An Energy-Dissipating System for Blast Mitigation in Structures

by

Martin Walker

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Department of Civil Engineering
University of Toronto

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Abstract

The design of buildings for extreme loads has traditionally been conducted on a life safety basis. As a result, buildings often need to be demolished after an extreme event since the cost of repairing the extensive damage is too great. In the field of earthquake engineering, this philosophy is beginning to change. However, buildings subject to blasts continue to be designed with only life safety in mind. For many buildings, especially critical infrastructure, continued operation after an explosive attack is essential. The use of energy-dissipating methods in a componentized system will enable the protection of a structure and occupants from a blast and permit the rapid repair and re-occupation of the building after an explosive attack.

The concept behind the system is the creation of panels that can be used as cladding for a structure. These panels are connected to the main structure using energy-dissipating component assemblies around the panel edge. When subjected to a blast load the panels will deform and transfer the blast pressure through the energy-dissipating component assemblies to the structure. These assemblies limit the peak loads transferred to the main structure allowing it to remain elastic. After an event, the panels and energy-dissipating component assemblies can be replaced quickly and easily allowing the building to be reoccupied in a very short time after an attack.

This study focuses on the characterization of energy-dissipating component assemblies using static and dynamic laboratory testing. From this characterization, a predictive theory, based on a single degree of freedom model, is developed and a general design method proposed. The predictive theory and design method are evaluated in field blast tests.
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Notation

\( \alpha \) Angle of incidence (Degrees)

\( \alpha_1 \) Constant

\( \dot{\varepsilon}^* \) Dimensionless plastic strain rate

\( \varepsilon_D \) Densification strain

\( \varepsilon_p \) Equivalent plastic strain

\( \rho \) Density of metallic foam (kg/m\(^3\))

\( \rho_0 \) Initial element density (kg/m\(^3\))

\( \rho_s \) Density of solid material (kg/m\(^3\))

\( \rho_t \) Element density at time t (kg/m\(^3\))

\( \sigma_f \) Plateau stress (MPa) for metallic foams

\( \sigma_{y,s} \) Yield strength of solid material (MPa)

\( A \) Static yield stress (MPa)

\( b \) Waveform parameter

\( B \) Strain hardening coefficient (MPa)

\( c \) Damping constant

\( C \) Strain-rate hardening coefficient (MPa)

\( C_D \) Drag coefficient

\( C_r \) Sound velocity in reflected region (m/s)

\( C_{\text{ra}} \) Reflected pressure coefficient

\( d \) Constant

\( f_s \) Resistance function

\( H \) Target height (m)

\( i_r \) Positive reflected impulse (kPa-ms)

\( i_s \) Positive specific impulse (kPa-ms)

\( i_{\text{-}} \) Negative specific impulse (kPa-ms)

\( I \) Reflected impulse multiplied by panel area (kN-s)

\( k_i \) Initial stiffness (kN/m)

\( M \) Panel mass (kg)

\( m \) Mass (kg)

\( n \) Strain hardening exponent

\( p \) Number of layers of EDCs

\( P_D \) Drag pressure (kPa)
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$P_s$ Side-on (incident) over-pressure as a function of time (kPa)

$P_{so}$ Peak positive side-on (incident) over-pressure (kPa)

$P_{so}^-$ Peak negative side-on (incident) over-pressure (kPa)

$P(t)$ Imposed load

$q_0$ Peak dynamic pressure (kPa)

$R$ Standoff distance (m)

$R_{pl}$ Plateau load (kN)

$s$ Number of EDCs in series

$t$ Time (ms)

$t_0$ Positive phase duration (s)

$t_0^-$ Negative phase duration (s)

$t_A$ Time of arrival of blast wave (s)

$t_c$ Clearing time (ms)

$t_{of}$ Fictitious positive phase duration (ms)

$t_{rf}$ Fictitious reflected pressure duration (ms)

$T$ Natural period of SDOF system (s$^{-1}$)

$T$ Temperature (K)

$T^*$ Homologous temperature

$T_{melt}$ Melt temperature (K)

$T_{room}$ Room temperature (K)

$U$ Shock front velocity (m/s)

$V_0$ Initial element volume (m$^3$)

$V_t$ Element volume at time t (m$^3$)

$X_{el}$ Displacement at beginning of plateau (mm)

$X_{max}$ Maximum displacement of SDOF system (mm)

$W$ Weight of charge (kg of TNT)

$w$ Target width (m)

$Z$ Scaled distance (kg/m$^{1/3}$)
1 Introduction

Civil structures are threatened by extreme loads due to natural or man-made hazards. As engineers became more adept at designing buildings to withstand these loads our built environment has become an increasingly critical component of our day-to-day lives. Our society relies on civil structures to shelter us from harm and support us in times of emergency. The recognition of this fact and the realization that structures must be designed to guarantee this capability has led to resilience becoming an increasingly common design requirement.

Resilience is the ability to quickly recover full functionality after the occurrence of an extreme event. Resilient structures are able to not only ensure the safety of occupants but also continue to support their activities after an extreme event. There are three components to a resilient structure: reduced probability of failure, reduced failure consequences, and reduced time for recovery. By addressing these components, the impact of an event can be greatly reduced and our infrastructure can continue to support communities in times of need.

1.1 Motivation

In addition to the usual gravity and wind loads, modern building codes require structures to be designed to resist extreme loads, such as earthquakes, hurricanes, or explosions. Due to the rarity of these events, the design of buildings for extreme loads has typically been conducted on a life safety basis. Therefore, if the occupants of a structure are able to exit the building safely after an extreme event, the continued operation of the building is not important. Often buildings must be demolished after an extreme event since the cost of repairing the extensive damage is too great, leaving the occupants without a home or workplace. In the field of earthquake engineering, this philosophy is beginning to change. However, buildings subject to explosions continue to be designed with life safety in mind. This philosophy is not acceptable for many buildings that form a region's critical infrastructure. Structures such as power plants, water treatment works, and hospitals can be the target of malicious explosive attacks simply due to the disproportionate effects that the failure, even if temporary, of one of these facilities has on the community it supports.

There has been a sharp increase in global terrorism incidents involving explosions over the last ten years, as shown in Figure 1-1.
This has led to a renewed interest in blast-resistant structures for civilian and military uses. For many buildings, especially critical infrastructure, continued operation after an explosive attack is essential. Traditional methods of hardening structures are often expensive to design and build and impractical for the retrofit of existing structures. The use of energy dissipating methods in a componentized system will enable the protection of occupants and contents from a blast and permit the rapid repair and re-occupation of the building after an explosive attack.

1.2 Scope

This thesis focuses on the characterization of a componentized energy-dissipating system for blast mitigation. Dynamic laboratory tests and finite element analysis were used to characterize this system. From this characterization, a predictive theory was developed and a general design method proposed. This theory and design method was then validated through full-scale blast tests.
2 Background

Although explosives have been used for hundreds of years, the comprehensive study of explosions and their mitigation only seriously began after the end of the Second World War. Due to the Cold War, these early investigations focused primarily on the effects of nuclear weapons and their mitigation. One of the first quantitative guidance documents for structures to resist the effects of blasts was published in 1969 by the US Army (United States Army, 1969). This document (TM5-1300), which was targeted towards military facilities, has undergone several revisions and is now available as UFC 3-340-02 (2008). UFC 3-340-02 provides planning, design, construction, sustainment, restoration, and modernization criteria for military departments and defence agencies. For designers of civilian structures, it is currently the main guide for design and analysis of structures subjected to blast loads. Other guidance documents have been developed for military use; however, access to these documents is restricted to official use.

The first design standard to be published for non-military use was ASCE/SEI 59-11: Blast Protection of Buildings (2011). This voluntary standard presents the current practice in blast protection for civilian structures. The Canadian Standards Association (CSA) also recently released its first edition of a design standard for blast loading of buildings: CSA S850-12 Design and Assessment of Buildings Subjected to Blast Loads (2012).

2.1 The Physics of Blasts

A blast can be the result of chemical, nuclear, or physical reactions. Chemical blasts are caused by the rapid oxidation of a fuel. Nuclear blasts are caused by the release of energy from the splitting (fission) or joining (fusion) of atoms. Physical blasts are caused by the sudden release of stored mechanical energy, such as the catastrophic failure of a pressure vessel. This thesis will focus on chemical explosives, but the same principles can be applied to other types of blasts.

The explosion of a chemical explosive produces gasses at high temperature and pressure. This process can be seen from the general oxidation reaction that occurs in chemical explosives:

\[ C_cH_hN_nS_sO_o + \left[ c + \frac{h}{4} + \frac{o}{4} + s \right] O_2 \Rightarrow c \ CO_2 + \left( \frac{h}{2} \right) H_2O + \left( \frac{n}{2} \right) N_2 + s \ SO_2 + Q \]

where c, h, o, and s are the numbers of each element in the compound and Q is the heat of combustion (Krauthammer, 2008). For example, TNT has: c=7, h=5, n=3, s=0, o=6. The explosive is oxidized with oxygen which can be from the atmosphere or part of the explosive fuel to form carbon dioxide (CO\textsubscript{2}), water (H\textsubscript{2}O), nitrogen gas (N\textsubscript{2}), and sulphur dioxide (SO\textsubscript{2}). In order for the explosive to reach its full
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potential, sufficient oxygen must be available for the reaction. This can be achieved by the inclusion of some form of chemical oxygen source, or by combining an oxygen deficient explosive with one which has an oxygen surplus.

The reaction progresses through the explosive material from the initiation point at the reaction front velocity, which depends on various factors such as the explosive material, fuel-oxygen mix, pressure, and temperature. If this velocity is much slower than the speed of sound in the explosive, the explosion is referred to as a deflagration. In a deflagration, the reaction process propagates through the explosive by thermal conduction (heating). When the reaction front velocity is greater than the speed of sound in the explosive, the explosion is referred to as a detonation. In a detonation a shock wave, characterized by large pressure and temperature gradients, is produced. Unlike a deflagration, the reaction occurs virtually instantaneously as the shock wave travels through the explosive. For TNT the reaction velocity of this shock wave is about 7km/s (Krauthammer, 2008). Deflagrations produce pressure pulses of smaller amplitude and longer duration than detonations.

The reaction generates hot gasses at high pressures and temperatures. The expansion of these gasses compresses the layer of air (or other medium) surrounding the explosive causing it to expand outwards due to the imbalance of pressure. This layer is called the blast wave.

2.2 Blast Waves in Air

The rise time across the blast wave front is essentially zero, resulting in the pressure changing from atmospheric pressure to a peak pressure almost instantaneously. The pressure decreases to atmospheric pressure over a positive phase duration \( t_0 \). This is followed by a negative phase with a duration of \( t_0 \). This negative phase is due to the blast wave momentum causing the gasses to over expand, necessitating a period of negative pressure in order to restore equilibrium. This negative phase is often ignored for design purposes since its magnitude is significantly smaller than the peak positive side-on (incident) pressure \( P_{so} \). However, in certain cases the negative phase can be important, for example, when a structure or element's natural frequency results in a rebound during the negative phase. Figure 2-1 shows the pressure-time history of a fixed point some distance away from the explosion centre.
Figure 2-1  Pressure-time history of a blast (adapted from UFC 3-340-02, 2008)

The blast wave travels at a velocity that decreases with increasing distance from the charge but is usually greater than the speed of sound.

The blast wave also accelerates gas (air) particles, which move at the particle velocity, which is lower than the blast wave velocity. This is often referred to as the blast "wind" and can cause additional drag loading (dynamic pressure) on structures that are engulfed by the blast wave. This dynamic pressure is dependent on the air density and particle velocity. The dynamic pressure has been experimentally correlated to the peak side-on pressure (Figure 2-7).

The area under the positive phase region pressure-time history is the positive specific impulse ($i_s$). The positive specific impulse can be calculated according to Equation 2-1.

\[
i_s = \int P_s(t) \, dt
\]  \hspace{1cm} 2-1

The negative specific impulse ($i_{-s}$) is the area under the negative phase region pressure-time history and is calculated in the same way as the positive specific impulse. The pressure-time history can be approximated using the modified Friedlander equation (Cormie et al., 2009):

\[
P_s(t) = P_{so} \left[ 1 - \frac{t}{t_0} \right] e^{-\frac{bt}{t_0}}
\]  \hspace{1cm} 2-2

where $b$ is a waveform parameter. The peak side-on over-pressure ($P_{so}$) can be calculated using Equations 2-3.
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\[ P_{so} = \frac{6.7}{Z^3} + 1 \text{ bar} \quad (P_s > 10 \text{ bar}) \] \hspace{1cm} 2-3

\[ P_{so} = \frac{0.975}{Z} + \frac{1.455}{Z^2} + \frac{5.85}{Z^3} - 0.019 \text{ bar} \quad (0.1 < P_s < 10 \text{ bar}) \]

where \( Z \) is the scaled distance (Brode, 1955). The commonly used Hopkinson-Cranz (cubed-root) scaled distance depends on both the standoff distance and the equivalent weight of TNT for the explosive in question, as shown in Equation 2-4,

\[ Z = \frac{R}{W^{1/3}} \] \hspace{1cm} 2-4

where \( R \) is the standoff distance (in metres) and \( W \) is the mass of the explosive in equivalent kilograms of TNT.

