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UNIVERSITY OF SOUTHAMPTON

FACULTY OF ENGINEERING AND PHYSICAL SCIENCES

Civil, Maritime, Environmental Engineering and Sciences Unit

Scaling Masonry Structures in a Blast Environment

by

Robert Vernon Johns

Thesis for the degree of Doctor of Philosophy

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ABSTRACT

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SCALING MASONRY STRUCTURES IN A BLAST ENVIRONMENT

By Robert Vernon Johns

Masonry construction has been globally ubiquitous for centuries due to its significant compressive strength and diverse architectural potential. Despite this, masonry is vulnerable to lateral loads such as those produced by explosions. The 21st century has been marred by a growing trend and threat of terrorist incidents, many of which involve explosive detonations. These generate blast waves which transmit substantial specific impulse to masonry buildings, resulting in significant damage and hazard to the occupants and those nearby. This highlights the need for methods to assess masonry structural resilience and human hazard due to blast.

Full-scale testing of masonry response to blast is challenging due to the need for a large testing site in tandem with high construction costs. The original contribution of this PhD has been to develop and experimentally benchmark a new scaling method for blast-loaded masonry built with commercially sourced materials. By adopting the concepts of dynamic similitude, a reciprocal scale factor can be applied to a reduced-scale (model) structure's density to replicate a full-scale (prototype) structure's lateral to vertical force ratio. Two model and prototype pairs were subjected to loading from 100kg TNT at two overpressures where results have shown that the models were able to qualitatively replicate the prototype cracking and collapse and quantitatively characterise the debris. This research has also examined blast flow and masonry response models via CFD and the Applied Element Method (AEM). CFD results demonstrated acceptable agreement with the blast trial. AEM solutions indicated an ability to model masonry cracking or debris generation, but results showed the need for additional work to develop a semi-rigid corner restraint to model both response phases sequentially. A series of future trials have been designed to test the scaling method with long duration, large impulse blasts. This trial data will permit further benchmarking of AEM to examine its predictive capabilities.

DECLARATION OF AUTHORSHIP

I, Robert Vernon Johns

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Nomenclature

Symbol and definition		Adopted Units
$\vec{a}$	Acceleration	m/s ²
a_0	Ambient speed of sound	m/s
A	Area of target loaded by blast	m ²
b	Waveform parameter	n.d
C_d	Drag coefficient	n.d
c_{TNT}	Detonation propagation speed of TNT	m/s
e_{det}	Specific detonation energy	J/kg
e_t	Specific total energy	J/kg
e_{TNT}	Specific energy of TNT	J/kg
$\vec{F}_d$	Drag force	N
$\vec{F}_h$	Horizontal forcing	N
$\vec{F}_v$	Vertical forcing	N
$F_{kx\ par}$	Flexural strength of masonry in vertical bending	N/m ²
$F_{kx\ perp}$	Flexural strength of masonry in horizontal bending	N/m ²
g	Gravitational acceleration	m/s ²
i_r	Reflected impulse	Pa-s
i_s	Positive phase impulse	Pa-s
i_s^-	Negative phase impulse	Pa-s
m	Mass	kg

P_o	Atmospheric pressure	Pa
P_r	Peak reflected static overpressure	Pa
P_s	Peak free-field static overpressure	Pa
P_{stag}	Stagnation pressure	Pa
$P_{(t)}$	Pressure as a function of time	Pa
P_{2nd}	Peak second shock static overpressure	Pa
Δp_{min}	Peak free-field static underpressure	Pa
Q	Heat	J
q_s	Peak dynamic pressure	Pa
R	Stand-off	m
R_c	Charge radius	m
r_{TNT}	Physical TNT charge radius	m
S	Scale factor	n.d
t	Time	s
t_a	Arrival time of blast	s
t_c	Clearing time	s
t_d	Positive blast duration	s
t^+	Positive phase duration	s
t^-	Negative phase duration	s
t_{start}	Simulation start time	s
U_s	Velocity of air	m/s
$\vec{u}_{flow}(t)$	Velocity of blast flow field as a function of time	m/s

U_s	Incident wave velocity	m/s
u_s	Particle velocity behind wavefront	m/s
$\vec{v}_{masonry-init}$	Initial horizontal velocity of masonry fragment	m/s
V	Volume	m ³
w	Self-weight	N
W	Explosive yield/ charge mass	kg
W	Work	J
Z	Scaled distance	m/kg ^{1/3}
γ	Ratio of specific heats	n.d
μ	Orthogonal ratio	n.d
∇	Partial del operator (nabla)	n.d
ρ	Density	kg/m ³
ρ_{exp}	Explosive density	kg/m ³
ρ_o	Density of air ahead of the wavefront	kg/m ³
ρ_s	Density behind the wavefront	kg/m ³
ρ_{TNT}	Density of TNT	kg/m ³
$\rho\vec{V}$	Specific momentum	kg/m ² s

Chapter 1

Introduction

The 21st century has been partly defined by a growing trend of global terrorist attacks with a significant number being due to the detonation of improvised explosive devices (IED). After detonation, these IEDs generate blast waves which undergo complex interactions with nearby structures, often resulting in devastating levels of damage. This was poignantly demonstrated by the Easter Sunday Bombings in Sri Lanka (April 2019) which led to 253 fatalities and a further 500+ sustaining injuries. In particular, the St. Sebastien's Church bombing in Negombo was found to produce a significant distribution of internal masonry debris as shown in figure 1.1.

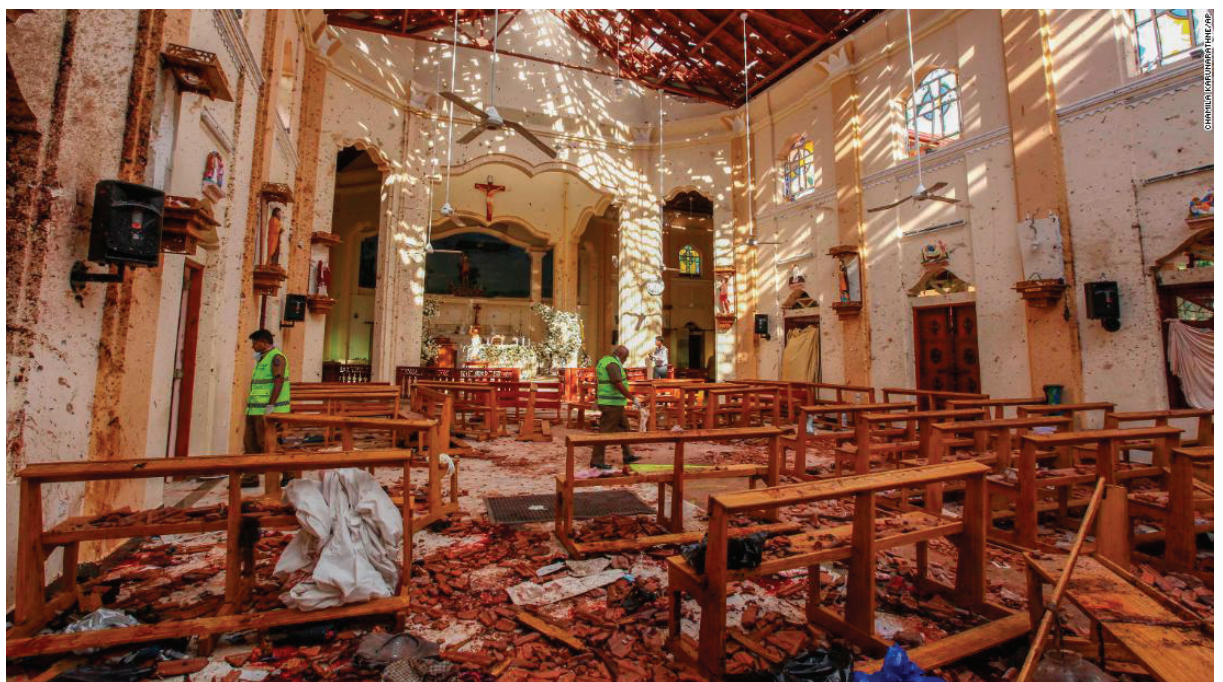


Figure 1.1: Masonry debris at St. Sebastien's Church, Sri Lanka, April 2019 (CNN, 2019)

The ongoing threat level from terrorist incidents is a global concern which emphasises the need for methods to determine the resilience of new and existing infrastructure alongside the associated human hazard for a range of blast events. Amongst the wide spectrum of structural types, masonry represents a significant proportion due to its sustained utilisation in developed and developing societies. In parallel with its widespread adoption, masonry construction has been implemented via a variety of methods, most of which can be classified as either dry or wet construction. Dry masonry relates to structures that have been built without a bonding agent whilst wet masonry is typically built by bonding courses of bricks with mortar. This research will focus on wet construction and in particular, the use of concrete bricks and blocks with mortar. The net result of which is a composite structure referred to as masonry.

The complex dynamic response of masonry to blast loading can be separated into two phases. The first of which is the lateral loading of an in-tact structure, resulting in cracking and deflection to characterise damage state. The second phase relates to the drag forcing of post-breakage projectiles, leading to a debris distribution and potential human hazard. The relative significance of these response modes is dependent upon explosive charge size and proximity, structural geometry and material properties. This PhD investigates the development and application of a new scaling methodology for masonry structures in an experimental blast setting, enabling the examination of structural response to a range of blast loads whilst reducing the economic implications and testing site size requirements that are typically associated.

1.1 Motivation and Scope

Full-scale blast testing of masonry structures can provide valuable data relating to structural resilience alongside potential hazard and risk due to masonry damage and debris. These trials can however be challenging to conduct with the need for a large and robust testing area in conjunction with substantial costs for masonry materials and construction. The aforementioned factors have been generally identified within the field of masonry structural dynamics, inspiring a number of investigations into scaling methodologies to circumvent these limitations. Principally, each of these studies has aimed to relate the response of a reduced-scale masonry structure (model) to its full-scale (prototype) equivalent.

A survey of the current body of knowledge has shown a diverse range of dynamic load cases as the basis for studies into masonry scaling. In particular, seismic loading studies led to the establishment of two scaling approaches, namely simple and complete modelling. The complete approach aims to utilise proprietary materials which scale the stress-strain relationship of the full scaled material via the scale factor. This can be difficult to achieve as it requires the design, manufacture and verification of material properties for reduced-scale bricks and mortar which can only be used in an experimental setting. Alternatively, the simple approach relies upon basic geometric scaling with the reduced-scale structure being built from full-scale versions of masonry materials. At present, the number of studies into the scaling of masonry in a blast environment is tremendously small with researchers having opted for basic geometric scaling and construction via full-scale masonry units. Whilst this approach is easily implemented, the accuracy of a simple modelling approach in a blast environment can be shown to be insufficient.

This PhD seeks to redress the current knowledge gap for scaling masonry in a blast environment by utilising the aerodynamic concepts of dynamic similitude to develop and experimentally benchmark a new scaling method. Specifically, this research outlines the development of a first principles scaling approach which utilises a reciprocal scale factor for the model structure's density to maintain the prototype's ratio of lateral to vertical forces as prescribed by dynamic similitude. In doing so, model structures can be deployed within smaller, cost-effective blast trials to characterise the damage (cracking and collapse) and debris distribution of a prototype structure. Data gathered during experimental trials has also enabled this research to benchmark computational models of masonry response and blast wave propagation produced with the Applied Element Method (AEM) and CFD respectively. It should be noted that this research has emphasised structural scaling and as such, it has not investigated the scaling of blast loads or the blast environment.

A significant aspect of this research is the utilisation of commercially available masonry units for construction at 1:2. In doing so, this research aims to improve upon the limited accuracy of a simple modelling approach whilst removing the manufacturing requirements of complete modelling. It is acknowledged that a 1:2 scale may still exceed the capacity of certain blast testing facilities, however, it is only currently possible to source commercial produced masonry units which conform to scaling requirements at 1:2 in the UK.

1.2 Aims

The principal aim of this PhD is to develop and experimentally benchmark a new scaling methodology for unreinforced masonry structures subject to blast loading by utilising the aerodynamic concepts of dynamic similitude. In doing so, reduced-scale (model) structures can be built using commercially available materials at a 1:2 scaling level and subsequently deployed within smaller, cost-effective blast trials to characterise the damage (cracking and collapse) and debris distribution of full-scale (prototype) structures.

This scaling methodology is intended to provide an alternative to the limited accuracy of simple geometric scaling and the complex manufacturing processes associated with a complete modelling approach. It will also enable the industrial sponsor, the Atomic Weapons Establishment (AWE), to design and conduct future masonry trials at the national blast test facility, MOD Shoeburyness. The results of which will refine their fast running engineering algorithms (FREA) for predicting masonry response to blast. These tools are an essential part of the UK's capability to respond to the ongoing threat of potential terrorist incidents.

The secondary aim of this PhD is to experimentally benchmark CFD and AEM models of blast flow propagation and masonry response to blast respectively. This will provide an insight into the capability of these techniques to serve in a predictive capacity when experimental trials are unavailable.

1.3 Objectives

The following objectives have been defined to fulfil the research aims:

1. Develop a scaling methodology for masonry structures subject to blast loading with a first principles approach which utilises the aerodynamic concepts of dynamic similitude and which enables construction via commercially available materials at a 1:2 scaling level
2. Design and conduct a short-duration blast trial to benchmark the scaling methodology's ability to accurately characterise masonry damage and debris distributions by examining model and prototype structural data sets over a range of blast overpressures

3. Conduct experimental benchmarking of CFD and AEM models of blast wave propagation and masonry response to determine predictive capacity of these computational methods
4. Design long-duration blast trials to examine the scaling methodology's ability to characterise masonry damage and debris distributions with larger impulse blast scenarios and conduct a precursory CFD and AEM investigation via potential trial results

1.4 Thesis Structure

The structure of this PhD thesis is outlined below on a chapter-by-chapter basis and the overall flow of work is shown in figure 1.2. This figure also demonstrates the contribution of each chapter to fulfil the four research objectives and thus address this PhD's aims.

Chapter 2

Details relevant background theory including blast formation, interactions with structures and blast parameters followed by the computational methods of CFD and AEM.

Chapter 3

Summarises masonry response to blast and scaling fundamentals before reviewing literature on masonry scaling for load cases including blast, seismic, wind and soil slippage.

Chapter 4

Development of a first principles scaling methodology which utilises the concepts of dynamic similitude. A brief discussion on scaled masonry response parameters followed by an examination of commercially available masonry which adheres to the scaling rules at 1:2. This chapter fulfils the PhD's first objective.

Chapter 5

Detailed examination of the experimental methodology for one 100kg TNT short-duration blast trial with the aim of benchmarking the scaling methodology developed in chapter 4, therefore partially fulfilling the second objective of this PhD.

Chapter 6

Thorough analysis of the short-duration blast trial results. Masonry damage is qualitatively assessed via visible cracking and degree of collapse. Debris distributions are quantitatively characterised via manual weighing and positional logging and with an experimentally validated inferential method. Blast environment data is analysed and compared with pre-trial predictions. The scaling method is benchmarked by comparing data sets for model and prototype structures, fulfilling the second objective of this PhD.

Chapter 7

Outlines the benchmarking of CFD models of blast flow and AEM models of masonry response using data from the PhD blast trial. A comparative study of CFD solvers Air3D and AUTODYN versus trial results precedes the remap of CFD data to configure AEM loading. Computational solutions are assessed to determine predictive capabilities when blast trials are not available, thus fulfilling the third objective of this PhD.

Chapter 8

Outlines long-duration blast trials to be completed at the national blast facility, the Air Blast Tunnel (ABT) at MOD Shoeburyness which aim to assess the PhD scaling methodology in higher impulse blast scenarios. Examines CFD and AEM models of blast flow and masonry response to long-duration blast to gain preliminary insights into the scaling method's performance. This chapter fulfils the fourth objective of this PhD.

Chapter 9

Concludes this PhD thesis by reviewing the research findings and the extent to which these have fulfilled the aims and objectives. This chapter also details areas of future work which were identified during the research period.

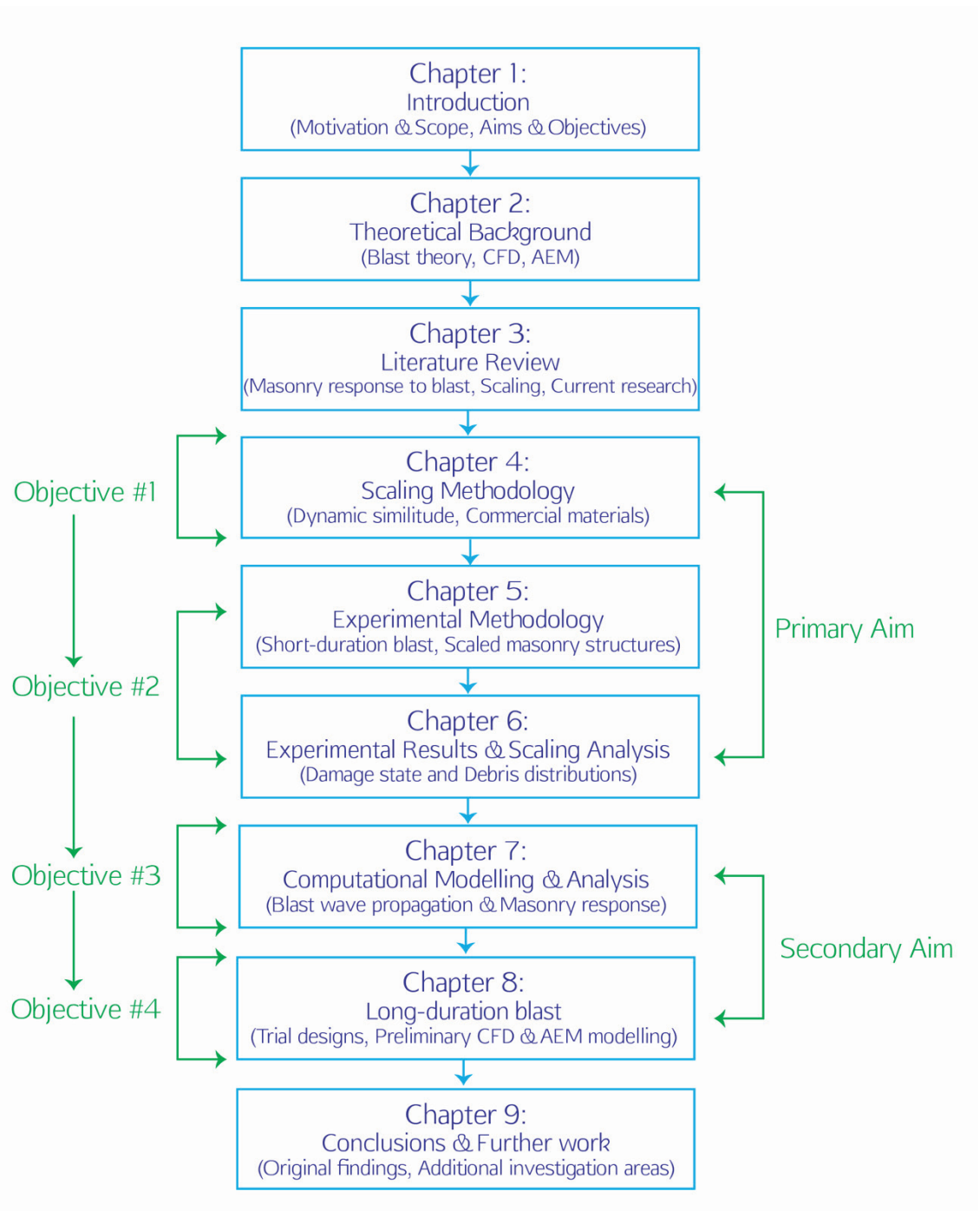


Figure 1.2: Flow diagram for PhD thesis structure.

Chapter 2

Theoretical Background

This chapter details the relevant theoretical background for this PhD research project. The first section outlines general blast wave interaction with structures followed by fundamental blast theory and essential blast parameters. The second section details the underlying theory of computational methods utilised within this research via CFD and AEM.

2.1 Essential blast theory

2.1.1 Explosive detonation

The detonation of high explosives such as trinitrotoluene (TNT) generates supersonic detonation shocks with a pressure range of 10-30GPa. As a result of chemical reactions, this detonation shock is a mixture of materials and densities referred to as 'detonation products'. The supersonic velocity enables a rapid breach of the explosive charge's physical surface, leading to the compression of ambient air and blast wave formation as shown in figure 2.1. This blast wave will be continually accelerated outwards, resulting in mass gas motion known as dynamic pressure. Detonation also generates a supersonic rarefaction wave which travels inwards, leading to self-reflection at 'ground-zero' and a trajectory reversal. At a similar instant, an outward-running second shock forms behind the blast wave. This is scarcely referenced in available literature with only Rose (Rose, 2001) theorising its cause to be the reflection of air from the detonation products. An alternative explanation may be deduced from shock tube analysis of a rarefaction wave which indicates that locally supersonic sections can produce additional shocks between compressions and rarefactions.

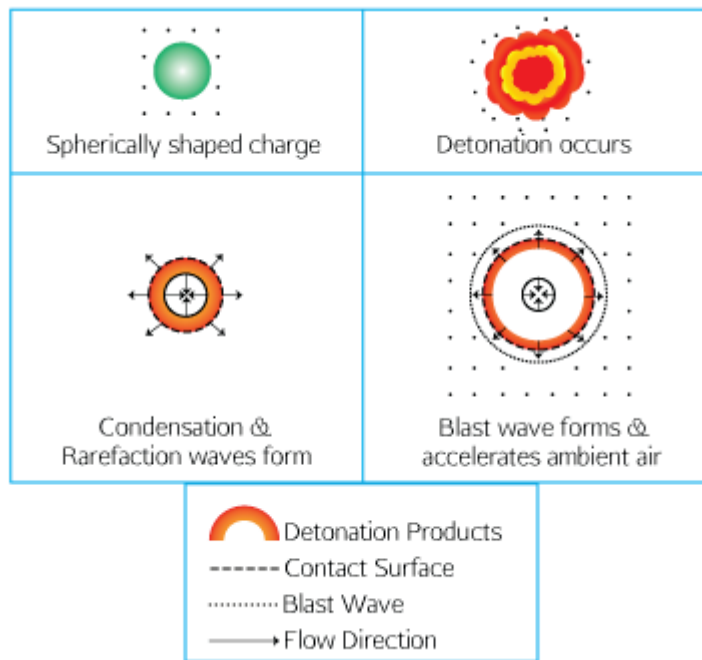


Figure 2.1: Explosive detonation process and blast wave formation.

Over the duration of a blast, the detonation product velocity continually decays as momentum is transferred to surrounding air. These products will then over-expand whilst slowing to rest, resulting in disconnection from the blast wave and the development of a negative pressure region. The detonation products and second shock will then travel inwards before the second shock self-reflects at ground-zero, resulting in a final outward motion within the negative pressure section of the blast as shown in figure 2.2.

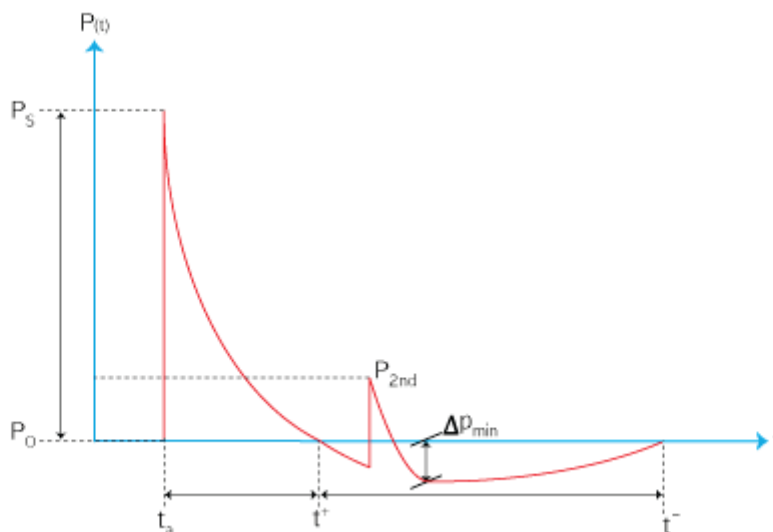


Figure 2.2: Blast profile time-history with second shock.

