing Detailing

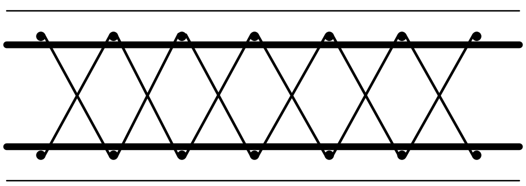

Figure 3-49. Typical Detailing for Reinforced Concrete Structural Element

Steel Universal Column (UC) and UC encased in concrete

The FE model for the steel UC are modelled simply using a beam element and importing the steel UC properties from the Strand7 materials library, shown in Figure 3-50. The optimised configuration of the UC is modified by encasing in concrete. This was achieved by constructing a beam to beam supposition, essentially utilising two beam elements overlaid with one element having steel UC properties and the other having concrete properties, as seen in Figure 3-51. As the overlaid beams share the same nodes, translations and rotations of each degree of freedom are identical. This effect coupled with identical deformation shapes assist in the beams section effectively working together as a composite section. This method greatly assisted in minimising computational times.

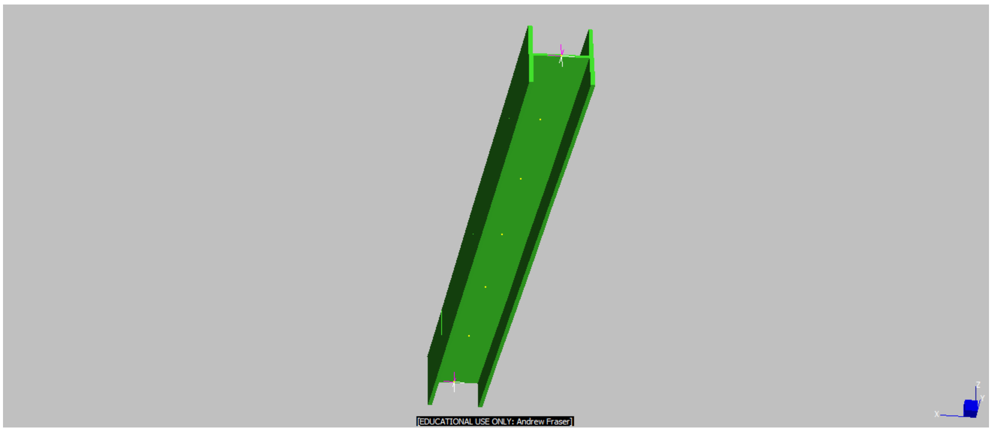

Figure 3-50. Universal Steel Column

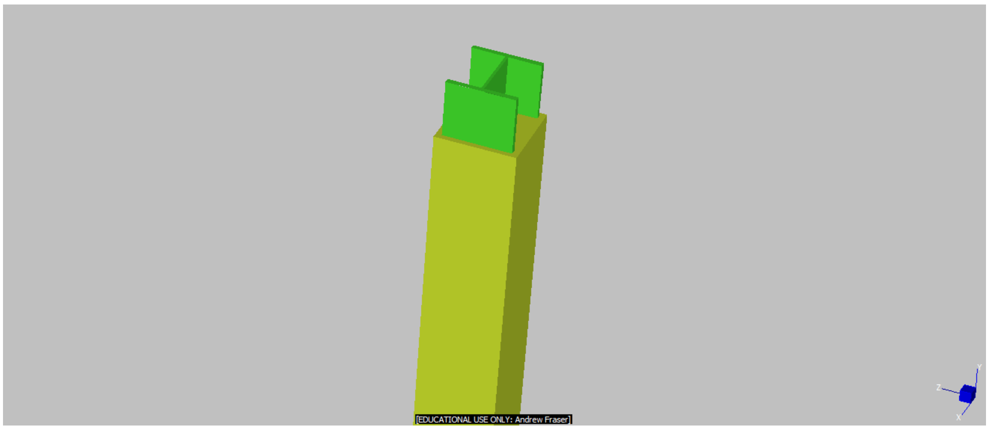

Figure 3-51. Universal Steel Column Encased in Concrete

Chapter 4 – Results

The raw data outputs of the NLTDA from the global effects study of the building FE models is contained in Appendix E.

Global effects results summary

Steel framed building

Scenario 1

The most significant structural response from scenario 1 was due the front wall pressure loading shortly after arrival. The stress responses on the structure were observed at the front columns on arrival and stress loading naturally followed the load path to the column foundations where the peak stress aggregated, as shown below in time steps 12 through 14.5ms. The resultant stresses peaked at 42 MPa in compression and 30.5 MPa in tension located in the base of the centre column on the ground floor in the flanges of the UC. The first floor base of columns and 2nd floor central column and beam joint experience the largest stress responses however, did not exceed yield limits and structure remained intact. Structural deformations showed only slight displacement for this scenario.

Scenario 2

The primary structural response from scenario 2 was due the roof pressure loading. The peak stress responses on the structure were observed in the roof supporting beams where the stresses exceeded the yield strength, as shown below in time steps 100ms at peak positive pressure and 200ms due the peak suction pressure. The result is likely to lead to roof collapse and potential to cause an internal collapse as the remaining floors will need to support the additional load. Lateral pressures on the structure due to front side and rear loadings were not significant to cause any significant response. The structure did however experience moderate deformations, slightly larger than scenario 1 due to the longer pressure phases attribute to the lager explosive charge.

Scenario 3

The most significant structural response from scenario 3 was due the front wall pressure loading shortly after arrival. The peak stress responses on the structure was observed at front columns on arrival and stress continued follow the path of resistance to the column foundations were the peak stresses aggregated, as shown below in time steps 22 through 24ms and eventually resulting in a peak response at time step 40ms. The resultant stresses peaked at 322 MPa in compression and 311 MPa in tension located in the base of the centre column on the ground floor in the flanges of the UC. Similar with scenario 1 the first floor base of columns and 2nd floor central column and beam joint experience the most significant stresses however, responses did not exceed yield limits. The structural deformations were the largest of the 3 steel frame building scenarios resulting in a 32mm horizontal displacement at the top of the building compared to 8.2mm and 6mm for scenarios 2 and 1 respectively.

Concrete framed building

Scenario 1

The most noteworthy structural response from scenario 1 was due the front wall pressure loading shortly after arrival. The peak stress responses on the structure were observed at the front columns on arrival and peak stresses aggregated at the column foundations, as shown below in time steps 12.5 through 16ms. The resultant stresses for concrete peaked at 5.77 MPa in compression and 4.06 MPa in tension located in the base of the centre column on the ground floor in the flanges of the UC. While the tensile stress in the concrete exceed the yield limit the steel reinforcement does not experience any significant stresses that are likely to lead to damage. The displacement of the buildings at the top of the front wall was 6.6mm.

Scenario 2

The structural response from scenario 2 appears to be relatively unaffected in terms of damage. The peak stress responses on the structure were observed at time step 37ms, shown below, at the front columns on as the front wall blast pressure is nearing the end of the positive phase duration. The stress responses for concrete peaked at 21.3 MPa in compression and were negligible in tension located in the base of the centre column on the ground floor in the flanges of the UC. The displacement of the buildings at the top of the front wall was 8.3mm only slightly higher than scenario 1.

Scenario 3

The most significant structural response from scenario 2 was due the front wall pressure loading shortly after arrival. The peak stress responses on the structure were observed front columns on at the foundations were the peak stress aggregated, as shown below in time 39ms. The resultant stresses peaked at 31.1 MPa in compression and 21.4 MPa in tension located in the base of the centre column on the ground floor in the flanges of the UC. The stresses experienced in the front columns have exceeded the tensile strength of the concrete and are likely to failure as the load will overcome steel reinforcement strength resulting in partial or total collapse. The displacement of the buildings at the top of the front wall was 34mm, the largest of the 3 scenarios, this is likely due to the concrete in the columns at the front wall yielding.

The raw data outputs of the NLTDA for the local effects study of the building FE models are contained in Appendix F.

Local effects and resilience results summary

The results contained in Table 4-1 summarise the data collected form the NLTDA for each column configuration for comparison.

Table 4-1. Summary of Data for Local Blast Effects Study

Column Configurations	Column Max Displacement (mm)	Principal Stress responses
Steel column 250UC	105	Tensile fibre stress 78.61MPa Compressive fibre stress 68.57MPa
Steel column 250UC encased in concrete	1.25	Concrete Tensile fibre stress 15.61MPa Compressive fibre stress 15.64MPa Steel UC Max tensile fibre stress 51.64MPa Max Compressive fibre stress 51.4MPa
RC Column with standard reinforcement	65	Concrete Tensile stress fibre 383 MPa Compression stress fibre 320MPa Steel Reo Tensile stress fibre 49.16 MPa Compression stress fibre 39.2 MPa
RC Column with standard reinforcement plus 3mm Steel Plate Wrap	1.3	Concrete Tensile stress fibre 7.26 MPa Compression stress fibre 7.1 MPa Steel Reo Tensile stress fibre 0.96 MPa Compression stress fibre 0.97 MPa Steel Plate Tensile stress fibre 0.56 MPa Compression stress fibre 0.54 MPa
RC Column modified shear reinforcement lacing	0.015	Concrete Mean stress fibre tensile 81.8 kPa Compression stress fibre 1.45 MPa Steel reo lacing Tensile stress fibre 53.1 kPa Compression stress fibre 140.7 kPa

Steel column 250UC vs Steel column 250UC encased in concrete

The Steel UC column performance sustains large deformation without yielding due to steel ductility. The modified column encased in concrete provides enhance rigidity reducing maximum displacement by over 100mm. The resultant stresses in the primary steel support is also reduced. However, the material stress has been exceeded in tension at the mid span and fixed ends. This level of damage could be seen as acceptable as the concrete is not intended to be a primary load bearer for the structure and is mainly focused on protecting the column against blast loads.

RC Column vs RC Column plus 3mm Steel Plate Wrap vs RC Column with shear lacing

The RC Column performed slightly better in terms of displacement with 65mm compared with Steel UC 105mm however, the column failed due to the material stress of concrete has been exceeded in both compression and tension at the mid span and fixed ends. With the addition of a steel plate wrap the displacement further reduced to displacement to 1.3mm and while exceeding the tensile stress for concrete at the fixed ends the are affected with negligible. The RC Column with shear lacing had the least amount of deformation of the column configurations at 0.015mm. The lacing also reduced overall stresses dramatically without leading to column failure.

Chapter 5 - Conclusion

Global effects of blast study

The global response of a structure due to blast pressure, is generally a consequence of lateral or out-of-plane loading. With longer phase durations tending to result in bending failures while impulsive loads (short pressure phase duration) lead to shear responses. The most susceptible structural elements were the permitter columns, in particular those closest to the blast centre. This is supported by blast incident case studies such as the Oklahoma bombing 1995.

The case study steel frame structures suffered the least damage and survived 2 of the 3 scenarios, personal and van delivered explosive, while the car bomb threat with longer pressure phase durations caused roof supporting beams to fail. The concrete framed building responses on the other hand suffered damage at base of the column at the front wall for all scenarios except scenario 2 during the front wall positive pressure phase duration. It was observed during analysis that often the overall response of blast loading is not fully developed by the end of the blast loading case; therefore, sufficient blast simulation durations need to consider the aftershock effects including the inertia of the structural mass. The steel framed building improved performance over concrete can be attributed to its ductility and ability to absorb stress as it deforms.

Results from the global displacements for the building models while small, approx. 30mm at the top, highlight the need to consider blast responses for taller building where excess displacement cause excess out of plane loading for columns and excess moments resulting in the structures toppling or collapsing.

Local effects of blast study

The local responses of the critical structural element such as a column are highly susceptible to failure from blast loading. As the extreme transvers loading for blast pressure are typically not considered during design. Resilience techniques including steel UC encased in concrete, RC steel plate wraps and RC shear reinforcement lacing have the potential to improve the robustness of structural elements reducing overall displacements and stress responses.

While majority of these techniques will need to be considered during the design phase of a building construction, steel plate wrap configurations has the potential to retrofit existing RC structures providing immediate benefit against blast loading. Of the resilience techniques considered in this study the shear reinforcement lacing method proved the best performance against reducing deformation and increasing shear and flexural capacity while enhancing confinement.

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Appendix A – Project Specification

Statement of project and broad aims

The focus of this research project is to study blast loading, the effects on structures and (if time permits) identify techniques for improved structural resilience through design. This is intended to be achieved through the understanding of the blast nature in identifying its causes, its effects and predicting the design loading based on a defined threat. The application of the blast threat on a structural model will be based on the identification of susceptible structures, common failure modes and simulating blast loading effects through Finite Element Analysis methods.

The aim of the project is to increase awareness on the need to design structures to cater for all the life cycle threats, including blast loadings where a credible threat exists. Possible industries that could benefit include structures at high risk of terrorist attack including government buildings, high value public structures and community assets whose destruction would cause widespread casualties.

Scope and objective

The objective of this research project is aimed at studying the effects of blast loading on structures. More specifically, developing an understanding of blast behaviour and generate a credible blast load case to be applied to structural system and study the effects of the interaction of the structure effects.

The scope is restricted to the blast pressure disturbance effects interacting with a structure and not considering the secondary effects of a blast incident including thermal and high velocity fragment effects. It is not the intention to investigate all structural elements but to investigate most commonly used building materials and elements for large public assets which may include reinforced concrete or steel.

Methodology

The overall methodology that will be pursued throughout the course of the project involves:

i. Conducting a comprehensive literature review to identify:
   a. historical cases of blast incidents,
   b. the blast loading behaviour, and
   c. a credible blast threat to be modelled.
ii. Gather historical evidence to identify susceptible structures to blast and development of a structural model using Strand7 FEA software.
iii. Conduct blast simulation analysis using Strand7 FEA software.
iv. Conduct an investigation on the behaviour of structural systems and assemblies through various parametric studies using the validated finite element model.
v. Where time permits, identify trends and possible resilience techniques for new designs.

Project safety

The safety aspects taken into consideration for the project were risks associated with conducting, completing and subsequent use of academic research paper beyond the project in fulfilment of the research project ENG4111 & 4112. The project involves predominantly computer based work i.e. no field work required. The safety analysis focussed on the long exposure times to office related activities and the physical effects as well as the risk of IT equipment failure. A thorough risk assessment, contained at Appendix B, covers all relevant safety aspects and risk management techniques to be implemented during the course of the project.

Project resources

Noting that the project involves predominantly computer based activities, therefore the bulk of the resources required are IT hardware/software and academic information. The requirements for conduct of the project include:

- Desktop and laptop (to account for working remotely)
- Back up storage devices locally and via cloud services
- Internet wired and mobile
- FEA software application and license ($10/month) for duration of project
- FEA software support such as:
    - Strand7 User manual
    - Strand7 Tech support
    - Strand7 Online troubleshooting
    - FEA Web notes for self-paced training
- Reference material (hard copies) of significant relevance ($220)
- Reference resources including:
    - USQ library eBooks and technical papers, online standards, guide and codes of practise
    - Blast related publication database including Blast consultants such as ORICA
- Subject matter experts including engineering professionals with backgrounds in Explosive Ordnance Engineering

While funds were required in order to undertake this project, all costs have been self-funded.

Project schedule

The project phases have been logically divided into work breakdown structures consistent with the objective and methodology with critical milestones highlighted, as illustrated in Appendix C.

Project justification and purpose

As engineers we have a duty to the public to preserve life and protect occupants within or the community that use structures that we build. All practicing engineers are obligated to foster the health, safety and wellbeing of the community and the environment (Engineers Australia, 2010). This involves acting on the basis of adequate knowledge and foreseeable risks that pose a potential hazard towards the built environment. Therefore, it is expected that engineers deliver outcomes that do not compromise the ability of future life to enjoy the same or better environment, health, wellbeing and safety as currently enjoyed (Engineers Australia, 2010). This also agrees with building codes including Performance based Building Code of Australia (BCA) which its main objective is the need to safeguard people and protect adjoining buildings or other property.

Newly built structures need to anticipate and consider all perceived load cases to determine a design that cater these loads over the life. This should also include credible special case threats where occurrence may be of low probability with high consequence in order to achieve due diligence. While most structural loading is well understood, blast loading falls into a unique category. Blast loading not associated with conflicts (war) has become more prevalent. This study is focussed on understanding the nature of blast effects on structures and identifying methods for analysing structures under blast load conditions. The study is intended to highlight deficiencies and developing ways for optimising a design in order to provide enhance resilience therefore damage to structures and preventing harm to personal occupying or using a structure.