Blast scaling is commonly used in practice to relate a particular charge weight and standoff configuration to previous tests conducted with different charge weights and standoff distances. The Hopkinson-Cranz scaling law has been shown to be accurate through many tests conducted over a wide range of charge weights and standoff distances. However, there is a lack of experimental validation at small scaled distances (Conrath et al., 1999). Huffington and Ewing (1985) have also shown that Hopkinson-Cranz scaling may not be applicable for scaled distances less than 0.16 m/kg\(^{1/3}\).

Hopkinson-Cranz scaling was only derived for spherical TNT charges. However, blast characteristics from all high explosives are very similar; therefore, Hopkinson-Cranz scaling is used for other explosives when they are converted to an equivalent weight of TNT.

2.2.1 TNT Equivalence

In order to standardize blast calculations an equivalent weight of TNT is used. By using a standardized explosive weight, a large quantity of experimental results and predictive models can be applied to different types of explosives. However, there is no standard method of determining this equivalent weight for a given explosive (Cormie et al., 2009). One method of determining the equivalent weight of TNT is to compare the heat of detonation of an explosive of interest to that of TNT according to Equation 2-5,

\[ W_{TNT} = \left( \frac{\Delta H_{exp}}{\Delta H_{TNT}} \right) W_{exp} \] \hspace{1cm} 2-5

where \( \Delta H_{exp} \) and \( \Delta H_{TNT} \) are the heats of detonation of the explosive and TNT respectively, \( W_{exp} \) is the weight of the explosive of interest and \( W_{TNT} \) is the equivalent weight of TNT. These heats of detonation
can be computed from the oxidation reaction of the explosives in question. Alternatively, many tests have been done to experimentally determine the TNT equivalency of various explosives. The values in Table 2-1 were determined from experimental measurements and may be more accurate than Equation 2-5 (Conrath et al., 1999). The TNT equivalency is often different for pressure and impulse, in practice these values are typically averaged to give a single TNT equivalency factor.

<table>
<thead>
<tr>
<th>Explosive</th>
<th>Pressure</th>
<th>Impulse</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANFO</td>
<td>0.82</td>
<td>-</td>
</tr>
<tr>
<td>C-4</td>
<td>1.37</td>
<td>1.3</td>
</tr>
<tr>
<td>Composition B</td>
<td>1.20</td>
<td>1.3</td>
</tr>
<tr>
<td>HBX-1</td>
<td>1.17</td>
<td>1.16</td>
</tr>
<tr>
<td>Picratol</td>
<td>0.90</td>
<td>0.93</td>
</tr>
</tbody>
</table>

Determining the TNT equivalence of improvised explosives can be particularly difficult, due to the lack of quality control in their manufacture (Cormie et al., 2009). If no information about the TNT equivalency is available, a value of 1.3 can provide a conservative estimate for most explosives (Conrath et al., 1999).

### 2.2.2 Reflected Blast Waves

When a blast wave encounters an obstacle that is not parallel to the motion of the blast wave it will reflect. The pressure-time history of the reflected wave is similar to that of the incident wave, however, the peak pressure and impulse is higher as shown in Figure 2-2. The reflected pressure and impulse are also dependent on the angle of incidence of the blast wave on the reflecting surface.

![Figure 2-2](adapted from UFC 3-340-02, 2008)
The peak reflected pressure \( (P_r) \) can be related to the peak side-on pressure by a reflected pressure coefficient \( (C_{r\alpha}) \) according to Equation 2-6 (UFC 3-340-02, 2008):

\[
P_r = C_{r\alpha}P_{so}
\]

2-6

The reflected pressure coefficient can range between about 1.5 and 12 depending on the angle of incidence and the peak side-on pressure. The angle of incidence \( (\alpha) \) is measured from the minimum distance line between the charge and the reflecting surface and the line connecting the centre of the charge to the point of interest, as shown in Figure 2-3.

**Figure 2-3**  Measuring the angle of incidence

The magnitude of \( C_{r\alpha} \) can be obtained from Figure 2-4 given the peak side-on pressure and the angle of incidence, while the magnitude of the peak reflected impulse can be determined from Figure 2-5.

**Figure 2-4**  Reflected pressure coefficients (UFC 3-340-02, 2008)
When an explosion occurs on the ground, the blast waves propagate in a hemispherical shape due to the instantaneous reflections from the ground plane. The hemispherical blast wave parameters are similar to those of free air explosions, although generally higher in magnitude due to the reflection from the ground surface. The different parameters for the blast wave can be determined using plots derived from experimental data. These plots are typically normalized based on the scaled distance from the target. For hemispherical explosions, the parameters can be obtained using Figure 2-6.
The peak dynamic pressure can be determined from the peak side-on pressure using Figure 2-7.
Blasts that occur in more complex situations, such as enclosed or confined spaces, can have substantially different behaviour. In these situations, the use of a hydrocode would be required in order to assess the blast wave behaviour and loading.

2.3 Blast Loading on Structures

The reflected blast parameters assume an infinite reflecting surface and therefore do not allow diffraction to occur. For relatively small targets subjected to large blasts, drag and clearing effects are important.

Drag forces are caused by the blast wave moving around an object. The drag pressure can be related to the peak dynamic pressure according to Equation 2-7 (Smith & Hetherington, 1994):

\[ P_D = C_D q_0 \]  

The drag coefficient is dependent on the shape of the target and can be obtained for simple geometries from Table 2-2.
Table 2-2 Drag coefficients (UFC 3-340-02, 2008)

<table>
<thead>
<tr>
<th>SHAPE</th>
<th>SKETCH</th>
<th>$C_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIRCULAR CYLINDER (LONG ROD), SIDE-ON</td>
<td>FLOW</td>
<td>1.20</td>
</tr>
<tr>
<td>SPHERE</td>
<td>FLOW</td>
<td>0.47</td>
</tr>
<tr>
<td>ROD, END-ON</td>
<td>FLOW</td>
<td>0.82</td>
</tr>
<tr>
<td>DISC, FACE-ON</td>
<td>FLOW OR</td>
<td>1.17</td>
</tr>
<tr>
<td>CUBE, FACE-ON</td>
<td>FLOW</td>
<td>1.05</td>
</tr>
<tr>
<td>CUBE, EDGE-ON</td>
<td>FLOW</td>
<td>0.80</td>
</tr>
<tr>
<td>LONG RECTANGULAR MEMBER, FACE-ON</td>
<td>FLOW</td>
<td>2.05</td>
</tr>
<tr>
<td>LONG RECTANGULAR MEMBER, EDGE-ON</td>
<td>FLOW</td>
<td>1.55</td>
</tr>
<tr>
<td>NARROW STRIP, FACE-ON</td>
<td>FLOW</td>
<td>1.98</td>
</tr>
</tbody>
</table>

The impulse experienced by a target is reduced by the propagation of a rarefaction, or clearing, wave from the nearest free edge (Tyas et al., 2011). The clearing time is the time it takes for the shock wave to clear the target ($t_c$), and can be calculated according to Equation 2-8 (ASCE 51-11, 2011):

$$t_c = \frac{4Hw}{(w + 2H)C_r}$$  \hspace{1cm} 2-8

The pressure vs. time loading on the front face of a structure can be constructed using Figure 2-8.
An Energy Dissipating System for Blast Mitigation in Structures

The parameters referenced in Figure 2-8 can be obtained from Figure 2-6 and Figure 2-7. In addition, the fictitious positive phase ($t_{of}$) and fictitious reflected pressure ($t_{rf}$) durations can be calculated using the Equations 2-9 (Krauthammer, 2008):

$$t_{of} = \frac{2i_{s}}{P_{so}} \quad t_{rf} = \frac{2i_{r}}{P_{r}}$$

The correct curve to use (triangular or bi-linear) is the one that gives the minimum impulse. The negative phase is often ignored for design calculations.

2.4 Blast Resistant Design

Most blast resistant design of structural members is done using a single degree of freedom (SDOF) analysis in conjunction with established maximum response limits. These response limits are based on the desired level of protection for the structure in question, determined by a threat analysis. ASCE 59-11 defines the allowable level of damage for building elements for different levels of protection (Table 2-3).

<table>
<thead>
<tr>
<th>Level of Protection</th>
<th>Primary Structural Elements</th>
<th>Secondary Structural Elements</th>
<th>Nonstructural Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>I (Very Low)</td>
<td>Heavy</td>
<td>Hazardous</td>
<td>Hazardous</td>
</tr>
<tr>
<td>II (Low)</td>
<td>Moderate</td>
<td>Heavy</td>
<td>Heavy</td>
</tr>
<tr>
<td>III (Medium)</td>
<td>Superficial</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
<tr>
<td>IV (High)</td>
<td>Superficial</td>
<td>Superficial</td>
<td>Superficial</td>
</tr>
</tbody>
</table>

Different building elements can have a different definition for the maximum response limits based on end use of the building. A sample of some maximum response limits is shown in Table 2-4.

<table>
<thead>
<tr>
<th>Element Type</th>
<th>Maximum Response Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Superficial</td>
</tr>
<tr>
<td>Single-reinforced concrete slab</td>
<td>1</td>
</tr>
<tr>
<td>Unreinforced masonry</td>
<td>1</td>
</tr>
<tr>
<td>Hot-rolled steel beam (compact)</td>
<td>1</td>
</tr>
<tr>
<td>Wood</td>
<td>1</td>
</tr>
</tbody>
</table>

The levels of protection and response limits in CSA S850-12 are virtually identical to those in ASCE 59-11.
After determining if the response of the member being designed meets the desired maximum response limits it must be checked to ensure that failure modes other than flexure do not govern the failure. For example shear failure, either direct or flexural shear, must be avoided. In addition, the element must be checked to ensure that failure will not result from load reversal or element rebound. Finally, all elements and connections must be detailed to ensure that they can attain the ductility ratios and end rotations assumed in the design.

Various SDOF software packages exist to help engineers to design building elements under blast loads. SBEDS (Single degree of freedom Blast Effects Design Spreadsheet), available from the USACE Protective Design Center, is one of the most popular packages.

2.5 Energy-Dissipating Methods for Blast Mitigation

In order to protect the main structural elements from damage due to blast loading, methods can be used to concentrate the energy imparted from blasts into devices that are not critical to the stability of the structure and are easily replaceable. This is analogous to the use of hysteretic dampers to improve the earthquake resistance of buildings. Energy-dissipating structures can be classified as one of two types: type I, which consists of a force-deflection curve characterized by a long plateau, and type II, which has a steeply falling load-deflection curve, as shown in Figure 2-9 (Calladine & English, 1984).

![Figure 2-9](image.png)

**Figure 2-9** Force-deflection characteristics (Calladine and English, 1984)

The impulse of these curves for a given loading is necessarily the same; however, the peak force experienced is drastically different. Guruprasad and Mukherjee (2000) stated that in order for energy absorption to be effective the objective is to "reduce the peak force and to extend the duration of the deformation." This behaviour corresponds to an energy-dissipating structure of type I.
Numerous solutions have been developed in order to achieve this ideal behaviour and can be generally characterized into sacrificial claddings and mechanical devices.

2.5.1 Sacrificial Claddings

Sacrificial claddings come in a variety of forms such as cellular structures, metallic foams, or collapsible energy absorbers. These techniques protect the underlying structure by reducing the peak force transferred, allowing the main structural elements to remain elastic.

2.5.1.1 Metallic Foams and Cellular Structures

The possibility of using metal foams for blast protection was suggested by Ashby et al. (2000). Metal foams (typically aluminium) are effective at absorbing blast energy due to the extensive plateau present in their stress-strain behaviour, which can be seen in Figure 2-10. Once the foam structure has been completely collapsed, it has reached what is called the densification strain ($\varepsilon_D$). As loading is continued the stress-strain behaviour will approach that of the solid material from which the foam was made.