Where:

P_s = Peak static overpressure (free-field overpressure)

P_0 = Ambient air pressure ahead of the blast wave

P_{2nd} = Peak second shock overpressure

ΔP_{min} = Peak free-field underpressure

t_a = Blast arrival time

t^+ = Positive phase duration

t^- = Negative phase duration

2.1.2 Blast interactions

Within this research, blast events will be classified by positive phase duration. Long-duration events will be characterised by positive pressure durations exceeding 100ms, resulting in significant impulses and high power dynamic loads as produced by large-scale TNT, hydro-carbon and nuclear explosions. Recent examples include the ‘Buncefield Disaster’ (2005), the West, Texas (2013) fertiliser plant explosion and the chemical explosions at the Port of Tianjin, China (2015) as shown in figure 2.3. In contrast, short-duration blasts will encapsulate all explosive events with a positive phase duration below 100ms. These shorter duration blasts can be shown to represent a large majority of explosive scenarios.

In many instances blast waves are likely to encounter a solid surface, resulting in the wave’s particles being halted, compressed, reflected and potentially diffracted. The process of blast interaction with finite structures can be visualised in three categories as shown by Smith and Hetherington (Smith and Hetherington, 1994). Firstly, large structures will be crushed by the extremely high-force loads associated with long-duration blasts. The second scenario involves smaller targets with long-duration blast loads, producing similar levels of damage with additional drag loading and structural translation. The third scenario represents smaller, short-duration blasts which interact with large structures. In these cases, the blast will diffract around the structure in a process known as blast wave clearing. This initiates a complex process where the incident blast wave will be reflected in the opposite direction, enabling the

reflected wave to oppose the incident wave and mass gas motion as shown in figure 2.4. As the blast bends around a structure it will produce compressive forces at the side and top surfaces as a function of the time varying free-field overpressure. In contrast, the front surface will be compressed by reflected overpressure until the clearing interval has elapsed. After which the structure is subject to a time varying stagnation pressure which is an equality to total pressure as shown in equation 2.1.



Figure 2.3: Blast damage of Port of Tianjin, China, August 2015, (The Telegraph, 2015)

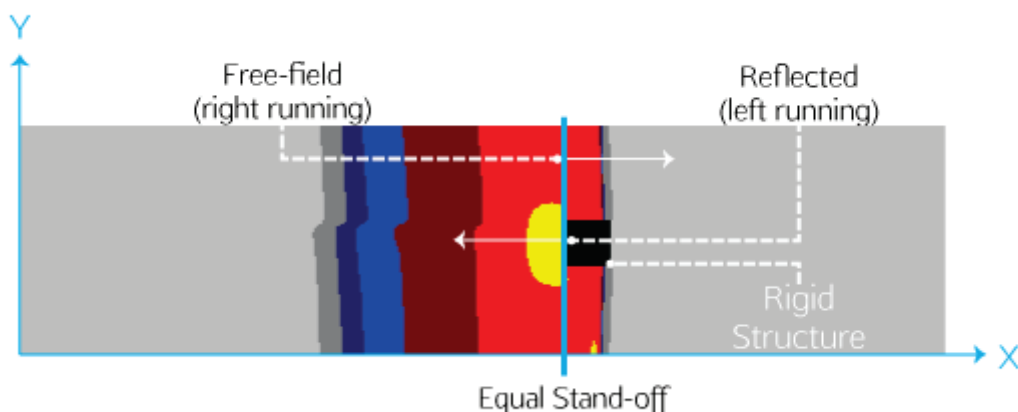


Figure 2.4: CFD visualisation of blast wave clearing, plan view.

$$P_{stag}(t) = P_s(t) + q_s(t) \quad (2.1)$$

Where:

$P_{stag}(t)$ = Time varying stagnation pressure

The free-field overpressure component in equation 2.1 can be shown to induce a translational force on the front surface of a structure until the structure has been enveloped by the wave, thus equalising the rear face pressure. The extent of this translation will be dependent upon structural dimensions, indicating that smaller geometries will be enveloped more rapidly to produce short duration pressure differentials. These instances may also increase the relative influence of viscous effects via skin friction and drag loading. Throughout the duration of a blast event, each of the tangentially orientated surfaces will be subject to skin-friction whilst the front surface will experience form drag. This can be calculated with the expression in equation 2.2 which is a function of time varying dynamic pressure, drag coefficient and presented structural area.

$$F_d = C_d \times q_s(t) \times A \quad (2.2)$$

Where:

F_d = Drag force

C_d = Drag coefficient determined by structural geometry

A = Presented structural area

Blast wave interaction with the ground can be visualised as the reflection of the wave from an infinite structure with the degree of reflections being determined by the height of detonation. As a result, blast events can be classified as either spherical or hemispherical bursts. This categorisation process is critical for calculating blast wave parameters as blast reflection from the ground can be shown to significantly amplify blast effects. In the case of a spherical burst, detonation occurs at an elevation which prevents any ground reflection, resulting in a fully spherical wave in three dimensional space. In contrast, hemispherical bursts occur at detonation heights below the limit for a spherical event, resulting in blast wave reflection and amplification of blast wave parameters by a factor of ~ 1.8 . Ground reflection can form craters to produce variable levels of ground shock. These additional effects result in an imperfect reflection to reduce the ideal increase factor of 2 down to ~ 1.8 .

2.1.3 Blast parameters

A comprehensive set of parameters derived from expressions to describe normal shocks (Rankine, 1870) enable the determination of a blast wave's essential characteristics. This is shown by equations 2.3-2.7 which calculate blast wave velocity, density of air behind the blast, maximum dynamic pressure, particle velocity behind the blast and time-varying dynamic pressure. Interestingly, the mass gas flow behind a blast wave is known as 'dynamic pressure' because it utilises the units of pressure. Examination of its expression in equation 2.7 shows it to be an equality for kinetic energy per unit volume, indicating that dynamic pressure can be viewed as be a property of the blast flow due to its proportionality to flow velocity. This contrasts with static overpressure which is a state variable for the blast wave.

$$U_s = a_0 \times \left(1 + \frac{6P_s}{7P_0} \right)^{1/2} \quad (2.3)$$

$$\rho_s = \frac{6P_s + 7P_0}{P_s + 7P_0} \times \rho_0 \quad (2.4)$$

$$q_s = \frac{5P_s^2}{2(P_s + 7P_0)} \quad (2.5)$$

$$u_s = \left(\frac{5P_s}{7P_0} \right) \left(\frac{a_0}{\left(1 + \frac{6P_s}{7P_0} \right)^{1/2}} \right) \quad (2.6)$$

$$q_s(t) = \frac{1}{2} \rho u_s^2 \quad (2.7)$$

Where:

U_s = Wave front velocity

ρ_s = Density behind the wave front

q_s = Maximum dynamic pressure

P_s = Peak static overpressure (free-field overpressure)

P_0 = Ambient air pressure ahead of the blast wave front

a_0 = Ambient speed of sound

ρ_0 = Density of air ahead of the wave front at ambient pressure

u_s = Particle velocity behind the wave front

$q_s(t)$ = Time varying dynamic pressure

ρ = Air density

Each of the expressions in equations 2.3-2.6 are a function of P_s which is known as peak static overpressure or free-field overpressure. Alongside the positive pressure duration parameter known as positive phase, free-field overpressure can be fed into the Friedlander function in equation 2.8 (Friedlander, 1946) to characterise an idealised pressure profile as shown in figure 2.5. The area beneath this curve represents specific blast impulse which is an equality for change in momentum of the ambient air due to blast propagation. This can be found by integrating the pressure curve as shown in equation 2.9.

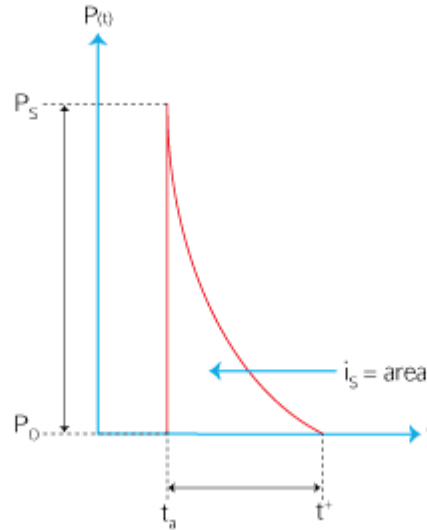


Figure 2.5: Friedlander function positive blast pressure profile.

$$p(t) = P_s \left[1 - \frac{t}{t^+} \right] \exp \left\{ -\frac{bt}{t^+} \right\} \quad (2.8)$$

$$i_s = \int_{t_a}^{t_a+t^+} P_s(t) dt \quad (2.9)$$

Where:

t = Time

t^+ = Positive phase duration

b = Waveform parameter (exponential decay rate)

t_a = Blast arrival time

i_s = Specific blast impulse

Equations 2.10-2.13 developed by Brode (Brode, 1954) enable the calculation of free-field overpressure and a range of negative blast phase characteristics. Each of these equations can be seen to be a function of the scaled distance parameter, Z , as shown in equation 2.14. This was derived by Hopkinson and Cranz (Baker, 1973) to demonstrate proportionality between independent charge masses which produce equal free-field overpressures at different stand-off distances.

$$P_s = \frac{0.975}{Z} + \frac{1.455}{Z^2} + \frac{5.85}{Z^3} - 0.019 \text{ bar} \quad (2.10)$$

$$\Delta p_{min} = - \frac{0.35}{Z} \quad (z > 1.6) \quad (2.11)$$

$$t^- = 1.25w^{1/3} \quad (2.12)$$

$$i_s^- \approx i_s \left[1 - \frac{1}{2Z} \right] \quad (2.13)$$

$$Z = \frac{R}{w^{1/3}} \quad (2.14)$$

Where:

Z = Scaled distance parameter, units $m/kg^{1/3}$

Δp_{min} = Peak free-field underpressure, units of bar

t^- = Negative phase duration

i_s^- = Negative phase impulse

w = Explosive yield or charge mass (kg)

R = Stand-off (radial distance from blast site)

When a blast wave encounters a solid surface, the particles within the wave are halted, compressed, reflected and diffracted. This will induce a peak reflected overpressure that exceeds the free-field overpressure value at the same stand-off distance. The calculation of this parameter can be made with the expression in equation 2.15. Examination of which

shows that reflected overpressure is a function of free-field overpressure and dynamic pressure. Short-duration blasts tend to introduce dynamic pressures which rapidly decay with increased stand-off whilst long-duration blasts tend to produce a significant dynamic pressure component. The presence of dynamic pressure therefore increases the kinetic energy and compressive forcing of blast wave particles against a reflecting surface, resulting in reflected pressures which are greater than the standard doubling effect that is found within general acoustic applications.

$$P_r = 2P_s + (\gamma + 1)q_s \quad (2.15)$$

Where:

P_r = Peak reflected overpressure

γ = Specific heat ratio of air (~ 1.4)

2.2 Computational methods

2.2.1 CFD governing equations

As a means of mathematically describing fluid flow, three fundamental physical laws can be applied to a theoretical fluid element as shown below. In their standard form, these laws describe a Lagrangian system which prohibits fluid flow across its boundary. This limitation can be removed by utilising stationary control volumes (CV) which enable fluid flow across the bounding surface within a Eulerian reference frame. These idealised CVs can represent finite and differential fluid elements where fluid property fluxes pass through an element's surface as shown in figure 2.6.

Conservation of mass; $\frac{dm}{dt} = 0$

Conservation of Momentum; Newton's Second Law; $\vec{F} = m \frac{d\vec{v}}{dt} = m \cdot a$

Conservation of energy; 1st law of thermodynamics; $dE_u = dQ - dW$

Where:

dE_u = Change in total internal energy

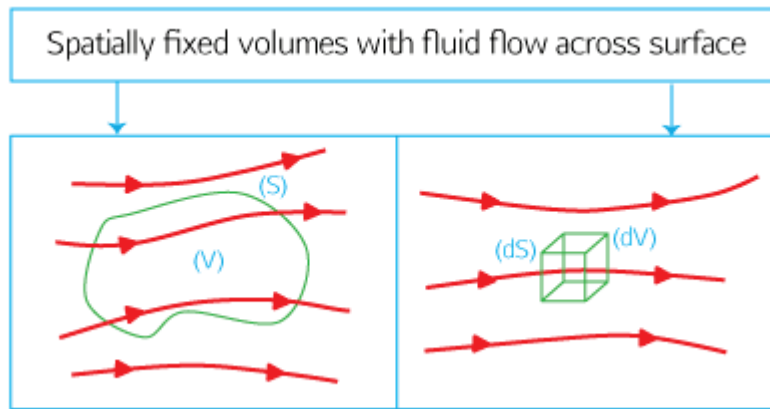


Figure 2.6: Fluid flow through the surface of a fluid element.

The mechanisms of transportation for fluid property fluxes across the control surface can be either convection via mass flow or diffusion due to molecular interactions caused by property gradients. The construction of a conservation equation for mass, momentum and energy can be accomplished by utilising the general form shown below.

$$\text{Accumulation Rate in CV} = \sum (\text{convection into CV}, \text{ diffusion into CV})$$

Fluid element motion can be described by four fundamental modes within two groups as shown in figure 2.7. Rigid translation and rotation represent non-viscous stresses whilst shearing and dilation involve the relative motion of fluid element sides to produce viscous stresses. These are distinguished by the non-reversible dissipative effects they exert upon a fluid which is particularly important within boundary layers of flow. By focussing on flow outside of boundary layers it is possible to model a notionally frictionless inviscid flow which disregards viscous stresses. These scenarios enable a ‘slip’ condition at the flow’s interface to a solid which simulates negligible friction whilst ensuring viscosity is not physically neglected.

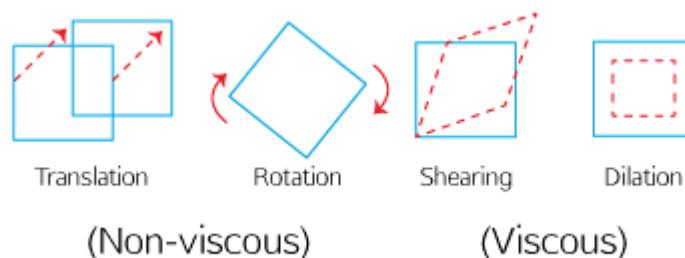


Figure 2.7: Categories of fluid element motion.

It can be shown that inviscid conditions provide a good approximation for blast propagation in the free-field. This can be extended to blast interaction with infinite and finite

structures and the subsequent application of static pressure at structural faces. The main limitation of this approach is the inability to model boundary layer regions or skin friction drag effects at structural faces due to viscous stresses. The mathematical representation of inviscid flow can be achieved by removing shear stresses and reducing diffusive surface forces to compressive normal stresses within the conservation expressions for momentum and energy. Conservation of mass requires no alteration for inviscid flow conditions. The resultant expressions are often termed the Euler equations as shown in equations 2.16-2.18.

$$\frac{d}{dt} \left[\iiint_V \rho dV \right] = - \iiint_V \nabla \cdot (\rho \vec{V}) dV \quad (2.16)$$

$$\iiint_V \frac{\partial(\rho \vec{V})}{\partial t} dV = - \iiint_V \nabla \cdot \vec{V}(\rho \vec{V}) dV + \iiint_V -\rho g dV - \iiint_V \nabla \cdot p dV \quad (2.17)$$

$$\iiint_V \frac{\partial \rho(e_t)}{\partial t} dV = - \iiint_V \nabla \cdot \rho(e_t)(\vec{V}) dV + \frac{dQ}{dt} - \frac{dW}{dt} \quad (2.18)$$

2.2.2 Air3D

Air3D (Rose, 2001) utilises the Euler equations to enable explicit modelling of inviscid blast flow in a Eulerian reference frame. Spatial discretization is conducted with a cubic mesh in a Cartesian coordinate system and boundaries can be modelled as flow out to permit fluid property convection or the inverse, termed reflecting. Air3D implements a standard shock tube methodology which utilises two distinct regions of gas with differing pressure values separated by a diaphragm. The instantaneous removal of which initiates a compression and rarefaction wave with opposing trajectories in one dimensional space. This enables the continual formation of finite waves in the zone directly behind the compression wave, leading to the formation of a discontinuous shock. Specifically, Air3D utilises an isothermal spherical driver gas which is configured with heat energy to replicate the chemical energy of a TNT mass. The spherical driver also enables the software to model blast wave curvature. The driver section's characteristics are determined via equations 2.19-2.20 where TNT values are referenced from Baker (Baker, 1973). The solution start time in equation 2.20 is set to simulate physical breach of the charge surface via instantaneous diaphragm removal. Program stability and solution time step is controlled by the Courant-Friedrichs-Levy (CFL) condition in equation 2.21 which prevents full wave propagation through a cell in one solution interval.

$$r_{TNT} = \left(\frac{3 \text{ charge mass}}{4 \pi \rho_{TNT}} \right)^{1/3} \quad (2.19)$$

$$t_{start} = \frac{r_{TNT}}{c_{TNT}} \quad (2.20)$$

$$\Delta t = cfl \frac{\Delta r}{\max(a + |u_i|)} \quad (2.21)$$

Where:

$\rho_{TNT} = 1.6 \times 10^3 \text{ kg m}^{-3}$ = Density of TNT

$e_{TNT} = 4.52 \times 10^6 \text{ J kg}^{-1}$ = Specific energy of TNT

$c_{TNT} = 6730 \text{ ms}^{-1}$ = Detonation propagation speed of TNT

r_{TNT} = Physical TNT charge radius

t_{start} = Simulation start time

Δt = Solution time interval

cfl = Courant-Friedrichs-Levy stability condition parameter (< 1)

Δr = 1D cell size

$\max(a + |u_i|)$ = maximum value of local sound speed in the i^{th} direction

Air3D utilises solution re-mapping to reduce computational overhead. 2D models re-map a 1D solution into a cylindrical domain where mesh coarseness can be shown to dictate re-map errors. Three dimensional models utilise the same approach with cubic cells. Non-responding structures can be defined in 2D and 3D via unavailable cells for a range of shapes including cuboids, cylinders and wedges. During its development, one dimensional benchmarking was performed with the Kingery and Bulmash predictive polynomials (Kingery and Bulmash, 1984), the results of which showed 1D model accuracy to saturate as a function of optimum 1D cell size for any TNT charge mass as shown in equation 2.22.

$$\text{1D cell size} = w^{\frac{1}{3}} / 10^3 \quad (2.22)$$

2.2.3 AUTODYN

AUTODYN utilises a Eulerian reference frame to explicitly model inviscid blast flow within a Cartesian co-ordinate system. Spatial discretization is conducted with a rectilinear mesh and boundaries can be modelled as flow out or reflecting. Much like Air3D, solid objects can be defined in 2D and 3D via unavailable cells to simulate infinitely rigid structures. AUTODYN also permits a graded zoning approach in 1D, 2D and 3D which enables regions of interest to be modelled with a finer cell size. This can be shown to impact the accuracy of the solution if the mesh transitions are too coarse. Solution time intervals are a direct function of the smallest cell size, emphasising the need to balance efficient run times with solution accuracy by utilising the optimum number of graded cells. Like Air3D, solution intervals are controlled by the CFL condition shown in equation 2.21 which prevents the blast wave fully propagating through a cell in one solution interval.

One dimensional blast analyses are conducted with a wedge shaped domain with axial symmetry where the divergent wedge shape enables blast wave curvature. The apex of the wedge is filled with TNT and the physical radius is determined by the volume of the explosive charge. One dimensional solutions utilise a Jones-Wilkins-Lee (JWL) equation of state versus the ideal gas equation used in Air3D which enables AUTODYN to accurately model TNT detonation and the propagation of mixed material detonation products. 2D and 3D models utilise re-mapping of a 1D solution after which AUTODYN remaps detonation products and atmospheric gas as a single ideal gas, namely air. These solutions are subsequently conducted with the ideal gas equation of state, replicating the approach used by Air3D in 2D and 3D. Unlike Air3D, AUTODYN does not conserve energy exactly throughout the solution with the developers citing issues with problem stability when attempting to do so. Energy errors can however be limited by a user-defined threshold to minimise the effect.

2.2.4 AEM theory

The Applied Element Method (AEM) is a computational structural dynamics methodology which utilises a continuum material approach with automatic material separation into discrete rigid body elements via 'virtual' discretization. Consequently, AEM implements a continuous and discrete structural method within a single solver, representing a

synergy of the Finite Element Method (FEM) and the discretized rigid bodies utilised by the Discrete Element Method (Lupoae and Bucur, 2009). Unlike the FEM which utilises a nodes located at element vertices and boundaries, AEM alters element connectivity by using a combination of face-connecting normal springs and shear springs in the x-z and y-z planes as shown in figure 2.8.

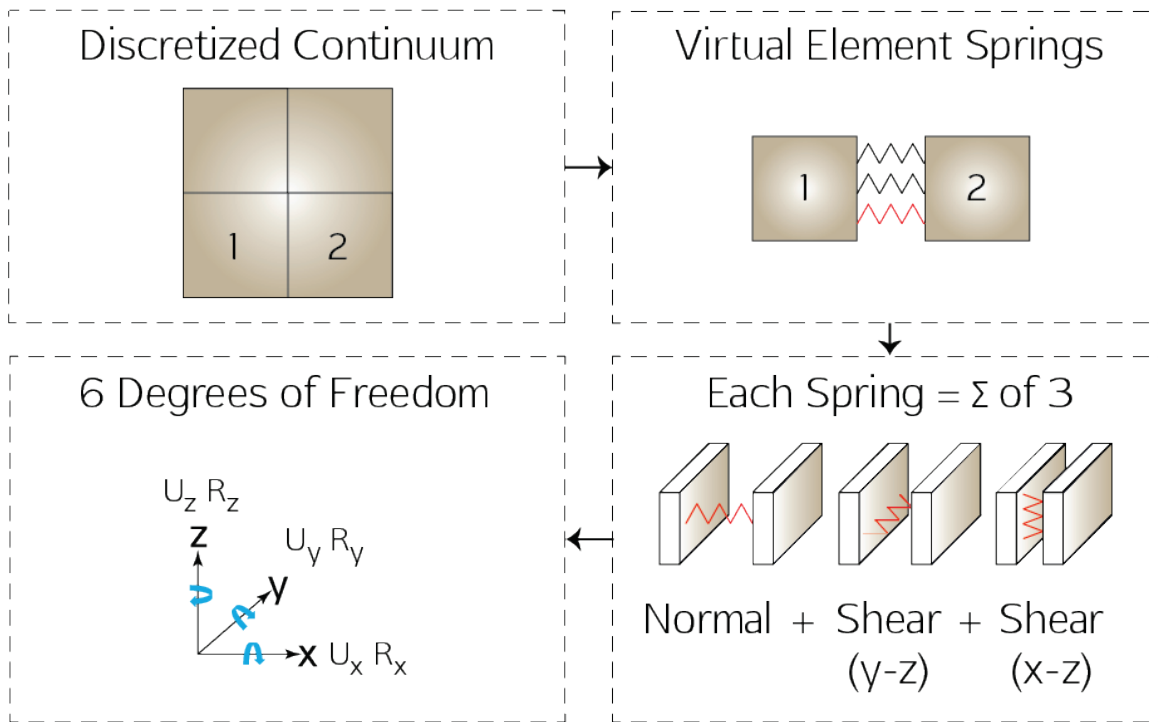


Figure 2.8: AEM material continuum discretization and element connectivity.