The term 'blast' is defined as a destructive wave of highly compressed air spreading outwards from an explosion (The Oxford English Dictionary, 2010). Blasts can be delivered by explosive events either deliberate, accidental or through indirect action. The blast type considered in this study is air blasts (i.e. excluding underwater and underground blasts). The main focus on threats includes deliberate acts such as terrorist's attacks and accidental blasts including the sudden release of high pressure gas or ignition of flammable source. Secondary effects of blasts are not considered in this study including fragmentation and temperature effects.

The terrorism threat has evolved rapidly in scale and occurrences in recent history where extremist groups are willing to explore insidious violent opportunities no matter how radical it may seem can be so committed, they are willing to die for their cause. Terrorism threat has become the norm in modern society and therefore counter terrorism measures including protection are becoming increasingly conscious in commercial, government and industrial projects. Who can forget the incident of the Oklahoma bombing 1995, World Trade Centre in New York and the Pentagon in 2001 which had an immediate effect on awareness (Cormine, Mays, & Smith, 2009). However, the majority of structures are rarely designed to resist the effects of blasts according to Dusenberry, 2010, which is considered as an increasing threat to structural safety considering the recent escalation of terrorist attacks. Therefore, the importance of resilient structural design becomes the first layer of defence against blast effects.

Motivation

This project was chosen partly due to having personal vested interest in the topic and making observations on the deficiency of blast loading information as it relates to structural design including Australian Standards, building codes and guidelines. This drove the motivation to embark on a large research project with an opportunity to provide useful research to the wider engineering community involved in blast design and gain experience with the use FEA software.

Appendix B - Research Project Risk Assessment

Risk assessment scope and objectives

This assessment takes into consideration the risks associated with conducting, completing and subsequent use of academic research paper beyond the project in fulfilment of the research project ENG4111 & 4112. The project is predominantly computer based work i.e. no field work required. The objective of the assessment is to identify the known and perceived risks likely to be encountered during the course of the research project, conduct a risk assessment and management the risk appropriately.

Risk assessment definitions

Hazards – A source or condition that poses a potential threat to health, property or project outcomes.

Likelihood - Probability of hazard occurrence that will have an effect on project objectives. Likelihood takes into consideration the frequency of hazardous event occurring but doesn't consider whether a hazard has been exposed or effected major objectives.

Exposure – How often or the duration of hazard is exposed. The concept of exposure is being directly affected be the hazard rather than indirectly. A hazard may have occurred but the effects may lay dormant and have a delayed effect on exposure.

Consequences – The severity or magnitude of effects caused by a hazard.

Risk – In the context of this project the risk is considered a function of likelihood, exposure and consequence. It can be described as the possibility and severity of a hazardous event occurring.

Risk Management process

Establish context

The context in which this risk assessment is applicable includes:

- Managing risks associated with the execution of the project or impeding progress towards project critical milestones.
- Risk beyond the completion of your project academic paper being used or interpreted by others including misused or misleading.

Identification of hazards

This phase aims at identifying all the sources or condition that has the potential to cause harm or affect the quality or completion of the project.

Key hazards considered:

- Injury from undertaking work activities and affects progress
- Illness that affects the completion of project objectives
- Critical equipment damage or loss of information affects project progress
- Misinterpreted or misrepresented information being published

Risk Assessment

Risk Analysis

The risk analysis phase considers all credible hazards towards the project and assesses each risk element including likelihood, exposure and consequence. The following assessment rating criterion was developed specifically for this project using risk management principles (AS/NZS, 2009) in order to determine the risk levels and identify area for risk reduction activities if required.

Likelihood Assessment

5 - Almost certain (expected to occur in most circumstances)

4 - Likely (expected to occur during project)

3 - Possible (may occur at some time in future)

2 - Unlikely (conceivable but not expected to occur)

1 - Rare (so unlikely it may never be exceptional circumstances)

Exposure Assessment

6 - Continuously

5 - Frequently (perhaps daily)

4 - Regularly (perhaps weekly)

3 - Occasionally (perhaps once or twice a month)

2 - Rarely (few times a year)

1 - Very rarely (once per year or less)

Consequence Assessment

5 - Severe (threatens project completion, death or permanent disability)

4 - Major (threaten project key milestones, serious injury or illness)

3 - Moderate (threaten project quality, injury illness requires medical treatment)

2 - Minor (reduced project efficiency or short term delay, medical attention non-emergency)

1 - Insignificant (minimal disruptions, first aid)

Risk Evaluation

Evaluation risk levels are used to assist in defining limits for acceptability and tolerability. This also identifies areas of improvement for risk reduction studies whether the risk itself can be tolerated at all. The following risk levels were developed specifically for this project.

Risk level is a function of likelihood, exposure, consequence which is divided into the following four categories, as shown in table 1A.

Risk Rating = likelihood x exposure x consequence

Table 1A. Risk level and acceptability table

Risk Level	Rating	Acceptability
Extreme	121 - 150	Considered intolerable
High	91 - 120	Tolerable with continuous review (on every occasion there is an exposure to the hazard) following a risk reduction study
Significant	61 - 90	Tolerable with periodic review following a risk reduction study
Medium	31 - 60	Acceptable with periodic review
Low	1 - 30	Acceptable (no further action needed)

Risk Treatment

Consider all available controls to reduce the likelihood, exposure or consequence related to the hazards. Controls can be either pre-event or post event controls. Prevent a hazard from occurring or reduce the hazardous effects after it has occurred.

Monitor and review

Where risks are considered acceptable/tolerable with review, monitoring and review techniques need to be considered to manage the risk throughout the project. These include proactive and reactive methods in other words ways in which a control condition can be monitored and remain effective to prevent hazard from occurring or provide recovery post hazard.

Table 2A. Hazard Risk Assessment

Hazard Description	Casual Factors	Likelihood (1 – 5)	Exposure (1 – 6)	Consequence (1 – 5)	Risk (1 – 150)	Controls (Risk Treatment)	Residual Risk (Post treatment)
Laptop failure	• Old or outdated hardware • Corrupted data • Computer virus • Physical damage (drop) • Overheating	Likely (4)	Regularly (4)	Severe (5)	Significant (80)	• Secondary Laptop or PC on standby • Laptop servicing & repair • Virus protection up to date • Protective case during laptop transit • Adequate ventilation during operation	Reduction in likelihood to Possible (3) Resulting Risk Medium (60)
Loss of Broadband Internet	• Data network fault • Service provider planned outages	Likely (4)	Regularly (4)	Major (4)	Significant (64)	• Mobile broadband on standby	Reduction in likelihood to Possible (3) Resulting Risk Medium (48)
Loss of project information including research and critical working files	• Human error; accidental deletion, misplaced storage device • Corrupted data	Likely (4)	Regularly (4)	Severe (5)	Significant (80)	• Back up PC regularly • Use of cloud storage to save critical files • Version control • Records management	Reduction in likelihood to Possible (3) Resulting Risk Medium (60)
FEA software fault	• Corrupted data • Inadequate training	Likely (4)	Regularly (4)	Major (4)	Significant (64)	• Strand7 User manual • Strand7 Tech support • Strand7 Online troubleshooting	Reduction in likelihood to Possible (3) Resulting Risk Medium (48)

Hazard Description	Casual Factors	Likelihood (1 – 5)	Exposure (1 – 6)	Consequence (1 – 5)	Risk (1 – 150)	Controls (Risk Treatment)	Residual Risk (Post treatment)
Office related injuries such as headaches, back aches, RSI and eye strain	• Insufficient lighting • Poor ergonomic setup	Likely (4)	Frequently (5)	Moderate (3)	Medium (60)	• Sufficient lighting • Ergonomic postural and visual setup • Regular breaks	Reduction in likelihood to Possible (3) and exposure to Occasionally (3) Resulting Risk Low (27)
Become ill and unable to complete tasks on time	• Virus or bacterial infection • Pre-existing condition	Likely (4)	Occasionally (3)	Moderate (3)	Medium (36)	• Request task extension on medical grounds • Maintain healthy lifestyle	Reduction in likelihood to Possible (3) Resulting Risk Low (27)
Family member becomes ill requiring care and being unable to complete tasks on time	• Virus or bacterial infection • Pre-existing condition	Likely (4)	Occasionally (3)	Moderate (3)	Medium (36)	• Request task extension on medical grounds, • Family support to assist with caring	Reduction in likelihood to Possible (3) and consequence to Minor (2) Resulting Risk Low (18)

Hazard Description	Casual Factors	Likelihood (1 – 5)	Exposure (1 – 6)	Consequence (1 – 5)	Risk (1 – 150)	Controls (Risk Treatment)	Residual Risk (Post treatment)
Mishandling of information without ethical clearance	• Human error; accidental misplacing information	Likely (4)	Regularly (4)	Major (4)	Significant (64)	• Avoid using classified information altogether and only use unclassified material released for public use.	Reduction in likelihood to Unlikely (2) and exposure to rarely (2) Resulting Risk Low (16)
Inadequate study environment inhibits study effectiveness	• Extreme temperature and humidity (Canberra climate) • Insufficient study space • Noise distractions	Likely (4)	Regularly (4)	Minor (2)	Medium (32)	• Sufficient study space • Temperature and humidity controlled environment • Sited away from noise distractions	Reduction in likelihood to Possible (3) and exposure to Occasionally (3) Resulting Risk Low (18)
Fatigue and stress	• Lack of sleep • Inactivity • Work and personal pressures	Likely (4)	Regularly (4)	Moderate (3)	Medium (48)	• Exercise regularly • Well rested • Time management • Regular study breaks	Reduction in likelihood to Possible (3) and exposure to Occasionally (3) Resulting Risk Low (27)
Disseminate/publish false or misleading information	• Inadequate reviews	Likely (4)	Occasionally (3)	Major (4)	Medium (48)	• Project supervision • Critical reviews	Reduction in likelihood to Possible (3) Resulting Risk Medium (36)

Risk Assessment Summary

The highest residual risks identified were:

- Laptop failure, resulting risk Medium (60). This was considered acceptable with periodic review. This includes monitoring including computer virus scanning, system diagnostics and check for regular software updates. As a recovery method a boot disk should also be provided.

- Loss of project information including research and critical working files, resulting risk Medium (60). This was considered acceptable with periodic review. This requires conducting regular backing up of working files to local hard drive and cloud as form of redundancy. In addition, version control and good records management practise will help reduce the risk of inadvertent deletion or misplacing data.

The remaining risks requiring monitoring and review were:

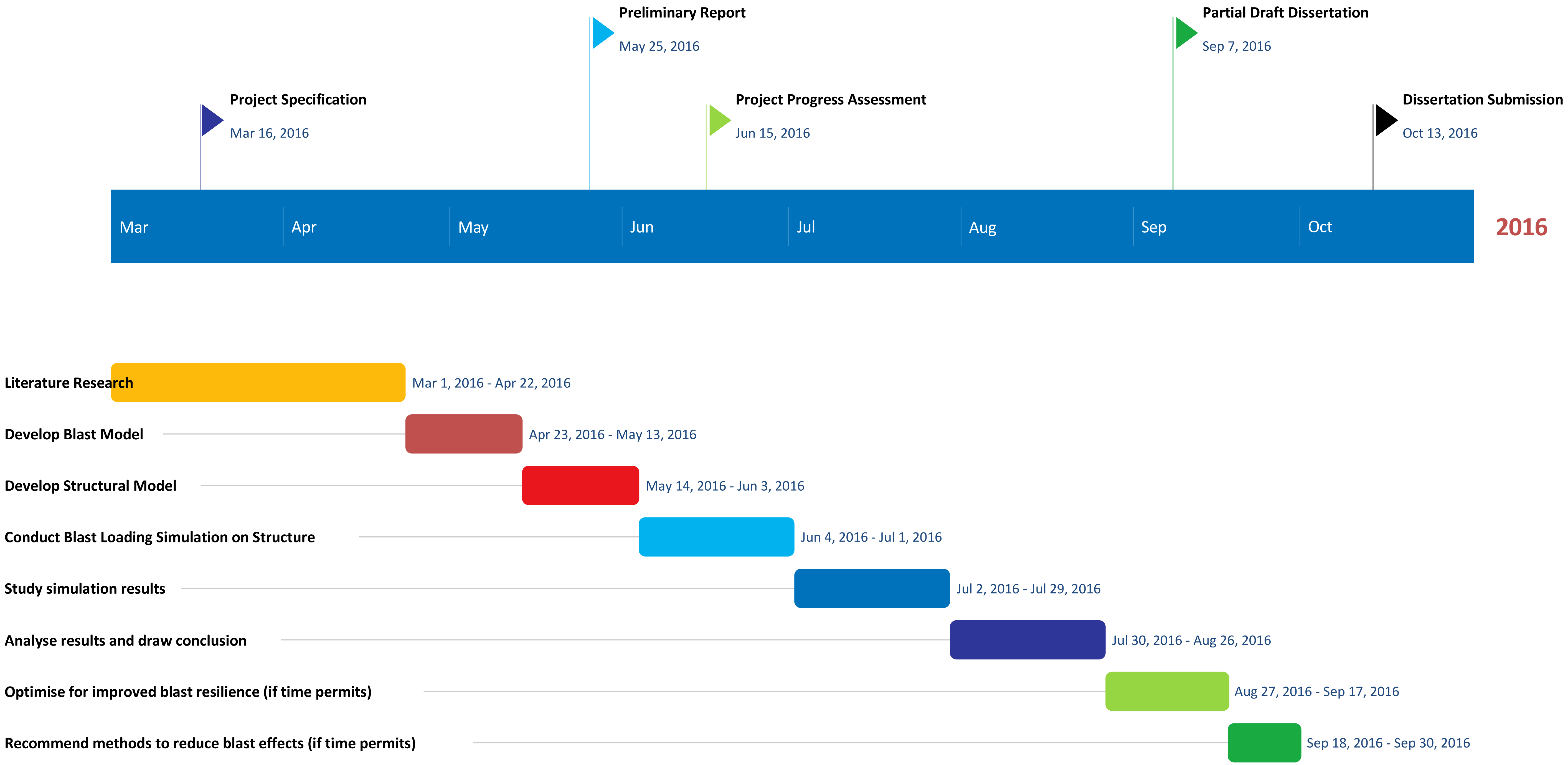
- Loss of Broadband Internet resulting risk Medium (48). The risk monitoring and review shall include frequent checking for internet outages and test standby mobile broadband regularly.

- FEA software fault resulting Risk Medium (48). The risk monitoring and review shall check software updates for fixes and conduct self-training to avoid user input errors.

General comments:

In order to ensure the project tasks are delivered on time all progress shall be monitored against project specification plan schedule on a regular basis to identify short falls and implement the necessary recovery actions.