![Deformation properties of aluminium foams (Hanssen et al., 2002)](image)

The plateau stress is dependent on the density of the foam and can therefore be customized to suit a particular loading scenario. The plateau stress and densification strain generally vary according to the relationships in equations 2-10 and 2-11 respectively (Ashby et al., 2000).

\[
\sigma_f \approx (0.25 \text{ to } 0.35)\sigma_{ys} \left(\frac{\rho}{\rho_s}\right)^d
\]

\[
\varepsilon_D \approx \left(1 - \alpha \frac{\rho}{\rho_s}\right)
\]
where $\sigma_f$ is the plateau stress, $\sigma_y, s$ is the yield strength of the solid from which the foam is made, $\rho_f/\rho_s$ is the relative density of the foam compared to the solid from which the foam is made, $d$ is a constant which typically lies between 1.5 and 2.0, $\varepsilon_D$ is the densification strain, and $\alpha_1$ is a constant which typically lies between 1.4 and 2.0. Ashby (2000) recommends values of $d = 1.6$ and $\alpha_1 = 1.5$ if more accurate information is not available.

By using the foam between the blast wave and the structure, the peak pressures experienced by the structure can be reduced to the plateau stress ($\sigma_f$) until the densification strain is reached. In order to conserve the impulse of the system the duration of the loading transferred to the structure is lengthened. When loaded statically a cellular structure collapses uniformly across the thickness. However, when loaded dynamically the structure collapses sequentially. This is due to inertia forces developed within the structure (Calladine & English, 1984).

A reduction in peak pressure transferred to the structure can drastically reduce the damage to structural elements during a blast. This can allow the structure to remain stable after an event and reduce the need for repair or demolition.

Hanssen et al. (2002) conducted an experimental and analytical analysis of aluminium foam panels subjected to blast loading using a ballistic pendulum apparatus. They showed that the loading time on the structure could be increased by a factor of 3 to 30 depending on the charge mass. However, they also found an increase in energy and impulse transferred to the structure when compared to the unprotected case. This was attributed to the change in shape of the aluminium foam layer into a double-curved shape, however, only a qualitative description of this phenomenon was provided. Theobald et al. (2010) conducted an experimental study of commercially available aluminium foam core panels from Alporas and Cymat. They investigated the influence of the face plate on the foam performance and found more localized foam core deformations when using thin face sheets. It was also found that brittle failure of the foam occurred at higher blast loads but was more predominant when using the Cymat panels.

Cellular sacrificial claddings work in much the same way as metal foams but are fabricated products typically made from thin gauge steel. Guruprasad and Mukherjee (2000) developed a layered sacrificial cladding system using a cellular steel structure and performed both experimental and analytical studies. Similar to the collapse of metallic foams, the layered cellular system collapsed sequentially from the surface closest to the blast. Other cellular systems have been developed for blast mitigation; for example, Palanivelu et al. (2011) developed a cellular "macro foam" using metal beverage cans.
An Energy Dissipating System for Blast Mitigation in Structures

Despite the beneficial attributes of cellular structures and metallic foams, they have not been implemented as a protection method for structures to date. According to Wadley et al. (2010) this is partly due to the fact that in order to provide sufficient protection for realistic blast threats excessively bulky systems are required. As a solution, Wadley et al. (2010) proposed using an active system in which a compressed cellular system is deployed after a blast has been detected, but before the blast wave impacts the structure.

2.5.1.2 Collapsible Energy Absorbers

A great deal of work has been done to develop collapsible energy absorbers, in particular for use as impact absorbers to mitigate the effects of vehicle collisions. Alghamdi (2001) has reviewed the common collapsible energy absorbing systems that have been investigated in the literature. These include tube deformation modes such as inversion, splitting, lateral indentation, lateral flattening, and axial crushing; axial crushing of frusta, crushing of struts and sandwich panels, as well as a variety of more unusual structures. While these systems were developed for vehicle collisions, several have also been adapted and shown to be useful as sacrificial claddings for blast protection.

Theobald and Nurick performed both a numerical (2007) and an experimental (2010) analysis of a sacrificial sandwich panel type system utilizing axially crushing thin-walled square hollow sections as the energy absorbing mechanism. While the authors found that this system had potential to be an efficient energy absorber, its stability depended heavily on the uniformity of loading and the layout. The system was tested under a narrow range of impulses (53 N s to 68 N s) therefore further study of this system is required to determine if scaling effects are important.

2.5.2 Mechanical Devices

Mechanical devices such as viscous dampers have been used for shock isolation in military systems for many years and since 1990 have been used extensively for seismic protection of structures (Miyamoto and Taylor, 2000). Very little research has been conducted on the use of dampers to protect structures from blasts. However, it is clear that structures protected against earthquakes using dampers should have characteristics beneficial for blast resistance as well (Miyamoto & Taylor, 2000).

In order for these mechanical devices to work for blast mitigation, the surface that the blast wave impacts must be hardened to withstand the blast and transfer the loading into the structural system. Miyamoto and Taylor (2000) showed that, by employing fluid viscous dampers in a conventional special moment resisting frame (SMRF), drifts and plastic rotations were reduced significantly, thus increasing
the survivability of the structure. Monir and Vahedi (2008) investigated using buckling restrained braces as hysteretic dampers in steel frames to reduce deflections and plastic rotations. They found that the overall performance of the frame was improved by incorporating the hysteretic dampers, however, midpoint deflections of the columns were not improved and needed to be addressed using cladding.

2.6 Explicit Dynamics Analysis

Explicit dynamics analysis software (hydrocodes) was created to simulate highly dynamic problems involving large strains and deformations, shocks, and wave propagation through a medium. Several formulations have been developed for explicit analyses including: Lagrange, Euler, Arbitrary Lagrange Euler (ALE), and Smooth Particle Hydrodynamics (SPH). Each of these formulations is suited to a particular range of problems. The Lagrange formulation is particularly suitable to solid mechanics problems since the material is fixed to a finite element mesh, and consequently the mass of a cell is invariant for the duration of the simulation (Anderson, 1987). Another advantage of the Lagrangian formulation is the ability to track the loading history of the material, allowing the modelling of strain hardening, and the computation of plastic work and material failure (Anderson, 1987). The Euler formulation is well suited to fluid flow problems since there is a fixed mesh and material flows between cells. The Arbitrary Lagrange Euler formulation combines properties of both Lagrangian and Euler meshes by allowing the mesh to deform independently of the material. This method can be an effective solution to problems where large deformations result in severe mesh distortions in Lagrangian models. The Smooth Particle Hydrodynamics formulation is a type of Lagrangian formulation that replaces the mesh with an assembly of particles, which are joined by attractive forces. SPH is particularly well suited to modelling fracture and fragmentation and is often used for modelling hypervelocity impacts.

All explicit codes solve dynamic problems through the application of the laws of conservation of mass, momentum, and energy. Conservation of mass is automatically satisfied in a Lagrangian system since the material moves and distorts with the finite element mesh. As a result, the density of an element at any time \( t \) can be determined using the initial mass \( (m) \), which is constant, and the volume of an element at time \( t \) \( (V_t) \) according to:

\[
\rho_t = \frac{\rho_0 V_0}{V_t} = \frac{m}{V_t}
\]

The conservation of momentum allows a relationship between acceleration and stress to be formulated (Century Dynamics, 2005):
Finally, the conservation of energy is formulated as (Century Dynamics, 2005):

\[ \dot{e} = \frac{1}{\rho} \left( \sigma_{xx}\ddot{x} + \sigma_{yy}\ddot{y} + \sigma_{zz}\ddot{z} + 2\sigma_{xy}\dot{x}\dot{y} + 2\sigma_{yz}\dot{y}\dot{z} + 2\sigma_{zx}\dot{z}\dot{x} \right) \]  

In order to ensure the stability of the solution algorithm, all the conservation laws cannot be held exactly due to round-off and truncation errors, as well as the introduction of pseudo-forces such as hourglass deformation modes and contact forces. In practice, the conservation of energy is relaxed slightly but monitored to ensure that the energy error does not get unreasonably large. By default, AUTODYN has a maximum energy error of 5%. (Century Dynamics, 2005).

In addition to the conservation laws, a material model is required in order to have a sufficient number of equations to solve the system of conservation equations. A material model includes two components: the equation of state, and a constitutive relationship. The equation of state defines the relationship between the three thermodynamic variables: pressure, volume, and internal energy (Anderson, 1987). This relationship can be stated in the form:

\[ P = f(E,V) \]

where \( P \) is the pressure in the material, \( V \) is the material volume, and \( E \) is the internal energy of the material.

A constitutive relationship relates stress and strain in the material. It can also include other material effects such as: strain-rate hardening, thermal softening, and strain hardening (Anderson, 1987). Linear elasticity is a simple example of a constitutive relationship:

\[ \sigma = E\varepsilon \]

The preceding equations form a highly nonlinear set of coupled partial differential equations that must be solved. Exact analytical solutions do not exist except for some very simple problems; therefore, in general a numerical solution technique must be used. The continuous problem definition must first be discretized into a set of points (nodes). Then, using the finite difference method (the finite element method is used in some codes) the continuous set of differential equations are transformed into difference equations:
Using the boundary and initial conditions a time-integration method is used to solve the system. The values of variables at a time \( t+1 \) can be found directly from the state at time \( t \). If the equations for the conservation laws are expressed generally as (Anderson, 1987):

\[
\frac{\partial U}{\partial t} = -\frac{\partial}{\partial x} F(U) \equiv G(U)
\]

then a variable at time \( t+1 \) can be determined according to:

\[
U(t + 1) = U(t) + \Delta t \cdot G(t)
\]

Unlike implicit analysis, explicit analysis is conditionally stable. In order to ensure the stability of the solution a maximum time-step must be imposed. This maximum is governed by the Courant-Friedrichs-Levy criterion (Century Dynamics, 2005):

\[
\Delta t \leq \frac{2d}{3 \sqrt{c}}
\]

where \( d \) is the characteristic length of an element (volume of element divided by square of longest diagonal), and \( c \) is the wave propagation speed through the material. This ensures that a stress wave will not pass through an element in a time less than the maximum time step.

### 2.7 System Overview

The concept behind the system is the creation of panels that can be assembled into a structure of any shape. The panels can consist of a variety of materials in order to suit different requirements (insulation, ballistic protection, etc.). They are connected to the structure using energy dissipating component assemblies around the panel edge. The panels do not serve any structural purpose beyond transferring wind (air pressure) loads to the structure.

When subjected to a blast load, the panels will deform and transfer the blast pressure through to the energy dissipating component assemblies that connect to the frame. The energy dissipating component assemblies will then limit the loads transferred to the structure. After the blast has occurred, the panels and energy dissipating assemblies can be replaced, since the main structure should remain undamaged. It will then be possible to reoccupy the structure in a very short time following an attack.
2.7.1 Infill Panels

The infill panels can be tailored for specific applications, but their main purpose is to transfer the blast pressure evenly between Energy Dissipating Assemblies. For the purposes of the current research the panel is assumed to be perfectly rigid and to transfer 100% of the blast pressure to the energy dissipating component assemblies. In reality, deformation of the panels will absorb some of the energy imparted from the blast; therefore, the assumption of rigid panels is conservative.

2.7.2 Energy Dissipating Component Assemblies

The energy dissipating component assemblies (EDCAs) which connect the panels to the structure consist of an assembly of galvanized steel units (Energy Dissipating Components - EDCs). These are assembled in series and/or parallel in order to obtain the required elongation and resistance for a specified maximum threat. An example EDCA with three EDCs in series and three in parallel (3EDCx3) is shown in Figure 2-11.

![Figure 2-11](image)

**Figure 2-11**  Energy dissipating component assembly (Explora Security Ltd, 2012)

The EDCs are assembled into configurations, such as those listed in Table 2-5. The first number in the identifier is the number of layers of EDCs; the second number is the number of EDCs in series.
Table 2-5  EDC configurations

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Identifier</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1EDCx2</td>
</tr>
<tr>
<td></td>
<td>1EDCx3</td>
</tr>
<tr>
<td></td>
<td>2EDCx3</td>
</tr>
<tr>
<td></td>
<td>3EDCx3</td>
</tr>
</tbody>
</table>

The displacement-resistance behaviour of a single EDC is shown in Figure 2-12. The EDCs exhibit a type I behaviour as described by Calladine and English (1984) which makes them ideal for use in energy-dissipating applications.

Figure 2-12  Static load-displacement relationship for a single EDC

By assembling the EDCs in series, the length of the plateau region can be extended substantially. By assembling the EDCs in parallel, the resistance of the plateau region can be increased. Therefore, by using EDCs in series and parallel, an assembly with a type I resistance function can be created to suit a wide variety of situations.
3 Laboratory Testing

Laboratory tests, conducted by the author, were used in order to obtain data to validate the finite element models and form the basis of the characterization. The tests were designed in order to load the EDCs dynamically in various configurations while recording the resistance-time and displacement-time histories.