Element springs are distributed in sets of three over each element face to represent stresses and deformations in six degrees of freedom for the monitored element area. This allows the springs to represent element distortions whilst ensuring elements remain as rigid bodies after ejection from the continuum. Element separation is accomplished by applying material properties to each spring, allowing AEM to cross-reference maximum resistance forces for each spring during loading. The exceedance of which results in spring removal and the onset of cracking at an element location. The removal of a full spring group at a particular element results in the separation and ejection of an element from the continuum. This sequence of elastic response followed by cracking and element separation enables the full spectrum of response modes to be modelled including element collisions with the ground surface and other structural elements (Gohel and Joshi, 2013). The stiffness of each spring within an AEM model is determined by equations 2.23-2.24 where it can be seen that the

springs represents the stiffness of the area of an elemental face. Stiffness matrices can be subsequently formed for each spring with the sum representing a global stiffness matrix for each element. AEM calculations are then governed by a force-displacement relationship as shown by equation 2.25. AEM therefore parallels the force-displacement calculations associated with FEM models.

$$k_{normal} = \frac{E \times D \times T}{A} \quad (2.23)$$

$$k_{shear} = \frac{G \times D \times T}{A} \quad (2.24)$$

$$[F] = [K_G] \times [\Delta] \quad (2.25)$$

Where:

k_{normal} = *virtual spring normal stiffness*

k_{shear} = *virtual spring shear stiffness*

E = *Young's Modulus*

G = *Shear Modulus*

D = *Distance between springs*

T = *Element thickness*

A = *Length of representative area*

F = *Applied load vector*

K_G = *Global element stiffness matrix*

Δ = *Displacement vector*

The modification of element connectivity versus the nodal approach of FEM was borne out of the need to enable automatic element separation whilst generating an accurate model of global structural failure for a range of extreme loading scenarios including blast. In particular, the developers of AEM aimed to overcome the constraints associated with the FEM's Discrete Crack Method for modelling concrete cracking and element separation. Specifically, the need to pre-define failure location was found to constrain the Discrete Crack

Method to the determination of fail/no-fail without the ability to represent global failure mode (Meguro and Tagel-Din, 2000). Recent attempts to combat these limitations with the FEM include a study which implemented an element deletion methodology (Pelfrene, 2016). This aimed to achieve automatic element removal after failure but encountered issues with model instability and the onset of large instantaneous cracks which compromised global structural behaviour.

During its development, Meguro and Tagel-Din (Meguro and Tagel-Din, 2000) conducted a sensitivity study to ascertain the influence of element size and spring quantity on AEM solution accuracy. This investigated a laterally loaded concrete cantilever where results indicated that spring quantity per element face had no effect on the accuracy of translational motion. In contrast, rotational motion errors as a function of spring quantity were found in the range of 1-25% as shown in table 2.1. These results indicate that a minor increase in spring quantity per element face has a sizable influence on error reduction as demonstrated by a value of 2.8% for six spring sets versus 25% for two. Further investigation by Meguro and Tagel-Din (Meguro and Tagel-Din, 2000) demonstrated that error amplification was due to large element sizes relative to the global structural geometry. The implementation of small element sizes was found to eliminate errors as the relative rotation between small elements was significantly reduced. The utilisation of large element geometries was also found to introduce unrealistic increases in the model stiffness, resulting in a greater failure load and further inaccuracy. This sensitivity study concluded that rotational motion and stiffness errors could be completely eliminated by reducing element geometries and increasing the structure's total element quantity. This demonstrates that solution accuracy is predominantly controlled by the element discretization resolution.

Table 2.1: AEM sensitivity study, rotational motion error as a function of spring quantity.

Springs	2	4	6	8	10	20
Error (%)	25	6.3	2.8	1.6	1.0	0.3

2.2.5 Extreme Loading for Structures

Extreme Loading for Structures (ELS) (Applied science international, 2010) is an explicit AEM solver which provides the ability to model continuous elastic and discrete inelastic

phases of masonry response to blast loading. This is accomplished by the virtual discretization of the material continuum and element separation into discrete bodies after the exceedance of a separation strain threshold. This allows ELS to model masonry debris distributions after cracking and collapse, thus encapsulating the two phases of masonry response outlined in section 3.1.1. ELS solutions are modelled within a Lagrangian reference frame and the underlying force-displacement methodology provides a parallel with the first principles scaling methodology adopted within this research.

To demonstrate ELS' ability to model structural response to extreme loading, nine blind test cases of walls subject to blast loading were undertaken for the US Defence Threat Reduction Agency (DTRA) (Applied Science International, 2009). This demonstrated that ELS was able to closely match FEM results produced with LS-Dyna for load-displacement and failure patterns. More recently, research conducted by Keys (Keys, 2016) investigated ELS models of free-standing masonry panel response to short-duration blast loads. The results of which demonstrated qualitative agreement between ELS and experimental trial data for masonry breakage patterns with two structural geometries at three overpressures.

At present, ELS cannot model fluid-structure-interaction, introducing limitations for models with small fragments and pronounced drag loading. Masonry breakage can be shown to produce fragments of variable shapes, indicating a potential drag load influence. This scenario is unlikely in short-duration blast events where dynamic pressure is typically negligible and masonry response duration greatly exceeds blast load duration. In the case of long-duration blasts, it is expected that masonry breakage will produce large fragment clusters, indicating that diffraction loads will be the primary influence versus drag loading. If drag loading is significant, it may be possible to calculate drag loading outside of ELS and implementing additional loads within a simulation. This was suggested by Keys (Keys, 2016) when conducting preliminary ELS investigations into masonry debris distributions for long-duration blasts. The development and implementation of this drag-loading methodology is beyond the scope of this PhD but it does represent an area of potential future work.

Chapter 3

Literature Review

3.1 Introduction

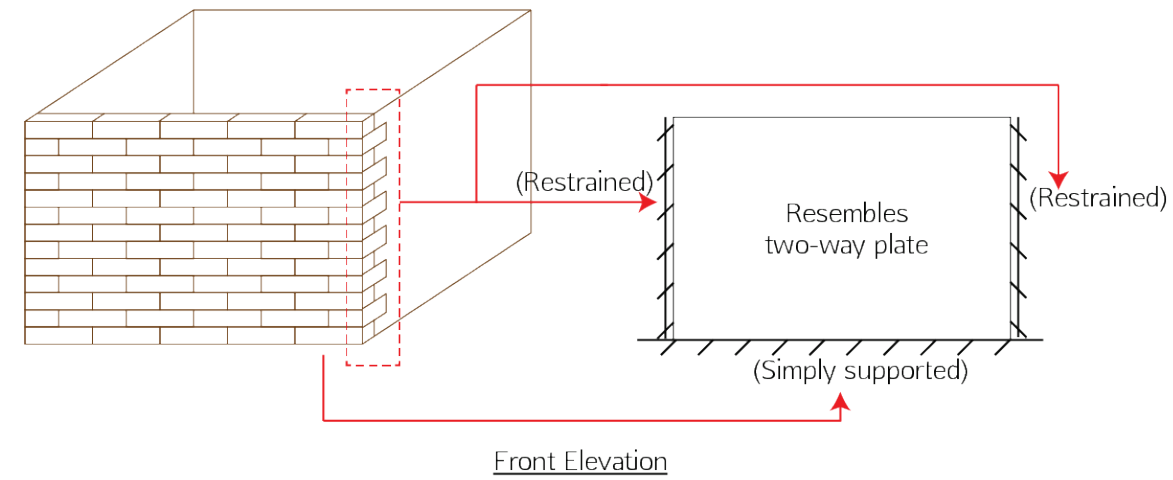
The first section of this chapter will outline masonry response to blast alongside the fundamentals of masonry scaling. The second section will review existing studies which investigate the scaling of masonry subject to blast loading. This will be followed by a chronological examination of current research into masonry scaling for a range of load cases including seismic loading, wind loading, soil excavation slippage and non-dynamic compressive loads.

3.2 Masonry structures

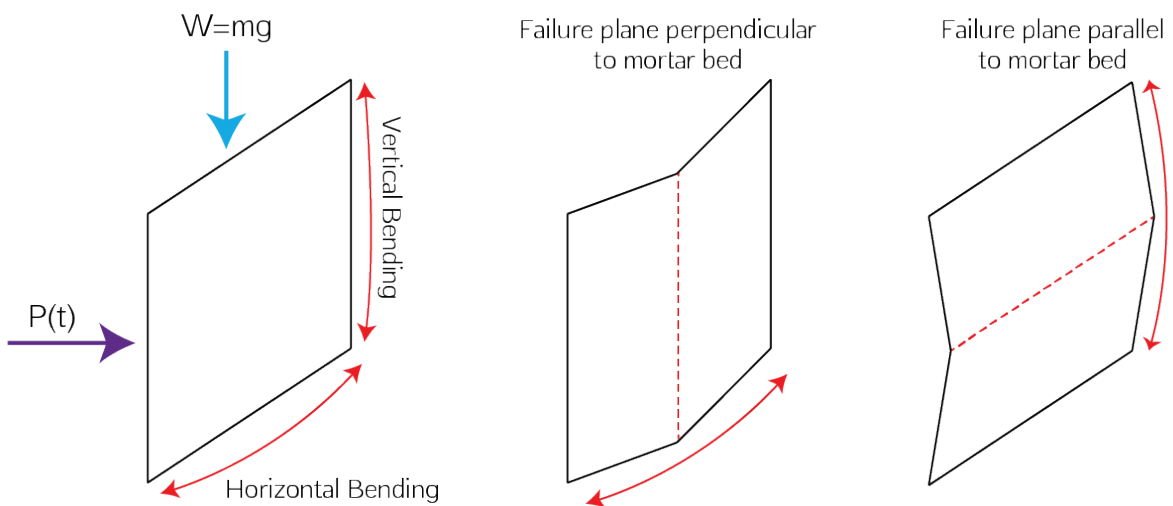
3.2.1 Response to blast

The complex dynamic response of masonry to blast loading can be subdivided into two sequential phases, namely the initial lateral loading of an in-tact structure followed by the drag forcing of post-breakage projectiles. Specifically, the first phase characterises structural damage state via cracking and varying degrees of collapse. This damage via the exceedance of a deflection limit which is a linear function of structural height will then sequentially initiate the second phase of debris distribution and associated hazard from airborne masonry fragments. In the simplest case, a non-load-bearing wall which is supported on multiple sides can be visualised as a panel wall (Arya, 2009). When subjected to the lateral loading of a blast

wave, these tend to behave in a similar manner to a two-way bending plate as shown in figure 3.1a. This can be visualised as having two separate bending modes as shown in figure 3.1b.



(a)



(b)

Figure 3.1: Masonry wall: (a) Panel wall assumption; (b) Panel wall failure modes.

Where:

$P(t)$ = Pressure as a function of time

W = Self-weight

m = Mass

g = Gravitational acceleration

Examination of figure 3.1b indicates two possible failure modes for a panel wall via vertical bending with failure parallel to the mortar bed and horizontal bending with failure perpendicular to the mortar bed. With respect to bending strength, panel walls can be defined by separate values in both the horizontal and vertical bending planes. It can be shown that the flexural strength in the horizontal direction will be significantly greater than that in the vertical direction. This occurs because the horizontal bending mode induces a mixture of shear and bending reactions whilst the vertical bending response typically fails when a crack forms at the interface between brick and mortar. The expression in equation 3.1 indicates that the ratio of flexural strengths is an approximate equality to 0.3, thereby underscoring the sizable difference in the two flexural strengths. Further inspection of figure 3.1b reveals that the blast load applied to the in-tact structure can be described as a time dependent lateral forcing, thus representing a horizontal force component. Similarly, structural self-weight can represent a vertical force component, the effect of which can appear more pronounced during the lateral deflection of the wall as it may be applied eccentrically and thus enhance the panel wall's buckling mode.

$$\mu = \frac{F_{kx\ par}}{F_{kx\ perp}} \approx 0.3 \quad (3.1)$$

Where:

μ = Orthogonal ratio of flexural strengths

$F_{kx\ par}$ = Flexural strength of masonry structure in vertical bending

$F_{kx\ perp}$ = Flexural strength of masonry structure in horizontal bending

The breakage mode of a laterally loaded panel wall can be shown to be a function of the degree of fixity at each of the wall's edges. Figure 3.2 illustrates a range of fracture patterns that are a function of edge restraint combinations. Closer examination of figure 3.2 indicates that the composite strength of a masonry structure can exceed that of the individual brick and mortar elements, resulting in the splitting of masonry units. The unit splitting behaviour identified in figure 3.2 can be observed at both the centre of a panel and at 'built-in' corners. These corner restraints are designed to induce unit failure in flexure before allowing the corner to rotate (Arya, 2009), thus representing a fixed structural support with inbuilt moment resistance. Due to the sequential nature of masonry response to blast

loading, it can be shown that edge fixity has a direct influence on the likelihood and extent of panel wall collapse. The degree to which this occurs can subsequently be shown to influence the possible onset of fragmentation and the final masonry debris distribution.

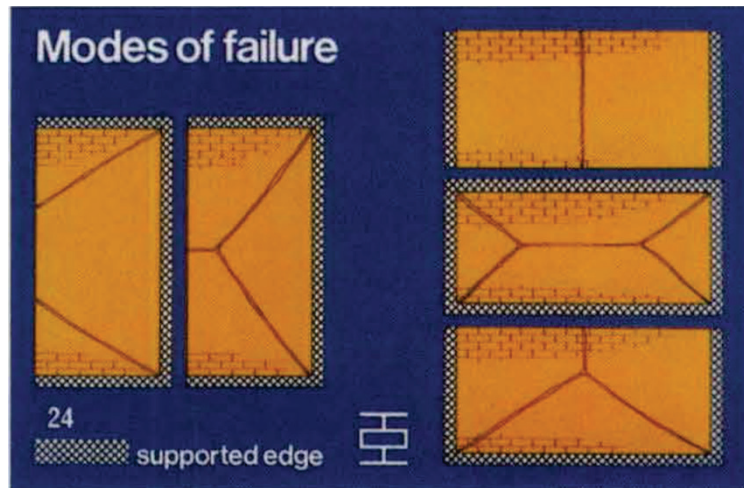


Figure 3.2: Masonry panel wall fracture modes as a function of edge restraint (Brick.org.uk,2019).

The post-breakage response of masonry fragment projectiles is demonstrated in figure 3.3 where it can be seen that fragments of variable geometry and mass will be propelled in the downstream direction. These have an initial horizontal velocity which is determined by pre-break forcing via transferred specific impulse. With relatively small geometries, masonry fragments tend to possess drag target characteristics, resulting in a relative increase in the influence of form drag. This can produce an additional horizontal velocity component due to the dynamic pressure and associated drag coefficient as shown in figure 3.3a. When visualised in one dimensional space, the direction of this additional velocity may vary if the fragment flight time exceeds the positive phase duration of a blast event. In these instances, fragments may be subject to a positive velocity during the positive blast phase and a negative velocity when dynamic pressure reverses in sense during the negative phase.

Further examination of figure 3.3a shows the vertical motion component of a masonry fragment to be a determined by self-weight, indicating that fragment flight times will be controlled by gravity. Consequently, it can be seen that the overall throw distance of a masonry fragment will be a function of its initial elevation within the structure , horizontal motion and acceleration due to gravity. At the instant of ground surface collision, each

fragment is likely to undergo variable levels of bounce and roll as shown in figure 3.3b. The extent of which will be determined by the collisional inelasticity of the fragments during contact with the ground. The final spatial debris distribution of a blast-loaded masonry structure will be the net effect of various fragment trajectories and the potential collapse of the original structure.

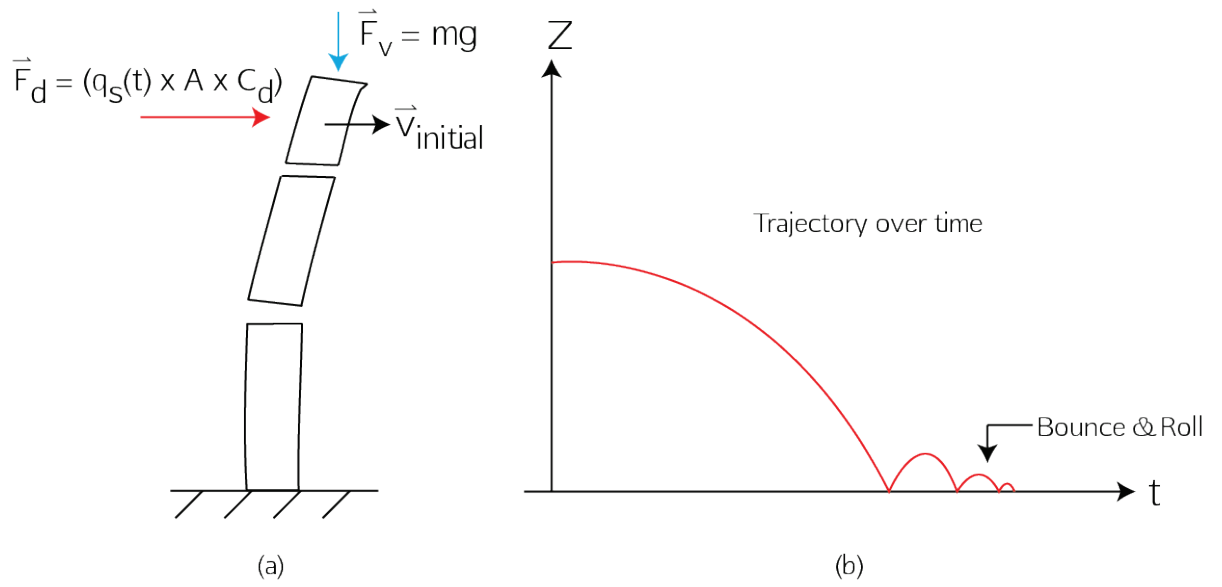


Figure 3.3: Masonry fragments: (a) Velocity components in-flight; (b) Bounce and roll.

Where:

$\vec{F}_d$ = Drag forcing

$q_s(t)$ = Time varying dynamic pressure

A = Area of wall or fragment

C_d = Drag coefficient

$\vec{F}_v$ = Vertical forcing

m = Mass

g = Gravitational acceleration

$\vec{v}_{initial}$ = Initial horizontal fragment velocity

3.2.2 Scaling fundamentals

The ability to relate the load-response of a reduced-scale structure (model) to its full-scale (prototype) counterpart represents the objective of structural scaling within this PhD. A number of experimental studies have been conducted to investigate the scaling of masonry structures in a dynamic loading environment within the field of earthquake engineering. The majority of which have utilised either a simple or complete modelling approach (Tomažević and Velechovsky, 1992). The simple approach which has been adopted by many utilises geometric similarity alongside masonry construction from full-scale versions of masonry materials. In contrast, a complete model aims to utilise bespoke materials which accurately scale the stress-strain relationship of the full-scaled material via the scale factor. These model materials also possess the equivalent specific weight, Poisson's ratio and damping coefficient of the prototype. This methodology is challenging to implement due to the need to design, manufacture and verify the material properties of reduced-scale bricks and mortar for a purely experimental purpose. As a result, there have been no studies which adopt a complete approach to scale masonry structures in a blast environment. There is also a degree of uncertainty over the performance of a complete approach in relation to masonry fragmentation. This introduces an additional level of complexity which is not typically examined within earthquake engineering studies that focus on global structural response.

In the case of blast-loaded masonry, a model structure must aim to replicate both the damage state and debris distribution of a prototype. With respect to damage state, this includes cracking modes at each panel wall alongside panel deflections which may induce variable degrees of collapse. The sequential onset of fragmentation and the associated debris distribution will be dependent upon the extent of the aforementioned damage behaviours. At the minimum, a model structure will need to be geometrically accurate to characterise the response of a prototype structure. Geometric scaling can be accomplished by maintaining accurate dimensional proportions as shown in figure 3.4. This utilises a scale factor as shown in equation 3.2 to produce an accurate model for length, area and volume. Another crucial aspect of masonry scaling is the need to adopt identical construction procedures for both the model and prototype structures. Specifically, this relates to the accurate construction of corner designs, panel wall edge restraints and equivalent levels of edge fixity. As described in section 3.1.1, any variation in edge restraint may result in unexpected panel fracture modes.

The wide ranging influence of which could be a sequential effect on the degree of collapse and subsequent debris distribution, thus introducing divergence between model and prototype response data. It is abundantly clear therefore that the accurate replication of essential design features and construction methods must be adhered to when producing a model representation of a prototype masonry structure.

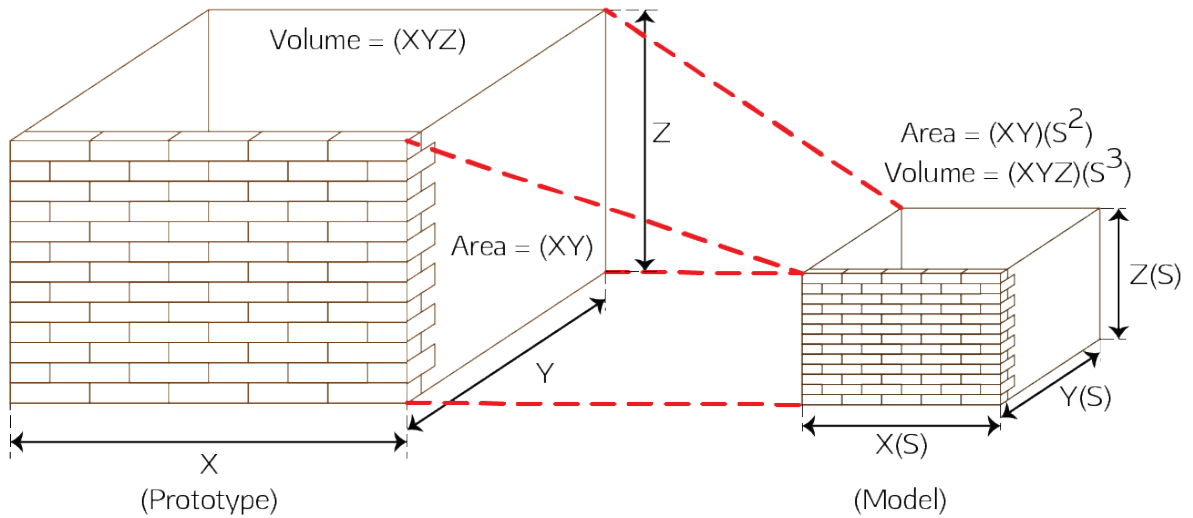


Figure 3.4: Geometric similitude of a masonry structure.

$$\text{Scaling Ratio of 1:2} \rightarrow S = \frac{1}{2} \quad (3.2)$$

Where:

$S = \text{Scale factor}$

Outside of the field of structural and civil engineering, aerodynamics can be found to place a particular emphasis on scaling methodologies as a means to determine the performance of a range of aerodynamic elements including aerofoils, fuselage structures and a wide ranging spectrum of concept designs relating to fluid-structure-interactions. Within this field, the spectrum of scaling approaches is defined at one end by basic geometric similitude which is equivalent to a simple modelling approach. At the other end of this spectrum is the highest standard in the form of dynamic similitude. This methodology requires geometrically similar bodies to be subject to equal ratios of applied forces as demonstrated by the expression in equation 3.3. At present, there are no structural engineering studies which aim to integrate the aerodynamic concepts of dynamic similitude

into a scaling methodology for dynamically loaded structures. Chapter 4 will redress this knowledge gap in more detail where it will be shown that the maintenance of equal applied force ratios for model and prototype structures can form the basis of a new scaling methodology. This will be shown to improve upon the limited accuracy of a simple modelling approach whilst removing the need to manufacture and test bespoke masonry elements as required with a complete modelling approach.

$$\left(\frac{\vec{F}_{h_{prototype}}}{\vec{F}_{v_{prototype}}}\right) = \left(\frac{\vec{F}_{h_{model}}}{\vec{F}_{v_{model}}}\right) \rightarrow \text{Dynamic Similitude} \quad (3.3)$$

Where:

$\vec{F}_h$ = Horizontal forcing

$\vec{F}_v$ = Vertical forcing

3.3 Existing research

At present, the level of research into the scaling of unreinforced masonry structures in a dynamic loading environment is very small. The growing utilisation of computational models has offered many researchers an alternative to experimental trials which provides rapid feedback and the capability to make infinite changes to a model without the cost implications of an experimental trial. These factors in conjunction with the constant increase and affordability of substantial computing power may have influenced the lack of studies into the development of experimental scaling methods for masonry. Despite this, it can be shown that the predictive capacity of a computational method is dependent upon comprehensive experimental benchmarking, especially when modelling the complex loads and damage modes associated with masonry structures subject to blast. Specifically, the interdependence of masonry fracture and collapse followed by the sequential onset of airborne fragments and debris distributions can be highly challenging to model computationally. This highlights the critical need for a symbiosis between computational and experimental methods which, when combined with an accurate scaling methodology can produce a substantial reduction in cost versus full-scale trials.