Appendix C - Project Schedule



Appendix D - Historical Cases of Blast Incidents

Table 1D. Accidental blasts incidents
(sourced from multiple news articles and government databases)

Date	Location	Structure	Damage	Cause of explosion
May 1968	Ronan Point London	18th floor tower block	Progressive collapse of corner of building due to structural precast walls. Note: Outcome of incident improved requirement for structures to be designed for notional column or transfer beam removal and min horizontal and vert trying provisions.	Kitchen gas explosion
February 1971	Woodbine, Georgia	Chemical plant		Accidental due to explosive pyrotechnic chemical mixture
June 1974	Flixborough disaster, England	Chemical Plant		Accidental due to release of flammable vapour mixture
February 1976	Galena Park Texas	Grain elevator	Dust explosion caused Partial collapse	Accidental Explosive grain dust atmosphere
October 1989	Pasadena Texas	Chemical Facility Plastics	Structural collapse	Accidental Flammable gas
May 1991	Louisiana US	Chemical nitro-paraffin plant		Accidental
June 1998	Haysville, Kansas	Grain elevator	Dust explosion caused Partial collapse	Accidental Explosive grain dust atmosphere
March 1999	Osseo, Michigan	Fireworks factory explosion	Levelled factory one wall remained standing	Accidental Explosive materials
September 2001	Toulouse France	Fertilizer Factory		Accidental

Date	Location	Structure	Damage	Cause of explosion
November 2002	Riobamba, Ecuador	Santa Barbara munitions factory	Shattered glass up to 1.5km	Accidental
2005	Texas, Texas City	Refinery		Accidental
March 2007	Maputo Mozambique	Arms depot	Buildings shook and windows broke	Arms depot explosion due to high heat
October 2009	Ottawa Canada	Heating and cooling industrial plant		Accidental boiler plant stored energy
April 2010	Anacortes, Washington	Petroleum Refinery		explosion and resulting fire when a heat exchanger ruptured
June 2010	Dhaka, Bangladesh	Residential buildings	Minor damage	Accidental (lightening) electrical transformer
January 2012	British Columbia Canada	Wood Mill		Accidental Combustible dust environment
March 2012	Brazaville Congo	Munitions depot	Levelled and Collapsed nearby buildings	Accidental Fire caused munitions
November 2012	Quebec Canada	biochemical plant		Accidental Flammable oil
April 2013	West Texas	Fertilizer Storage		Accidental
June 2013	Quebec Canada	fireworks warehouse explosion		Accidental Explosive materials
August 2014	Beijing China	Metal factory		Accidental Investigation suggested the blast was triggered by a flame lit in a dust-filled room
August 2014	Kunshan Taiwan	Metalwork factory car parts	Glass was shattered up to 500 meters away	Accidental Triggered by a flame in a dust-filled workshop
Sept 2015	China Tianjin	Chemical Warehouse		Accidental two large explosions, investigation concluded in that an overheated container

Date	Location	Structure	Damage	Cause of explosion
				of dry nitrocellulose was the cause of the initial explosion
April 2016	Texas US	Fertilizer manufacturing Plant		Unknown
May 2016	Dombivili Mumbai	Chemical Factory		Chemical chain reaction
August 2016	Dangyang China	Power plant		A high-pressure steam pipe exploded
August 2016	Florida USA	chemical plant		Accidental Explosion likely originated near a holding tank in the Airgas loading dock where two semis were holding nitrous oxide
Sept 2016	Yantai, China	Chemical plant		blast occurred during maintenance work at a methylene diphenyl diisocyanate (MDI) plant
Sept 2016	Bangladesh	Cigarette packaging factory	Caused near total collapse of the factory building	Accidental Boiler explosion

Table 2D. Deliberate blasts incidents
(sourced from multiple news articles and government databases)

Date	Location	Structure	Damage	Cause of explosion
April 1983	Beirut	US Embassy	Total destruction	Suicide bomber Massive truck bomb
October 1983	Beirut	US Marine HQ Airport	Levelled 4 story building	Suicide bomber truck
September 1984	East Beirut	US embassy		Suicide bomber van
April 1992	St Mary Axe London	Chamber of shipping	extensive collapse after key columns were severed	IRA attack
February 1993	World trade center tower One	Underground carpark	1000 feet wide crater four sub levels of Reinforced concrete	Suicide bomber van
April 1993	Bishopsgate London	Kansallis house 8 storey in situ RE frame with RC perimeter beam and	3 load bearing columns lost at corner but did not collapse	IRA attack vehicle bomb
April 1995	Oklahoma City	Federal building	Eighth storey office block Destroyed transfer beam running length of building causing a progressive collapse loss of major part over full height	Parked vehicle Fuel oil bomb
August 1998	East Africa	US embassy		2 Suicide bomber truck
September 11 2001		World trade centre twin towers		Two commercial jets
June 2002	Karachi Pakistan	US consulate,	Hole in wall	Suicide Truck bomb Fertilizer bomb
October 2002	Kutta Beach Bali	Night Club		Large vehicle bomb and a possible suicide bomber
October 2002	Indonesian island of Bali	Paddy bar Sari club		1st Back pack bomb 2nd bomb in van
May 2003	Riyadh, Saudi Arabia	Residential Compounds		detonated vehicle-borne improvised explosive devices (VBIEDs) in the compounds
May 2003	Riyadh Saudi	Foreigner housing compound	4 and 5 story building façade sheared off Crater 20 feet across	7 car bombs

Date	Location	Structure	Damage	Cause of explosion
			6-7 Single story houses with 5oft destroyed	
August 2003	Jakarta, Indonesia	JW Marriott Hotel	blast caused extensive damage to the hotel and an adjacent office building	vehicle-borne improvised explosive device (VBIED) exploded in front of the JW Marriott Hotel
August 2003	Emeryville, California	Chiron Life Science Centre	Damaging the building and the surrounding area	an improvised explosive device (IED) was detonated near the front door, second device detonated in another Chiron building
August 2003	Jakarta, Indonesia	JW Marriott hotel	Severe damage	Suicide Car bomb
August 2003	Iraq	Canal hotel	Destroyed building	Suicide truck bomb
September 2003	Pleasanton, California.	Shaklee Corporation, subsidiary of Yamanouchi Pharmaceutical Co. Ltd		improvised explosive device was detonated at
November 2003	Riyadh Saudi Arabia	Residential compound		Suicide Car bomb
October 2004	Taba Egypt	Hilton Hotel		2 Suicide Car bomb
July 2005	London, United Kingdom	London Transportation System		four suicide bombers “home-grown” terrorists
October 2005	Bali, Indonesia	Raja Restaurant in Kuta Square		vests or carried backpacks containing the explosives used improvised explosive devices
March 2006	Karachi Pakistan	Marriott hotel US consulate		Suicide Car bomb
January 2009	Hernani, Spain.	Television station in Hernani causing damage		
March 2009	Athens, Greece	government office of the ruling party in Greece, causing damage		homemade bomb exploded
September 2009	Athens, Greece	Athens Stock Exchange	bomb went off outside a government building in Thessaloniki, causing minor damage	A bomb in a van explodes

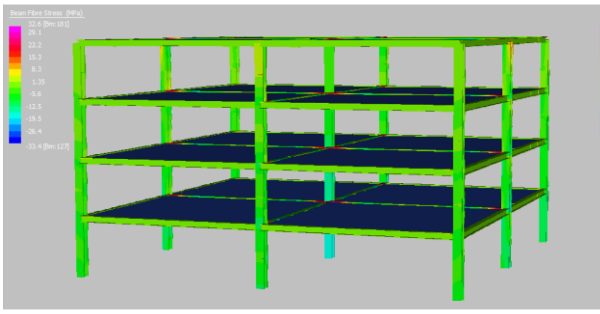
Date	Location	Structure	Damage	Cause of explosion
January 2016	Kabul, Afghanistan	French restaurant called 'Le Jardin'	blast also left a building engulfed in flames	Suicide bomber detonated himself
January 2016	Mogadishu, Somalia	popular restaurant near the National Theatre of Somalia		suicide bomber detonated himself
January 2016	Camp Speicher, Iraq	Camp Speicher, a former U.S. base		bombers detonated their vehicle-borne explosives
January 2016	Kabul, Afghanistan	Armoured gates of a compound for civilian contractors near Kabul's airport	Smashing windows and sending glass flying and badly damaging nearby houses	truck packed with explosives
January 2016	Zliten Libya	police training camp Al-Jahfal		Suicide truck bomb
January 2016	Ra's Lanuf, Libya	checkpoint in the Libyan oil port of Ras Lanuf		car bombing
January 2016	Istanbul, Turkey	The blast struck at a park that is home to the landmark Obelisk of Theodosius, when the bomber walked up to a tour group standing in Sultanahmet Square and blew himself up. The last major attack on Sultanahmet Square occurred on 6 January 2015, when a suicide bomber detonated herself at a police station.		A suicide bomber blew himself up near Hippodrome of Constantinople
January 2016	Jalalabad, Afghanistan	Near the Pakistani, Indian and Iranian consulates		A suicide bomber detonated its explosives
January 2016	Quetta, Pakistan	Near security personal vehicles close to a polio vaccination centre		Suicide bomber detonated himself
January 2016	Kouyape, Cameroon	Mosque		A suicide bomber blew himself
January 2016	Diyarbakır Province, Turkey	Police headquarters		A massive bomb blast, followed by rocket and long gun fire
January 2016	Jakarta, Indonesia	Starbucks and a police station		Several explosions followed by gunfire

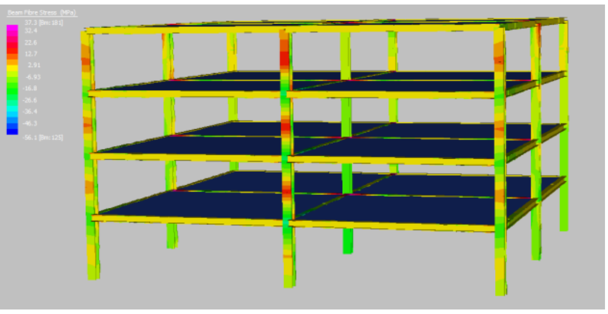
Date	Location	Structure	Damage	Cause of explosion
January 2016	Jalalabad, Afghanistan	Home of politician		suicide bomber
January 2016	Aden, Yemen	Entrance of the residence of Aden police chief		suicide bomber detonated his explosives while within a car
January 2016	Nguetchewe, Cameroon	Mosque		suicide bomber
January 2016	Peshawar, Pakistan	National Highway		suicide bomber driving a motorcycle
January 2016	Quetta, Pakistan	FC's Margat Checkpoint		IED was detonated
January 2016	Kabul, Afghanistan	Russian embassy in Kabul		A suicide car bomber detonated his explosives
January 2016	Aden, Yemen	Presidential palace in Aden		suicide bomb
January 2016	Al-Ahsa, Saudi Arabia	mosque of Imam Reza		Suicide bombings
January 2016	Aden, Yemen	Checkpoint in the southern Yemen city		suicide car bomber
January 2016	Damascus, Syria	Sayyidah Zaynab Mosque shrine		two suicide bombs and a car bomb exploded
Feb 2016	Kabul, Afghanistan	Headquarters of the Afghan National Civil Order Police in Kabul.		A suicide bomber blew himself within a queue
Mar 2016	Belgium, Brussels	Airport Train Station	Building system and glazing damage and deformed structures	Deliberate Suicide Person borne IED
Aug 2016	Pakistan Quetta	Civil hospital		explosive blast detonated 8kg of explosives at the gate of the emergency department

Appendix E – Global blasts effects results

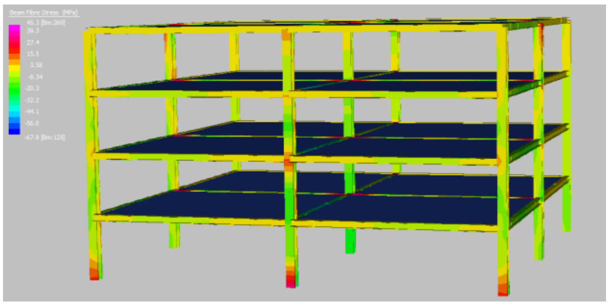
Scenario 1 - Steel Frame Building

Scenario 1 steel frame building critical stress responses contained below.



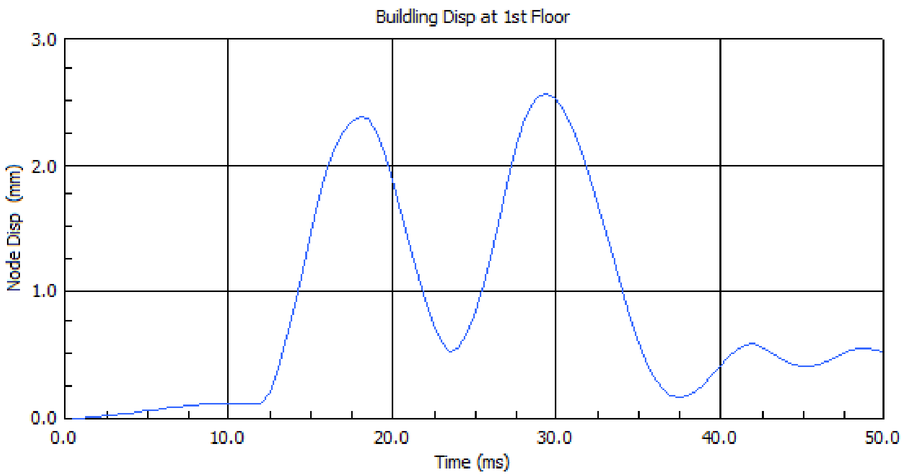


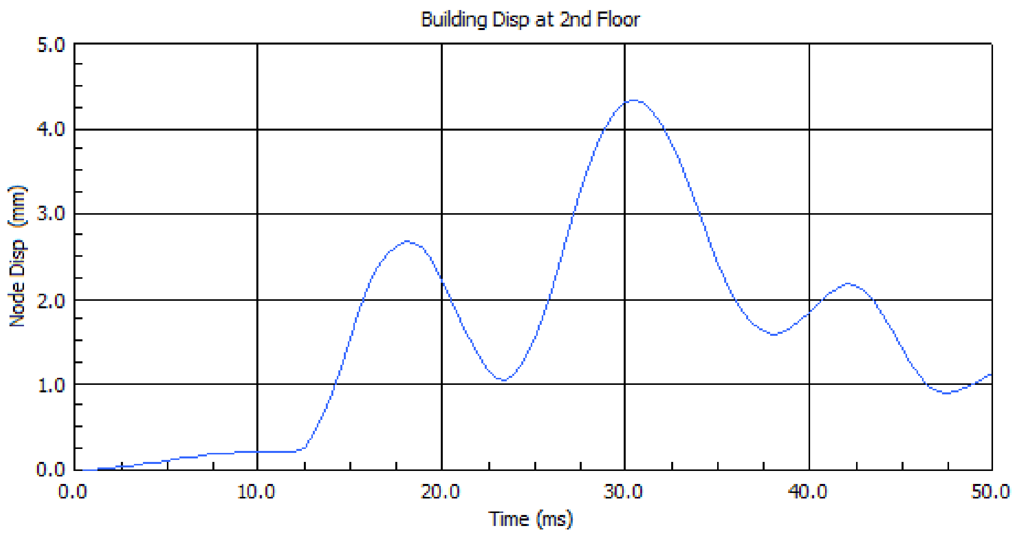
Time step 12ms (arrival of front wall blast pressure) Time step 13ms

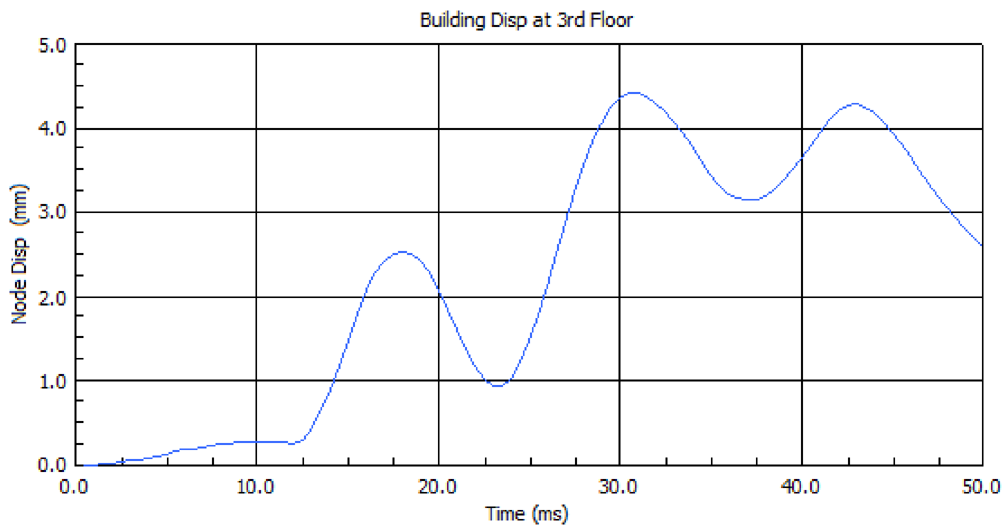


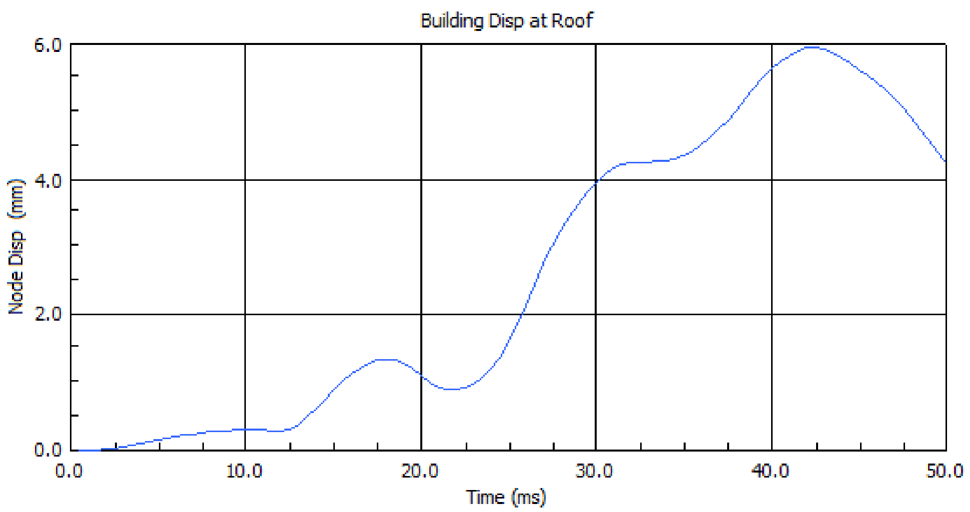
Time step 14.5ms (peak stress response)

Scenario 1 steel frame building displacements at floor levels contained below.