3.1 Test Setup

The testing rig consisted of a pneumatic cylinder connected to a loading frame. One end of an EDC assembly was connected to the loading frame while the other end was secured to a fixed support. Between the EDCA and the fixed support, a load cell was used to measure the support reaction load-time history.

A 0.46L reservoir was charged with compressed hydrogen until the seal between the reservoir and pneumatic cylinder failed. Figure 3-1 shows a schematic of the test setup.

![Test setup schematic](image)

Figure 3-1 Test setup schematic

A 0.2 millimetre thick brass disk (Figure 3-2) was used as a seal between the reservoir and piston. Once the cylinder reached a sufficient pressure the seal burst (Figure 3-3) allowing the hydrogen to pass from the reservoir to propel the piston and load the sample. Due to variances in material properties of the brass disks, there was some variation in the pressure required to fail the seal. The average pressure required to fail the seal was 8650 kPa with a standard deviation of 900 kPa.
The test rig attained loading rates of approximately 18.5 m/s.

The force-time history was measured using a load cell on the fixed end of the EDCA. A laser displacement gauge was used to measure the displacement-time history of the loading frame. A phantom v4.2 high-speed video camera recording 2000 frames per second was used during the tests to record the behaviour of the EDCAs. The completed test setup is shown in Figure 3-4.

3.2 Test Program

The cost of the tests restricted the number of samples to eight. In order to get the maximum information from the test program four EDCA arrangements were chosen and two tests of each arrangement were conducted. The test program is summarized in Table 3-1.
During the commissioning of the rig it was found that a charge pressure of approximately 9000 kPa was most appropriate for the range of tests being conducted. At pressures significantly below this, the loading was quasi-static, while pressures significantly higher caused erratic behaviour of the test apparatus, which made capturing the load-time and displacement-time histories difficult.

Due to the mass of the test rig and the driving force, the tests were effectively displacement-controlled. However, there was some slowing of the loading rate as the number of EDCs in an assembly increased, as shown in Figure 3-5.

### 3.3 Deformation Mechanism

The deformation mechanism is critical to how the ideal type I behaviour is achieved. The elements begin in a layered planar configuration and undergo three separate stages of deformation: in-plane stretching,
out-of-plane rotation, and full extension to failure. When an initial load is applied, any play in the connections is quickly removed and the EDCAs begin to stretch in plane as shown in Figure 3-6.

![Unloaded and In-plane stretching]

**Figure 3-6** Deformation mechanism - Stage 1

The connections between the supports then begin to rotate out-of-plane. This occurs sequentially from the loaded side to the support side as shown in Figure 3-7. This sequential deformation is analogous to that observed in metallic foams.

![Deformation mechanism - Stage 2](image)

**Figure 3-7** Deformation mechanism - Stage 2

Once the out-of-plane rotation has started, the force plateau region begins. When all the EDCAs are fully extended, the force transferred through the element begins to increase until failure, which usually
occurs as shown in Figure 3-8. The areas of stress concentration in the corners fail and the EDC typically splits into two pieces.

**Figure 3-8**  Deformation mechanism - Failure

As the number of layers increases, the apparent complexity of the deformation increases. However, the same stages are present despite the number of layers. When three layers are used, the in-plane stretching of stage one is apparent in Figure 3-9.

**Figure 3-9**  Stage 1 - three layers

This in-plane stretching is very similar to that observed in Figure 3-6. The out-of-plane rotation of stage two is also apparent, although the layers begin to separate as shown in Figure 3-10.
The failure happens in a similar way to that shown in Figure 3-8, however, all three layers do not necessarily fail at the same location as shown in Figure 3-11.

The out-of-plane mechanism allows a very consistent plateau force to be achieved despite variances in yield stress of the steel. It also reduces the influence of strain-rate effects, which is discussed in section 3.5.

The deformation mechanism indicates that the loading regime was dynamic and not quasi-static. Under quasi-static loading, the EDCs would be expected to deform in a random sequence. However, under dynamic loading the deformation is sequential due to inertia effects. Some cellular structures, which the load-displacement history of the EDCAs resembles, have been shown to collapse uniformly under quasi-
An Energy Dissipating System for Blast Mitigation in Structures

static loads and non-uniformly under dynamic loads (Calladine & English, 1984). Since the deformation mode of the EDCAs is sequential, it is clear that the loading regime is in the dynamic range.

The rivets used to connect the EDCs together had a neck diameter very close to the diameter of the connection holes in the EDCs and extended on the back side by approximately 19mm as shown in Figure 3-12.

![Rivets connecting EDCA](image)

**Figure 3-12** Rivets connecting EDCA

When the EDCAs were at full extension it was observed that often the rivet pulled out before the EDCs failed. This was not an issue since the behaviour of interest is the plateau region. When the EDCA is fully extended and the plateau region ends the system has effectively failed since the forces transferred to the structure begin to increase. However, in the future, the EDCAs should be assembled with a washer on the reverse side in order to prevent rivet pull-out and shorter rivets should be used as shown in Figure 3-13.

![EDCA assembled with shorter rivets and washers](image)

**Figure 3-13** EDCA assembled with shorter rivets and washers

3.4 Results

The force-time and displacement-time histories from the tests were filtered using a 50-point floating average in order to eliminate much of the high-frequency noise inherent in the data collection system.
The unfiltered and filtered force-time histories for Test 3 are shown in Figure 3-14 and Figure 3-15 respectively.

![Figure 3-14](image1.png)  ![Figure 3-15](image2.png)

**Figure 3-14**  Force-time history (unfiltered)  **Figure 3-15**  Force-time history (filtered)

The first test to be conducted was an assembly of two EDCs in series (Figure 3-16). Two tests were conducted on this arrangement. This configuration is the simplest to model and therefore an excellent candidate for use as a model verification test.

![Figure 3-16](image3.png)

**Figure 3-16**  Test 1/2 - two EDCs in series

The experimental load-displacement curves for the first two tests are shown in Figure 3-17. There is a noticeable difference in the initial peak load between the first and second tests. The initial peak and plateau loads of the second test are lower due to initial deformations from the manufacturing of the element. This removed some of the in-plane stretching stage, which occurs before the connection rotates and the deformation mode switches to the out-of-plane rotation stage. The onset of the full extension to failure stage begins at an extension of 80mm.
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**Figure 3-17** 1EDCx2 load-displacement test results

This difference is only significant in the single layer elements, since the resistance to out-of-plane deformations relies on the bending resistance of a single element. With more layers, the bending resistance increases and the effect of initial deformations is reduced, which can be seen in the remaining results. Despite the variance in the peak load, the three stages of planar deformation, out-of-plane deformation and full extension to failure are apparent. Ignoring the lower peak load of the second test it is clear that the plateau load for the 1EDCx2 configuration is approximately $p_1$.

The second assembly to be tested was three EDCs in series (Figure 3-18). Two tests were conducted on this arrangement.

**Figure 3-18** Test 2/3 - three EDCs in series
The experimental load-displacement curves for tests 2 and 3 are shown in Figure 3-19. The Load-Displacement results show excellent consistency of the initial peak load as well as the plateau load and the start of the full extension to failure stage.

![Figure 3-19 1EDCx3 load-displacement test results](image)

By having three EDCs in series, the length over which the plateau occurs is increased. In comparison to two EDCs in series the three EDCs in series plateau extends for approximately another 70 mm, to 150mm. This results in additional energy being dissipated as well as additional movement. The plateau load for the 1EDCx3 is approximately $p_2$, which is slightly higher than the 1EDCx2 configuration.

The third assembly to be tested consisted of two parallel layers of three EDCs in series (Figure 3-20). Two tests were conducted with this arrangement.

![Figure 3-20 Test 5/6 - Two layers of three EDCs in series](image)
The experimental load-displacement curves for tests 5 and 6 are shown in Figure 3-21. These results also show excellent consistency of the peak and plateau loads. However, the onset of the full extension differs slightly between the two tests.

![Figure 3-21](image)

**Figure 3-21**  2EDCx3 load-displacement test results

In comparison to the 1EDCx3 tests, the length over which the plateau region extends is slightly smaller at 140mm. In addition, the plateau load has approximately doubled to \( p_3 \). This is also expected since the second layer of EDCs should approximately double the resistance of the connections.

The final configuration tested consisted of three parallel layers of three EDCs in series (Figure 3-22). There were three tests of this configuration.

![Figure 3-22](image)

**Figure 3-22**  Test 7/8 three layers of three EDCs in series
The experimental load-displacement curves for this configuration are shown in Figure 3-23. As with the previous tests, excellent consistency was achieved. The peak and plateau loads, as well as the onset of the full extension stage, were found to be approximately the same for all three tests.

**Figure 3-23** 3EDCx3 load-displacement test results

In comparison to the 1EDCx3 and 2EDCx3 tests, the onset of the full extension stage was slightly earlier (125mm), indicating that the added resistance to rotation due to the increased number of layers has decreased the overall elongation capacity of this arrangement. As expected, the plateau load has increased to approximately $p_4$. While testing this configuration, it was discovered that the orientation of the element influenced the direction of the deformation sequence. It was found that the deformation progressed from the lowest side (right side in Figure 3-22) to the highest side. Tests 7 and 8 were conducted with the low side fixed while test 9 was conducted with the high side fixed. Despite this difference in deformation direction, the force-displacement results were not affected, as confirmed by Figure 3-23.

### 3.5 Analysis of Results

The effect of strain rate on the performance of the EDCAs was found to be minimal. This can be seen by the comparison of the 2EDCx3 arrangement under both static and dynamic loadings, as shown in Figure 3-24.
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Figure 3-24  Static vs. dynamic loading comparison

The dynamic plateau load is slightly higher than the static load; however, the difference is minimal especially considering the difference in support conditions and variations in material properties between the two tests.

The energy dissipated by each configuration is dependent on both the number of layers and the number of EDCs in series. Table 3-2 shows the energy absorbed until the end of the plateau for each configuration. As expected, the energy absorbed increases as the number of EDCs increases. Increasing the number of EDCs in series from two to three increases the energy absorbed by a if there is one layer of EDCs. Adding a second layer of EDCs more than doubles the energy absorbed for three EDCs in series. However, adding a third layer only increases the energy absorbed by about 3a. This is due to the reduction in the length of the plateau region, which occurs as the resistance to out-of-plane deformation increases.

Table 3-2  Energy absorbed by each configuration

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Energy Absorbed (J)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1EDCx2</td>
<td>e_1</td>
</tr>
<tr>
<td>1EDCx3</td>
<td>e_2</td>
</tr>
<tr>
<td>2EDCx3</td>
<td>e_3</td>
</tr>
<tr>
<td>3EDCx3</td>
<td>e_4</td>
</tr>
</tbody>
</table>
An Energy Dissipating System for Blast Mitigation in Structures

The plateau load is the most important quantity for the EDC assemblies since this is the force transferred into the supporting structure (unless full extension and failure is reached). The plateau loads for each configuration are shown in Table 3-3.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Plateau Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1EDCx2</td>
<td>$p_1$</td>
</tr>
<tr>
<td>1EDC3</td>
<td>$p_2$</td>
</tr>
<tr>
<td>2EDCx3</td>
<td>$p_3$</td>
</tr>
<tr>
<td>3EDCx3</td>
<td>$p_4$</td>
</tr>
</tbody>
</table>

The plateau load increases as the number of layers increase since this adds to the resistance to out-of-plane rotations. The initial stiffness of the EDC assemblies is another important quantity. The initial stiffness values for each configuration tested are shown in Table 3-4.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Initial Stiffness (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1EDCx2</td>
<td>$k_1$</td>
</tr>
<tr>
<td>1EDC3</td>
<td>$k_2$</td>
</tr>
<tr>
<td>2EDCx3</td>
<td>$k_3$</td>
</tr>
<tr>
<td>3EDCx3</td>
<td>$k_4$</td>
</tr>
</tbody>
</table>

The initial stiffness generally increases as the number of layers increases since this adds to the total cross section of steel that is loaded, and increases the resistance to out-of-plane rotations. The displacement at the onset of stage 3, full extension to failure, for each configuration is shown in Table 3-5.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1EDCx2</td>
<td>$d_1$</td>
</tr>
<tr>
<td>1EDC3</td>
<td>$d_2$</td>
</tr>
<tr>
<td>2EDCx3</td>
<td>$d_3$</td>
</tr>
<tr>
<td>3EDCx3</td>
<td>$d_4$</td>
</tr>
</tbody>
</table>

The displacement at the onset of the full extension to failure stage increases as the number of EDCs in series increases, however, adding additional layers reduces this displacement.