Existing research into the scaling of masonry in a blast environment has focussed upon simple models via geometric similarity. This was demonstrated by Dennis et. al (Dennis et. al, 2002) when investigating the response of $\frac{1}{4}$ scale masonry walls with one-way spanning supports and single running mortar bonds over a series of five short-duration blast trials. Masonry structures were constructed from full-scale and full-size concrete blocks with global structural dimensions reduced to $\frac{1}{4}$ scale. The utilisation of full-scale blocks indicates that the ratio of lateral to vertical forcing structure was unlikely to be maintained versus a full-scale structural counterpart. Each of the five trials conducted by this study utilised high-speed video to monitor masonry response and deflection behaviour of the walls. The analysis of which showed that each structure 'failed' due to partial collapse in-situ. Trial data was subsequently utilised to address the principal aim of the authors, namely the validation of FEM models produced with DYNA3D. These computational models utilised pressure data captured during the trials to configure blast loading. A comparative study was undertaken where results indicated that FEM models were unable to accurately model wall failure modes. The cause of which was attributed to an inability to remove mortar elements from the analysis during wall deflection. These limitations underline the difficulties in using FEM to model masonry breakage due to the inability to automatically remove elements after breakage without compromising global structural behaviour as discussed in section 2.3.1.

An experimental and computational study conducted by Iqbal (Iqbal, 2014) sought to examine the blast response of a nuclear reactor containment chamber constructed from a coarse grained rock known as schist at $\frac{1}{8}$ scale. This study utilised geometric similitude to scale the length, area and volumetric dimensions of the structure. Uniquely, this study utilised geometric similarity to calculate a hypothetical scaled distance parameter as a function of the geometrically scaled stand-off distance and charge mass. The presentation of which underwrites a demonstration of the variable ways in which geometric scaling can be applied to units with the dimension of length. Five blast trials were conducted at a range of stand-off distances for charge masses of 3-100kg TNT equivalence. Post-trial visual analyses were conducted to qualitatively determine the extent of cracking incurred by the containment unit due to blast loading. Trial data for 50kg TNT at 5m stand-off was specifically utilised to conduct a comparative study with FEM results produced via SAP2000. The results of which were shown to demonstrate acceptable agreement for crack patterns and the qualitative

extent of cracking. Based on this agreement, a number of additional computational models were undertaken to examine the response of a full-scale containment unit built from schist for long-duration blast loads ranging from 30-70 tonnes of TNT at 10 tonne increments for 100m stand-off. These models were analysed for cracking to indicate potential failure, allowing the author to examine the predictive capabilities of SAP2000 for blast response of nuclear containment chambers constructed from schist. The computational benchmarking undertaken by this study was however linked to a single experimental comparison, indicating the need for a larger comparative study with experimental data to fully benchmark the suggested computational approach.

Outside of the field of blast engineering, an investigation by Laefer et. al (Laefer et. al, 2011) examined the manufacture and assembly of masonry walls at a 1/10 scale as part of a wider study. These were used to model the deflection and cracking of real-world masonry structures as a result of nearby soil excavation and subsequent structural slippage. This study aimed to implement a complete modelling approach by scaling the flexural strength of bricks and mortar. The curing time and workability of the mortar was also scrutinised to ensure a scalable and workable material. As a result, the researchers were able to successfully scale the brick compressive strength, brick modulus of rupture, mortar tensile strength and masonry shear strength with each found to closely match the target reduction factor of 10. Prior to construction, each of the model bricks were subject to visual inspection for cracks and defects, enabling similar quality bricks to be grouped. Based on this sorting method, approximately 50% of the manufactured bricks were selected for construction. These units were utilised to carefully build scaled representations of historic structures within a weather-sealed and highly controlled environment. Due to the small scale geometries within this study, mortar was applied to each brick with a small knife and 'buttering' process whilst facing was accomplished with a small spatula. Post-trial analyses demonstrated an accurate representation of the crack patterns and vertical motion recorded by the historic buildings at full-scale due to adjacent soil excavation. The carefully controlled nature of this study's masonry construction does however indicate that 1/10 scale may be too small in a typical blast testing environment due to the potential for atmospheric variability via moisture and strong winds.

Research by Mohammed et. al (Mohammed et. al, 2011) investigated the presence of 'scale effects' within reduced-scale masonry structures as a result of non-negligible increases in structural strength when utilising full-scale masonry materials. Simple structures took the form of masonry testing triplets at $\frac{1}{2}$, $\frac{1}{4}$ and $\frac{1}{6}$ scale and masonry units were cut from full-scale versions whilst mortar was defined as M4 due to its medium level of compressive strength. This study emphasised compressive strength tests to represent standard structural loading versus the dynamic load cases which are the primary interest of this PhD. Nonetheless, the results of this research demonstrated that $\frac{1}{2}$ scale structures built from full-scale materials cut to $\frac{1}{2}$ scale sizes did not exhibit an increase in compressive strength when tested as individual units or as part of a triplet. Thus indicating that 'scale effects' may not be a factor for non-dynamic load cases at a $\frac{1}{2}$ scale level when resizing full-scale bricks. In contrast, increased compressive strengths were found for $\frac{1}{4}$ and $\frac{1}{6}$ scale structures both for the individual units and the triplet structures, inferring a production of greater strength versus structures at full scale.

An investigation conducted by Petry and Beyer (Petry and Beyer, 2012) investigated the production of four different $\frac{1}{2}$ scale hollow masonry brick units with the same material properties as those at full scale. The authors aimed to develop a brick production methodology that enabled the simulation of masonry response to seismic loading via shake table tests. An emphasis was placed on the control of mechanical properties of the scaled units to limit the influence of 'scale effects' as discussed above with Mohammed et. al (Mohammed et. al, 2011). To minimise manufacturing variability, model units were produced at the same plant as the prototypes with the same clay and brick firing procedure. This study tested each of the model units under compression in the parallel and perpendicular directions and flexural strengths were analysed via 3-point bending tests. The results of which revealed modest differences of -5% in compressive strength and up to +25% in tensile strength for the model brick units. The authors concluded that these differences were a function of the clay void ratio, indicating the need to carefully match this property between prototype and model to achieve similar mechanical properties. The influence of mortar on the 3-point bending tests of masonry triplets was identified to be an important factor as the models were found to produce increases in tensile failure resistance of up to 80%. This led the authors to conclude that further development of reduced-scale mortar may be required.

Research by Herbert et. al (Herbert et. al, 2014) aimed to experimentally assess the lateral capacity of six masonry structures when subject to wind loading at 1/6 scale. These were constructed by cutting full-scale bricks to the appropriate size to achieve geometric similitude. The application of a lateral load was accomplished by utilising a pressure bag rig which was securely fixed within a swinging gondola centrifuge. This enabled the researchers to model the effect of self-weight whilst applying the modelled wind loading. Each of the masonry structures were simply supported along each edge and the lateral load was applied to a single face up until the moment of failure. Deflections and cracking were measured in three dimensions by utilising a novel digital image correlation (DIC) rig. Post-trial analyses indicated the failure mode to be caused by vertical bending, causing the mortar to crack parallel to the brick units with each structure, demonstrating similar failure results to those obtained via a literature search. Failure loads were also found to closely correlate with those predicted by Eurocode EC6 in four of the six trials. In the remaining two instances, differences were reported with the introduction of a small aperture to simulate a window. In these scenarios, the authors found the opening to exert minimal influence on the failure load, indicating the EC6 prediction may be conservative.

A study by Milani et. al (Milani et. al, 2016) investigated the experimental testing of a 1/5 scale masonry cross vault structure by the implementation of 3D printed plastic brick units. These were manufactured to possess a friction coefficient which approximates the lime mortar adhesion to Italian clay bricks utilised in the full-scale structure. In order to scale the mass of the reduced-scale structure, steel inserts were utilised to increase the lightweight plastic block masses, resulting in an equivalent density to the full-scale structure. Tilting plane tests were conducted to simulate seismic loading actions at varying degrees of inclination, the results of which were utilised to underwrite the benchmarking of an FEM modelling procedure developed by the authors. The lack of full-scale or real-world data comparisons can however be shown to limit the benchmarking aspect of both the experimental scaling methodology and the computational procedure.

Szyszkowski et. al investigated the utilisation of 3D printed materials to conduct testing of historical masonry structures subject to seismic loads at 1/10 scale (Szyszkowski et. al, 2018). Dry masonry construction was simulated via gypsum powder brick units produced via 3D printing. The friction coefficient of the printed units was emphasised as it represented the main

determinant in replicating unit-to-unit interlocking that underpins dry masonry. Initial efforts to model this parameter resulted in a friction values which greatly exceeded requirements with significant scatter amongst the recorded values. These issues were limited by implementing a 'sanding' procedure which yielded the desired friction value whilst reducing the coefficient of variation of the data set to 11% of the mean friction value. A range of tilting plane tests were subsequently conducted for 'two-layered' masonry walls, enabling the determination of acceleration values and the associated collapse modes via high-speed video. The results of which were found to yield good agreement with previous experimental trials, enabling the authors to determine that existing theoretical methods for calculating failure acceleration values may be conservative. It was not possible however to verify this study's level of agreement with the cited trial data as the study was written in Italian and it was not accessible.

Research conducted by Drougkas et. al aimed to examine the vertical in-plane shear failure of fifteen $\frac{1}{4}$ scale, single leaf masonry walls subject to loads associated with seismic events (Drougkas et. al, 2019). The results of which were used to refine FEM models of masonry response with the objective of demonstrating the combined need for experimental and computational methods to assess the complex shear failure modes of masonry walls. The scaling method used by this research involved the manual forming of reduced-scale units from the 'clay' mixture used to produce full-scale bricks, thus diverging from the standard method of cutting full-scale units to size. Experimental results were found to show that structural openings such as window apertures reduced the shear resistance of single-leaf masonry walls whilst decreases in structural height were found to introduce minor increases in shear resistance. Experimental values for shear stress and qualitative failure mode were used to benchmark computational models produced with FEM, indicating a greater degree of consistent agreement with experimental data versus results obtained via closed form expressions or design guidelines. However, the authors were not able to demonstrate the accuracy of the $\frac{1}{4}$ scale shear loading tests versus full-scale results or real-world data for seismic shear failure of single-leaf masonry walls, indicating the need for additional benchmarking.

3.4 Summary

This chapter began by summarising the concepts of masonry response to blast loading alongside a synopsis of fundamental scaling theory. This was followed by an examination of the few available studies into experimental scaling of unreinforced masonry structures in a blast environment. Existing blast research was found to utilise basic geometric similarity with construction via full-scale masonry materials with the intention of benchmarking computational models to provide predictive capacity. The utilisation of full-scale masonry materials may however introduce an inequality between full-scale and reduced-scale ratios of lateral to vertical forcing, thereby corrupting experimental data sets.

This chapter then reviewed a range of tangentially related masonry scaling research for a variety of dynamic loads. A relatively large amount of which was conducted with an emphasis on seismic loading. These studies were found to utilise two general classifications of experimental scaling approach, the first of which implemented geometric similarity alongside construction via reduced-scale bricks which were typically cut from full-size bricks. The second methodology involved the use of 3D printing to produce brick units which utilised a coefficient of friction to simulate wet or dry masonry construction. This chapter also reviewed the application of masonry scaling for wind loading. This study was found to implement the first of the two aforementioned masonry scaling approaches in tandem with a novel gondola-based centrifuge to model the effect of self-weight. Despite its novelty, this approach may only be useful for dynamic loading setups which do not involve masonry fragmentation as seen with blast loading.

Only one study was found to implement a complete modelling approach for unreinforced masonry with the manufacture of scaled brick units which integrated scaled material properties. As the most difficult scaling approach, it is perhaps not surprising that this was implemented within a highly controlled environment where the aim was to replicate the cracking mode of historic masonry due to nearby soil excavation. This load scenario may therefore be viewed as the least 'dynamic' of the alternative dynamic load cases. Finally, a small number of studies investigated the influence of 'scale effects' which can be shown to increase the unit and overall strength of a structure due to the use of full-scale materials at a

reduced size. The results of which indicated that these strength increases are typically found at scale levels smaller than $\frac{1}{2}$.

A large proportion of the identified studies within this literature review have combined experimental research with the benchmarking of computational models. In spite of this, none of the studies aimed to demonstrate the accuracy of the implemented scaling method in a comparative study with full-scale data. This crucial stage can be shown to be vitally important to provide credence for both the scaling method and all associated experimental data produced at reduced-scale. Subsequent computational models can therefore be presented with a high degree of confidence if benchmarking has been completed via accurate experimental results at either full or reduced-scale.

Finally, it can be said that the lack of research into masonry scaling in a blast environment underlines the need for this PhD. This is underscored by a growing reliance upon computational models which have not been fully benchmarked to model the complex structural response of masonry to blast alongside the substantial risk posed by the ongoing threat of terrorist incidents. By aiming to develop a new scaling methodology which improves upon the limited accuracy associated with a simple model, this PhD research intends to enable a range of blast trials to be conducted which can further the collective knowledge base for masonry response to blast. The results of which can be used in conjunction with benchmarked computational methods to provide a robust 'toolbox' for assessing masonry response to any blast event whilst minimising the cost and space implications associated with full-scale blast trials.

Chapter 4

Scaling Methodology

Representing the principal aim of this PhD and the first objective, this chapter details the development of a new scaling methodology for unreinforced masonry structures in a blast environment. As a means of fulfilling this objective, a numerical investigation was undertaken to determine how the concepts of dynamic similitude can be integrated into a scaling scenario involving the unique elements of blast loading. An emphasis is subsequently placed on the accurate replication of damage via cracking and degrees of collapse which in turn may sequentially initiate airborne fragments and a final state debris distribution. Throughout this chapter and the remainder of this thesis, full-scale structures will be referred to as prototypes and reduced-scale structures will be referred to as models as is customary in scaling studies.

4.1 Derivation

In the simplest cases, the implementation of geometric similarity enables a model structure to accurately represent a scaled down version of a prototype counterpart. This can be accomplished by using the pre-defined scale factor to scale each length dimension of the original prototype structure. This important step represents an essential component of any scaling approach, but its limitations become apparent when attempting to utilise the geometrical model to replicate the complex response of a masonry structure subject to blast loading. Perhaps the most important consideration when aiming to deploy a geometric model in a dynamic load environment is the selection of construction material. As discussed in chapter 3, much of the existing research has opted to use full-scale versions of masonry materials to produce the masonry units used to construct a geometric model. Specifically, this

can take the form of a geometrically scaled brick unit which has been physically cut from a full-scale unit used in a prototype. These individual units are subsequently used to construct a model which implements geometric scaling for both the constituent parts and the global structural dimensions. Alternatively, some researchers and in particular, those in the field of blast engineering have elected to use full-scale masonry units to build a model which only implements geometric scaling for the global structural dimensions. On first inspection, it would be logical to assume that the more comprehensive of the two construction methods would enable a more faithful representation of prototype masonry response to blast loading. It can however be shown that regardless of the lengths taken to produce a geometric model, the limitations of utilising full-scale masonry materials becomes apparent when considering the force ratio requirements of dynamic similitude.

As discussed in chapter 3, the field of aerodynamics utilises dynamic similitude to define the gold standard in scaling methodologies. In particular, the underlying concept specifies the requirement for two geometrically similar structures to be subject to equal ratios of lateral to vertical forcing. This rule-set will form the basis of this PhD's scaling methodology as it enables the lateral and vertical forcing components of a blast environment to be expressed in a non-dimensional format. In doing so, the cross-examination of model versus prototype forcing can be conducted by determining whether the structures are subject to an equal ratio figure. The application of an equal force ratio will therefore mitigate any concerns of non-representative force application to a geometric model which may produce inaccurate representations of masonry damage and debris. Expanding upon this, it can be shown that utilisation of full-scale masonry materials and the subsequent maintenance of prototype material density for the geometric model produces an inequality of blast force ratios for prototype and model structures.

Figure 4.1 in conjunction with the derivation below yields equations 4.1-4.3 which mathematically demonstrate that the maintenance of equal structural densities results in unequal blast force ratio values when assessing an in-tact masonry structure. Note that the subscripts 'proto' and 'model' denote prototype and model structures respectively. Examination of figure 4.1 shows that the horizontal forcing component can be represented by the application of time varying pressure from the blast wave. In a similar manner, the self-weight of the structure can be shown to represent the vertical force component. By

utilising these assumptions, the first stage of the derivation below demonstrates that for the same blast environment, the model structure's horizontal forcing component is equal to the prototype force multiplied by the scale factor squared as shown in equation 4.1. This is a result of the model area being scaled by this same figure. The second stage of the derivation demonstrates that the model structure's vertical force component is equal to the prototype force multiplied by the scale factor cubed as shown in equation 4.2. This is a result of the model volume being scaled by the same figure when assuming that the model density is equal to that of the prototype structure. The combination of equations 4.1 and 4.2 indicate that the ratio of lateral to vertical forcing is not equal for the model and prototype structures as shown by the reciprocal scale factor in equation 4.3. As this research has chosen not to scale the blast environment, it can be shown that the vertical force component is the only manipulatable parameter to produce an equality in force ratio.

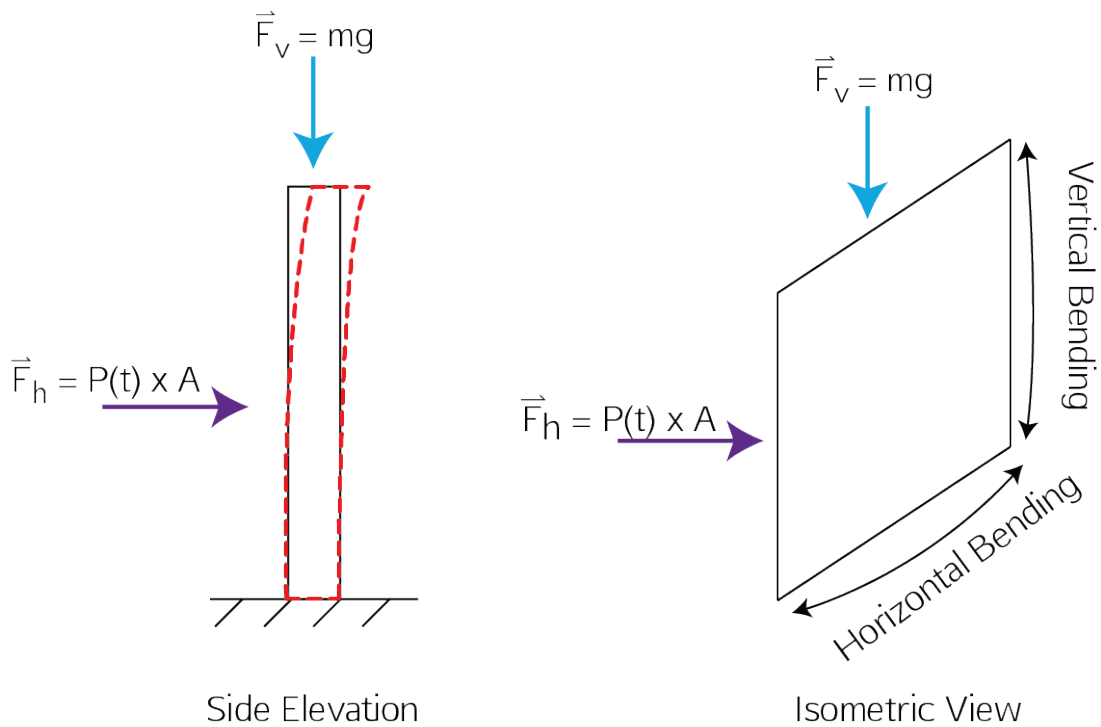


Figure 4.1: Masonry structure force components.

$$P(t) \cdot A = \vec{F}_h$$

$$A_{model} = A_{proto} \cdot S^2$$

$$\therefore \vec{F}_{h_{model}} = \vec{F}_{h_{proto}} \cdot S^2 \quad (4.1)$$

$$\vec{F}_v = m \cdot g = w$$

$$\rho_{proto} = \frac{m_{proto}}{V_{proto}}$$

$$if \rho_{model} = \rho_{proto} \rightarrow \frac{m_{proto} \cdot S^3}{V_{proto} \cdot S^3}$$

$$m_{model} = m_{proto} \cdot S^3$$

$$\therefore \vec{F}_{v_{model}} = \vec{F}_{v_{proto}} \cdot S^3 \quad (4.2)$$

$$\left(\frac{\vec{F}_{h_{proto}}}{\vec{F}_{v_{proto}}} \right) \neq \left(\frac{\vec{F}_{h_{model}}}{\vec{F}_{v_{model}}} \right) \equiv \left(\frac{\vec{F}_{h_{proto}} \cdot S^2}{\vec{F}_{v_{proto}} \cdot S^3} \right) \equiv \left(\frac{\vec{F}_{h_{proto}}}{\vec{F}_{v_{proto}}} \right) \cdot S^{-1} \quad (4.3)$$

Where:

$P(t)$ = Pressure as a function of time

A = Area of wall

$\vec{F}_h$ = Horizontal forcing

$\vec{F}_v$ = Vertical forcing

m = Mass

g = Gravitational acceleration

w = Self-weight

ρ = Material density

V = Volume

An examination of the vertical force component derivation indicates that the two available variables for manipulation are mass and gravitational acceleration. It is not practical to scale the effect of gravity via a centrifuge or other methodologies due to the challenges associated with fielding these techniques within a harsh blast environment. This research will therefore examine the mass parameter via material density as this combines the scaled volume parameter alongside structural mass. By aiming to manipulate the geometric model density to yield an equal force ratio to a prototype, this research will adopt a first principles

approach where the masonry structure will be represented as an inertial resistance to the momentum of a blast wave.