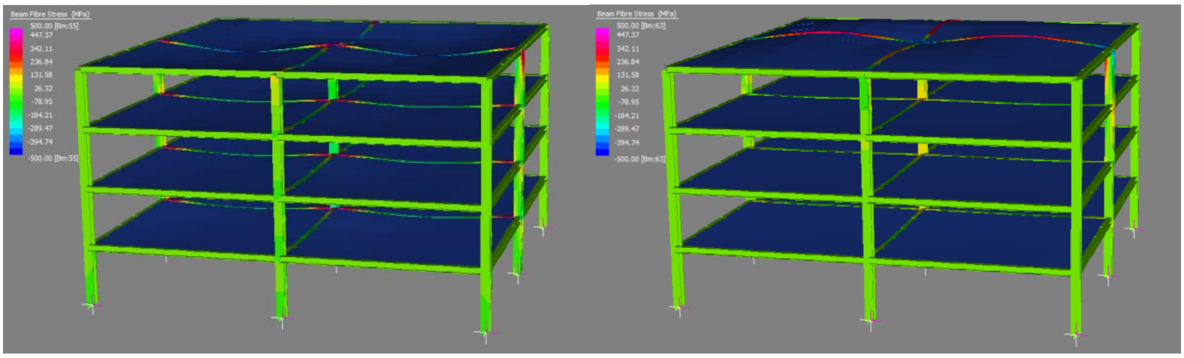

Building Disp at 2nd Floor
Node Disp (mm)
Time (ms)
5.0
4.0
3.0
2.0
1.0
0.0
0.0
10.0
20.0
30.0
40.0
50.0


Building Disp at 3rd Floor
Node Disp (mm)
Time (ms)
5.0
4.0
3.0
2.0
1.0
0.0
0.0
10.0
20.0
30.0
40.0
50.0


Building Disp at Roof
Node Disp (mm)
Time (ms)
6.0
4.0
2.0
0.0
0.0
10.0
20.0
30.0
40.0
50.0

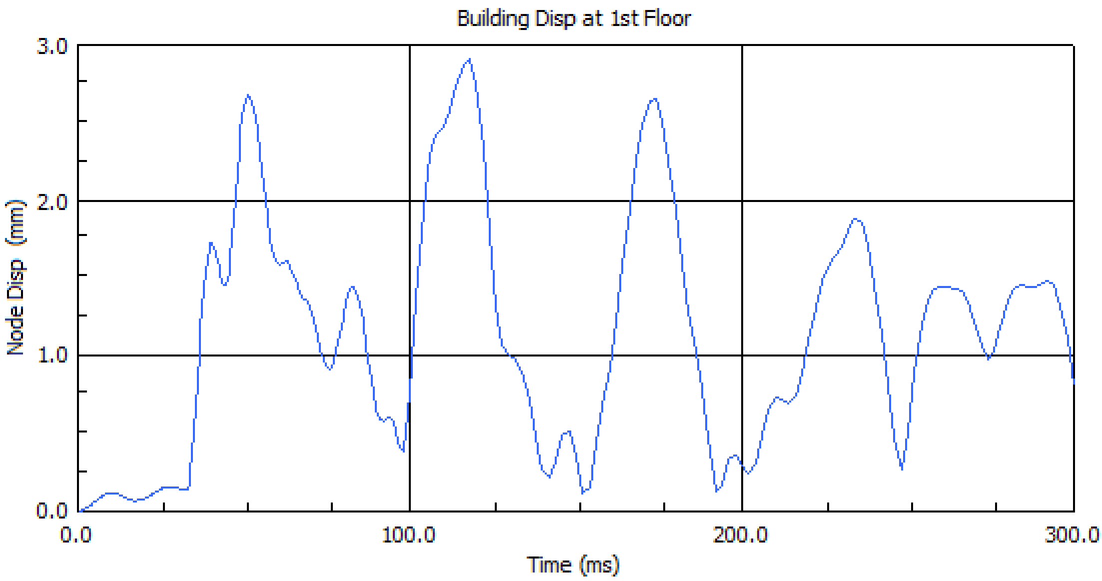
Scenario 2 - Steel Frame Building

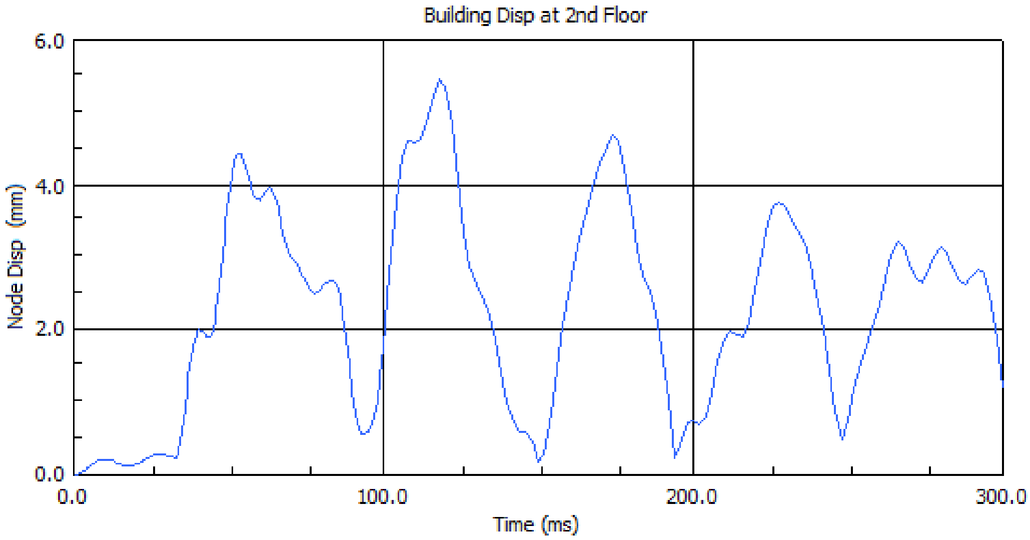
Scenario 2 steel frame building critical stress responses contained below.

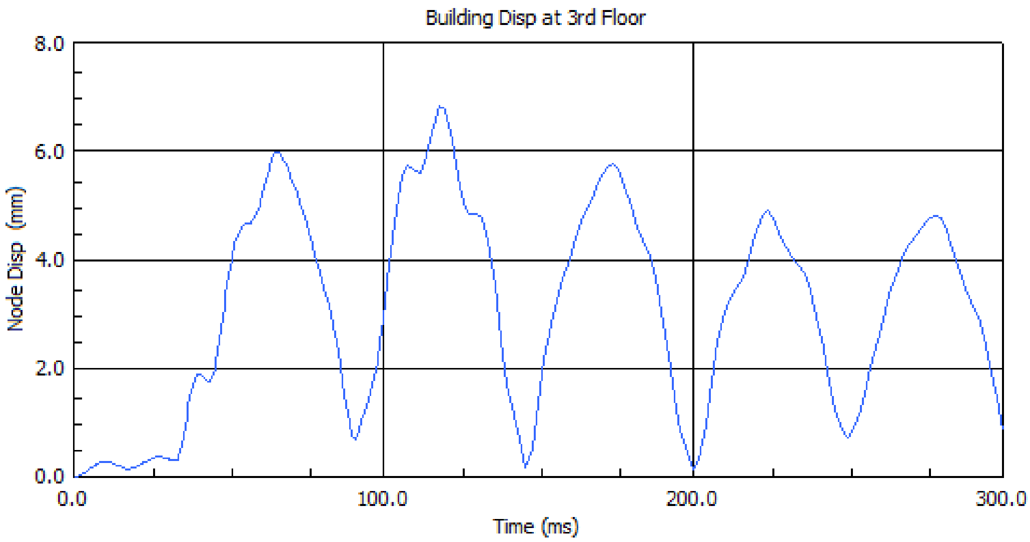


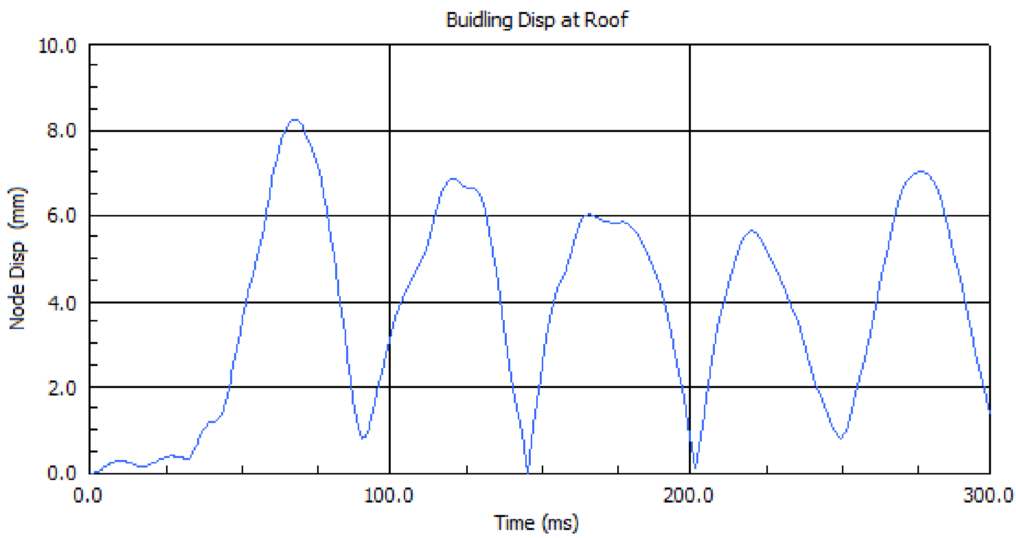
Time step 100ms Time step 200ms

Scenario 2 steel frame building displacements at floor levels contained below.



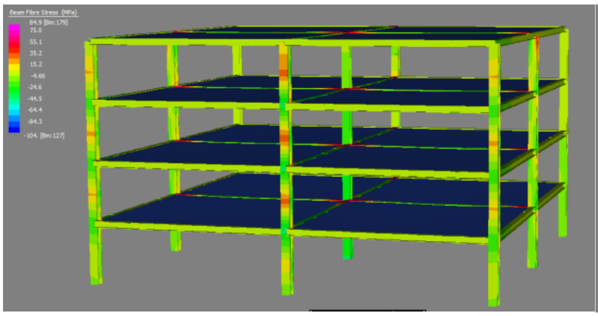

Building Disp at 2nd Floor
6.0
4.0
2.0
0.0
Node Disp (mm)
0.0
100.0
200.0
300.0
Time (ms)


Building Disp at 3rd Floor
8.0
6.0
4.0
2.0
0.0
Node Disp (mm)
0.0
100.0
200.0
300.0
Time (ms)

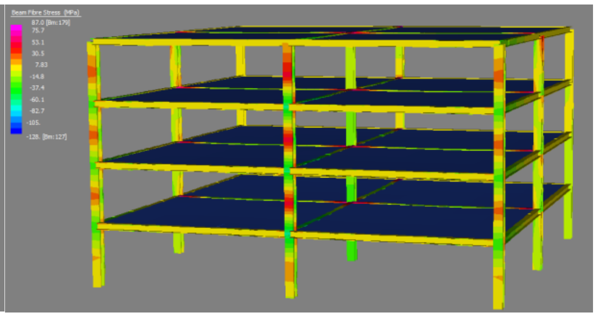

Buidling Disp at Roof
10.0
8.0
6.0
4.0
2.0
0.0
Node Disp (mm)
0.0
100.0
200.0
300.0
Time (ms)

Scenario 3 - Steel Frame Building

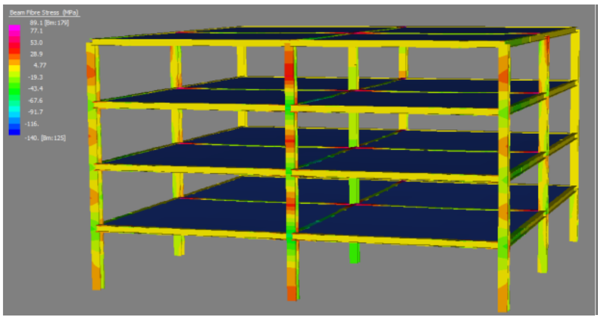
Scenario 2 steel frame building critical stress responses contained below.