By adopting a configuration with an energy absorption capacity adequate for a specified blast threat the design loads for the structure can be obtained from the plateau load. This gives some capacity for overloading the assembly since the full-extension and failure stage will absorb additional energy. The
structure should behave elastically in the plateau range. The supporting structure should also be designed to have some reserve capacity to accommodate the increased load transfer during the full-extension and failure stage in case of overloading.
4 Finite Element Study

A finite element study was attempted in order to determine the relationship between EDC configuration, deformation, energy absorption and applied peak pressure and impulse. The goal of this study was to extrapolate the laboratory data to EDCAs which were not tested in the laboratory testing program.

4.1 Material Model

Several material models have been proposed for ductile metallic materials. The most popular of these models are the Johnson-Cook constitutive model, Zerilli-Armstrong constitutive model, and the Cowper-Symonds strength model. The Johnson-Cook and Cowper-Symonds models treat strain hardening the same way, however, the strain-rate term is represented in a different form. The Johnson-Cook model also includes temperature softening while the Cowper-Symonds model does not. The Zerilli-Armstrong model is a more complex model based on dislocation mechanics. The Johnson-Cook model is the most widely used for mechanics and impact problems, due to its simplicity and the availability of material parameters for a wide variety of metals. As a result, it was selected as the material model for this study.

Johnson and Cook developed their empirically based material model in the 1980s. The model considers strain hardening, strain-rate hardening, as well as temperature effects in a simple formulation as shown in Equation 4-1 (Johnson & Cook, 1983):

\[
\sigma = \left[ A + B\varepsilon^n \right] \left[ 1 + C \ln \dot{\varepsilon}^* \right] \left[ 1 - T^*m \right]
\]

where \( A \) is the static yield stress, \( B \) and \( n \) represent the effects of strain hardening, \( C \) is a strain hardening coefficient, \( T^* \) and \( m \) are temperature parameters, \( \varepsilon \) is the equivalent plastic strain, \( \dot{\varepsilon}^* \) is the dimensionless plastic strain rate and \( T^* \) is known as the homologous temperature and is calculated according to the following equation:

\[
T^* = \frac{T - T_{room}}{T_{melt} - T_{room}}
\]

Since the melt temperature of steel is high (~1500°C), this term is small at normal temperatures and is often ignored.

Johnson and Cook performed both torsion and Split Hopkinson Pressure Bar tests on various materials over a range of temperatures in order to develop the constants for their model. Parameters for some different steels are summarized in Table 4-1.
Table 4-1  Johnson-Cook material model parameters

<table>
<thead>
<tr>
<th>Material</th>
<th>A (MPa)</th>
<th>B (MPa)</th>
<th>n</th>
<th>C</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>4340 Steel</td>
<td>792</td>
<td>510</td>
<td>0.26</td>
<td>0.014</td>
<td>1.03</td>
</tr>
<tr>
<td>S-7 Tool Steel</td>
<td>1539</td>
<td>177</td>
<td>0.12</td>
<td>0.016</td>
<td>1.00</td>
</tr>
<tr>
<td>Structural Steel</td>
<td>360</td>
<td>635</td>
<td>0.114</td>
<td>0.075</td>
<td>1.00</td>
</tr>
<tr>
<td>C350 steel</td>
<td>430</td>
<td>250</td>
<td>0.60</td>
<td>0.075</td>
<td>1.00</td>
</tr>
<tr>
<td>ST-37 Steel</td>
<td>297</td>
<td>320</td>
<td>0.32</td>
<td>0.03</td>
<td>1.00</td>
</tr>
</tbody>
</table>


Two criticisms of the Johnson-Cook model are the uncoupled strain rate and temperature, and its empirical basis (Dey et al., 2011). However, it remains one of the most popular material models for metals due to its simplicity and the availability of model parameters published in the literature.

4.2 Material Model Validation

The mill certificate for the steel used to manufacture the EDCs specified galvanized ST-37 steel with a minimum yield stress of 298MPa and a minimum ultimate stress of 366MPa with a minimum elongation at fracture of 38%. However, in order to validate the material model, full stress-strain curves were produced using tensile coupon tests on coupons machined from EDCs. Due to the size of the EDCs, three very small tensile coupons were machined as shown in Figure 4-1.

![Tensile coupons](image)

The dimensions of the three coupons are summarized in Table 4-2. Due to the small dimensions Mitutoyo Digimatic Micrometer Model FLA-03-11-5LT strain gauges were used. These had a gauge length of 0.3mm and a gauge factor of 2.35. All tests were conducted at an extension rate of 0.0085mm/s (0.02 in/min).
The engineering stress-strain curves for all three specimens are shown in Figure 4-2.

All three tensile coupons showed very similar stress-strain behaviour. The elastic modulus and 0.2% offset yield stress are summarized in Table 4-3. The average elastic modulus was 240 700 MPa and the average yield stress was 310 MPa. The average rupture strain was 23.6%.

The engineering stress-strain curves were converted to true stress-strain curves with the procedure outlined by Matic (1985) used to adjust for post-necking behaviour. The plastic stress-strain curves for each coupon, after undergoing the transformation outlined by Matic, are shown in Figure 4-3. The Johnson-Cook material model was fitted to each curve as shown in Figure 4-3.
The Johnson-Cook model parameters for each tensile coupon are summarized in Table 4-4.

<table>
<thead>
<tr>
<th>Coupon</th>
<th>A (MPa)</th>
<th>B (MPa)</th>
<th>n</th>
<th>C</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>331</td>
<td>155</td>
<td>0.25</td>
<td>0.011</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>300</td>
<td>79</td>
<td>0.28</td>
<td>0.003</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>328</td>
<td>185</td>
<td>0.24</td>
<td>0.017</td>
<td>1</td>
</tr>
<tr>
<td>Ave.</td>
<td>319</td>
<td>140</td>
<td>0.26</td>
<td>0.01</td>
<td>1</td>
</tr>
</tbody>
</table>

The model parameters obtained all lie within the range of values for steel cited in the literature (Table 4-1). The average model parameters were used in the finite element model.

### 4.3 Finite Element Model

A finite element model was created which modelled the geometry of the EDCs exactly. The snug-fitting rivets were simplified as cylinders and were modelled as rigid parts bonded to the EDC surface. Two rigid surfaces were modelled on either end of the assembly in order to replicate the boundary conditions in the dynamic laboratory tests. The completed finite element model can be seen in Figure 4-4.
The model was created using half-symmetry in order to save computation time. The left hand rivet and surface were modelled as fixed supports. The force reaction for the left hand rivet was recorded to measure the loads transferred to the supporting structure (the load cell in the laboratory tests). The right hand rivet and surface had a displacement control that imposed the displacement time history measured in the laboratory tests. There were frictionless contacts between the EDCs, as well as between the EDCs and the supports.

4.4 Meshing

A patch conforming tetrahedral mesh was used for the model. This was chosen over a more computationally efficient hexahedral since the large strains experienced in the corner regions were causing hourglass modes of deformation, as shown in Figure 4-5. Hourglass modes are a zero-energy deformation mode, which can induce significant errors into an analysis. While hourglass damping can be used to apply an artificial stiffness to hexahedral elements to combat hourglassing, it is not 100% effective. Many strategies were used to attempt to control the hourglass modes in the EDC model including increasing the mesh density, adjusting the hourglass damping, and using fully integrated elements. However, none successfully eliminated this source of error.
Since tetrahedral elements do not suffer from hourglass modes of deformation, a tetrahedral mesh was used for the EDC model, as shown in Figure 4-6. The mesh was refined in the regions of high strain in order to reduce energy errors and increase the accuracy of the model. The mesh was composed of 20741 elements.

As discussed in Section 2.6, the time step in an explicit dynamics analysis is controlled by the characteristic length of the smallest element and the wave propagation velocity in the material. In order to reduce the effect of a small number of very small elements controlling the time step, mass scaling was used in the model. This adds non-physical mass to small elements in order to reduce the wave
propagation velocity in that element, thus increasing the time step and allowing for a faster analysis. The mass of any individual element was not increased by more than 5%, in order to ensure the results were not affected significantly due to the added mass.

4.5 Validation of Finite Element Models

The 1EDCx2 configuration was used for the model validation since it was the simplest arrangement tested in the laboratory. In order to evaluate the finite element model, it was subjected to the same displacement-time history that was measured in the laboratory tests, shown in Figure 4-7.

![Figure 4-7](image)

Displacement-time control used to validate 1EDCx2 model

The boundary conditions of the laboratory tests were replicated in the model, as shown in Figure 4-4. The reaction load-time history was measured at the opposite end of the EDCA from the displacement control and compared to the experimental results. A comparison of the 1EDCx2 laboratory results and model is shown in Figure 4-8.
Figure 4-8  1EDCx2 comparison of model to laboratory results

While the model matches the Test 2 plateau load reasonably well, the Test 1 results are more likely representative of the actual behaviour, as discussed in Section 3.4. A model of the 1EDCx3 configuration was created in order to determine how the model compared to the laboratory results in this configuration, since the results were more consistent, as shown in Figure 4-9.

Figure 4-9  1EDCx3 comparison of model to laboratory results

It is clear that the plateau load determined in the model under-predicts the laboratory measured plateau load. The model predicted plateau load was $a_m$, approximately the same as the model-predicted plateau load for the 1EDCx2 configuration, as expected, while the laboratory measured plateau load was $p_2$. 
The cause of this discrepancy was not discovered, despite a thorough investigation of factors such as mesh density, material model, support conditions, and others. One possible cause is that the deformation is concentrated in a very small region of the model. Practically, it is not possible to mesh this region with a large number of elements since the analysis time quickly becomes unreasonable (hundreds of hours). Further analysis will need to be carried out in order to improve the accuracy of the finite element model.
5 Characterization

The laboratory test data was used to develop a characterization of EDC assemblies. The effect of the number of layers of EDCs as well as the number of EDCs in series on the design parameters of the plateau load, initial stiffness, and maximum extension were determined by using linear regression to fit the laboratory data.

The plateau load of EDC configurations showed a very strong linear correlation to the number of layers as shown in Figure 5-1. The fit was made to pass through the origin, therefore making sure that the theoretical configuration of zero layers would have a plateau load of zero. The coefficient of determination value for the fit was 0.9810, which indicates a very good fit to the laboratory data.

![Plateau load vs. number of layers](image)

**Figure 5-1** Plateau load vs. number of layers

The linear fitted equation relating the number of layers to the plateau load is provided as Equation 5-1.

\[ R_{pl} = Fp \]  

The initial stiffness of EDC assemblies also showed a very strong linear correlation to the number of layers, as shown in Figure 5-2. The fit has a coefficient of determination of 0.9999, which is virtually a perfect fit.
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The linear fitted equation relating the number of layers to the plateau load is provided as Equation 5-1.

\[ k_l = Gp + J \]  

5-2

The relationship between the EDC configuration and the maximum extension was more complex as shown in Figure 5-3. An analysis of the laboratory tests suggested that the maximum extension was dependent both on the number of layers as well as the number of EDCs in series. In order to determine the relationship between these three quantities a linear combination of terms was assumed, according to Equation 5-3.

\[ x_{max} = As + Bp + C \]  

5-3

An optimization routine was used to determine the coefficients A, B, and C by minimizing the square-root-sum-of-squares difference between the maximum extensions predicted by Equation 5-3 and those determined in the laboratory testing. Equation 5-3 using these coefficients had a coefficient of determination of 0.9986, which indicates a very good correlation to the laboratory results.

**Figure 5-2**  Initial stiffness vs. number of layers

The linear fitted equation relating the number of layers to the plateau load is provided as Equation 5-1.

\[ k_l = Gp + J \]  

5-2

The relationship between the EDC configuration and the maximum extension was more complex as shown in Figure 5-3. An analysis of the laboratory tests suggested that the maximum extension was dependent both on the number of layers as well as the number of EDCs in series. In order to determine the relationship between these three quantities a linear combination of terms was assumed, according to Equation 5-3.

\[ x_{max} = As + Bp + C \]  

5-3

An optimization routine was used to determine the coefficients A, B, and C by minimizing the square-root-sum-of-squares difference between the maximum extensions predicted by Equation 5-3 and those determined in the laboratory testing. Equation 5-3 using these coefficients had a coefficient of determination of 0.9986, which indicates a very good correlation to the laboratory results.
Equations 5-1, 5-2, and 5-3 define the important quantities of behaviour for EDC assemblies. Using these equations, a model can be developed in order to predict the behaviour of EDC assemblies under any loading condition.
6 Design Methodology

6.1 Single Degree of Freedom Model

A single degree of freedom (SDOF) analysis can be constructed of a panel supported by assemblies of EDCs. The panel is assumed to be rigid and attached to springs with resistance functions corresponding to those of an assembly of EDCs. Minimal damping (2% of critical) is added to the spring-mass system, which accounts for energy losses during plastic deformation, as well as other factors such as air resistance. The equivalent SDOF model is shown in Figure 6-1.