Equation 4.3 showed that a constant material density for the model and prototype structures resulted in an inequality for force ratio with the model value being equal to the prototype multiplied by a reciprocal scale factor. In order to remove this additional multiplier, the vertical force component of the model needs to be multiplied by the scale factor squared to match the lateral forcing scale multiplier, thus cancelling out the scale factors and matching the force ratio of the prototype. By starting at this point, it is possible to reverse engineer the required manipulation of model material density to achieve an equality in force ratios as shown in the derivation below. Closer inspection of this shows that by aiming to apply a scale factor squared, the mass of the model needs to be scaled by the same value. However, the need to maintain geometric similarity has not been removed, indicating that the volume of the structure needs to be scaled by scale factor cubed. By combining these two requirements it can be seen that the density of the model needs to be equal to the prototype density multiplied by a reciprocal scale factor as shown in equation 4.4. If this condition is met, it can therefore be shown in equation 4.5 that the model force components are both multiplied by the scale factor squared, producing a cancellation and an equal force ratio to the prototype structure.

$$\text{For equal force ratios} \rightarrow \vec{F}_{v_{model}} \equiv \vec{F}_{v_{proto}} \cdot S^2$$

$$\therefore \text{if } \vec{F}_{v_{model}} = m_{model} \cdot g \rightarrow m_{model} = m_{proto} \cdot S^2$$

$$\text{For geometric similarity} \rightarrow V_{model} = V_{proto} \cdot S^3$$

$$\therefore \rho_{model} = \frac{m_{proto} \cdot S^2}{V_{proto} \cdot S^3} = \rho_{proto} \cdot S^{-1} \quad (4.4)$$

$$\Rightarrow \left(\frac{\vec{F}_{h_{proto}}}{\vec{F}_{v_{proto}}} \right) = \left(\frac{\vec{F}_{h_{model}}}{\vec{F}_{v_{model}}} \right) \equiv \left(\frac{\vec{F}_{h_{proto}} \cdot S^2}{\vec{F}_{v_{proto}} \cdot S^2} \right) \quad (4.5)$$

The implementation of an increased density for the model structure therefore underwrites this PhD's scaling methodology for masonry structures in a blast environment. In doing so, it can be said that the scaling method will implement a pseudo-increase in the model structure's inertia which is essential to match the vertical forcing of a prototype

structure. Neglecting this fundamental issue has been shown to produce an inequality in force ratios for the same blast environment, the result of which would be an under-application of vertical forcing to model structures. The consequences of which would be an overestimation of masonry damage, resulting in an unrealistic amount of deflection and reduced time to collapse, which in turn would likely result in an overestimation of airborne fragment throw and a wider, inaccurate spatial spread of masonry debris.

Up until now, an emphasis has been placed on producing equal blast forcing ratios for in-tact model and prototype structures. This sequence was chosen as it can be shown that the lateral loading of an in-tact structure can result in fracture and varying degrees of collapse before the potential and sequential initiation of airborne masonry fragments and a spread of masonry debris as discussed in chapter 3. The fragmentation and debris phase of masonry response can be characterised by the exceedance of a deflection limit which is a linear function of structural height. At which point, the initial horizontal velocity of a fragment will have been determined by pre-break lateral blast forcing as shown by figure 4.2.

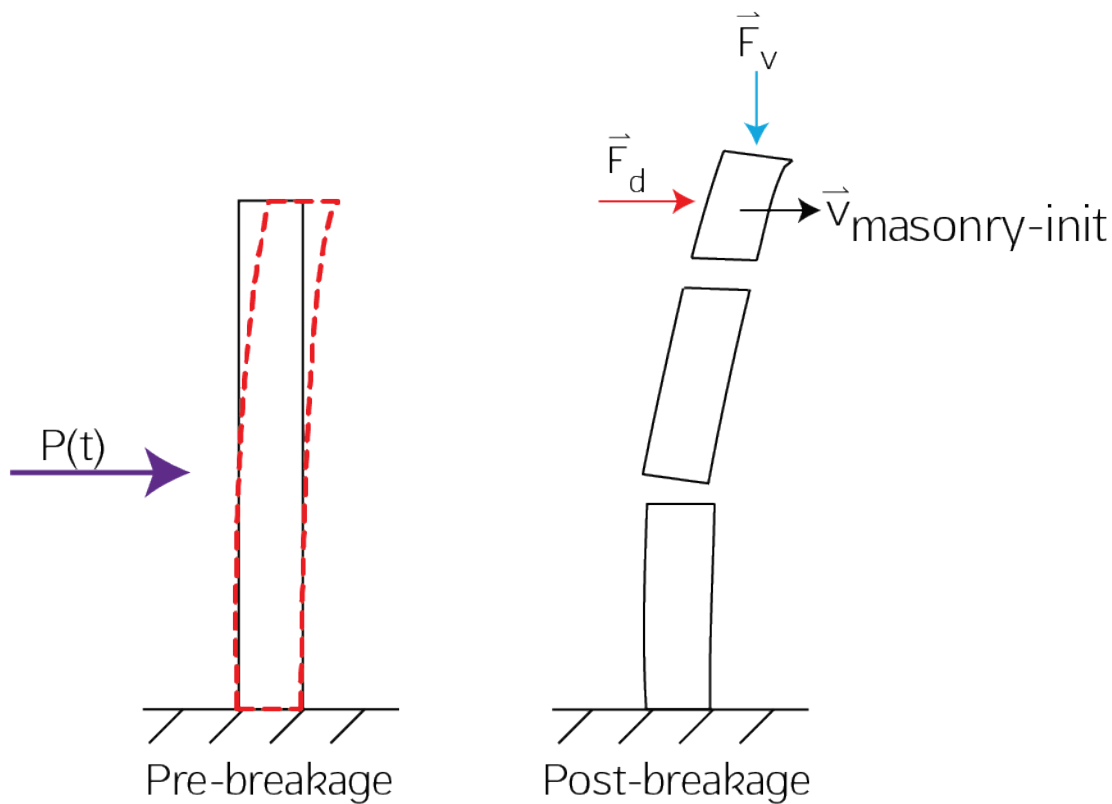


Figure 4.2: Masonry fragmentation with initial force components.

By implementing geometric similarity, it can be shown that the deflection limit for breakage of a model structure will be lower than the prototype due to its correlation with structural height which has been reduced by the scale factor. As a result, the model structure is likely to have a decreased break time and lower potential initial fragment velocity than a prototype in the same blast environment. This decrease in velocity may also reduce the potential fragment throw distance. Similarly, the lower actual elevation of masonry units versus taller prototype structures indicates that vertical acceleration due to gravity will result in faster collapse and reduced fragment flight time versus a prototype. The onset of structural collapse in the instances where it occurs can also be shown to produce potential inelastic brick-to-brick collisions alongside inelastic ground contacts. These will be followed by random 'bounce and roll' motions before a final debris distribution is established.

In general, the post-break phase of masonry response is likely to represent one of two scenarios. The first being related to short-duration blasts which provide an inertial shunt to a structure, resulting in a relatively slow process of deflection, fracture and potential degrees of collapse with debris and without airborne fragmentation. The second scenario represents long-duration blasts which deliver a large impulse with a sizable dynamic pressure. These events will produce significant masonry damage and a high likelihood of collapse, the result of which is a far higher likelihood of airborne fragments which will increase the spread of masonry debris. In the case of short-duration blast events which will be examined within chapters 5-6 of this research, the absolute effects of reduced breakage time and breakage deflection will be non-dimensionalised within the debris distribution analysis via percentages of total mass within a radius that is a percentage of the structural dimensions. The application of this analytical approach will also dampen any pronounced effects due to random fragmentation or ground interactions for both the model and prototype structures.

The effect of long-duration blast loading on masonry response and in particular, the initiation of airborne fragments introduces the potential of drag loading as shown by figure 4.3. In these instances, it can be shown that the scaling methodology derived within this research can yield equal rates of forcing in the post-break phase as demonstrated by the derivation below. Initially, it can be seen that drag loading is caused by the application of dynamic pressure with an associated variable velocity expression as shown in equation 4.6. This accounts for the potential varying sense and magnitude of dynamic pressure over the

duration of a fragment's flight. The utilisation of geometric scaling can be shown to produce a model area which is equal to the prototype multiplied by the scale factor squared. As a result of this multiplier and the constant blast environment, it can be shown that the model drag forcing component is equal to the prototype forcing multiplied by the scale factor squared. When combined with the increased density of the model structure, the vertical forcing component can be shown to be equal to the prototype forcing multiplied by the scale factor square. The net effect of which can be seen to an equal ratio of lateral drag forcing and vertical forcing for both the model and prototype as shown in equation 4.7. It is noted that the magnitude of the lateral drag and vertical force components is dictated by the geometry and subsequent mass of a fragment. This indicate that levels of correlation between the shape and size of model and prototype fragments may influence the levels of agreement for debris distribution results. The examination of this scaling approach within a long-duration blast is detailed within chapter 8 which outlines a series of trial designs which will be conducted in the future. This will therefore enable an experimental assessment of the influence of drag loading and airborne fragments on the debris distribution of model and prototype debris distributions.

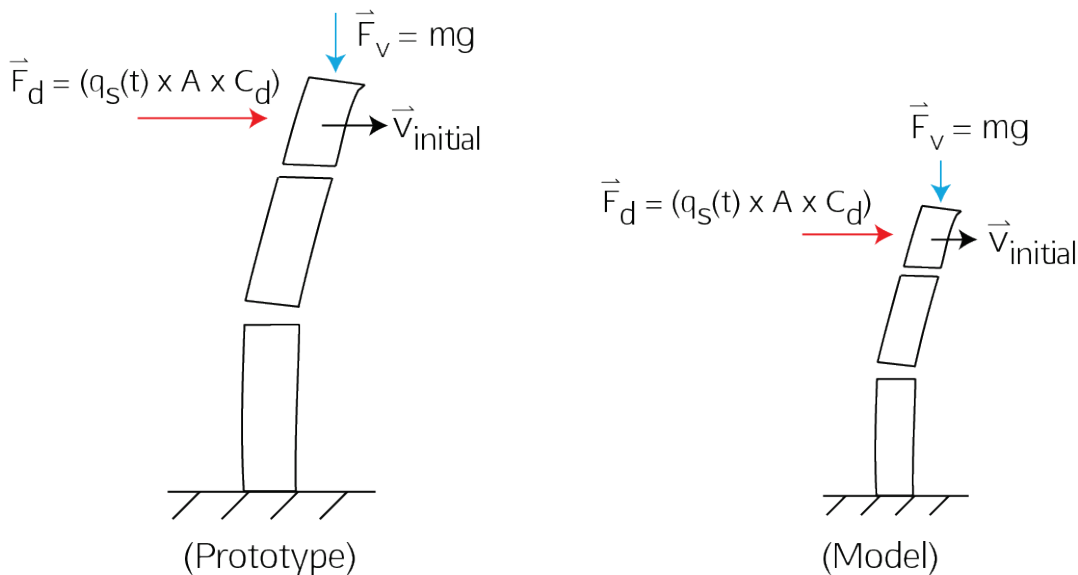


Figure 4.3: Masonry fragment drag loading.

$$\vec{F}_d = q_s(t) \cdot C_d \cdot A$$

$$q_s(t) = \frac{1}{2} \rho(t) (\vec{u}_{flow}(t) - \vec{v}_{masonry-init})^2 \quad (4.6)$$

$$if A_{model} = A_{proto} \cdot S^2 \Rightarrow \vec{F}_{d_{model}} = \vec{F}_{d_{proto}} \cdot S^2$$

$$if \rho_{model} = \rho_{proto} \cdot S^{-1} \Rightarrow m_{model} = m_{proto} \cdot S^2 \Leftrightarrow \vec{F}_{v_{model}} = \vec{F}_{v_{proto}} \cdot S^2$$

$$\Rightarrow \left(\frac{\vec{F}_{d_{proto}}}{\vec{F}_{v_{proto}}} \right) = \left(\frac{\vec{F}_{d_{model}}}{\vec{F}_{v_{model}}} \right) \equiv \left(\frac{\vec{F}_{d_{proto}} \cdot S^2}{\vec{F}_{v_{proto}} \cdot S^2} \right) \quad (4.7)$$

Where:

$\vec{F}_d$ = Drag force in lateral direction

C_d = Drag coefficient

$\vec{u}_{flow}(t)$ = Velocity of blast flow field as a function of time

At this stage, the scaling methodology developed by this PhD can be mathematically summarised by the expressions shown by equations 4.8-4.9. These demonstrate the implementation of geometric similarity alongside a reciprocal scale factor for the model structure's density. The combination of which results in a geometrically faithful model of a prototype masonry structure which is subject to an equal ratio of lateral to vertical blast forcing components within a constant blast environment. This therefore adheres to the core rule-set of dynamic similitude from the field of aerodynamics as specified at the beginning of this section.

$$L_{model} = L_{proto} \cdot S \quad (4.8)$$

$$\rho_{model} = \rho_{proto} \cdot S^{-1} \quad (4.9)$$

4.2 Scaled parameters

The scaling methodology derived within section 4.1 focussed on accurate modelling of masonry response within a constant blast environment. In doing so, it can be shown that a range of masonry design parameters and associated forcing expressions relating to a prototype can be re-written with the addition of a scaling parameter operation to describe a model. Table 4.1 details a range of these parameters in primary dimension format (Mass,

Length, Time) for the prototype column. The model column applies a scaling parameter operation to the corresponding value from the prototype. Inspection of table 4.1 demonstrates that a geometric model can be produced by applying a scale parameter index which matches the prototype's dimensional parameter. The model material density is shown to represent the prototype density multiplied by a reciprocal scale factor whilst each of the prototype force parameters has been multiplied by the scale factor squared to produce an equal forcing ratio for the model.

Table 4.1: Masonry design and force parameters, prototype and model.

Parameter	Prototype (MLT)	Model (MLT)·(S)
Length, L	L	L·S
Area, A	L ²	L ² ·S ²
Volume, V	L ³	L ³ ·S ³
Material Density, ρ	M·L ⁻³	(M·S ²)·(L ⁻³ ·S ⁻³) = (M·L ⁻³)·S ⁻¹
Free-field Pressure, P _s	M·L·T ⁻² /L ²	M·L·T ⁻² /L ²
Horizontal Force, F _h = P _s ·A	(M·L·T ⁻² /L ²)·L ² = M·L·T ⁻²	(M·L·T ⁻² /L ²)·(L ² ·S ²) = (M·L·T ⁻²)·S ²
Vertical Force, F _v = m·g	M·L·T ⁻²	(M·S ²)·L·T ⁻² = (M·L·T ⁻²)·S ²
Drag Force, F _d = q _s (t)·A·C _d	ML ⁻³ ·L ² T ⁻² ·L ² = M·L·T ⁻²	ML ⁻³ ·L ² T ⁻² ·(L ² ·S ²) = (M·L·T ⁻²)·S ²

A similar methodology can be used to describe a range of masonry response parameters as shown by table 4.2. Structure clearance time relates to the interval required for a blast to reach the rear of a structure and it can be seen that the model represents the prototype value multiplied by the scale parameter. This effect of this parameter is likely to be more pronounced in a short-duration blast where the blast wavelength may be less than the structural geometry. In contrast, long-duration blast waves tend to possess very large wavelengths, limiting the influence of this parameter on masonry response. Fragment flight-time represents a derivation of simple mechanics for projectile motion with constant acceleration, which in this case is represented by gravity. As a result, the model can be shown to represent the prototype value multiplied by the square root of the scale factor due to the initial application of a scale factor to the model's height dimension. Breakage deflection was shown to scale with structural height in section 4.1, indicating that the model value is equal to the prototype value multiplied by the scale factor.

Table 4.2: Response parameters, prototype and model.

Parameter	Prototype (MLT)	Model (MLT)·(S)
Structure Clearance Time, $t_c = L \cdot U_s^{-1}$	$L \cdot L^{-1} T = T$	$(L \cdot S) \cdot L^{-1} T = (T) \cdot S$
Flight-Time, $F-t = (2s)^{1/2} \cdot g^{-1/2}$	$L^{1/2} \cdot (LT^{-2})^{-1/2} = T$	$(L \cdot S)^{1/2} \cdot (LT^{-2})^{-1/2} = (T) \cdot S^{1/2}$
Breakage Deflection, δ	L	$(L) \cdot S$
Breakage Time, t	T	$(T) \cdot S^{1/2}$
Initial Horizontal Velocity, v	LT^{-1}	$(LT^{-1}) \cdot S^{1/2}$
Throw Distance, $T-D = F-t \cdot (u_{flow} + v)$	$T \cdot LT^{-1} = L$	$T \cdot S^{1/2} \cdot LT^{-1} \cdot S^{1/2} = (L) \cdot S$

Scale factors for breakage time and initial horizontal velocity were derived by assuming that model and prototype structures are subjected to the same linearly increasing velocity up until breakage. This is shown in the proof below where it can be seen that the reciprocal scale factor for model density yields an equal rate of lateral forcing to the prototype, producing equal values of acceleration until breakage. This enabled the utilisation of an idealised velocity-time profile to relate breakage time and initial horizontal velocity to a pre-defined deflection limit as shown in figure 4.4. Analysis by similar triangles for breakage time and initial horizontal velocity can therefore be used to show that the model values are equal to prototype values multiplied by the square root of the scale factor.

$$P_{model}(t) = P_{proto}(t) \rightarrow \text{equal rates of forcing}$$

$$\vec{F}_{h_{model}} = m_{model} \cdot \vec{a}_{model} \equiv (m_{proto} \cdot S^2) \cdot \vec{a}_{model}$$

$$\vec{F}_{h_{model}} = \vec{F}_{h_{proto}} \cdot S^2 \Rightarrow \vec{a}_{model} = \vec{a}_{proto}$$

Where:

$\vec{a}$ = Acceleration due to lateral forcing from blast loading

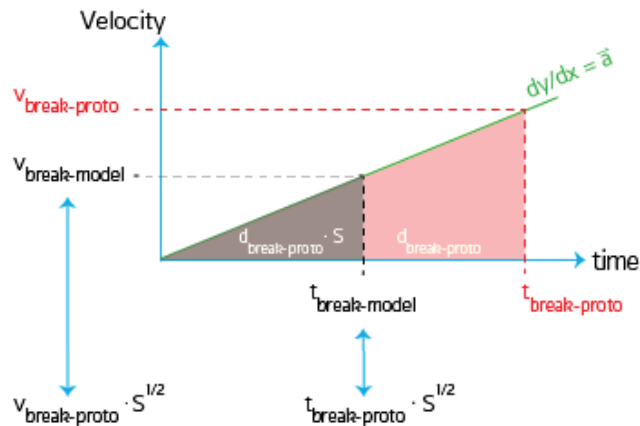


Figure 4.4: Similar triangle analysis of break time and initial fragment velocity.

Finally, fragment throw distance in table 4.2 is the product of flight time and fragment velocity. The combination of scaled model values for these two parameters can be shown to yield a throw distance value equal to the prototype value multiplied by the scale factor. The accuracy of this parameter is likely to be influenced by inelastic brick and ground collisions as described in section 4.1.

4.3 Commercial masonry materials

An important aspect of this research is the ability to implement the scaling methodology with commercially produced masonry materials. By conducting an investigation into a range of industry standard bricks and blocks for masonry construction it was possible to source two 'off-the-shelf' masonry units which adhere to the geometric and density requirements of this PhD's scaling methodology at a 1:2 scaling level as shown in figure 4.5.

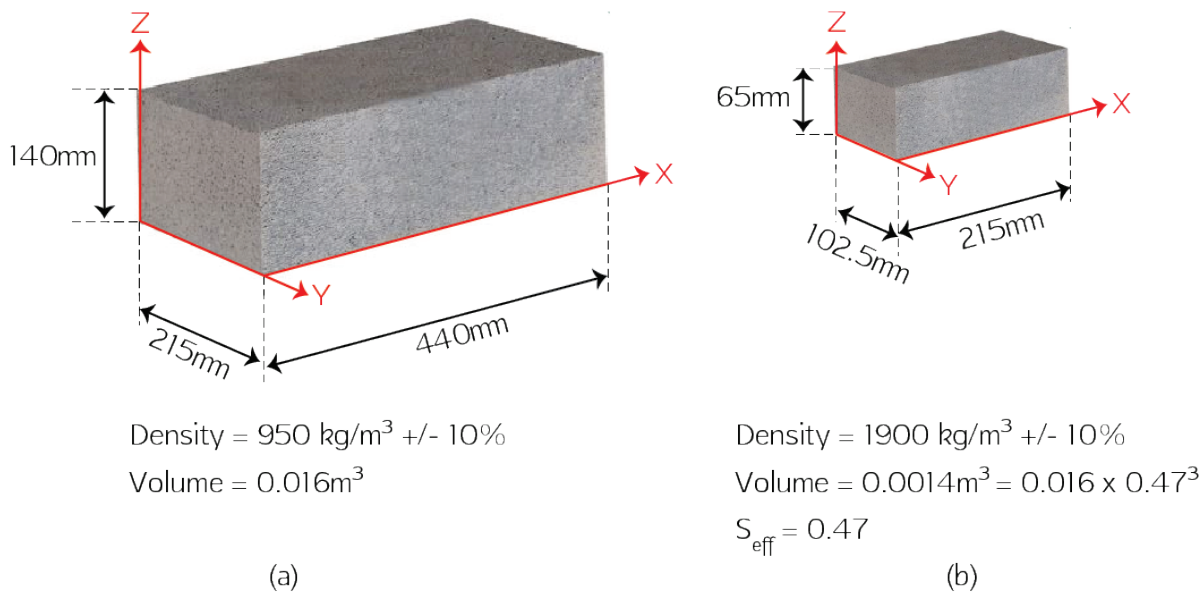


Figure 4.5: Masonry unit specifications: (a) Concrete block; (b) Concrete brick.

Examination of figure 4.5 shows that prototype structures can be built via solid concrete blocks with $\sim 950 \text{ kg/m}^3$ density and model structures can be constructed from solid concrete bricks with $\sim 1900 \text{ kg/m}^3$ density. As a result, it can be seen that the concrete brick density value is an equality to the concrete block density multiplied by the reciprocal of the 0.5 scale factor as shown below. Further examination of these masonry materials also shows that the mean volume of the concrete bricks represents a value which has been scaled by an

effective factor of 0.47, which can be seen to closely match the desired scaling factor of 0.5 at a 1:2 scale level.

$$\rho_{model} = 1900 \equiv 950 \times 1/0.5 \equiv 950 \cdot S^{-1}$$

Where:

$$S = 0.5$$

$$\rho_{proto} = 950 \text{ kg/m}^3$$

$$\rho_{model} = 1900 \text{ kg/m}^3$$

4.4 Summary

Representing the principal aim and first objective of this PhD, this chapter has examined the development of a new scaling methodology for masonry structures subject to blast loading. The first section began by showing that the implementation of geometric similarity underpins all structural scaling methodologies. A numerical investigation was then conducted to demonstrate the limitations of a geometric approach which relies upon construction of a model structure via full-scale masonry materials. The results of this showed that the ratio of lateral to vertical blast forcing was not maintained, introducing the potential for inadequate vertical forcing which in turn, infers a high likelihood of overexaggerated damage and a greater spread of debris for a model structure. In order to remove these unwanted response effects, additional numerical derivations were presented which adopted the main concept of dynamic similitude, namely the equal application of lateral to vertical force ratios for geometrically similar structures. It was subsequently possible to demonstrate that a model structure's density could be scaled to introduce a pseudo-increase in structural inertia, thereby overcoming the challenges associated with full-scale materials. This was shown to be possible via the implementation of a reciprocal scale factor for the model density. The combination of this with geometric similarity was shown to satisfy the core requirement of dynamic similitude by applying an equal ratio of lateral to vertical blast forcing for geometrically similar structures.

The applicability of the developed scaling approach was initially demonstrated for the blast loading of an in-tact structure due to the sequential nature of masonry response. This

can be characterised by fracture and potential collapse of an in-tact structure before the potential and sequential ejection of airborne fragments and the establishment of a debris distribution. It was then shown that the post-break phase of masonry response may differ as a function of whether the blast event is short or long-duration, with the latter representing the greatest likelihood of drag loading of projectile fragments. As a result, it was identified that the extent of correlation for the shape and size of projectile fragments ejected from model and prototype structures may influence levels of agreement for debris distributions in long-duration blasts. In the case of short-duration blasts, it was shown that the post-break phase is likely to be defined by relatively slow deflection and possible collapse of a structure, resulting in less dynamic fragment behaviour and a smaller debris spread. As a means of investigating the aforementioned response modes, chapters 5-6 will aim to demonstrate the capability of this PhD's scaling methodology for a short-duration blast trial with an emphasis on masonry damage and debris distribution data. Similarly, a series of long-duration blast trials have been designed in chapter 8 with the aim of assessing the scaling methodology's performance in a long-duration blast setting in the future.