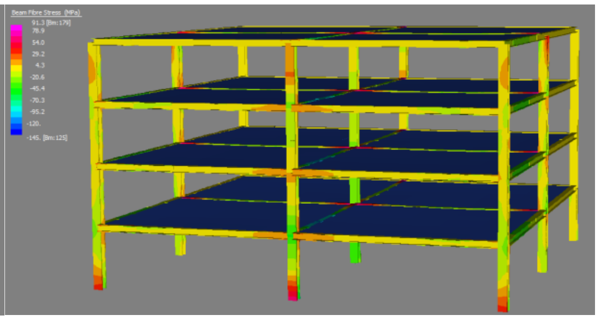
Time step 22.5ms



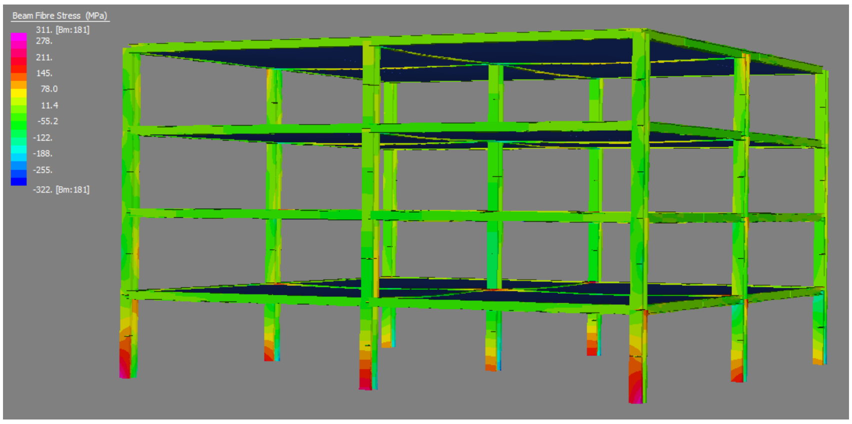
Time step 23ms



Time step 23.5ms

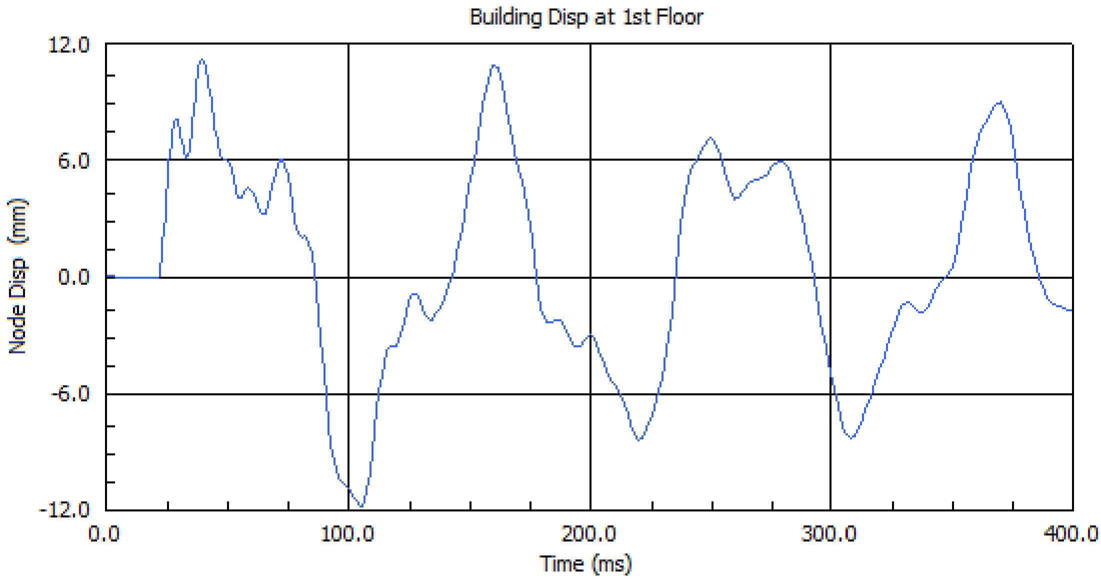


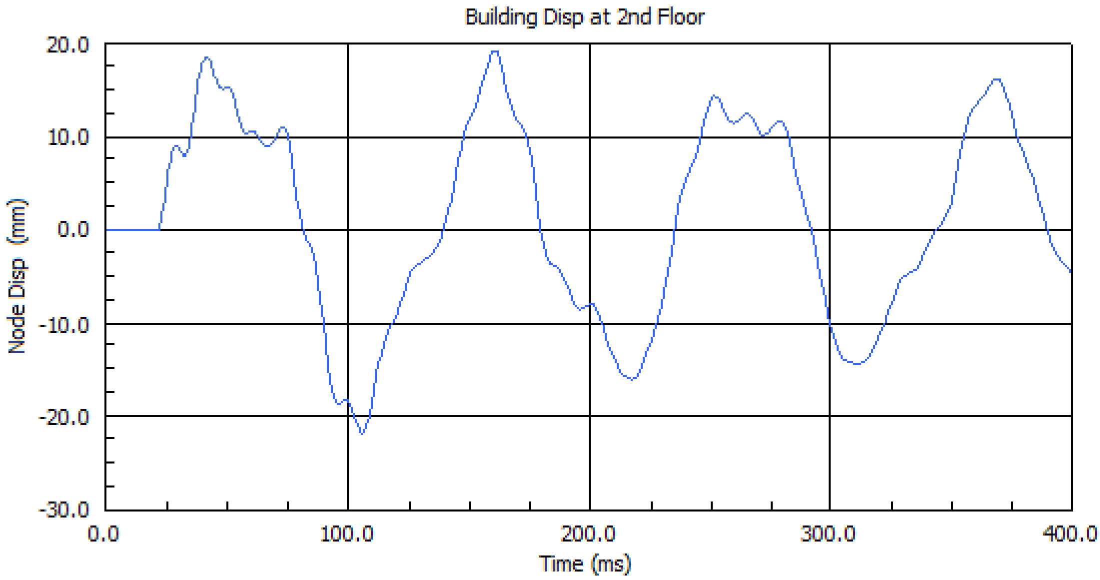
Time step 24ms

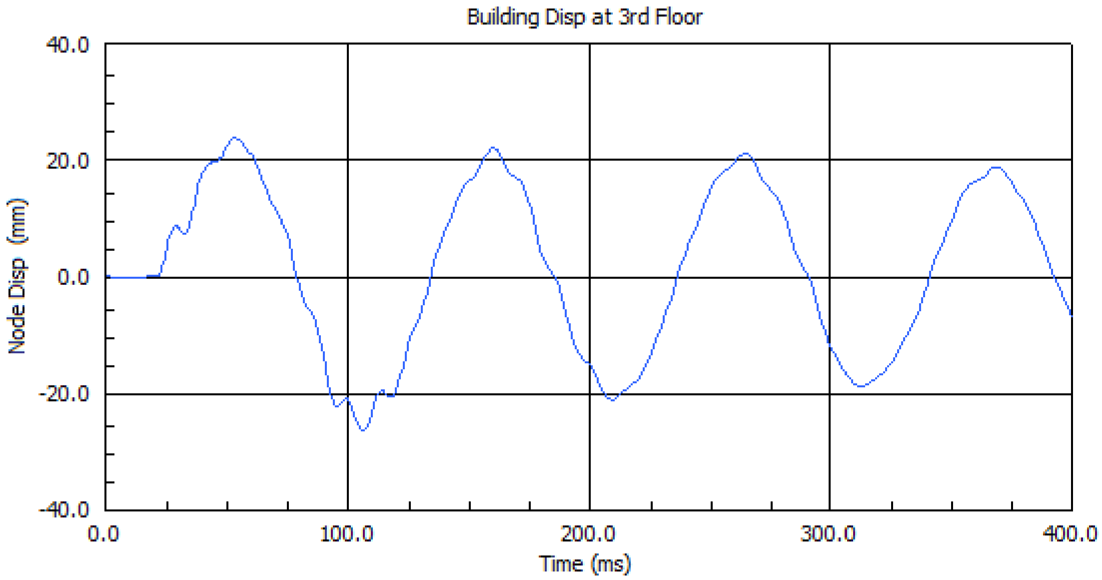


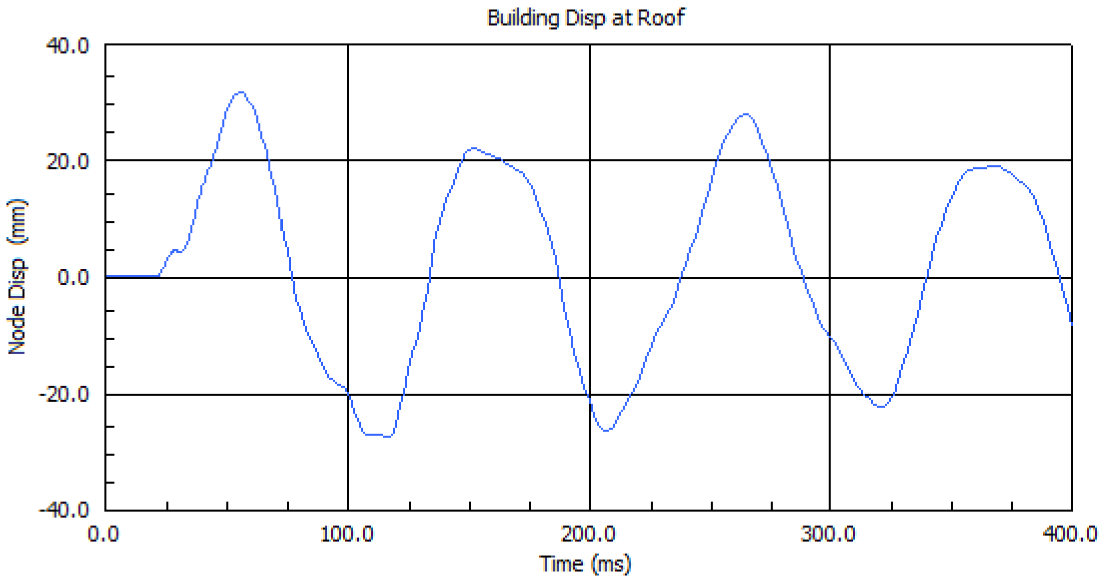
Time step 40ms end of front wall positive phase duration

Scenario 3 steel frame building displacements at floor levels contained below.


Building Disp at 1st Floor
12.0
6.0
0.0
-6.0
-12.0
Node Disp (mm)
0.0
100.0
200.0
300.0
400.0
Time (ms)

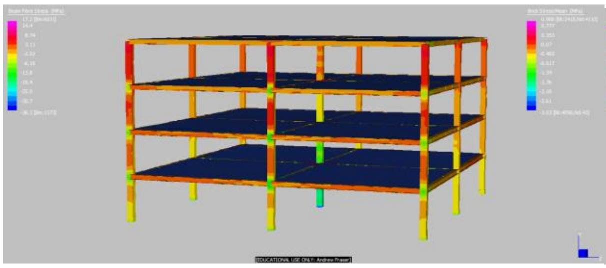

Building Disp at 2nd Floor
20.0
10.0
0.0
-10.0
-20.0
-30.0
Node Disp (mm)
0.0
100.0
200.0
300.0
400.0
Time (ms)

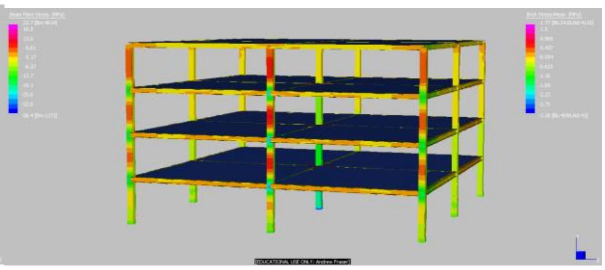

Building Disp at 3rd Floor
40.0
20.0
0.0
-20.0
-40.0
Node Disp (mm)
0.0
100.0
200.0
300.0
400.0
Time (ms)

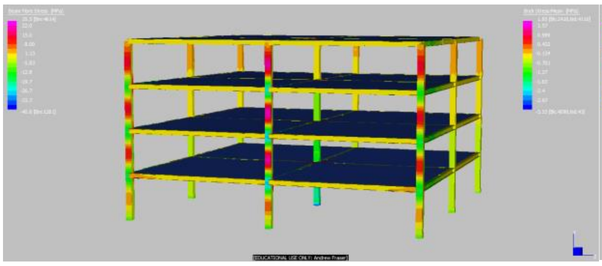

Building Disp at Roof
40.0
20.0
0.0
-20.0
-40.0
Node Disp (mm)
0.0
100.0
200.0
300.0
400.0
Time (ms)

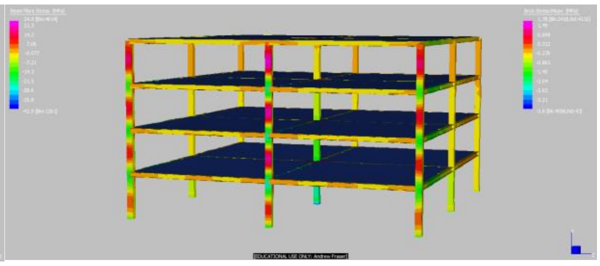
Scenario 1 - Concrete Frame Building

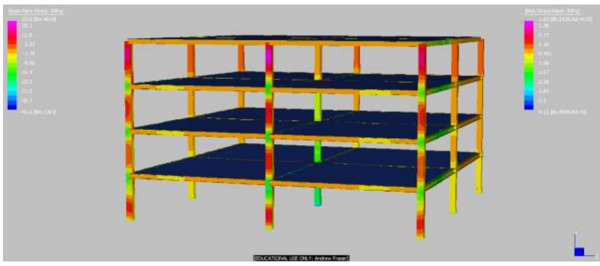
Scenario 1 concrete frame building critical stress responses contained below.