![Figure 6-1 SDOF model](image1)

The response function of the EDCAs is approximated as elastic perfectly plastic as shown in Figure 6-2. The initial stiffness is determined using equation 5-2.

![Figure 6-2 Elastic perfectly plastic response](image2)

In applications which use multiple assemblies of EDCs in parallel (such as in a panel configuration) the total initial stiffness can be determined by multiplying the stiffness of an EDCA by the total number of EDCAs attached to the panel.

The governing equation of motion for inelastic systems such as this is:

$$m\ddot{u} + c\dot{u} + f_s(u, \dot{u}) = p(t)$$  \hspace{1cm} 6-1

where \(m\) is the mass, \(c\) is the damping constant, \(f_s\) is the resistance function, \(u\) is the displacement, and \(p\) is the imposed loading.

Response spectra have been derived for elastic-plastic systems subjected to triangular (Biggs, 1964) and exponential (Gates and Pnevmatikos, 2004) blast loads which can be used to determine the peak
response of this SDOF system. However, if a full displacement-time history is desired, numerical methods, such as Newmark's method (Chopra, 2007), must be used in order to obtain a solution.

6.2 Design Method

There are three considerations governing the selection of a particular configuration of EDCs: the total number of EDC assemblies to be used, the peak displacement, and the reaction (plateau) load. The design parameters, which serve as inputs to the design method, are the reflected peak pressure and impulse as well as the panel geometry (area) and mass.

The parameter governing the design is the maximum load per EDC assembly, which will depend on the supporting structure. Once the maximum load per EDC assembly is known, the number of layers of EDCs can be determined. The number of layers of EDCs determines the plateau load, and consequently the maximum load per EDC assembly. The relationship between the plateau load and the number of layers is linear. Therefore, the required number of layers can be determined using Equation 6-2, which is a rearranged form of the linear curve fit in Equation 5-1.

\[ \text{Layers} = \frac{1}{F} (R_{pl}) \]  

Once the number of layers is known, the total number of EDC assemblies needs to be determined. In order to minimize the loads transferred to the structure, the number of assemblies should be maximized. The EDC assemblies should also be spaced as evenly as possible around the perimeter of the panel.

When the number of EDC assemblies and the number of layers are known the pressure-impulse diagrams can be used to determine the number of EDCs required in series. One of these diagrams is shown in Figure 6-3.
Reflected peak pressures and impulses should be used in the pressure-impulse diagram. Using these P-I diagrams will ensure that a safe choice of assembly is made.

These diagrams were generated by running SDOF analyses of various charge weight and standoff combinations (including the negative phase) and generating a line indicating the combinations that result in the maximum extension of the EDCA. The impulse was normalized using the mass of the panel in order to allow the diagrams to be used for any mass. Different diagrams were created for between 20 and 60 EDCAs in intervals of 10 with the number of layers ranging from 2 to 5 and the number of EDCs in series ranging from 2 to 6. This should cover most common design configurations. If a design is not accounted for with these diagrams (more than 60 EDCAs or more than 5 layers), a SDOF analysis must be undertaken to determine the displacement under a particular loading scenario and compare it to the displacement limits listed in Table 3-5.

Once a configuration of EDC assemblies has been chosen, the peak displacement can be estimated using the elastic-plastic response spectrum for exponential loading in Figure 6-4 or by using numerical methods.
The reflected blast wave parameters $t_0$ and $P_r$ can be determined using Figure 2-6 or using a software package such as Conwep.

### 6.2.1 Limitations

This proposed design method does have some limitations that must be considered. The characterization used for the design is based on a range of laboratory tests conducted on EDC assemblies ranging from 1EDCx2 to 3EDCx3. As a result, the design method becomes more unreliable when extrapolated to large numbers of EDCs. The design method also does not consider any energy dissipation due to panel deformations. This energy dissipation can be considerable, especially when panels contain deformable materials such as foam. Not considering this energy dissipation is a conservative assumption; however, for highly deformable panels it may be overly conservative. The design of the panels is also not addressed in the design method. The panels must be able to transfer the blast loads to the EDCAs without failing. In order to determine the required strength of a panel, the plateau load of the EDCAs can be used as the reaction loads for the panel. Substantial extra reserve capacity should be included in the panel in order to withstand any overloading.
7 Field Testing

In order to evaluate the accuracy and suitability of the prediction model and design methodology, large-scale arena blast tests using 500kg of ANFO (Ammonium Nitrate and Fuel Oil) were conducted at the Rafael Shdema Firing Range located south of Mitzpe Ramon, Israel.

7.1 Target Design and Construction

In order to isolate the behaviour of the EDCs a panel was designed to transfer the blast pressure to the EDCAs located around its perimeter. The panel was designed to be extremely stiff in bending so that the energy used in bending the panel would be minimized. In order to accomplish this, a lattice of 4mm steel stiffener plates was welded to the back of a 5mm steel plate as shown in Figure 7-1. The panels were 2074mm tall and 933mm wide with a depth of 124mm. A heavier version of the panel was also created in order to investigate the influence of mass on the behaviour. The heavy version had a steel face plate thickness of 25mm instead of 5mm with all other elements remaining the same. Manufacturing drawings of the panel are included in Appendix A.

![Panel stiffener arrangement](image)

**Figure 7-1** Panel stiffener arrangement

EDC assemblies were spaced equally around the perimeter of the panel and connected to the test frame shown in Figure 7-2. Panels for the first test were installed on the front side and panels for the second test were installed on the back side of the test frame. Therefore, in order to re-set for the second test the target was lifted with a crane, located closer to the charge and rotated. This minimized the time and resources required on site to re-set the target.

The test frame was a welded steel moment frame made from rectangular hollow sections. The corner sections were HSS150x150x5 while the top and bottom beams were HSS120x60x3.6 and the internal columns were HSS100x100x4.
The roof and ends of the test frame were covered with 4 mm steel sheets welded to the frame in order to prevent the blast wave entering the test structure. In order to limit clearing effects concrete cubes were stacked along the ends of the test structure as shown in Figure 7-3.

The top of the structure was left as shown in Figure 7-3. As a result, clearing effects would affect the loading on the panels; however, this would be equal for all the panels.
7.2 Test Setup

The test panels were installed in a 2-1-2 configuration. The heavy panel (designated by (H) in Table 7-1) was in the middle bay. The left pair and the right pair each had the same EDC configuration. The test arrangement is summarized in Table 7-1 along with the predicted peak pressure and impulse loading.

<table>
<thead>
<tr>
<th>Test</th>
<th>EDC Configuration</th>
<th># of EDCs</th>
<th>Top/Bottom</th>
<th>Side</th>
<th>Total</th>
<th># of Panels</th>
<th>Panel Mass (kg)</th>
<th>Charge Weight (kg-TNT)</th>
<th>Standoff (m)</th>
<th>Predicted Peak Pressure (kPa)</th>
<th>Predicted Impulse (kPa-ms)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>EDC1</td>
<td>6</td>
<td>14</td>
<td>40</td>
<td>2</td>
<td>136</td>
<td>412</td>
<td>35</td>
<td>106.1</td>
<td>928.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EDC1 (H)</td>
<td>6</td>
<td>14</td>
<td>40</td>
<td>1</td>
<td>433</td>
<td>412</td>
<td>35</td>
<td>106.1</td>
<td>928.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EDC2</td>
<td>8</td>
<td>16</td>
<td>48</td>
<td>2</td>
<td>136</td>
<td>412</td>
<td>35</td>
<td>106.1</td>
<td>928.5</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>EDC3</td>
<td>6</td>
<td>14</td>
<td>40</td>
<td>2</td>
<td>136</td>
<td>412</td>
<td>15</td>
<td>1021</td>
<td>2662</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EDC4 (H)</td>
<td>8</td>
<td>16</td>
<td>48</td>
<td>1</td>
<td>433</td>
<td>412</td>
<td>15</td>
<td>1021</td>
<td>2662</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EDC4</td>
<td>8</td>
<td>16</td>
<td>48</td>
<td>2</td>
<td>136</td>
<td>412</td>
<td>15</td>
<td>1020</td>
<td>2662</td>
<td></td>
</tr>
</tbody>
</table>

The heavy panels had the same configuration as one of the other pairs of panels so that a comparison could be made considering the mass as the only variable.

7.2.1 Instrumentation

The test target and panels were instrumented with reflected pressure gauges and displacement gauges. In addition, a free-field gauge was located near to the target. Two Kulite pressure gauges with a maximum rating of 350 kPa were mounted into the steel frame as reflected pressure gauges for the first test. These were located on the first and fourth interior columns. The gauges were mounted into a threaded nylon mount. The mounts were then installed into a threaded steel pipe welded through the column as shown in Figure 7-4.

![Reflected pressure gauge mount](image)
The reflected pressure gauges were changed to 6895 kPa maximum rating Kulite piezoresistive gauges for the second test due to the much smaller standoff distance. A Kulite piezoresistive free field pressure gauge with a maximum rating of 175kPa was located approximately 3m to the side of the target at the same standoff distance at a height of approximately 1.5m for the first test. This was changed to a 350 kPa maximum rating Kulite piezoresistive pressure gauge for the second test.

The displacement gauges consisted of an aluminium channel with gear teeth fixed inside. This rod turns a dashpot as it moves through a guide as shown in Figure 7-5. The displacement gauges had a maximum stroke of approximately a metre.

![Displacement gauge](image1)
![Installed displacement gauge](image2)

**Figure 7-5** Displacement gauge  **Figure 7-6** Installed displacement gauge

The displacement gauges were mounted to a 5mm steel plate that spanned between the middle two stiffeners at the centre of each panel on the internal side, as shown in Figure 7-6. The mounting bracket on the panels was free to rotate about the vertical and horizontal axes minimizing any bending in the rod. The displacement gauges were mounted to a steel tube with a diameter of 200mm and a thickness of 5mm. The mounting tubes were welded to the roof frame of the test stand in order to make them as rigid as possible as the floor framing was not accessible and the flooring too flexible. Figure 7-7 shows all the displacement gauges installed in the test structure.
All gauges were connected to a Hi-Techniques meDAQ data acquisition system with built-in signal conditioning for piezoresistive gauges. Data was recorded at a rate of two million samples per second over 500ms. The data acquisition system was triggered using an opto-isolated circuit and a break-wire wrapped around the charge.

### 7.2.2 Charge Configuration

The charge was composed mainly of 495kg of commercial Ammonium Nitrate and Fuel Oil (ANFO). A C4 booster charge with a mass of 5kg was located in the centre of the ANFO as shown in Figure 7-8. The ANFO was poured into a cylindrical nylon bag and had a final diameter of 0.93m and a final height of 0.93m.
The TNT equivalency of C4 is 1.3 while the TNT equivalency of ANFO is 0.82, which results in a total TNT equivalency of 412kg of TNT.

Predictions of blast pressures are typically made using either spherical or hemispherical charges. However, it has been known for some time that the charge shape can have a significant effect on both pressure and impulse (Stoner and Bleakney, 1948). Knock and Davies (2012) reviewed the existing test data, performed tests using PE4 and RDX cylinders, and found that as the L/D ratio increases for a constant charge mass and standoff distance the peak pressure increases while the impulse remains constant. However, there was not enough experimental data to confirm this finding. Esparza (1992) analyzed test results from Plooster (1982) in order to develop spherical equivalency factors for cylindrical charges. Using the charts developed by Esparza (1992) the mass equivalency factors for peak side-on pressure and impulse for tests 1 and 2 were determined and are shown in Table 7-2.

**Figure 7-8** Charge composition
Table 7-2  Mass equivalency factors for cylindrical charges

<table>
<thead>
<tr>
<th>Test</th>
<th>Charge Weight (kg-TNT)</th>
<th>Standoff (m)</th>
<th>L/D</th>
<th>Mass Equivalency Factor (side-on pressure)</th>
<th>Mass Equivalency Factor (side-on impulse)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>412</td>
<td>35</td>
<td>1</td>
<td>0.77</td>
<td>1.03</td>
</tr>
<tr>
<td>2</td>
<td>412</td>
<td>15</td>
<td>1</td>
<td>1.00</td>
<td>0.98</td>
</tr>
</tbody>
</table>

For the predictions, an average of the mass equivalency factors for side-on pressure and impulse was used. This results in an equivalent charge weight of 370.8 kg-TNT for test 1 and 407.9 kg-TNT for Test 2.