The second section of this chapter analysed a range of essential masonry design, blast forcing and masonry response parameters to demonstrate the effect of this PhD's scaling methodology on each value. This was shown by tabulating a range of parameters in primary dimensional format for prototype structures with the addition of a scale parameter operation for the equivalent model value. The final section of this chapter identified two commercially available masonry construction units which adhere to the geometric and density scaling requirements at a 1:2 scale level. These masonry units were shown to closely match the desired geometric scaling level with an effective value of 0.47 versus the idealised value of 0.5 at 1:2. The density of these commercial masonry units were also examined to show that the model value is equal to the prototype multiplied by the reciprocal of the 0.5 scale factor.

Chapter 5

Experimental Methodology: Short-duration Blast

5.1 Introduction

This chapter details the design of one short-duration high explosive (HE) blast trial. This was devised in collaboration with the industrial sponsor, the Atomic Weapons Establishment (AWE), to benchmark the masonry scaling methodology derived in chapter 4, thereby fulfilling the second objective of this PhD. This trial was constructed and conducted on the Windrush Arena site at MOD Shoeburyness over the period of January-May 2018 where it was shown that commercially available masonry materials could be utilised to construct and deploy two pairs of prototype and model structures at a 1:2 scaling level. This trial enabled the capture of masonry damage and debris distribution data for two overpressure regions with 100kg TNT equivalence, enabling the comparison of model and prototype response data. These results were subsequently utilised to assess the capability of the scaling method to accurately scale masonry structures in a short-duration blast environment.

This trial also measured blast propagation in the free-field and reflected environments, enabling CFD models to be experimentally benchmarked as shown in chapter 7. These results also enabled the use of CFD to configure blast loading within AEM models of masonry response in chapter 7. The combination of these steps enabled an assessment of these computational methods for providing a predictive capacity, thus addressing the third objective of this PhD.

5.2 Methodology

5.2.1 Trial design

One short-duration blast trial was designed to detonate 98kg TNT flake and 2kg PE4 at an altitude of 1m above the ground surface to produce 100kg TNT equivalence and a hemispherical blast. This trial was conducted in May 2018 on the Windrush Arena site at MOD Shoeburyness as shown in figure 5.1. This location was selected because it possesses a robust concrete ground surface which can withstand the significant detonation shocks produced by 100kg TNT. This trial aimed to capture masonry damage and debris distribution data for two pairs of model and prototype structures at two overpressure regions, thus enabling the experimental benchmarking of the scaling methodology derived in chapter 4.

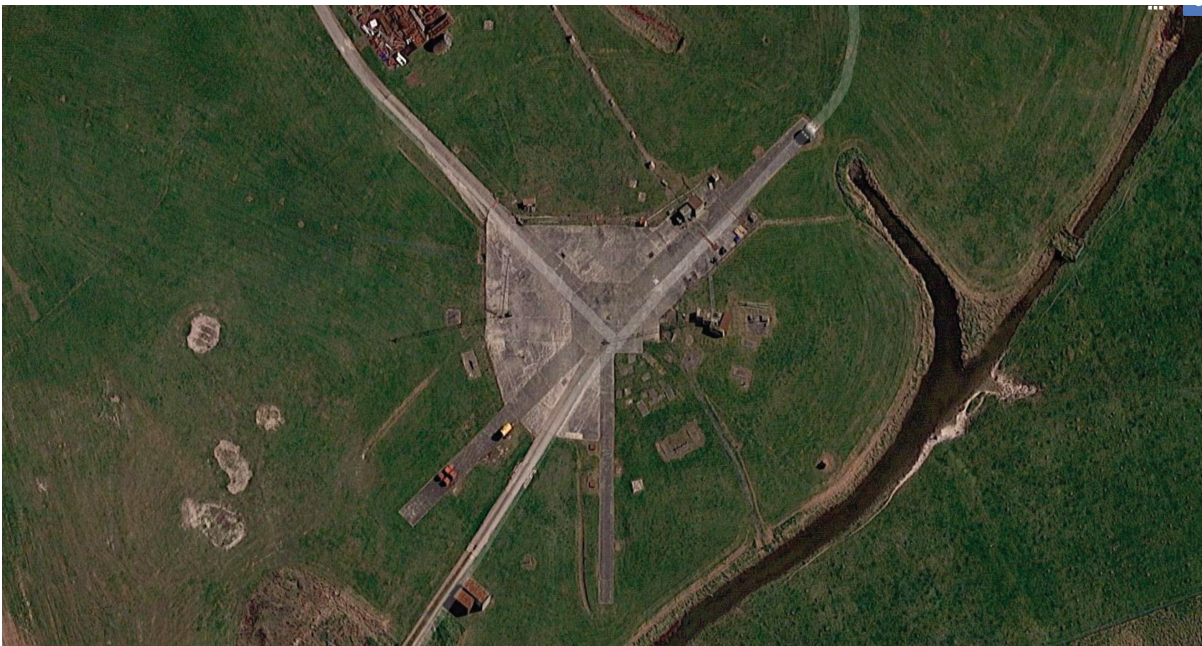


Figure 5.1: Windrush Arena testing site, MOD Shoeburyness (Foulness Island, 2019).

Two pairs of prototype and model masonry structures were built using the commercially available masonry units described in chapter 4 and shown in figure 5.2. Prototype structures utilised solid concrete blocks with 950kg/m^3 manufacturer specified density and model structures utilised solid concrete commons bricks with 1900kg/m^3 density. The manufacturer specifications for these masonry units indicated a potential density variability of $\pm 10\%$ due to a porous structure which may permit additional moisture absorption. A sample of ten bricks and ten blocks were subsequently placed in a temperature

controlled environment for 60 days to ascertain the accuracy of the supplied density values. The results of which demonstrated a mean block density of 1055kg/m^3 with a standard deviation of 3% of the mean. The mean brick density was found to represent 2060kg/m^3 with a standard deviation of 2% of the mean value. Both of these measured densities exceed the manufacturer provided values, indicating that the atmospheric conditions related to winter construction may have resulted in additional moisture absorption. Despite this, it can be shown that the measured densities produced an effective scale factor of 0.51 which closely matches the desired value of 0.50. As a result, it was determined that the masonry units met both of the scaling methodology's core requirements via geometric similarity and a reciprocal scale factor for density at a 1:2 scale level. To control any additional variances in masonry density, pallets of prototype blocks and model bricks were stored in the same weather-sealed facility during the 4 month construction phase. It was also ensured that equal quantities of bricks and blocks were relocated to the testing arena during the bricklaying stage to further mitigate any moisture absorption imbalances. Finally, each of the four structures fielded within this trial were exposed to the same atmospheric conditions on the Windrush Arena test site for approximately 4 weeks after construction, after which the trial was conducted.

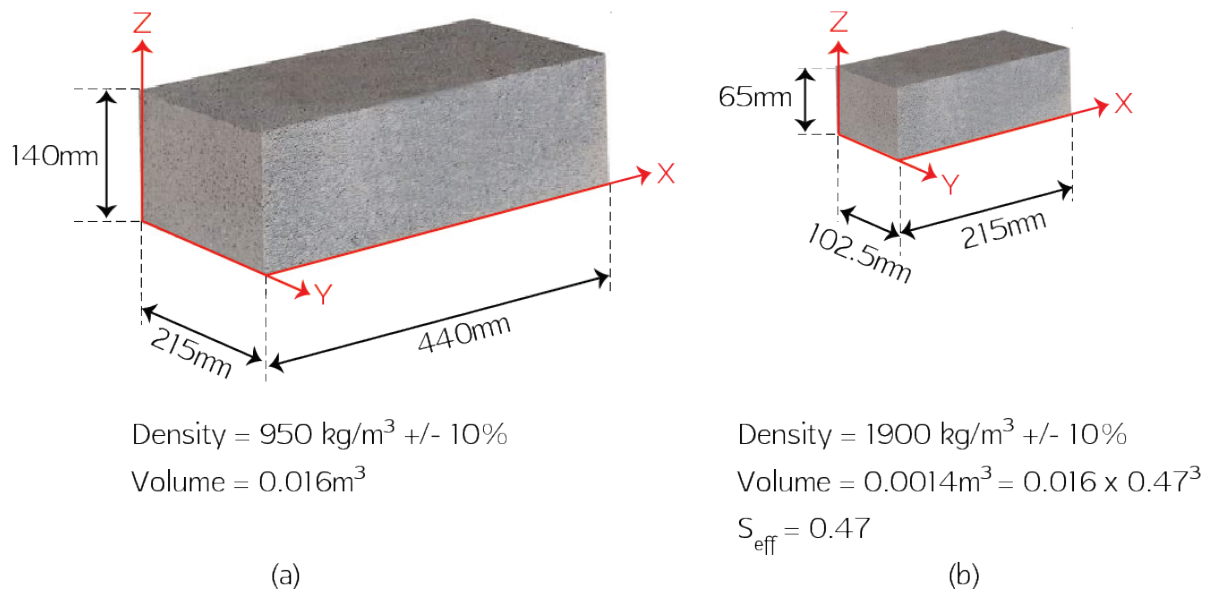


Figure 5.2: Masonry unit specifications: (a) Concrete block; (b) Concrete brick.

This trial utilised two model and two prototype masonry structures with a design which aimed to replicate simplified residential construction. This was accomplished via single-storey enclosed structures with half-thickness stretcher bonds and cuboid dimensions

as shown in table 5.1 and figures 5.3-7. Prototype masonry dimensions were chosen to represent a single UK storey at full-scale as shown by the 3.14m dimension in the Z axis. Half-thickness stretcher bond structures were selected for this trial to represent the simplest and most widely implemented form of masonry construction throughout developed and developing societies. It is acknowledged that masonry structures can be built via a range of structural bonds which also introduce the potential for full-thickness and cavity wall designs. This research has however emphasised the simplest complete structural design to enable the initial experimental benchmarking of this PhD's scaling methodology. Future work may investigate a wider diversity of structural designs alongside the additional complexities these may introduce to the damage and debris phases of masonry response.

Table 5.1: Short-duration trial masonry design summary.

Structure Type	Dimensions in x, y, z (m)	Bond Type
Prototype	3.14 x 3.14 x 3.14	Stretcher
Model	1.57 x 1.57 x 1.57	Stretcher

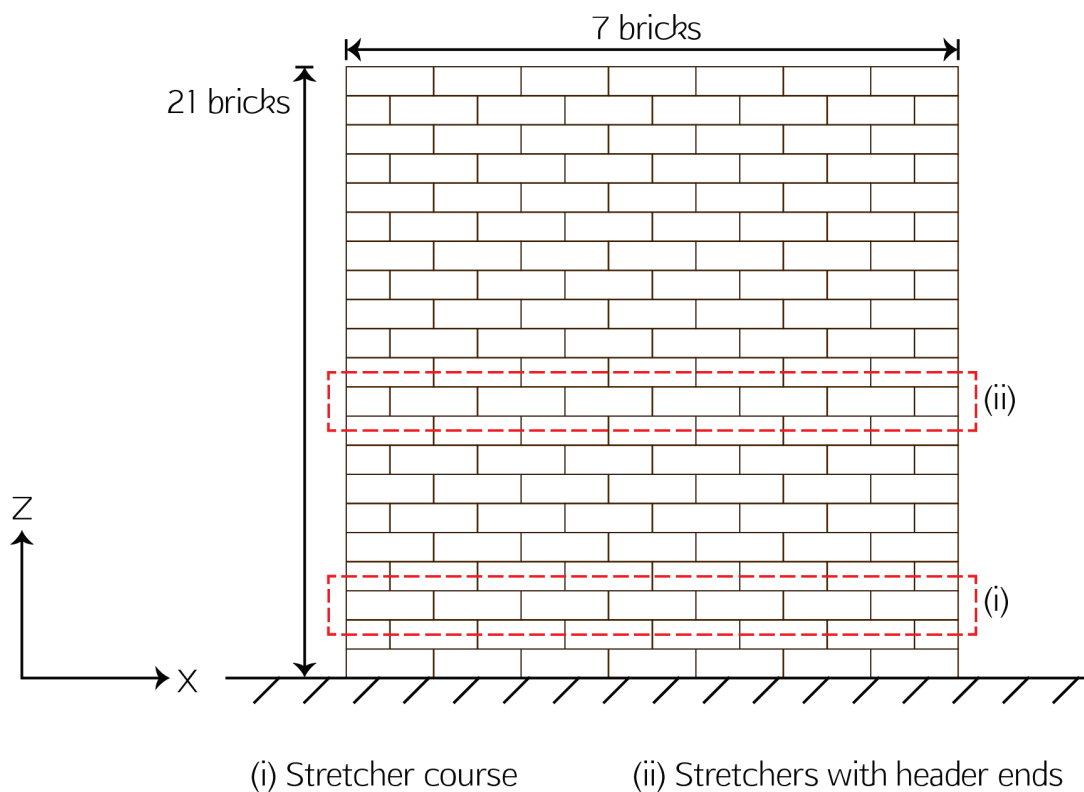


Figure 5.3: Front elevation schematic of single-storey stretcher bond courses with brick requirement in lateral and vertical directions.

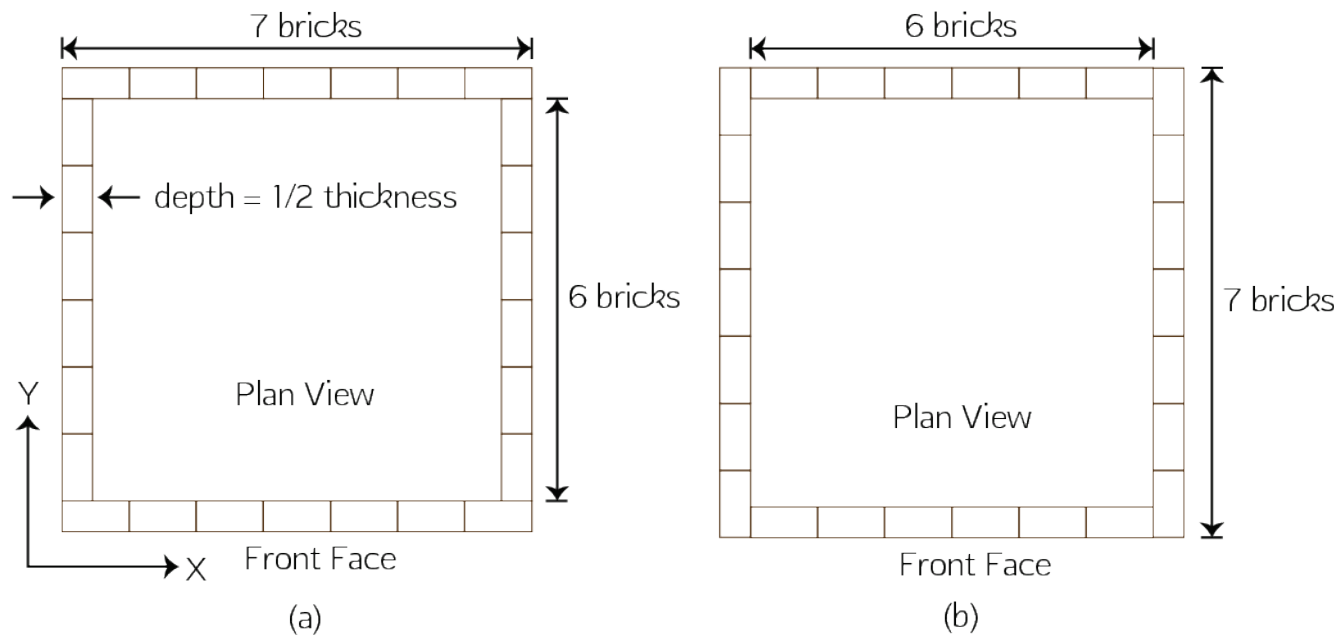


Figure 5.4: Plan schematic of stretcher bond courses: (a) Stretcher course; (b) Header course.



Figure 5.5: Pre-trial comparative image of model and prototype masonry structures.

Scaling Masonry Structures in a Blast Environment



(a)



(b)



(c)



(d)

Figure 5.6: Experimental masonry structures, model elevation images: (a) 55kPa front; (b) 55kPa rear; (c) 110kPa front; (d) 110kPa rear.



(a)



(b)



(c)



(d)

Figure 5.7: Experimental masonry structures, prototype elevation images: (a) 55kPa front; (b) 55kPa rear; (c) 110kPa front; (d) 110kPa rear.

Each of the prototype and model structures were designed to implement a type (iii) mortar to provide a predicted crushing strength of 4N/mm^2 . The constituent properties of the mortar were Ordinary Portland Cement, building sand and plasticizer with the materials being combined via: Cement-to-sand with plasticizer: **ratio by volume of 1:5-6**.

To determine the physical mortar properties used within this trial, a range of 40 cubes with 100mm dimensions were cast for each new batch of mortar during the construction phase. These were allowed to cure alongside the structures to ensure their physical properties were a product of the same atmospheric conditions as the masonry. These cubes were crushed on the day of the trial to yield the data shown in table 5.2. The results of which show a mean curing period of 71 days which exceeds the minimum design requirement of 28 days. A mean crushing strength of 6.2N/mm^2 was found to be conservative versus the design requirement of 4N/mm^2 , resulting in a type (ii) mortar versus the originally designed type (iii). The source of this increase is likely linked to the use of a professional mason for whom, additional strength is often viewed as a positive attribute. A standard deviation of 16% of the mean for strength indicates that the mason exerted an high degree of quality control over the mortar production process. This is further shown by a standard deviation value of 2% of the mean for density which indicates a very small level of mixture variation. A low level of density variation is particularly important as the scaling method depends upon a first principles representation of the masonry structure as an inertial resistance to blast loading.

Table 5.2: Material properties of mortar utilised to construct experimental masonry.

	Curing Time (days)	Density (kg/m^3)	Crushing Strength (N/mm^2)
Mean Value	71	1718	6.2
Standard Deviation (% of μ)	32%	2%	16%

The physical application of mortar within each of the model and prototype structures was achieved via a constant 10mm bed thickness to meet the durability criteria specified by BS5628 (The British Standards Institution, 2005) for external walls. The decision not to scale the mortar material density or bed thickness was made after developing the proof shown below. This demonstrates that the practical approach of utilising an industry-standard mortar bed thickness was able to yield a mass scaling factor of S^2 which matches the idealised scaling

scenario for the model. The utilisation of a constant mortar material for the model and prototype also mitigates potential issues with workability, curing and crushing strength which could arise from modifying the density. Similarly, the maintenance of 10mm bed thickness moderated the issue of bed thickness uniformity which would be more challenging with a scaled thickness of 5mm.

$$\text{Ideal: if } \rho_{\text{mortar}_{\text{model}}} = \rho_{\text{mortar}_{\text{proto}}} \cdot S^{-1}$$

$$V_{\text{mortar}_{\text{model}}} = V_{\text{mortar}_{\text{proto}}} \cdot S^3$$

$$\therefore m_{\text{mortar}_{\text{model}}} \equiv m_{\text{mortar}_{\text{proto}}} \cdot S^2$$

$$\text{Practical: } V_{\text{mortar}_{\text{model}}} = (\text{Width}_{\text{proto}} \cdot S) \cdot (\text{Breadth}_{\text{proto}} \cdot S) \cdot \text{Thickness}_{\text{proto}}$$

$$V_{\text{mortar}_{\text{model}}} = V_{\text{mortar}_{\text{proto}}} \cdot S^2$$

$$m_{\text{mortar}_{\text{model}}} = \rho_{\text{mortar}_{\text{model}}} \cdot V_{\text{mortar}_{\text{model}}}$$

$$\text{if } \rho_{\text{mortar}_{\text{model}}} = \rho_{\text{mortar}_{\text{proto}}} \rightarrow m_{\text{mortar}_{\text{model}}} \equiv \rho_{\text{mortar}_{\text{proto}}} \cdot V_{\text{mortar}_{\text{proto}}} \cdot S^2$$

$$\therefore m_{\text{mortar}_{\text{model}}} \equiv m_{\text{mortar}_{\text{proto}}} \cdot S^2$$

As discussed in chapter 4, the scaling methodology derived by this PhD research utilises first principles to represent a masonry structure as an inertial resistance to blast loading. It is therefore critical to determine the actual applied scaling factor for the composite model masonry structures as a whole. Tables 5.3-5 calculate the total masonry structure volume, mass and density for the actual model structure fielded within this experimental trial. These values are shown alongside an idealised theoretical model at 1:2 and the values for the actual experimental prototype structure. Examination of table 5.3 shows that the volume of the experimental model structure has been scaled by an effective factor of 0.49, which closely matches the desired scale factor of 0.50. By utilising the total structural volume from table 5.3 and total mass from table 5.4 it was possible to calculate total structural density in table 5.5. As a result, it can be seen by the highlighted cells in table 5.5 that the density of the experimental model structure has been scaled by an effective factor of 0.55 versus the experimental prototype. This value closely matches the ideal design value of 0.50, resulting in a 1:1.81 scaling level versus the ideal design level of 1:2.

Table 5.3: Effective scale factor of model structure volume, Windrush Arena trials.

Structure type	Structure volume (m ³)	Effective Scale Factors	
Prototype	7.94	$S^3 = (\text{Volume}_{\text{model}} / \text{Volume}_{\text{prototype}})$	$S_{\text{volume}} = (S^3)^{1/3}$
Ideal Model	0.99	0.125	0.5
Model	0.95	0.119	0.49

Table 5.4: Effective scale factor of model structure mass, Windrush Arena trials.

Structure type	Structure mass (kg)	Effective Scale Factors	
Prototype	8844	$S^2 = (\text{Mass}_{\text{model}} / \text{Mass}_{\text{prototype}})$	$S_{\text{mass}} = (S^2)^{1/2}$
Ideal Model	2211	0.25	0.5
Model	1902	0.22	0.46

Table 5.5: Effective scale factor of model structure density, Windrush Arena trials.

Structure type	Structure density (kg/m ³)	Effective Scale Factors	
Prototype	1114	$S^{-1} = (S_{\text{mass}} / S_{\text{volume}})$	$S = (S^{-1})^{-1}$
Ideal Model	2228	2	0.5
Model	2014	1.81	0.55

Two design free-field overpressures of 55kPa and 110kPa were selected for this trial to represent a range of potential masonry response modes including cracking, deflection and minimal collapse at 55kPa versus more severe damage and structural collapse at 110kPa. Table 5.6 details the stand-off distance and design blast parameters for the model and prototype structures at these overpressure regions.

Table 5.6: Short-duration trial design blast parameters.

Structure Type	Charge, W (kg)	Structure Position	Stand-off, R (m)	Design Blast Parameters		
				Overpressure, P _s (kPa)	Impulse, i _s (kPa.ms)	Positive Phase, t ⁺ (ms)
Prototype	100	Front	18.5	65	336	15.9
		Mid	20	55	314	16.5
		Rear	21.5	50	294	17
Model	100	Front	19.25	61	324	16.2
		Mid	20	55	314	16.5
		Rear	20.75	53	304	16.8
Prototype	100	Front	12.75	140	466	12
		Mid	14.25	110	422	13.4
		Rear	15.75	90	386	14.4
Model	100	Front	13.5	124	442	12.8
		Mid	14.25	110	422	13.4
		Rear	15	99	403	13.9

Examination of table 5.6 shows differing stand-off distances for the front and rear faces of the model and prototype structures which contrasts with mid positions which remained constant as shown by the highlighted rows. These positional differences are due to the relatively small blast wavelength compared to the masonry dimensions. The result of which is the application of a pressure gradient to a structure as a function of its length. It can also be shown that this gradient would differ for the model and prototype structures due to their different lengths. To counter this issue, this blast trial was designed to apply an average pressure gradient to the model and prototype structures by positioning the centroids at equal stand-off distances with a face-on orientation to the blast as shown in figure 5.8.