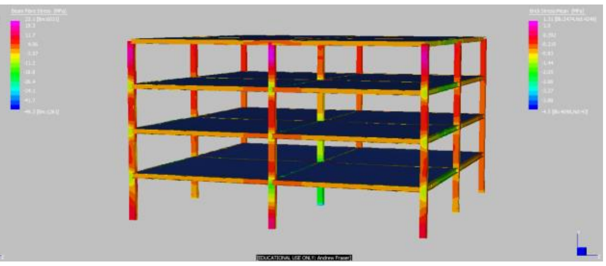

Time step 12.5ms

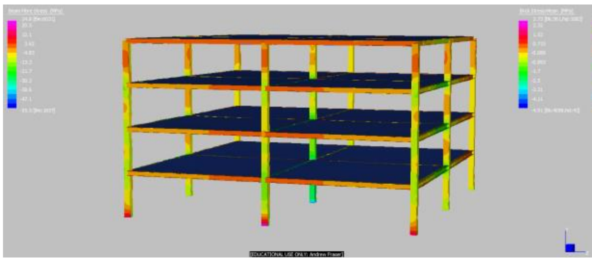

Time step 13ms

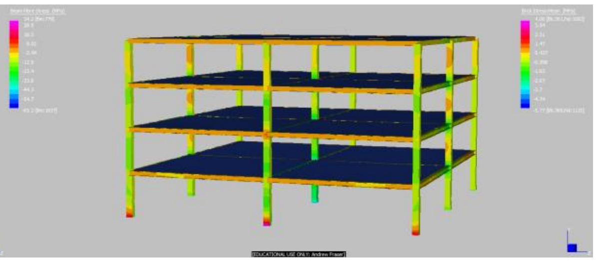

Time step 13.5ms


Time step 14ms

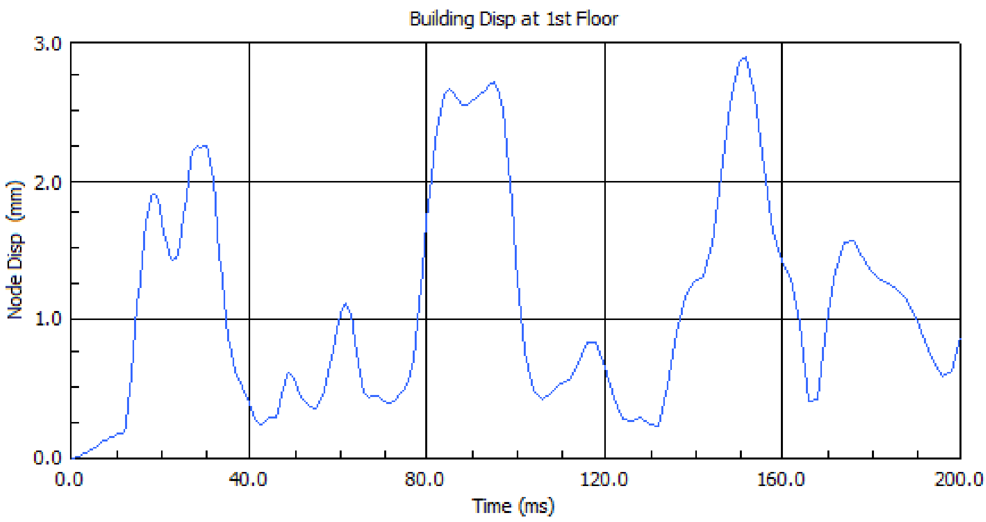

Time step 14.5ms

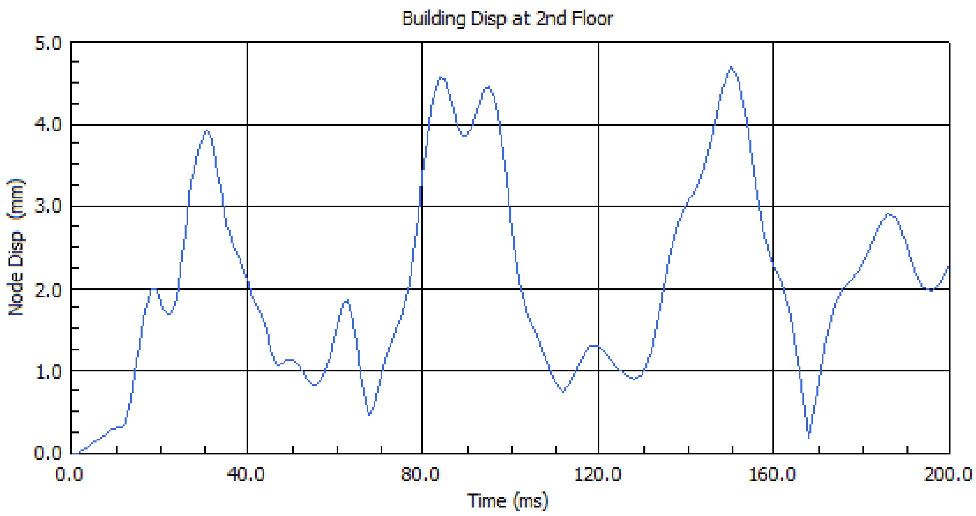

Time step 15ms

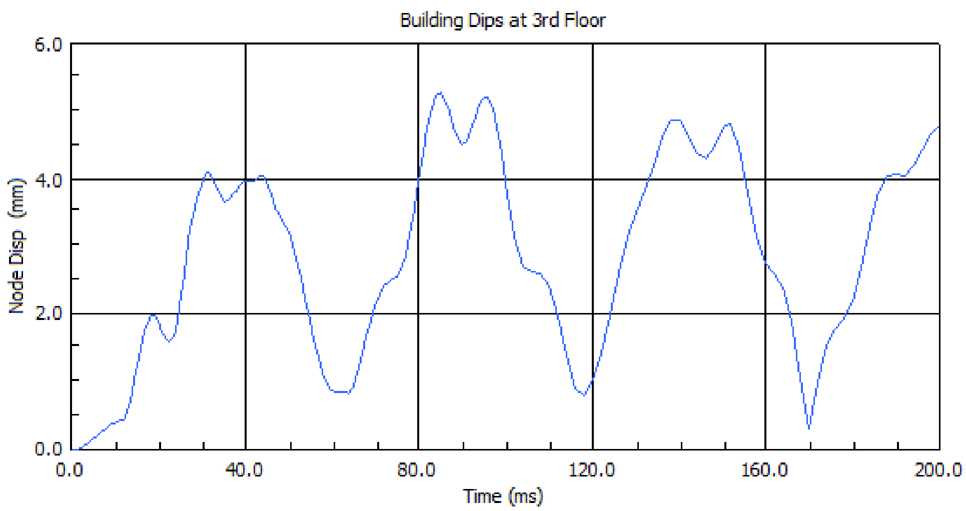

Time step 15.5ms

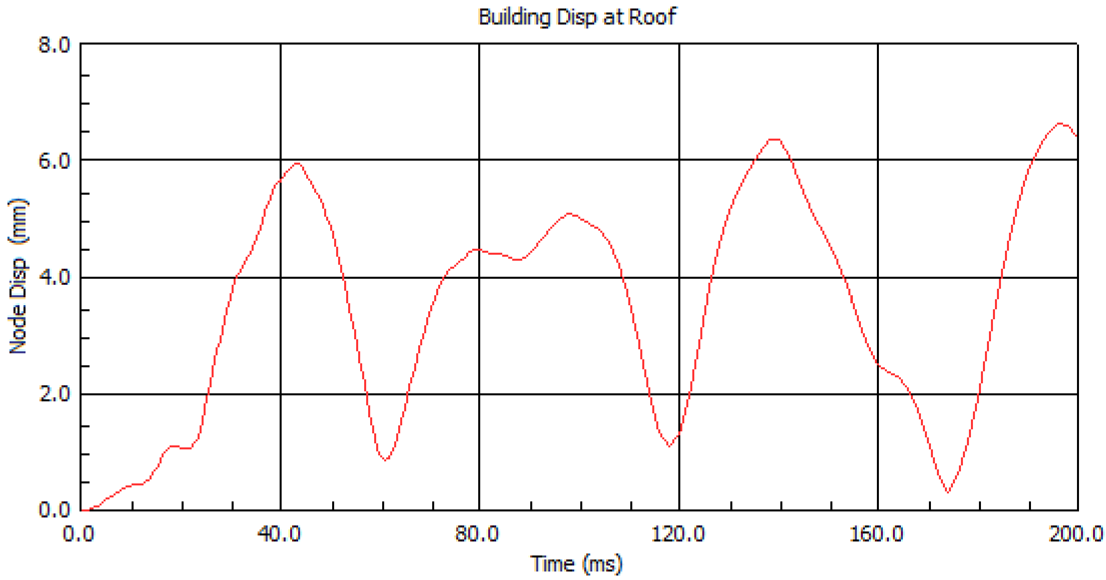

Time step 16ms

Scenario 1 concrete frame building displacements at floor levels contained below.


Building Disp at 1st Floor
Node Disp (mm)
Time (ms)
3.0
2.0
1.0
0.0
0.0
40.0
80.0
120.0
160.0
200.0

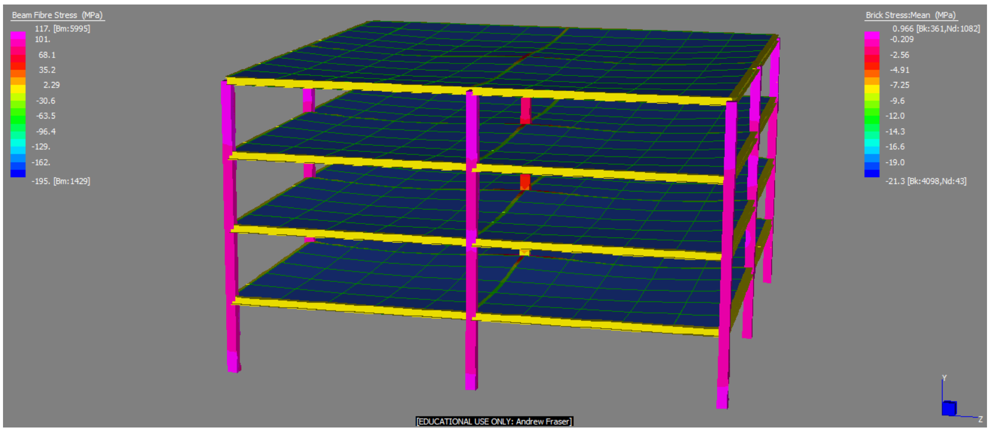

Building Disp at 2nd Floor
Node Disp (mm)
Time (ms)
5.0
4.0
3.0
2.0
1.0
0.0
0.0
40.0
80.0
120.0
160.0
200.0


Building Dips at 3rd Floor
Node Disp (mm)
Time (ms)
6.0
4.0
2.0
0.0
0.0
40.0
80.0
120.0
160.0
200.0


Building Disp at Roof
8.0
6.0
4.0
2.0
0.0
Node Disp (mm)
0.0
40.0
80.0
120.0
160.0
200.0
Time (ms)

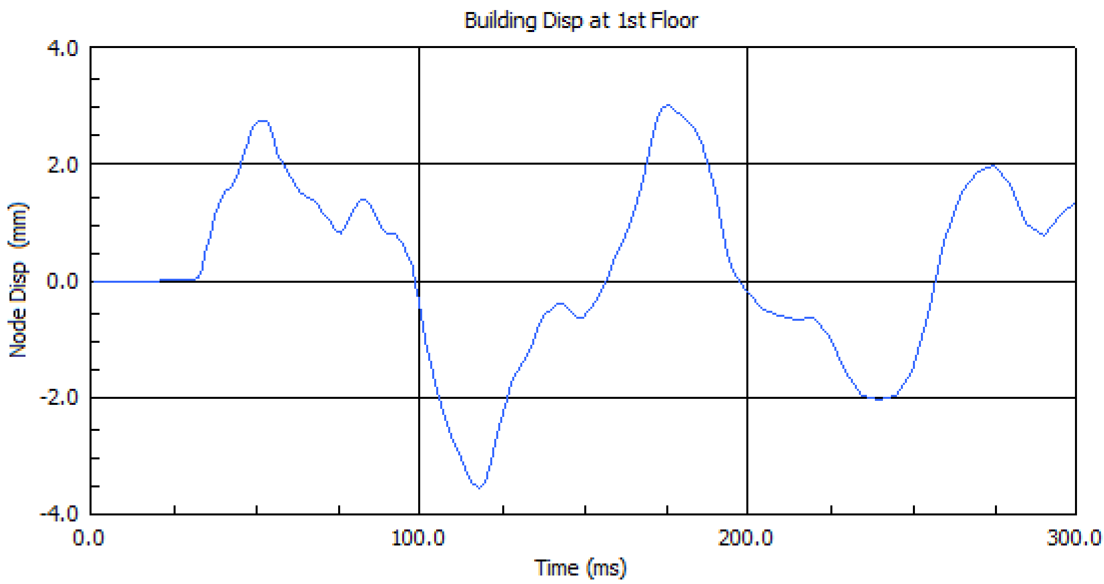
Scenario 2 - Concrete Frame Building

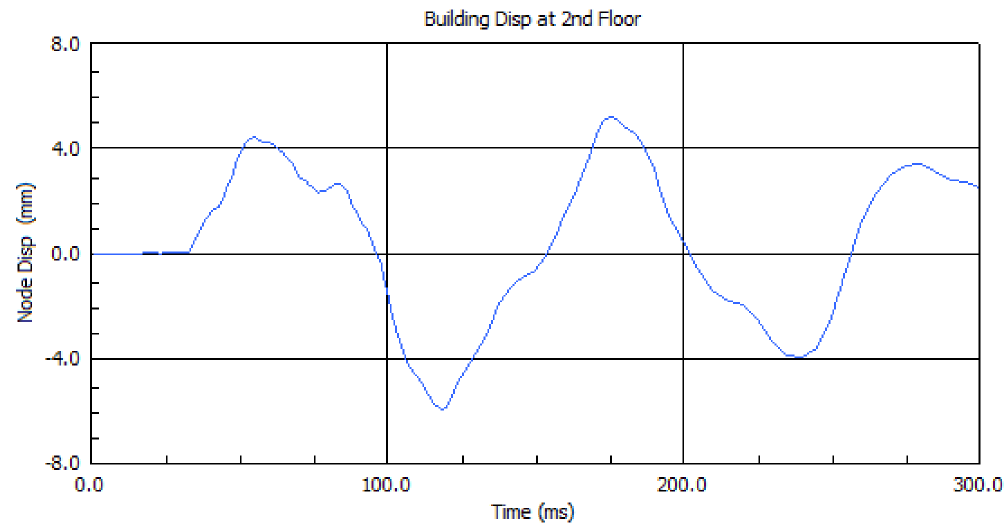
Scenario 2 concrete frame building critical stress responses contained below.

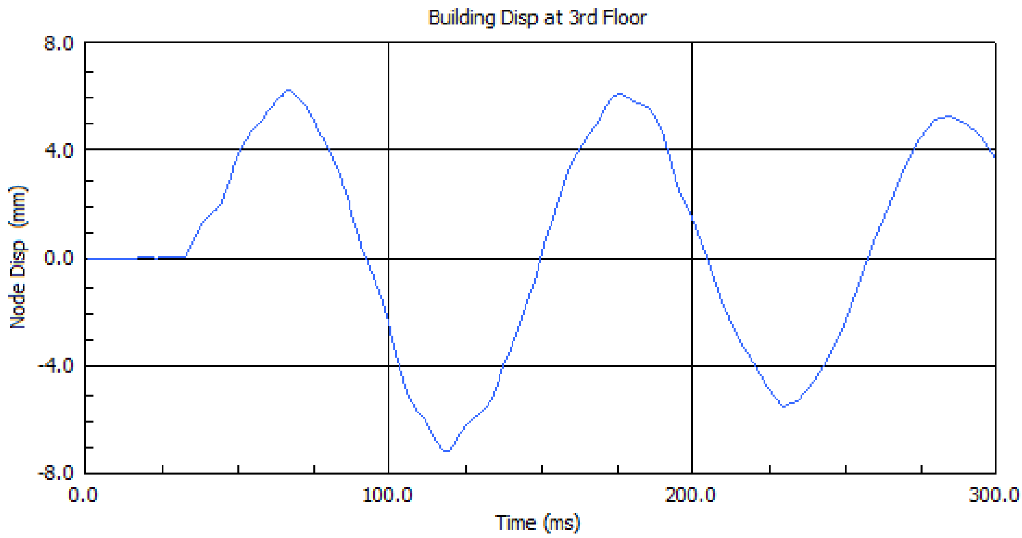


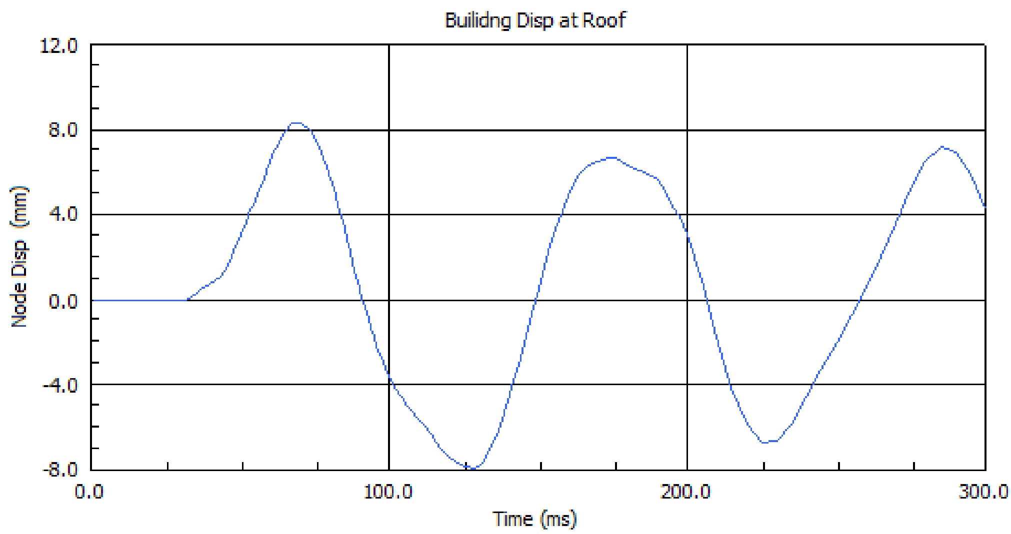
Time step 37ms

Scenario 2 concrete frame building displacements at floor levels contained below.



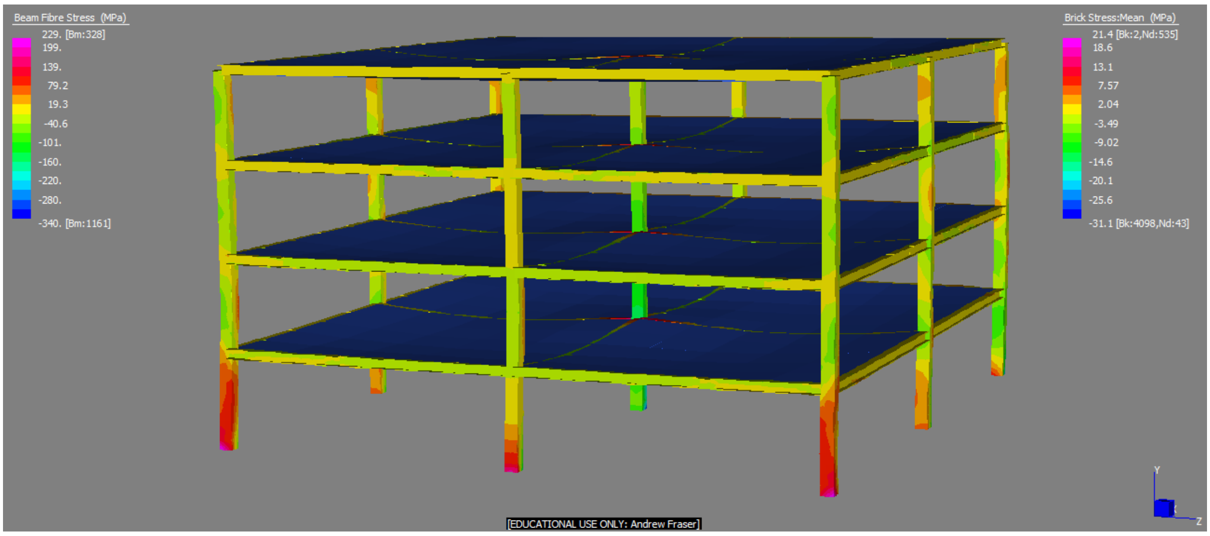

Building Disp at 2nd Floor
8.0
4.0
0.0
-4.0
-8.0
Node Disp (mm)
0.0
100.0
200.0
300.0
Time (ms)


Building Disp at 3rd Floor
8.0
4.0
0.0
-4.0
-8.0
Node Disp (mm)
0.0
100.0
200.0
300.0
Time (ms)


Builidng Disp at Roof
12.0
8.0
4.0
0.0
-4.0
-8.0
Node Disp (mm)
0.0
100.0
200.0
300.0
Time (ms)