7.3 Results

7.3.1 Test 1

The temperature for the first test was 35°C and the atmospheric pressure was 100.7 kPa with clear sky conditions. The wind speed was 9.4 km/h from the North. All gauges triggered and recorded data with no gauges malfunctioning. The time of arrival of the shock wave on the target was 57 ms, which matches the predicted time of arrival. The measured free-field pressure-time history and the predicted (by Conwep) pressure-time history are shown in Figure 7-9.

![Figure 7-9 Test 1 free-field pressure](image)

The predicted and measured pressure-time histories correlate very well, with the predicted peak pressure being slightly higher than the measured value as summarized in Table 7-3.
Table 7-3  Test 1 Free field pressure and impulse

<table>
<thead>
<tr>
<th>Curve</th>
<th>Peak Pressure (kPa)</th>
<th>Impulse (kPa-ms)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ch 29 - FF</td>
<td>41.8</td>
<td>341.6</td>
</tr>
<tr>
<td>Prediction</td>
<td>45.2</td>
<td>435.5</td>
</tr>
</tbody>
</table>

While the peak free-field pressure matched closely, the measured impulse was 22% lower than predicted. This is most likely due to variations in the TNT equivalency of ANFO used to make the predictions. The TNT equivalency of any explosive is not constant, and therefore errors as high as 50% have been found for published TNT equivalency factors (Locking, 2011).

The predicted and measured reflected pressure time histories for Test 1 are shown in Figure 7-10. The measured time of arrival was 49ms, which was slightly earlier than the predicted time of arrival of 57ms. This discrepancy could be the result of several factors such as non-uniformity of the blast wave, a time shift in the reflected pressure gauges, or variations in the TNT equivalency of ANFO. Ultimately, this difference in the time of arrival does not affect the results of the test, as the most important quantities are the reflected pressure and impulse.

![Figure 7-10 Test 1 reflected pressure-time history](image)

The two reflected pressure-time histories matched quite closely. This shows that there was a uniform blast load across the width of the target. When compared to predictions, as shown in Table 7-4, the peak reflected pressures were approximately 10% higher than predicted, while the measured reflected impulse
was 34% lower than predicted. Much of this difference is due to clearing effects over the top of the target as well as variances in the TNT equivalence of ANFO.

<table>
<thead>
<tr>
<th>Curve</th>
<th>Peak Pressure (kPa)</th>
<th>Impulse (kPa-ms)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ch 15 - RP</td>
<td>117.8</td>
<td>618.5</td>
</tr>
<tr>
<td>Ch 16 - RP</td>
<td>114.6</td>
<td>609.3</td>
</tr>
<tr>
<td>Prediction</td>
<td>106.1</td>
<td>928.5</td>
</tr>
</tbody>
</table>

If clearing effects are included, the predicted peak reflected pressure is 103.2 kPa and the peak reflected impulse is 734.9 kPa-ms. This results in the measured impulses being 16% lower than predicted.

The measured displacement-time histories for the panels with 48 EDC3 assemblies located around the perimeter are shown in Figure 7-11. The displacement behaviour is characterized by an initial peak displacement followed by a return towards the original position then movement back towards a permanent displacement. The peak inwards displacement was very similar for the inner and outer panels.

![Displacement-Time History](image)

**Figure 7-11** Test 1 - Panels 1 and 2 (48-EDC2) displacement-time history

The outer panel continued to move outwards to a maximum before moving back towards its final permanent position. The inner panel only moved outwards by a small amount before moving back towards its final position. The final displaced state of the panel is shown in Figure 7-12.
Part of the reason for the return to the original panel position is the negative phase of the blast pressure-time history, which coincides with this return behaviour. This shows that the negative phase is important in this case and should be considered in SDOF analyses. The reflected pressure-time history has been overlaid onto Figure 7-11 to illustrate this correlation. Ultimately, the two most important parameters, the peak displacement and final permanent position, matched very well for the two panels.

The panels with 40 EDC1 assemblies located around the perimeter behaved in a similar way to the EDC3 panels. The displacement-time history for these two panels is shown in Figure 7-13. The panels both displaced outwards from their original position with the inner panel outwards more than the outer panel.
Figure 7-13  Test 1 - Panels 4 and 5 (40-EDC1) displacement-time history

The final displaced state of the EDC1 panels is shown in Figure 7-14.

Figure 7-14  EDC1 panel after test 1

The heavy version of the 40 EDC1 panel had a similar behaviour as the other panels in the first test, as shown in Figure 7-15.
The panel had a final position approximately equal to its starting position.

All five panels showed a similar displacement-time history where there was an initial peak displacement followed by a return to the original position with all panels passing the original position and moving outwards, some by as much as they moved inwards, before moving back towards a final inwards displacement. Comparing the heavy weight EDC1 panel to the normal weight panel it is clear that the mass of the panel has a large effect on reducing peak and final displacements, as well as the maximum velocity of the panel.

### 7.3.2 Test 2

The temperature for the second test was 39°C and the atmospheric pressure was 100.9 kPa with clear sky conditions. The wind speed was 20.8 km/h from the North. All gauges triggered and recorded data except one reflected pressure gauge and the free-field gauge. The predicted time of arrival of the blast wave on the target was 12.8 ms while the measured time of arrival was 10.6 ms. The measured and predicted (by Conwep) reflected pressure-time histories are shown in Figure 7-16.
The predicted and measured peak reflected pressure matched well, with the measured peak pressure being 5% larger than predicted. Similarly, the measured reflected impulse matched the predicted impulse well with the measured impulse being 3% larger than predicted. This is summarized in Table 7-5.

<table>
<thead>
<tr>
<th>Curve</th>
<th>Peak Pressure (kPa)</th>
<th>Impulse (kPa-ms)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ch 15 - RP</td>
<td>1069</td>
<td>2735</td>
</tr>
<tr>
<td>Prediction</td>
<td>1021</td>
<td>2662</td>
</tr>
</tbody>
</table>

The displacement-time response of the panels in Test 2 did not show the return to the original position. Instead, the panels reached a peak displacement and then returned directly to their final position. The displacement-time history for the 40 EDC3 panels is shown in Figure 7-17.
An Energy Dissipating System for Blast Mitigation in Structures

Figure 7-17  Test 2 - panels 1 and 2 (40-EDC3) displacement-time history

The reason for the absence of a return to the original position as observed in Test 1 can be seen from the reflected pressure-time history overlaid on the displacement-time histories of the panels in Figure 7-17. The negative phase of the blast occurs while the panel is still moving towards its peak displacement and not during the rebound. As a result, the panel is not pulled back to its original position.

The condition of the EDC3 panel after Test 2 can be seen in Figure 7-18. It is apparent from the final condition of the panel that the system reached its maximum displacement limit since some of the EDCAs had detached from the panel. It can also be seen that the panel did not deform noticeably, therefore ensuring that the entire blast load was transferred to the EDCs.
An Energy Dissipating System for Blast Mitigation in Structures

Unfortunately, the displacement gauge disconnected from the inner EDC4 panel and therefore did not record the displacement-time history. However, the displacement-time history was recorded for the outer panel. The displacement time-history for the outer 48-EDC4 panel is shown in Figure 7-19.

![Figure 7-18 EDC3 panel after Test 2](image)

As with the EDC3 panels the EDC4 panel reached a peak displacement before returning to its final position. The condition of the panel after Test 2 is shown in Figure 7-20.
It is clear from the figure that the panel had reached its maximum displacement since some of the EDCAs had separated from the panel. It can also be seen that the panel did not deform noticeably and therefore transferred the entire blast load to the EDCs.

The displacement-time history for the heavy version of the 48-EDC4 panel is shown in Figure 7-21. The displacement gauge detached from the panel after it had reached its final position, which can be seen by the large jump in the displacement-time history. Despite this, the peak displacement and final inwards displacement could still be determined.
Figure 7-21  Test 2 - panel 3 (48-EDC4 (H)) displacement-time history

The condition of the panel after Test 2 is shown in Figure 7-22. In this case, it can be seen that the panel did not reach its maximum displacement, none of the EDCAs separated from the panel and they were not completely stretched.

Figure 7-22  Heavy EDC4 panel after test 2

The displacement-time histories measured from the first and second tests differed due to the return to the original position, which occurred in the first test and not the second test. This is due to the negative phase of the blast occurring during the rebound of the panel in the first test, but during the inwards
movement in the second test. Additionally, it was clear that the panels behaved rigidly and transferred the entire blast load to the EDCs.

7.4 Observations

After the first test, it was observed that the bottom corner of the frame of the outer 48 EDC2 panel had been pulled off the RHS support frame. Six of the self-tapping screws were pulled out during the negative phase of the blast as shown in Figure 7-23. While this did not make a significant difference to the results, corrective action was taken to ensure that it did not happen again on the second test. The frame was spot welded to the test structure in order to prevent it from separating as shown in Figure 7-24.

![Frame separation in Test 1](image1)

![Modification of frame for Test 2](image2)

Figure 7-24 Modification of frame for Test 2

Figure 7-23 Frame separation in Test 1

No damage happened to the displacement gauges in the interior of the target in the first test, as shown in Figure 7-25.
In contrast to the first test, there was significant damage to the target and displacement gauges after the second test due to the much higher blast loads. The interior of the target after the second test is shown in Figure 7-26. The middle three displacement gauges detached from their mounts, however, it seems that this occurred after the recording time had elapsed, except for the middle panel.
A few of the displacement gauge rods bent during Test 2, the most extreme case was at the inner 48 5EDCx5 panel where the rod had an extreme amount of bending, as seen in Figure 7-27. This bending was due to binding which may have occurred in the displacement gauge mechanism.
In addition to the rods bending, the plates which attach to the panels also bent, by between 17mm and 43mm, as shown in Figure 7-28. However, the plate attached to the heavy central panel did not bend significantly.

![Displacement gauge attachment plate bending](image)

**Figure 7-28** Displacement gauge attachment plate bending

Despite the damage in the second test, enough data was obtained to evaluate the predictions and design method.

### 7.5 Comparison to Predictions

Predictions were made using a numerical analysis to solve the single degree of freedom system discussed in Section 6.1. For the predictions, a number of assumptions were made. The elastic-perfectly plastic system described in Section 6.1 was used together with the properties described in Table 3-3 and Table 3-4. The stiffness and plateau load values were input as being the same in both the positive and rebound directions. This assumption will allow the maximum displacement to be obtained, but will not accurately predict any rebound or residual displacement. The stiffness on the rebound phase depends on several factors including the EDC configuration, the length of elongation, and the boundary conditions. In most cases, the stiffness in the rebound phase is likely to be lower than the stiffness in the elongation phase.

Damping equal to 2% of critical was assumed. The blast loading used for the SDOF system included both the positive and negative phase as shown in Figure 7-29.
The positive phase of the blast load is represented by the modified Friedlander equation (Equation 2-2) and the negative phase is represented by equation 7-1 (USACE, 2008).

\[
\frac{P_s^-}{P_{so}} = \frac{27}{4} \left( \frac{t^-}{t_0^-} \right) \left( 1 - \frac{t^-}{t_0^-} \right)^2
\]  

7-1

The predictions for the peak displacement match reasonably well to the measured peak displacements for the first test. However, the residual displacements were greatly over estimated since the SDOF predictions do not capture the return to the original position observed in the test due to the assumption that the initial stiffness and plateau load is the same in the positive and negative directions. Comparisons of the predicted and measured displacement-time histories for Test 1 are shown in Figure 7-30, Figure 7-31, and Figure 7-32.
Figure 7-30  Comparison of EDC2 panel to prediction

Figure 7-31  Comparison of EDC1 panel to prediction
The peak displacement was over predicted for the EDC1 and EDC2 panels while it was slightly underestimated for the heavy EDC1 panel. The predictions for the second test were not as successful as the first test. Comparisons of the predicted and measured displacement-time histories for Test 2 are shown in Figure 7-33, Figure 7-34, and Figure 7-35.