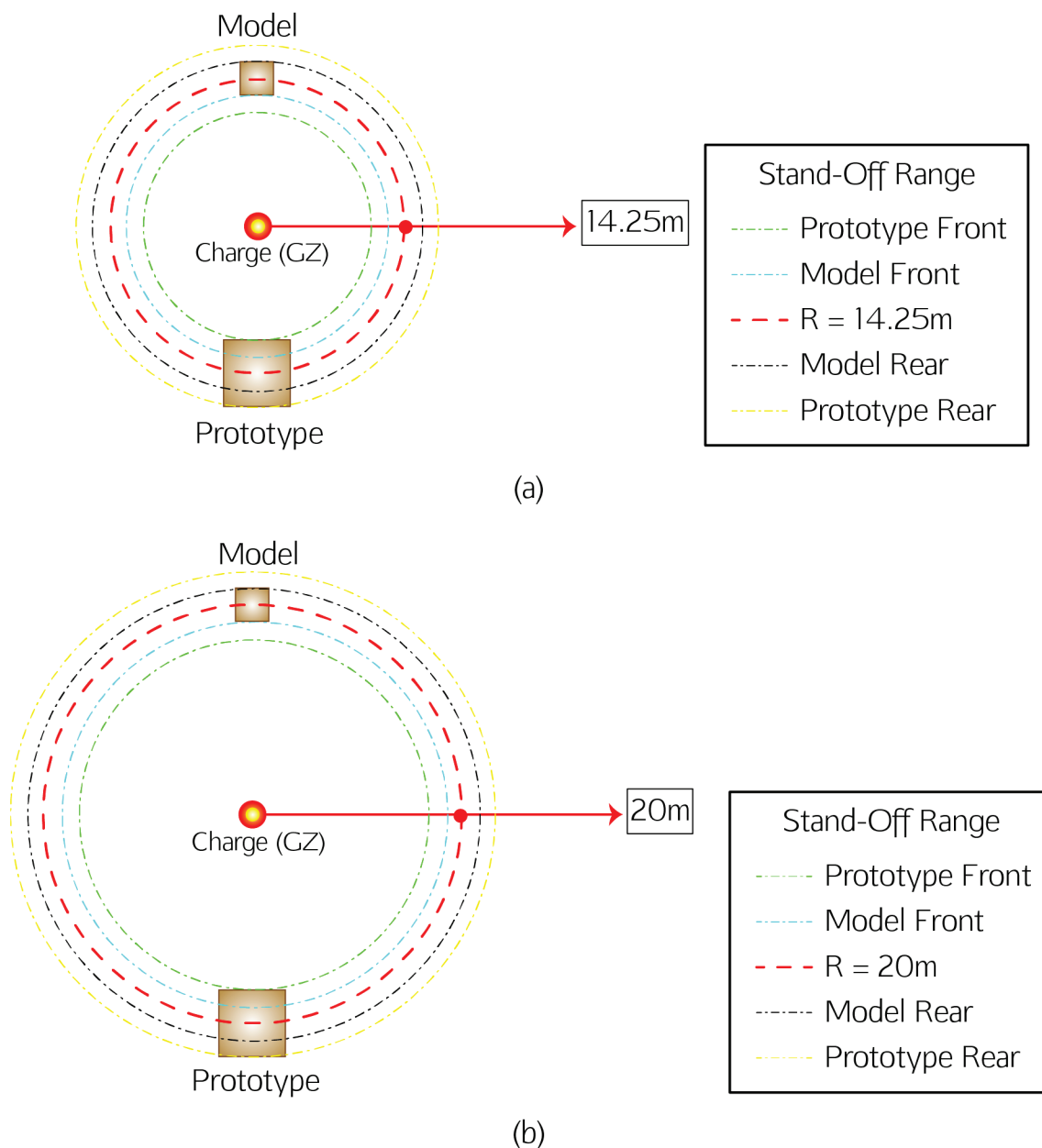


Figure 5.8: Stand-off distances variance due to pressure gradient effect: (a) 110kPa; (b) 55kPa.

The final locations of the model and prototypes on the Windrush Arena test site can be seen in figure 5.9. This layout was selected to prevent blast wave interactions and reflections in the spaces between the structures as these could influence masonry response. As a result, the distance between each structure at least greater than four times the blast wavelength. Figure 5.10 details a range of pre-trial still images which illustrate the structural layout shown in figure 5.9. Figure 5.10a provides a panoramic view of the entire Windrush Arena test site with each of the four masonry structures alongside the 100kg TNT charge at the centre. Figure 5.10b provides a similar perspective from the opposite side of the testing site. Figure 5.10c demonstrates the physical spacing between the model and prototype structures at the 14.25m stand-off location with a 'charge-eye-view' perspective. Similarly, figure 5.10d demonstrates this for the model and prototype at 20m stand-off.

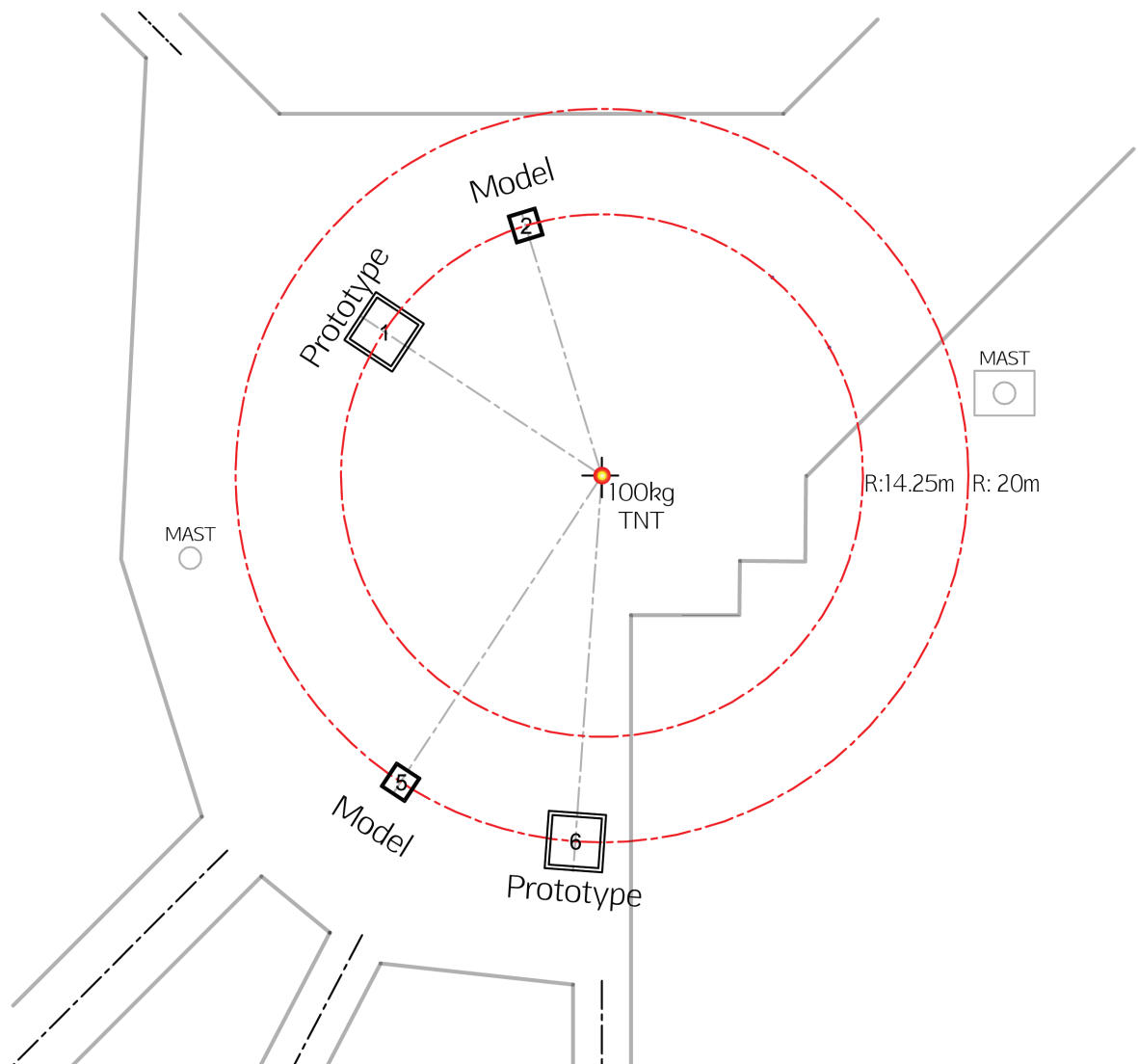


Figure 5.9: Plan schematic of prototype and model structure locations on Windrush Arena.



(a)



(b)



(c)



(d)

Figure 5.10: Pre-trial still images of Windrush Arena site: (a) Panoramic view #1 of trial; (b) Panoramic view #2 of trial; (c) Prototype and model at 14.25m; (d) Prototype and model at 20m.

Each of the masonry structures were equipped with non-responding roof enclosures that were designed to be stiff, yet lightweight as demonstrated by figure 5.11. The frame sections of these enclosures were constructed using 10mm steel and they were securely bolted to the ground surface within the plan area of the masonry structures as shown by figures 5.11a and 5.11c. Masonry construction was subsequently completed around the enclosure frames as shown in figure 5.11b and 5.11d. The top section of the roof enclosures were built with lightweight interlocking polyurethane panels that were reinforced by steel cross-braces as shown in figure 5.12



(a)



(b)



(c)



(d)

Figure 5.11: Roof enclosure images: (a) Model frame bolted to ground; (b) Model construction around enclosure; (c) Prototype frame bolted to ground; (d) Prototype construction around enclosure.



Figure 5.12: Polyurethane roof panels with steel cross-support frames.

Each of the roof enclosure systems were designed to incorporate an adjustable threaded mechanism which enabled the enclosure to be lowered into contact with the top course of the masonry structure. In doing so, these enclosures sealed the interior of the masonry structures to limit blast pressure infill that may otherwise influence masonry response. This process is demonstrated in figure 5.13. Importantly, the self-weight of the roof enclosure systems were fully supported by the steel framework and transmitted to the ground, ensuring no additional loads were applied to the masonry structures. This was an important requirement for this trial as the initial phase of experimental benchmarking of the PhD scaling methodology was designed to minimise the number of potentially random variables. As a result, it was important to ensure that the roof enclosures did not apply any additional loading to the masonry as this could result in eccentric compressive loading during panel wall deflections due to blast loading. The result of which would likely be additional masonry response complexity which is beyond the scope of the first phase of experimental benchmarking of the scaling methodology.



Pre-adjustment



Threaded adjustment of roof panel



Post-adjustment

Figure 5.13: Demonstration of roof enclosure adjustment process for model structure.

5.2.2 Data acquisition

This trial aimed to capture a range of data relating to masonry response and the blast environment. A series of 12 free-field gauges were located at stand-off distances to represent the front, mid and rear positions of each model and prototype structural location as detailed in table 5.6. Free-field blast environment data was captured by proprietary B12 static overpressure gauges that were developed by the Foulness Trials Group of Spurpark Ltd. Each pressure device was calibrated to enable time sequencing with the detonation trigger, defining accurate blast arrival times within the pressure data. These gauges were distributed over two radial rows as shown in figures 5.14 and 5.15a-b. Two instrumentation sheds were also located at a 26m stand-off distance to serve as a central hub for the gauges utilised within the trial. As a means of protection, a wall was erected between the sheds and the blast wave as seen in figure 5.15b.

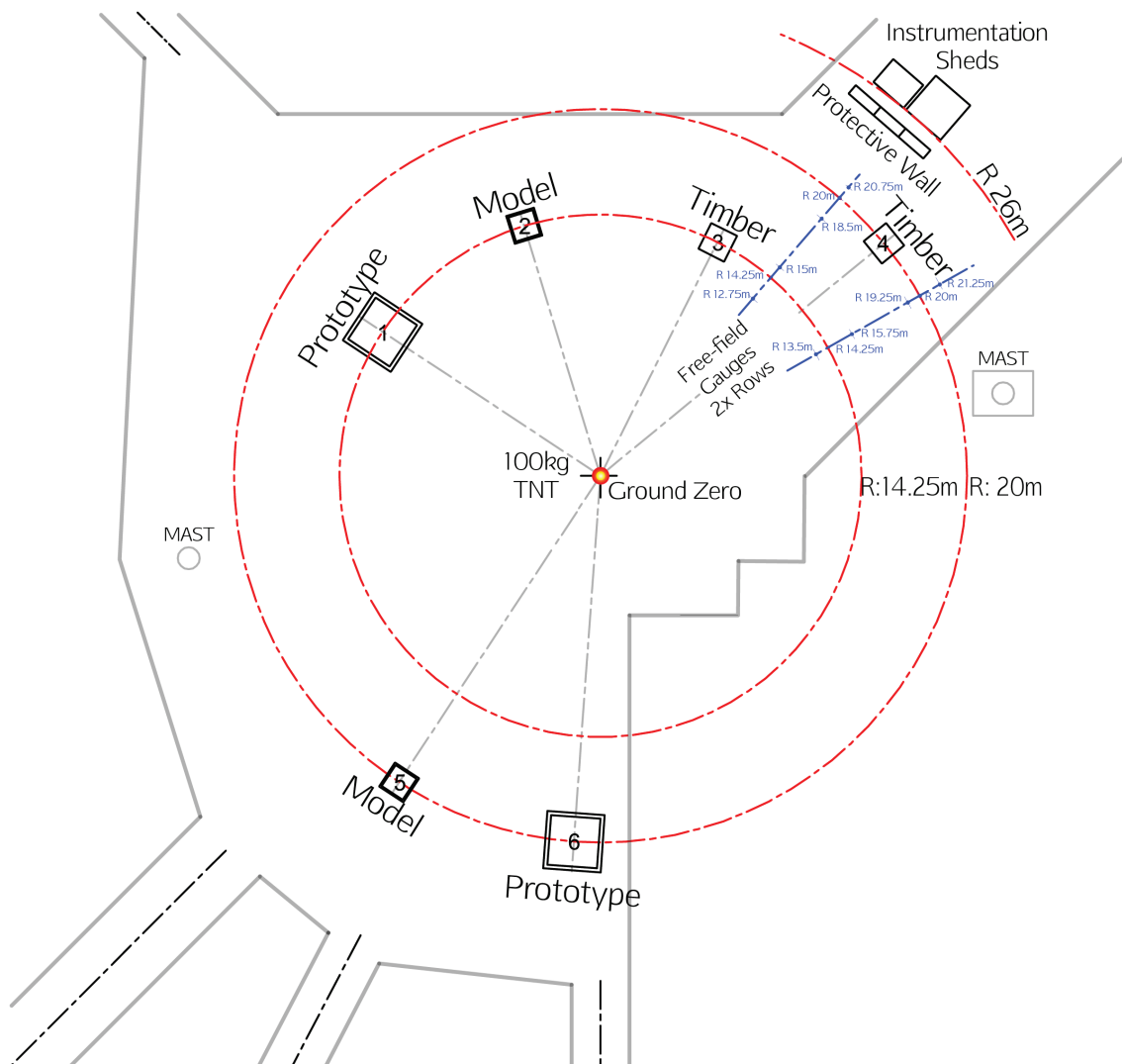


Figure 5.14: Plan schematic of pressure gauge locations on Windrush Arena.



(a)



(b)

Figure 5.15: Images of instrumentation on Windrush Arena: (a) free-field pressure gauge rows; (b) Free-field pressure gauges and protective concrete wall.

Closer inspection of figure 5.14 shows that this trial included two timber structures to provide non-responding structural geometries which represented the model masonry structures at 14.25m and 20m stand-off locations. These were constructed via 50mmx200mm studwork for the front face and 50mmx100mm studwork for the side and rear faces with each using 200mm centres. The studwork frames were finished with 18mm plywood facing as shown in figure 5.16. Examination of figures 5.16a-b illustrate the level of timber reinforcement utilised to build these elements and thus increase their chances of survival during the blast event. After being positioned on the test site, these structures were securely

bolted to the ground surface to prevent translation during the blast event as shown in figure 5.16d. Non-responding structures were also considered for the prototype geometries but it was not feasible to construct 3m timber cuboids inside the available workshops at MOD Shoeburyness.

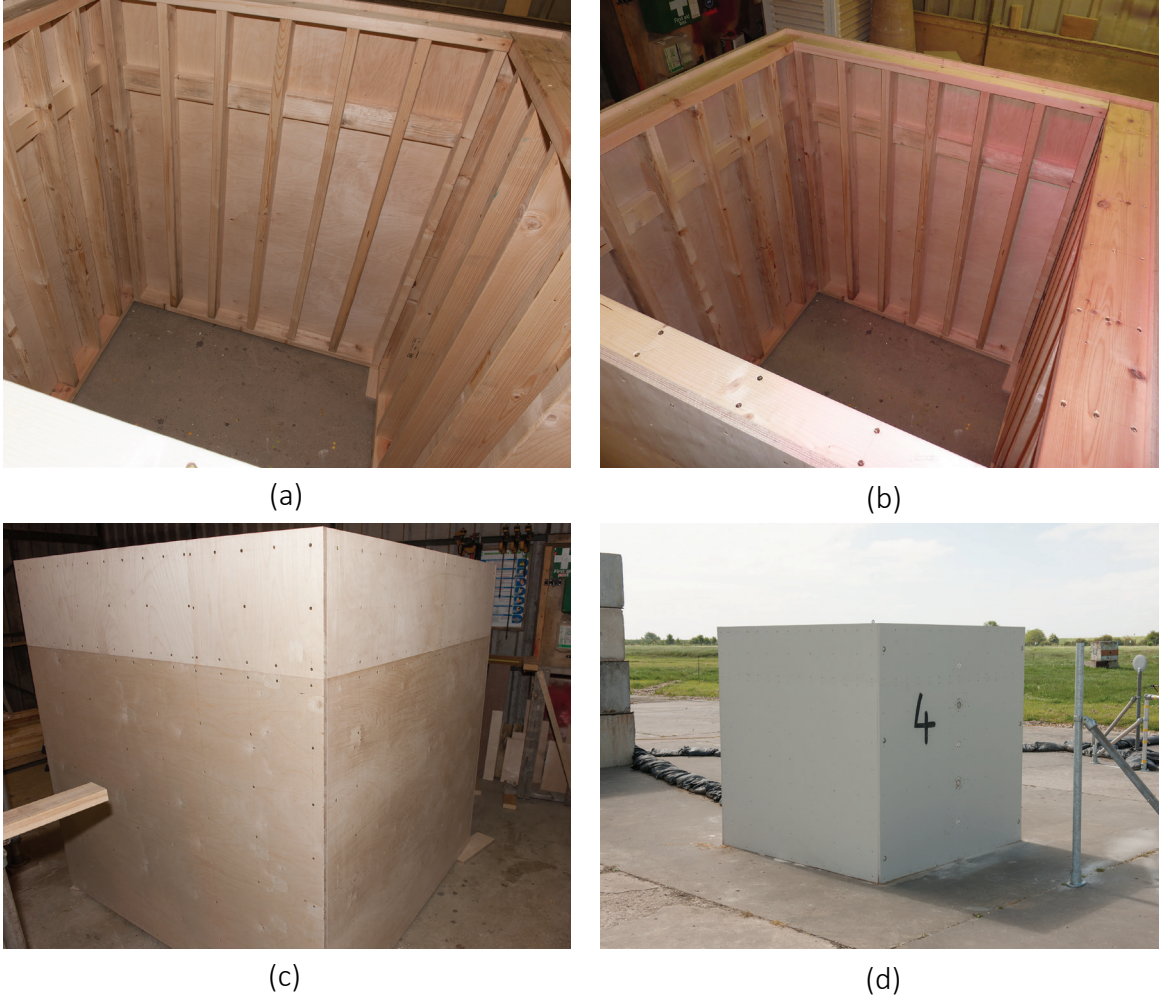


Figure 5.16: Images of non-responding timber structures: (a) Internal studwork view #1; (b) Internal studwork view #1; (c) Completed structure; (d) Structure located on test site.

By utilising non-responding timber structures, it was possible to instrument the front, side and rear faces with a series of Endevco 8510C reflected pressure gauges as shown in figures 5.17-19. Each of these pressure devices were time sequenced with the detonation trigger to define an accurate time of blast arrival within the pressure records. Thus enabling this trial to measure the interaction of the blast wave with non-responding model structure geometries. Front face gauge locations were selected to investigate the level of variation in reflected blast pressure over the structure's height. Side face gauge locations were chosen to monitor blast clearing near to the leading edge and measure pressure variation along the

structure's length. Rear face gauge positions were selected to assess blast clearing over the top of the structure and the application of pressure creep at the rear face due to blast clearing. The overall design of these non-responding structures and associated pressure gauges provided the potential for additional computational benchmarking of CFD models of blast interaction with model structure geometries in chapter 7.

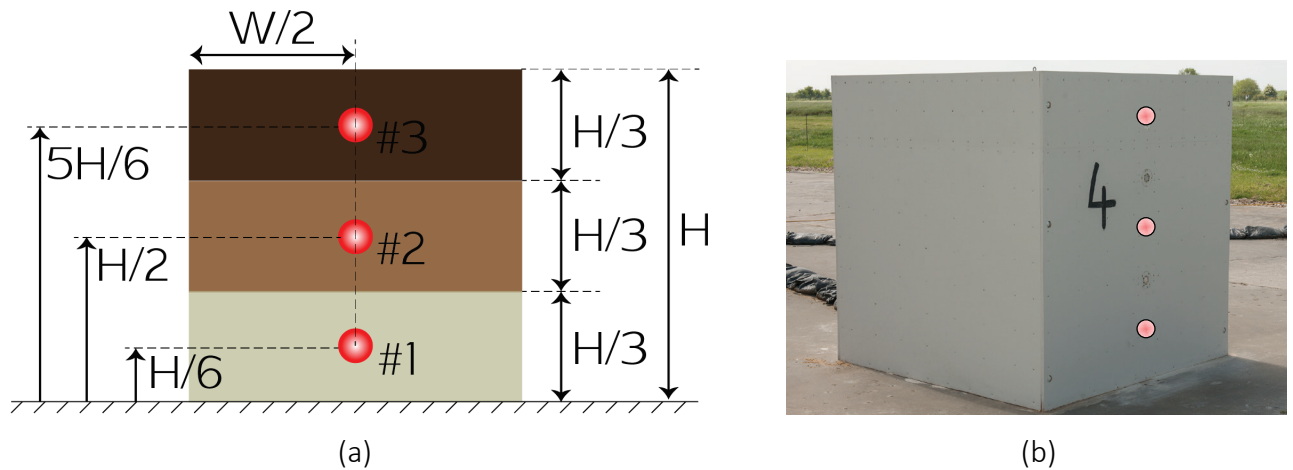


Figure 5.17: Front face reflected pressure gauge: (a) Gauge position schematic; (b) Pre-trial image of front face instrumentation.