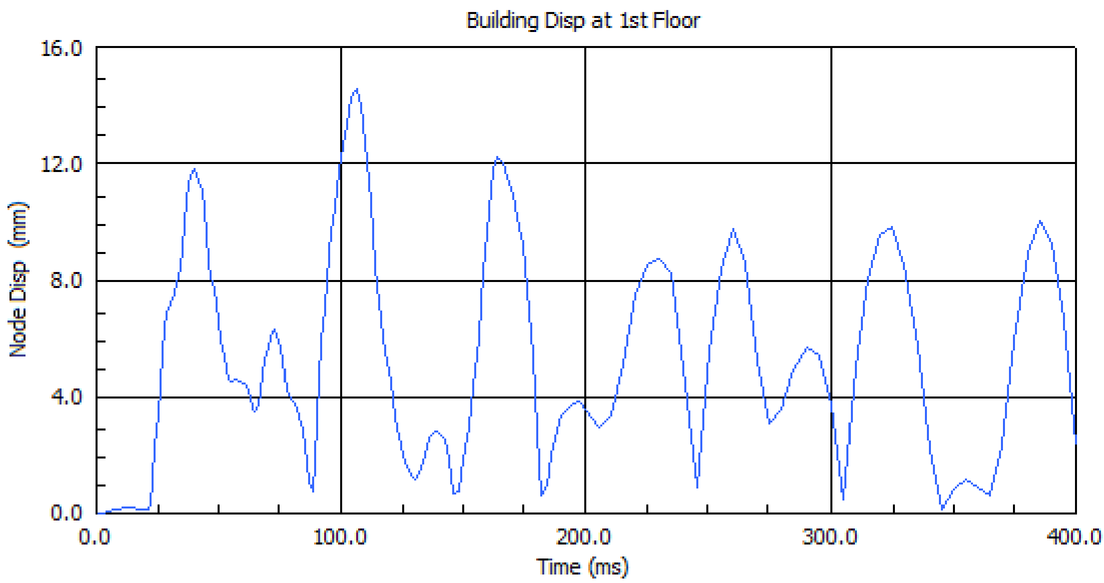
Scenario 3 - Concrete Frame Building

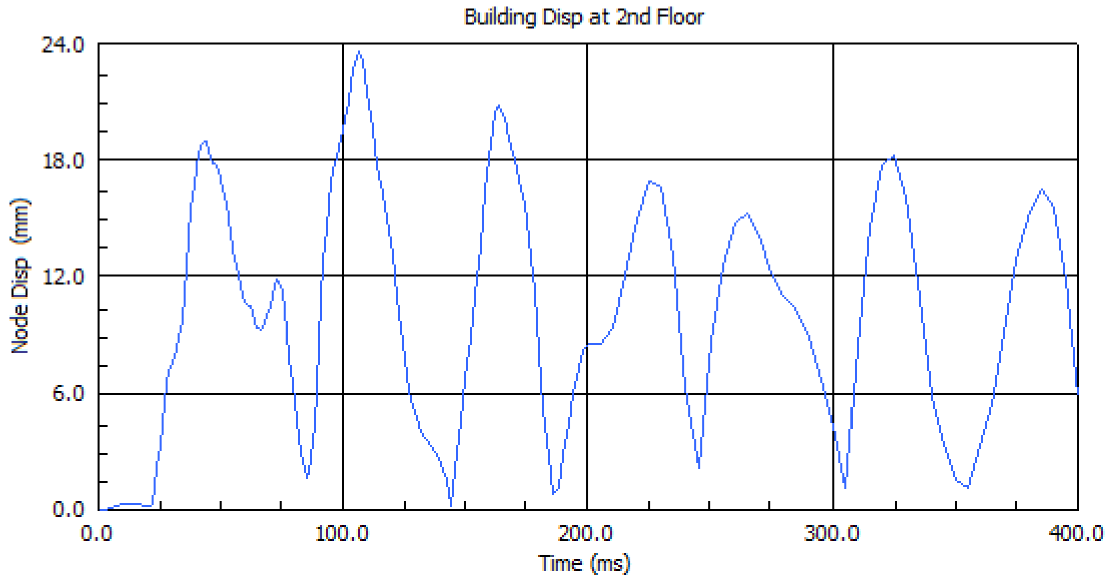
Scenario 2 concrete frame building critical stress responses contained below.

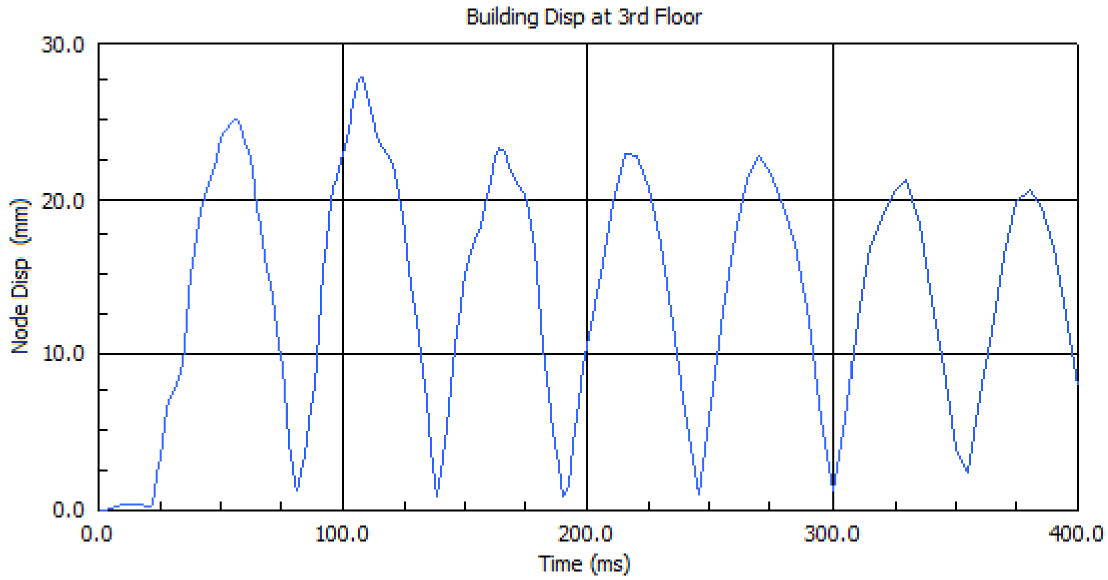


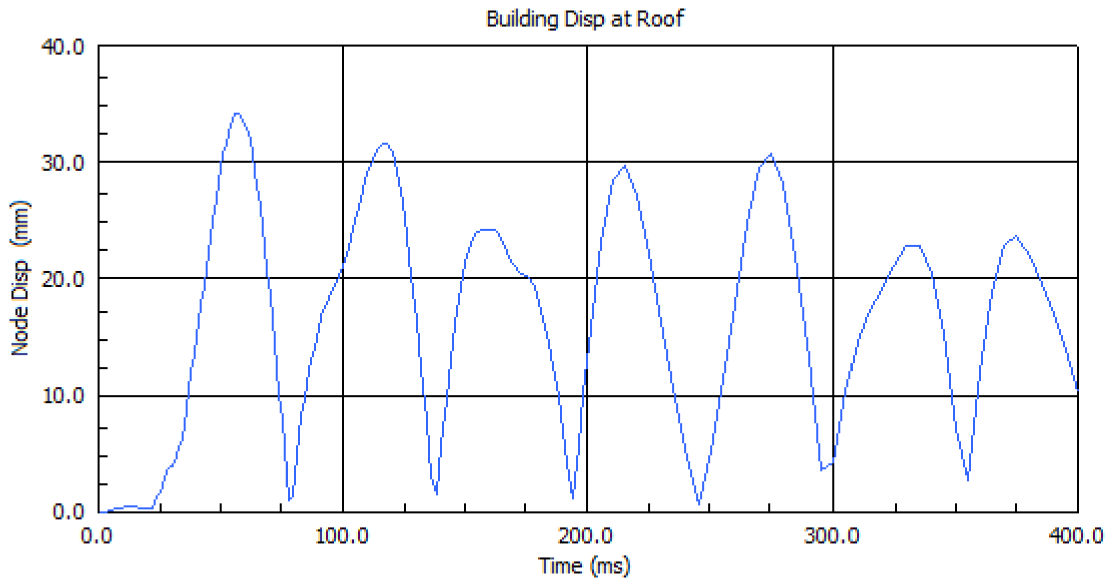
Time step 39ms

Scenario 3 Concrete frame building displacements at floor levels contained below.




Building Disp at 2nd Floor
24.0
18.0
12.0
6.0
0.0
Node Disp (mm)
0.0
100.0
200.0
300.0
400.0
Time (ms)

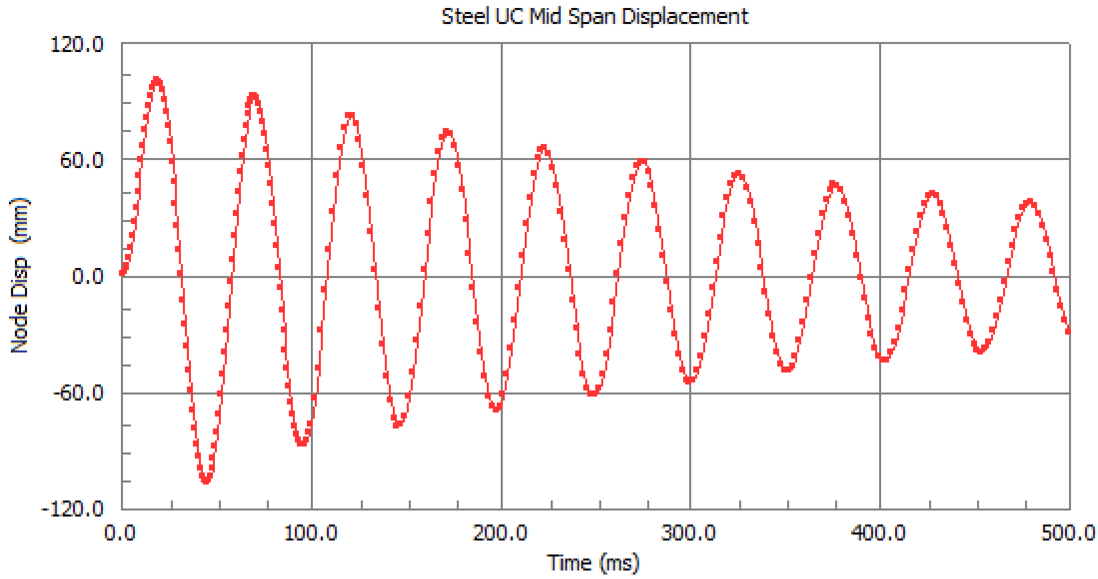

Building Disp at 3rd Floor
30.0
20.0
10.0
0.0
Node Disp (mm)
0.0
100.0
200.0
300.0
400.0
Time (ms)


Building Disp at Roof
40.0
30.0
20.0
10.0
0.0
Node Disp (mm)
0.0
100.0
200.0
300.0
400.0
Time (ms)

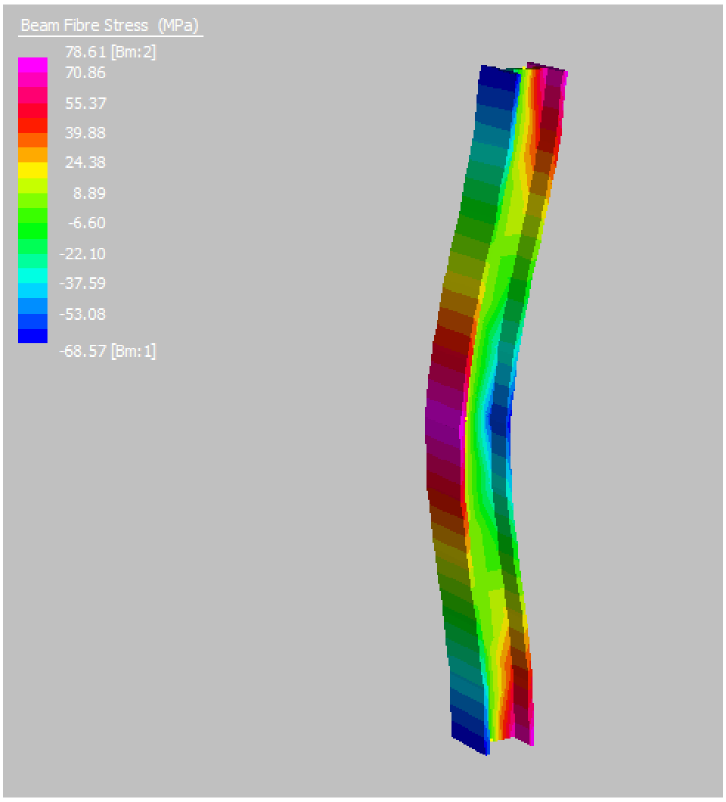
Appendix F – Local blasts effects results

Steel Universal Column (UC)

Steel UC displacements is contained below.



Steel UC peak stress response is contained below.

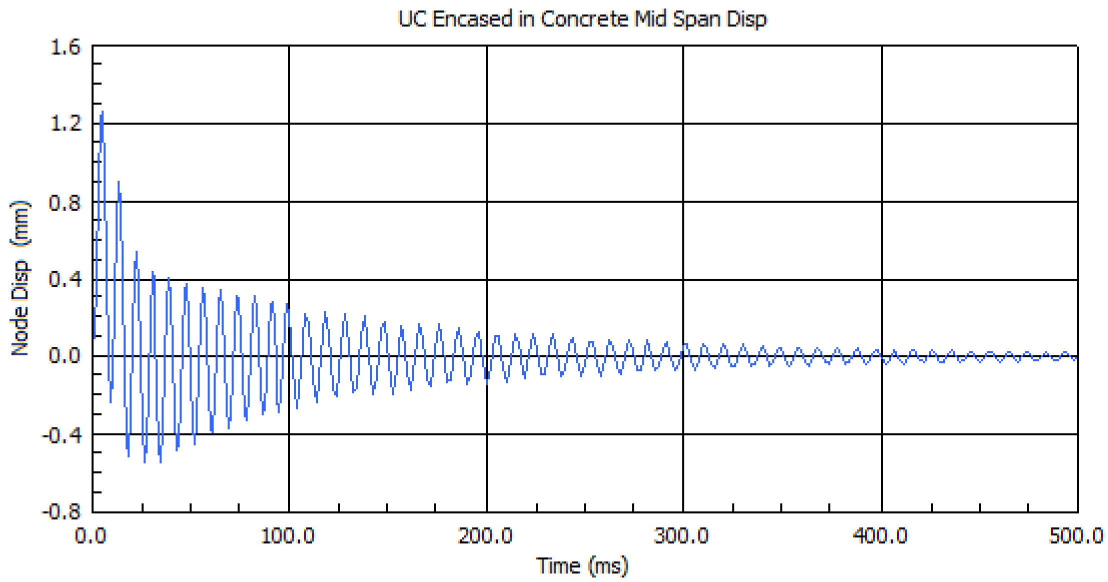


Time step 45ms

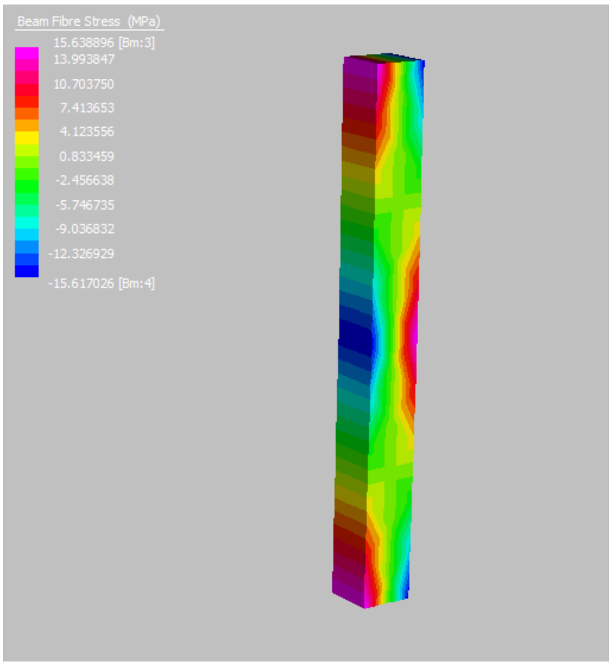
(Peak stress responses: Tensile fibre stress 78.61MPa and Compressive fibre stress 68.57MPa)

Steel Universal Column (UC) encased in concrete

Steel UC encased in concrete displacements is contained below.