**Figure 7-32** Comparison of the heavy EDC1 panel to prediction

**Figure 7-33** Comparison of EDC3 panel to prediction
The predictions for the EDC3 and EDC4 panels substantially over-estimated the peak displacements. However, the predicted and measured displacements matched closely for the heavy EDC4 panel. The reason for the over-predictions is that the EDC3 and EDC4 panels both exceeded their maximum displacement and entered into the hardening zone. The SDOF model does not consider this hardening and therefore will over-predict the displacement.
The peak displacements were generally predicted well, except in situations where the EDC assemblies were extended past the plateau range into the hardening region (EDC3 and EDC4 configurations in Test 2). In this case, the peak displacements were over-predicted. This over-prediction can be corrected by including the hardening region in the SDOF model; however, this could obscure the fact that the EDC assembly was being over-extended. For this reason, and for simplicity, the hardening region has not been included in the SDOF model. Elastic-plastic response spectra for blast loading can also be used to conservatively obtain the maximum response of the EDC assemblies instead of performing a full nonlinear SDOF analysis. Gantes and Pnevmatikos (2004) have derived elastic-plastic response spectra for exponential blast loading, including the negative phase. These response spectra can be used to get a quick indication of the maximum extension of the EDC assemblies without the need for non-linear SDOF software.

The predictions did not accurately predict the residual displacements. The reason for this discrepancy is that both the positive and negative directions of loading were treated as having the same initial stiffness and plateau load. As discussed in Section 7.5, this is not the case for the real system.

7.6 Evaluation of Design Method

In order to determine how well the design method performed in the blast tests the actual peak reflected pressure and impulse values can be input into the appropriate P-I diagrams for the different configurations of panel tested. In the first test, two different configurations were used: EDC1 and EDC2, with the EDC1 configuration also having two panel weights.

In the first test, the EDC2 panels were designed to reach their maximum displacement, as shown by the design location in Figure 7-36. The 50 EDCA P-I diagram was chosen because it was closest to the 48 EDCs which were selected. The reflected impulse was lower than predicted in Test 1; therefore, the panels only reached about 40% of their predicted maximum displacement, as shown in Figure 7-12. As a result, the measured loading puts the design well below the capacity of the EDC2 arrangement, as shown in Figure 7-36. This represents well what occurred in the test.
The design for both the normal weight and heavy version of the EDC1 arrangement is shown on Figure 7-37. The normal weight panel was designed to slightly exceed its maximum elongation so that the heavy panel would also be extended significantly. Since the reflected impulse was lower than predicted, the normal weight EDC1 panel only reached about 57% of the predicted maximum displacement, as shown in Figure 7-14. This can also be seen in Figure 7-37, the measured loading point is well below the maximum capacity curve for the EDC1 arrangement. A similar trend is apparent for the heavy version of the EDC1 arrangement. This shows that the design method was successful in providing safe designs for the first test.
In the second test another two configurations were tested: EDC3 and EDC4, with the EDC4 configuration also having two panel weights. The design for the EDC3 panel was chosen to exceed the maximum displacement capacity for this arrangement. As a result, the design was chosen to be well over the capacity of the EDC3 arrangement as shown in Figure 7-38. The measured peak reflected pressure and impulse was very close to the predicted peak pressure and impulse used for design. As a result, the measured location of the design is on top of the design location in Figure 7-38. As expected, the EDC3 arrangement exceeded its maximum displacement in Test 2, as shown in Figure 7-18.

**Figure 7-37** EDC1 panel design evaluation

**Figure 7-38** EDC3 panel design evaluation
The design for the EDC4 panels is shown in Figure 7-39. The intent of this design was to also exceed the maximum displacement of this configuration, as a result the design point on the P-I diagram is well above the EDC4 line for the normal weight panel and slightly above the EDC4 line for the heavy panel. The predicted peak pressure and impulse used for the design was very close to the measured peak pressure and impulse, therefore the measured and design locations on Figure 7-39 are virtually on top of each other.

![EDC4 panel design evaluation](image)

**Figure 7-39**  EDC4 panel design evaluation

In Test 2, the normal weight EDC4 panel exceeded its maximum displacement, as shown in Figure 7-20. However, the heavy version of the EDC4 panel did not exceed its maximum displacement as shown in Figure 7-22. The design and measured points are beyond the capacity of the EDC4 configuration. This shows that the P-I curves used for design may be overly conservative for EDC assemblies consisting of a large number of EDCs. The cause of this over-conservatism is the extrapolation of the laboratory data. The EDC4 arrangement has almost twice as many EDCs as the largest EDC assembly tested in the laboratory and as a result, the characterization is being extrapolated significantly. This causes the predicted maximum displacement capacity prediction of this configuration to be too low.

Despite the slightly over-conservative design resulting from using the design curves, these curves result in a safe design and allow for a simple design process. As more configurations are tested, either using further laboratory testing or through finite element modelling, the characterization can be improved resulting in more efficient designs.
8 Summary and Conclusions

The componentized energy-dissipating system for blast mitigation described limits the blast loads transferred to a structure and enables the protection of occupants and contents from a blast, permitting the rapid re-occupation of the structure after an explosive attack. The energy-dissipating components which, when joined into assemblies form the critical components of this componentized energy-dissipating system, were investigated in this research program.

The investigation began with a laboratory testing program which provided the basis for characterizing the behaviour of these assemblies. The testing program tested four different assemblies with a dynamic testing rig. The specimens were confirmed to have been loaded dynamically using high-speed video. The video showed that the specimens deformed sequentially, instead of in a random order as would be expected in a static test. This sequential deformation mode has also been observed in dynamic tests of metallic foams and is due to inertia effects. The load-displacement behaviour of the energy dissipating component assemblies was characterized by a long plateau region. The existence of this plateau region is ideal for energy-dissipating applications since it limits the peak force transmitted to the structure and extends the duration of loading. This behaviour allows the system to protect the primary structural components from damage while absorbing the blast loading.

Using the data obtained from the laboratory testing program a characterization of the energy-dissipating component assemblies was developed. Curve fits of the laboratory test results provided the relationship between the number of layers of EDCs, the plateau load and the initial stiffness. The curve fits also provided the relationship between both the number of layers and the number of EDCs in series to the maximum displacement capacity of an assembly. The number of layers showed a very strong linear relationship to both the plateau load and the initial stiffness. The maximum extension capacity of the energy-dissipating component assemblies was more complicated to characterize as it depended on both the number of layers as well as the number of energy-dissipating components in series. As a result, a linear combination of these terms was assumed and the coefficients determined by minimizing the difference between the assumed equation and the laboratory data.

From the relationships developed by the characterization a single degree of freedom model was developed. The single degree of freedom system assumed that the panel connected to the energy dissipating component assemblies was perfectly rigid, and therefore transferred the entire blast load to the energy dissipating component assemblies. The resistance function of the energy-dissipating component assemblies was idealized as bi-linear (elastic-perfectly plastic). The magnitude of the plateau
load was obtained from the characterization, as was the initial stiffness. This bi-linear single degree of freedom system was then solved using numerical methods to predict the displacement-time behaviour of an energy-dissipating component assembly under a given blast load.

The single degree of freedom model was used to develop a design method for the energy-dissipating component assemblies. The key design aid that was developed was a series of normalized pressure-impulse diagrams, which allow the selection of an energy dissipating component assembly. The use of these diagrams provides a safe selection of energy-dissipating component assembly for a particular design blast load.

In order to evaluate the validity of the characterization and design method two full-scale blast tests were performed. Panels, which were designed to be as rigid as possible, with energy-dissipating component assemblies installed around the perimeter were installed in a testing frame. Five panels were tested in each blast test. In each test, two different energy-dissipating component assemblies were used, with two panels of each assembly. The fifth panel was a heavier version of with the same arrangement of energy-dissipating component assemblies as one of the other panels. The panels were instrumented with displacement gauges in order to measure the displacement-time history of the panels. Additionally, two reflected pressure gauges were mounted on the surface of the structure and one free field pressure gauge was positioned at the same standoff. The blast tests showed that the panels moved inwards to their peak inwards displacement, and then moved outwards towards a final permanent displacement. In the first test, the panels actually moved outwards beyond their initial position, due to the negative phase of the blast wave corresponding to the rebound phase of the panel, before then moving inwards again to the final displaced position. In contrast, the panels in the second test rebounded directly to their final displaced position. This was due to the negative phase of the blast occurring during the initial inwards displacement and not during the rebound.

The predictions of the panel behaviour made using the single degree of freedom model matched the peak displacement of the panels for the first test well. However, the predictions for the second test greatly over-estimated the peak displacement of the panels. This was due to the model not considering the third phase of deformation, full extension to failure, of the energy-dissipating component assemblies.

Comparing the design points on the pressure-impulse diagrams to the actual behaviour, the diagrams were found to be a conservative way of designing energy-dissipating component assemblies. As a result, the design methodology proposed provides a safe and easy method to design a system using energy-
dissipating component assemblies. It should be noted, however, that the proposed design approach is empirical and based on a very limited number of assemblies.

9 Recommendations

There is a need for additional research in order to address some of the limitations identified in this research project. In particular, it would be very beneficial to have more development work done on the finite element model in order to validate it to experimental results. This model could then be used to populate the characterization with more data points resulting in a more accurate representation of the energy-dissipating component assemblies. This in turn could be used to improve the design method and ensure that designs are not overly conservative. Additionally, more laboratory tests could be conducted on higher numbers of energy-dissipating component assemblies in order to refine the prediction model and design method.

The single degree of freedom model does not predict the permanent displacement of the energy-dissipating component assemblies. This is due to the inwards and outwards initial stiffness and plateau load properties being the same in the model. This is not actually the case in reality and more research should be done to properly characterize the rebound behaviour.

This research project assumed that the panels connected to the energy-dissipating component assemblies did not deform and dissipate any of the energy from the blast. It would be advantageous to allow the panels to deform and dissipate additional energy from the blast. However, some means of calculating the energy dissipated by the panel and the energy directed to the energy-dissipating component assemblies needs to be developed.

This research program has proven that energy-dissipating methods for blast protection of structures are an innovative way to ensure the resilience of critical infrastructure. Continuing research in this area will help ensure that the people who depend on structures for their safety and way of life will be able to continue to rely on them despite the threat posed by an explosive attack.
References


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Appendix A:
Manufacturing Drawings
Part A - Steel plate
**Part B6 - Panel edge angle**
Part C14 - Panel edge angle
Part C16 - Panel edge angle
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**Part D - Stiffener**
Part E - Stiffener
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Part F - Stiffener
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Part G6 - Test frame ("Picture Frame") angle
Part G8 - Test frame ("Picture Frame") angle
Part H14 - Test frame ("Picture Frame") angle
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Part H16 - Test frame ("Picture Frame") angle
An Energy Dissipating System for Blast Mitigation in Structures

Panel Assembly
Panel assembly sections
Test frame assembly ("Picture Frame")
Panel installation into Test Frame
Appendix B:

SBEDS Parameters
Using SBEDS to predict ECDA response

Predictions can be made using the software package SBEDS (Single degree of freedom Blast Effects Design Spreadsheet) published by the U.S. Army Corps of Engineers Protective Design Center (US Army, 2009). The General SDOF capability of SBEDS can be used to solve an arbitrary SDOF system subjected to blast loads.

Once the General SDOF program has been initialized the blast load can be entered to suit a particular scenario. The Dynamic Shear Factors are left blank. The SDOF properties, shown in Figure D-1, need to be entered.

![Figure D-1](SBEDS SDOF properties input form)

The mass of the panel should be input in the appropriate cell. However, the mass of the panel must be divided by the area of the panel (m²) before being input into SBEDS.

A load-mass factor of 1 should be used since the full panel is loaded and moves. The load-mass factor (K_{LM}) is a ratio of the mass factor (K_M) to the load factor (K_L). The mass factor is the ratio of the...
equivalent lumped mass system to the actual mass of the element. The load factor is the ratio of the equivalent load to the actual load on the member, which is determined by setting the external work done by the equivalent load on the equivalent system to the work done by the actual loads on the actual system.

The stiffness value $k_1$ can be calculated using Equation 5-2. This stiffness value must then be multiplied by the total number of EDCAs around the panel edge to obtain the total system stiffness. The resulting stiffness value must then be divided by the panel area ($m^2$) before being entered into SBEDS. Care must be taken with the units as Equation 5-2 results in a stiffness value in units of kN/m. This must be multiplied by 1000 in order to get units of kN/mm then divided by the panel area ($m^2$) in order to get the required units of kPa/mm. The remaining stiffness values, $k_2$ to $k_5$, should be entered as 0.

The resistance values $R_1$ to $R_4$ should all be equal and can be calculated using Equation 5-1. This resistance value must be multiplied by the total number of EDCAs around the panel edge to obtain a total system resistance. The resulting stiffness must then be divided by the panel area ($m^2$) before being entered into SBEDS.

The equivalent elastic displacement $X_E$ can be calculated by dividing the resistance value $R_1$ by the initial stiffness value $k_1$. This is the displacement at the onset of the plateau load. The remaining SDOF parameters are not used in the analysis.