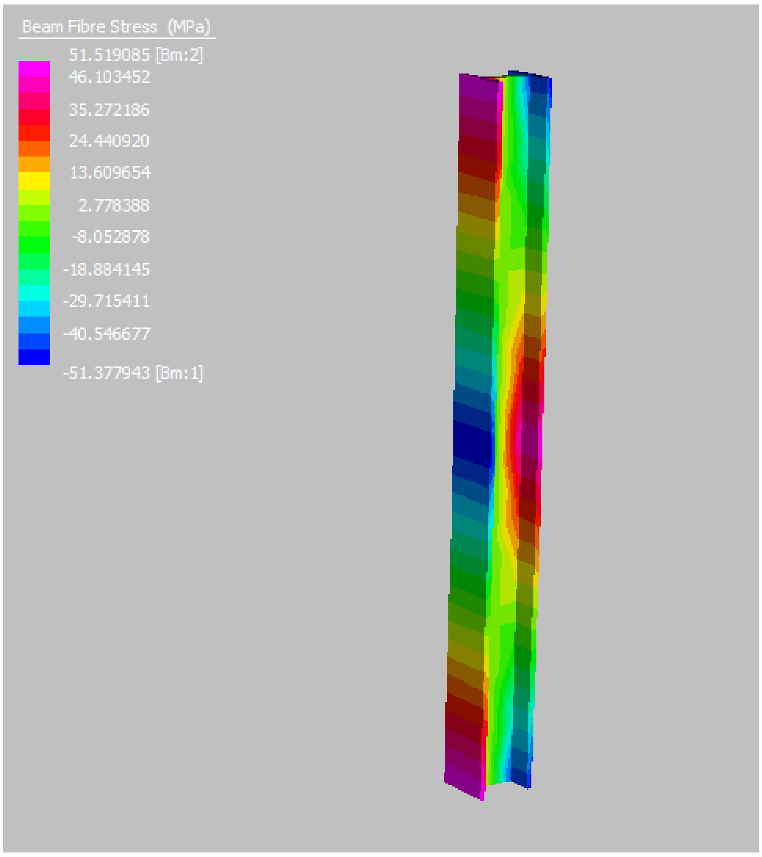


Steel UC encased in concrete peak stress response of elements is contained below.



Time step 5ms (Concrete elements)

(Peak stress responses: Tensile fibre stress 15.61MPa and Compressive fibre stress 15.64MPa)

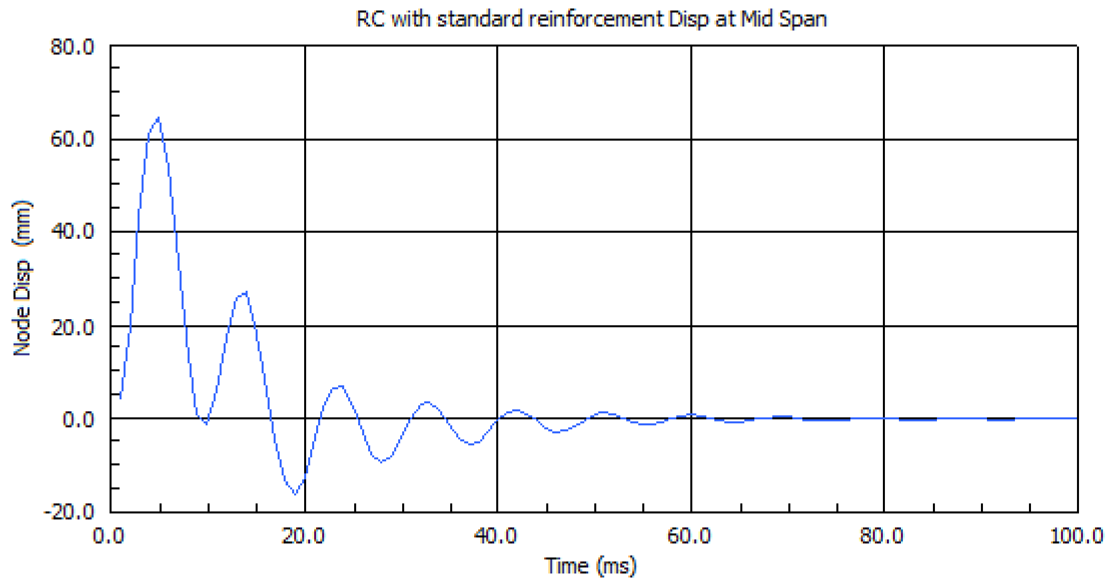


Time step 5ms (Steel UC elements)

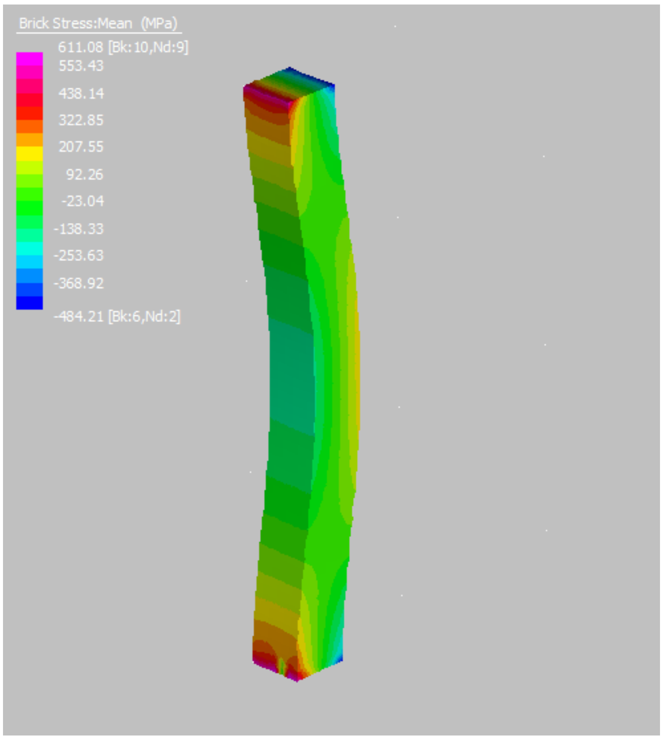
(Peak stress responses: Tensile fibre stress 51.64MPa and Compressive fibre stress 51.4MPa)

Reinforced Concrete (RC) with standard reinforcement

RC with standard reinforcement displacements is contained below.

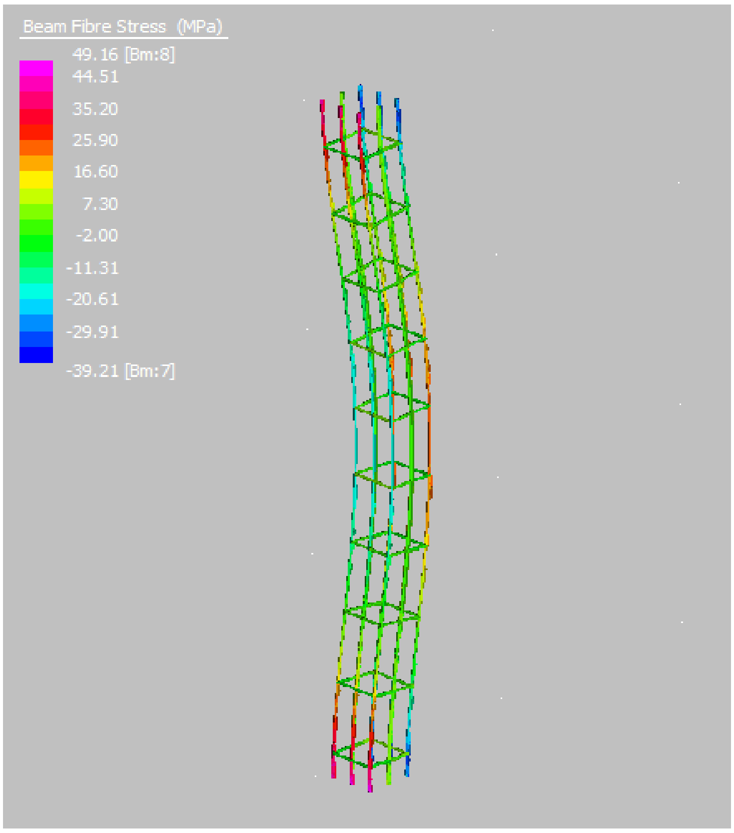


RC with standard reinforcement peak stress response of elements is contained below.



Time step 5ms (Concrete elements)

(Peak stress responses: Tensile fibre stress 383 MPa and Compression fibre stress 320MPa)

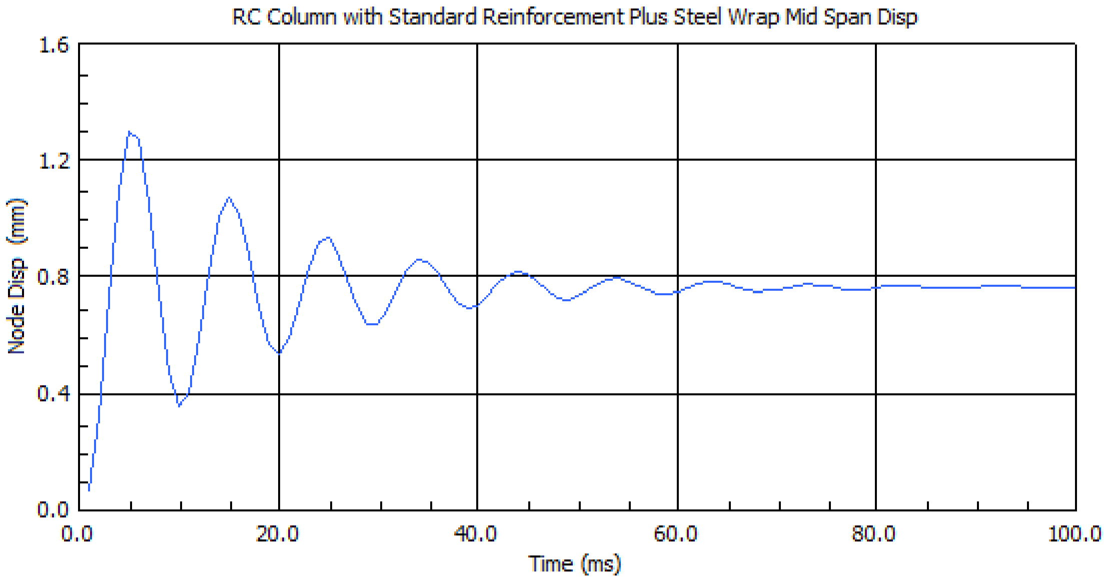


Time step 5ms (steel reinforcement elements)

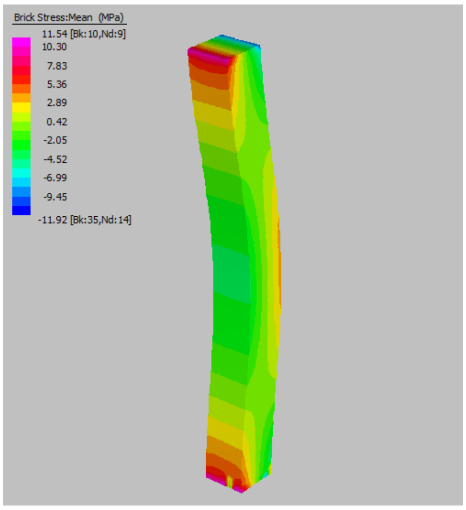
(Peak stress responses: Tensile stress fibre 49.16 MPa and Compression stress fibre 39.2 MPa)

Reinforced Concrete (RC) with standard reinforcement plus steel plate wrap

RC with standard reinforcement plus steel plate wrap displacements is contained below.

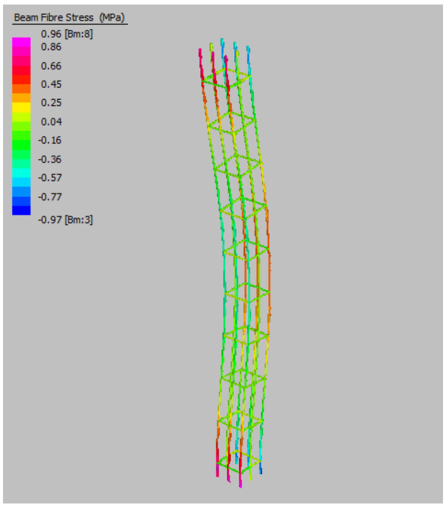


RC with standard reinforcement plus steel plate wrap peak stress response of elements is contained below.



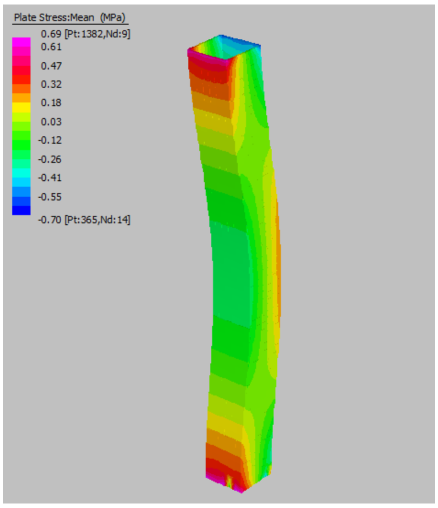
Time step 5ms (Concrete elements)

(Peak stress responses: Tensile stress fibre 7.26 MPa and Compressive stress fibre 7.1 MPa)



Time step 5ms (steel reinforcement elements)

(Peak stress responses: Tensile stress fibre 0.96 MPa Compression stress fibre 0.97 MPa)

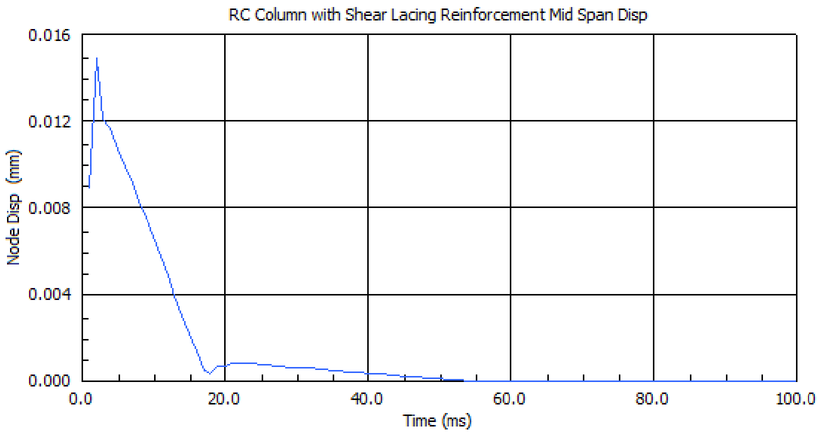


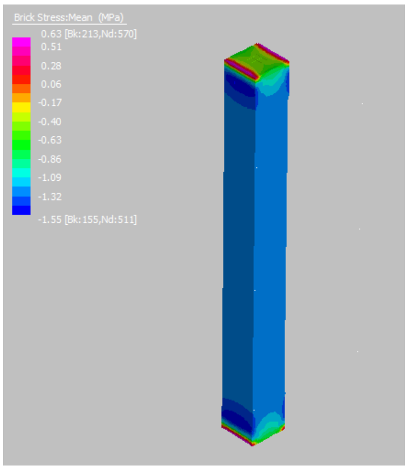
Time step 5ms (steel plate elements)

(Peak stress responses: Tensile stress fibre 0.56 MPa and Compressive stress fibre 0.54 MPa)

Reinforced Concrete (RC) with shear lacing reinforcement

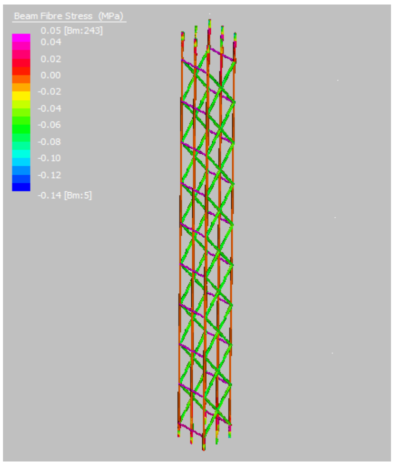
RC with shear lacing reinforcement displacements is contained below.





Time step 2ms (Concrete elements)

(Peak stress responses: Tensile stress fibre 81.8 kPa and Compression stress fibre 1.45 MPa)



Time step 2ms (steel shear lacing reinforcement elements)

(Peak stress responses: Tensile stress fibre 53.1 kPa and Compressive stress fibre 140.7 kPa)