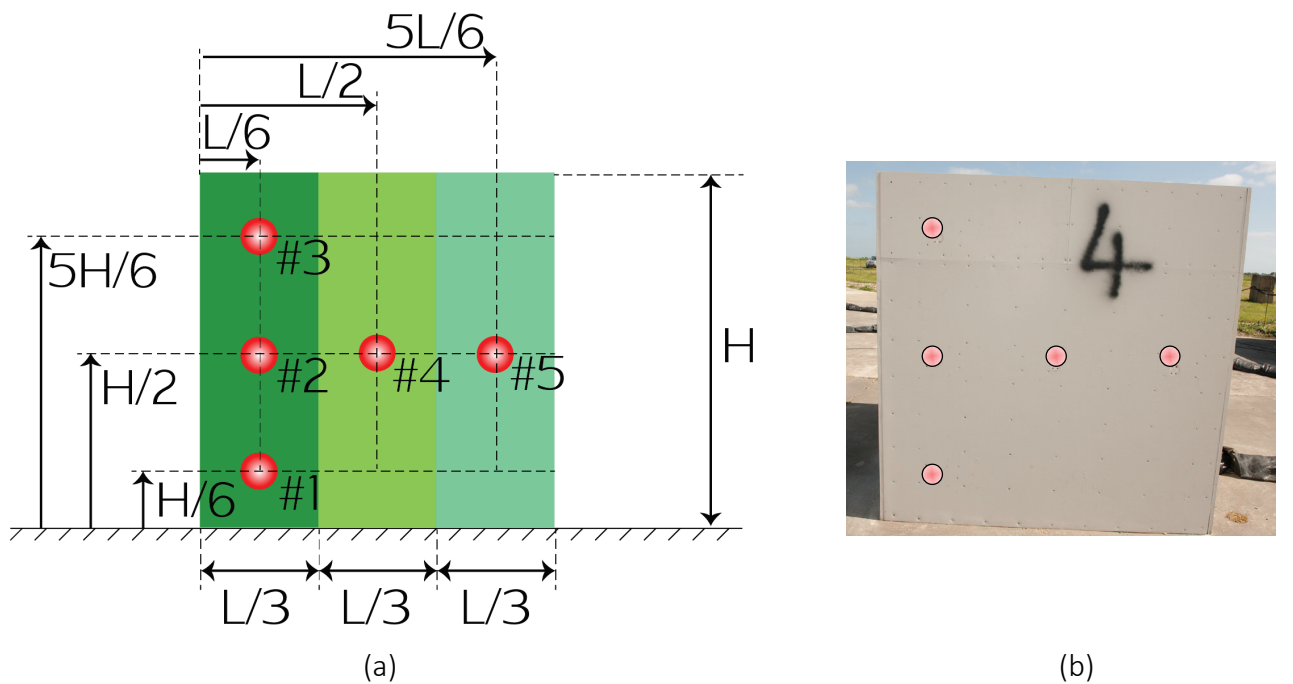


Figure 5.18: Side face reflected pressure gauge: (a) Gauge position schematic; (b) Pre-trial image of side face instrumentation.

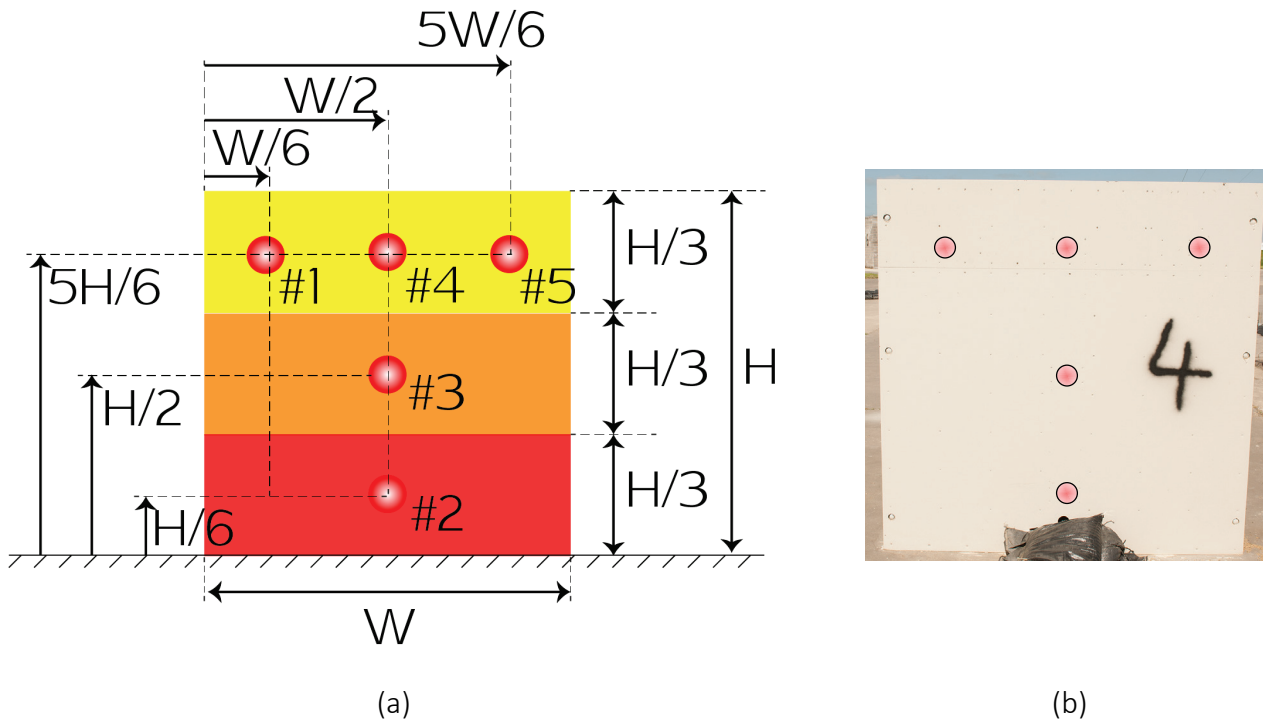


Figure 5.19: Rear face reflected pressure gauge: (a) Gauge position schematic; (b) Pre-trial image of side face instrumentation.

The qualitative assessment of masonry damage via cracking and degrees of collapse was achieved by utilising a combination of high-speed video and still photography. A series of eight high-speed Phantom v7.3 cameras were deployed to monitor masonry response at 2000fps as shown in figure 5.20. These were positioned circumferentially around the Windrush Arena test site with two dedicated cameras per masonry structure. These two cameras each captured a 45 degree perspective of two panel walls where the first documented front and side panels and the second monitored rear and side faces. Each of the eight cameras were housed in armoured containers to prevent the application of blast loading directly to the camera bodies as shown in figure 5.21. It was important to position the cameras at stand-off distances outside of the main testing area to limit the likelihood of camera damage due to blast pressure and heat from the fireball. As a result, each of the front panel wall camera views would likely be partly obstructed by the fireball due to an unavoidable 'cross-site' perspective. To assist the assessment of post-trial video files, pressure-triggered light sources were deployed for each structure as shown in figure 5.22. It can be seen that reflected pressure gauges were adhered to the front of each masonry structure to enable pressure-triggered activation of the light sources, thus visually identifying the instant of blast arrival within the Phantom video data.

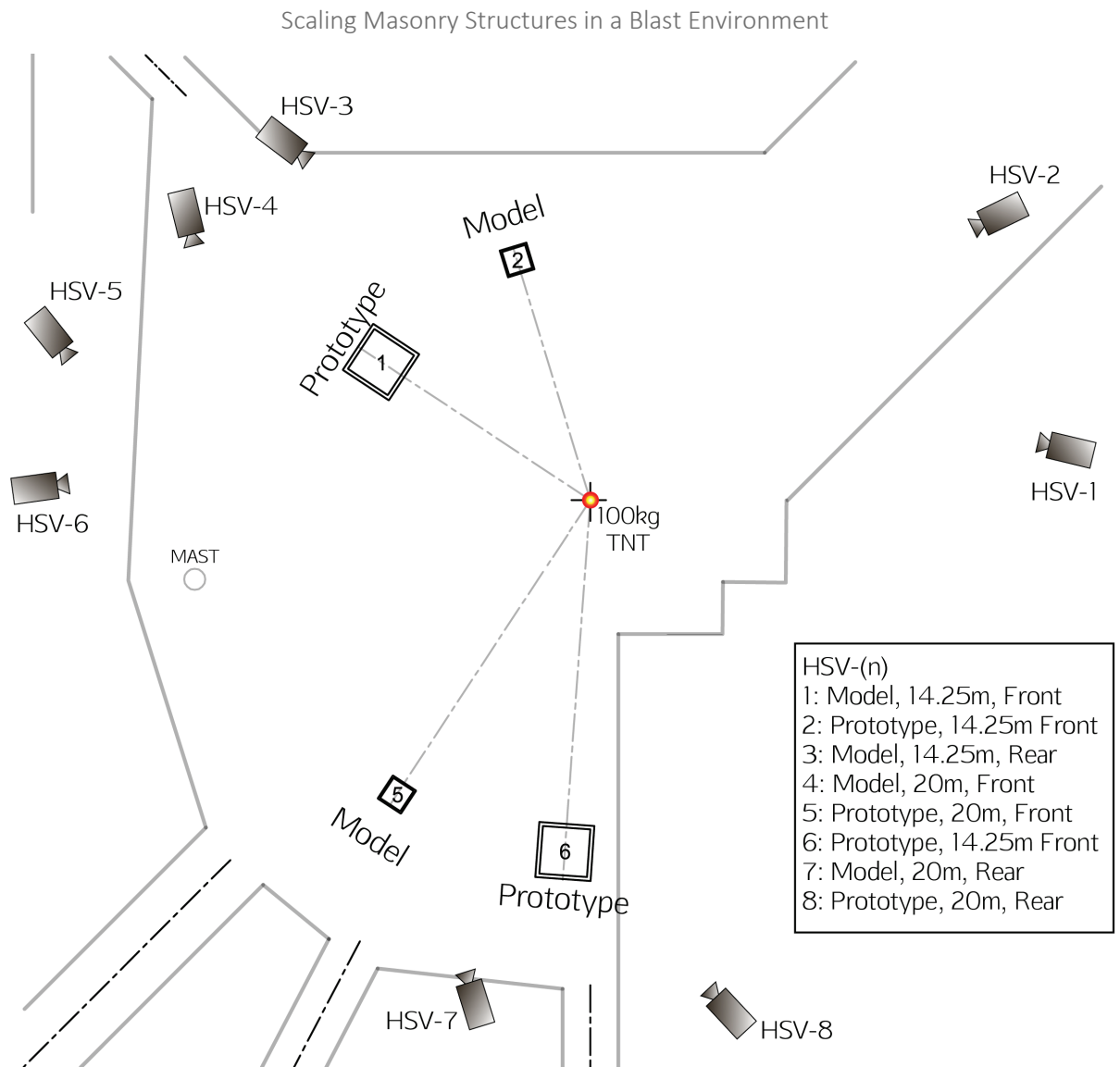


Figure 5.20: Plan schematic of high-speed camera locations on Windrush Arena.



Figure 5.21: Image of armoured housing for high-speed camera.

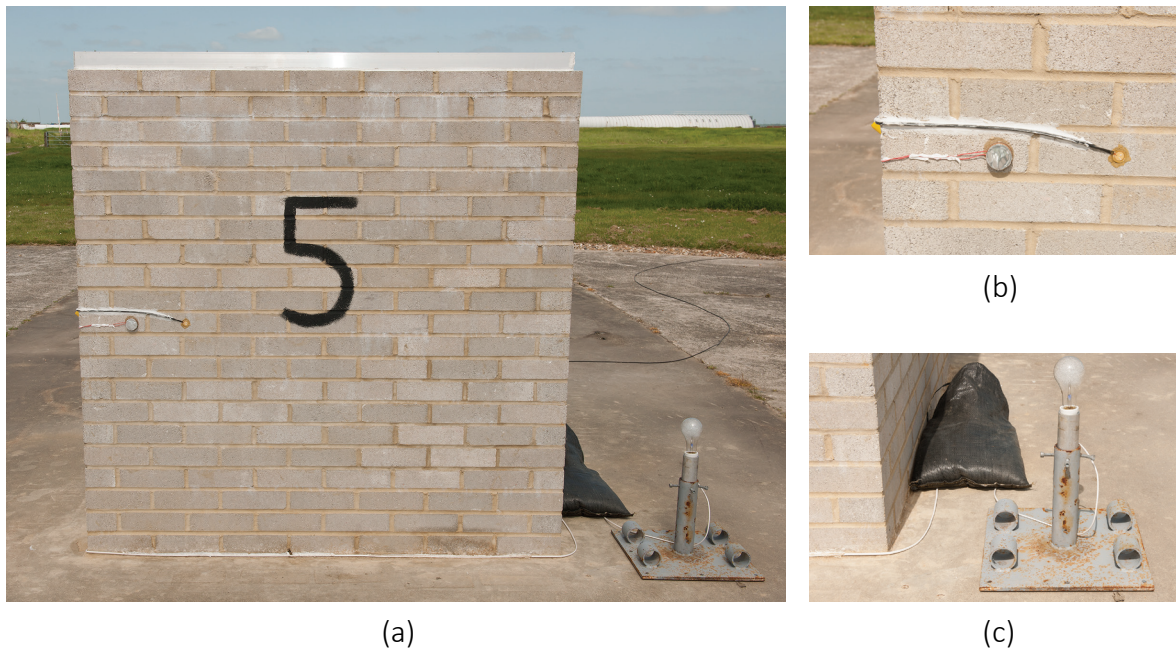


Figure 5.22: Pre-trial images of pressure-driven light sources: (a) Model structure with pressure gauges and light source; (b) Magnified view of reflected pressure gauge; (c) Magnified view of light source.

Quantitative assessment of post-trial debris distributions was accomplished via manual weighing and positional logging of masonry debris for model masonry structures. Debris mass could then be divided by the total mass of the original structure to produce a percentage of total mass value. These values were assigned to a spatial grid location in the local X and Y axes where the grid resolution was 50% of the structural width and 50% of the structural length as shown in figure 5.23.

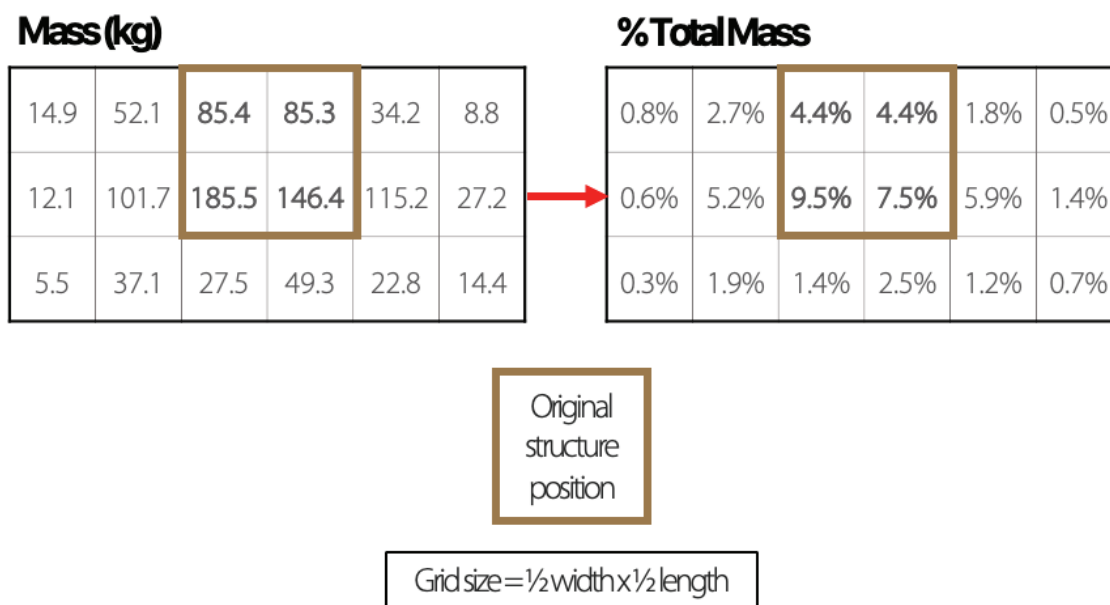


Figure 5.23: Spatial quantification of model debris distributions with non-dimensionalised grid.

Whilst the utilisation of a non-dimensionalised grid system enabled direct comparisons with prototype debris, the enormous masses associated with prototype debris required an alternative method to collect quantitative debris data. This implemented a manual block counting procedure to determine the spatial distribution of blocks as a percentage of the total block quantity for the prototype. It was then possible to directly infer the percentage of total structural mass from this data as shown in figure 5.24. This procedure was verified by benchmarking the percentage of total brick distribution for a model structure against the manually acquired actual debris spread as percentage of total structural mass as shown in figure 5.25. A close level of agreement between these two distributions therefore indicated the appropriateness of this methodology to infer actual debris spread for a prototype structure via spatial block counting. The accuracy of this methodology can be linked to the high likelihood that mortar will remain attached to brick or block units, indicating that brick or block spread is highly representative of total masonry debris spread. As a result, future debris collection procedures can be greatly simplified by utilising a brick or block counting procedure within a spatial grid, enabling the inference of quantitative debris spread as a percentage of the original structural mass.

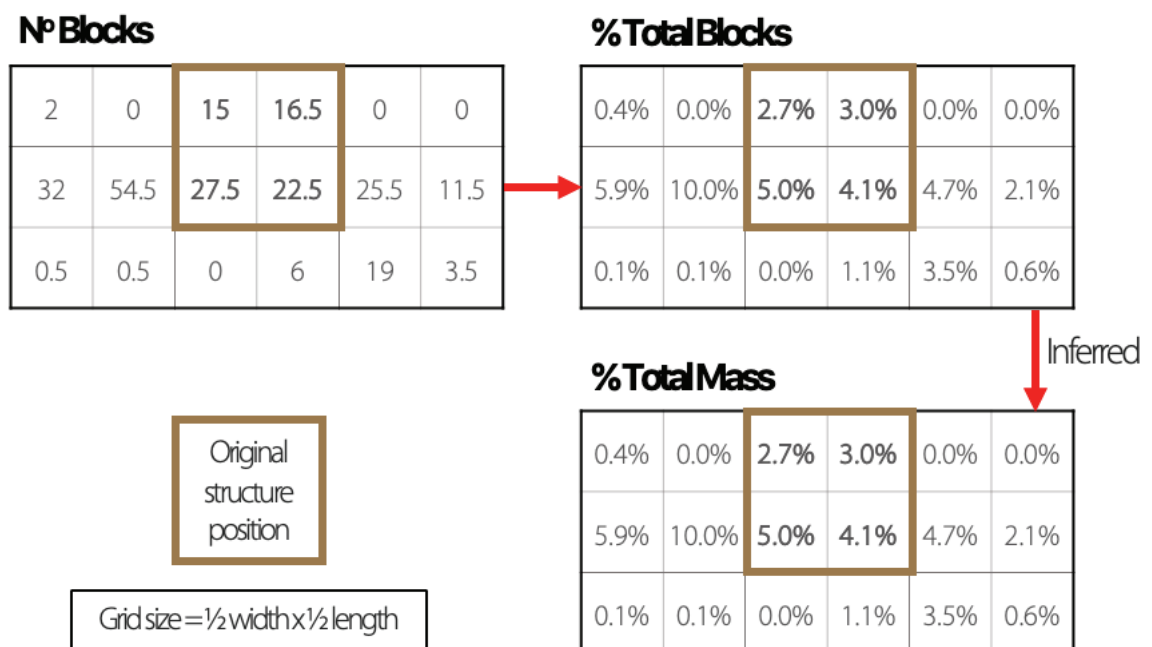


Figure 5.24: Block counting procedure to infer actual debris distribution.

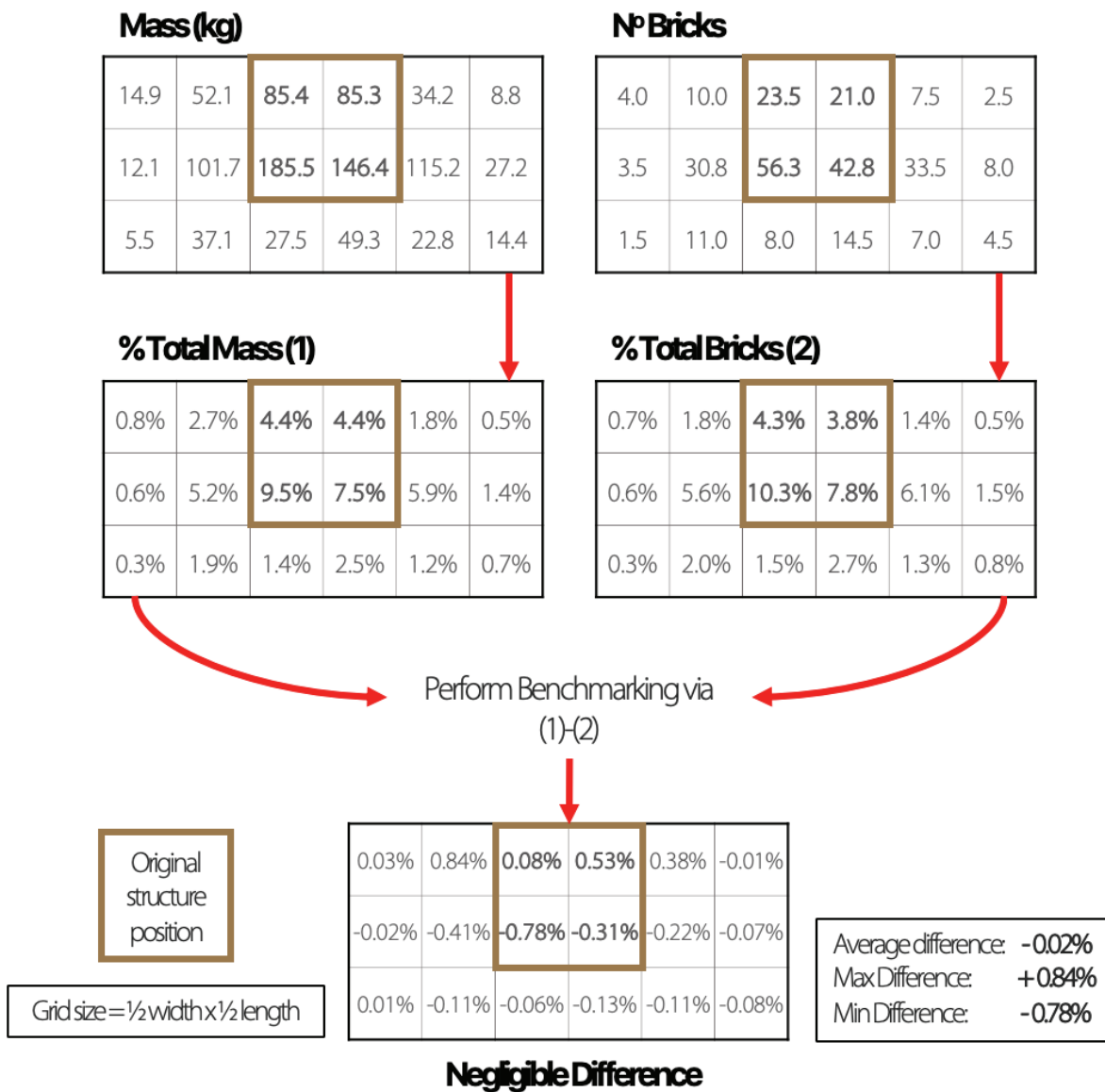


Figure 5.25: Benchmarking of brick counting procedure to infer actual debris distribution.

This trial also aimed to measure equivalent brick motion via bespoke housings for a six degree of freedom DTS-6X Pro accelerometer in the front face of the model and prototype at the 14.25m stand-off as shown in figure 5.26. The housing design for the model and prototype differed with the model utilising a hollow steel brick element and a 3D printed plastic insert for the accelerometer and data capture device. The exterior of this brick was coarsely sanded and finished with a coarse gravel exterior to simulate the surface roughness of a concrete brick element as shown in figure 5.27a. This element was designed to replicate the geometry and mean mass value of a concrete brick unit to ensure the geometry and density requirements of the scaling method were maintained. The accelerometer element was finally built into the masonry structure via the application of mortar to simulate and capture the dynamic motion of a brick at this location as shown in figures 5.27b-e.

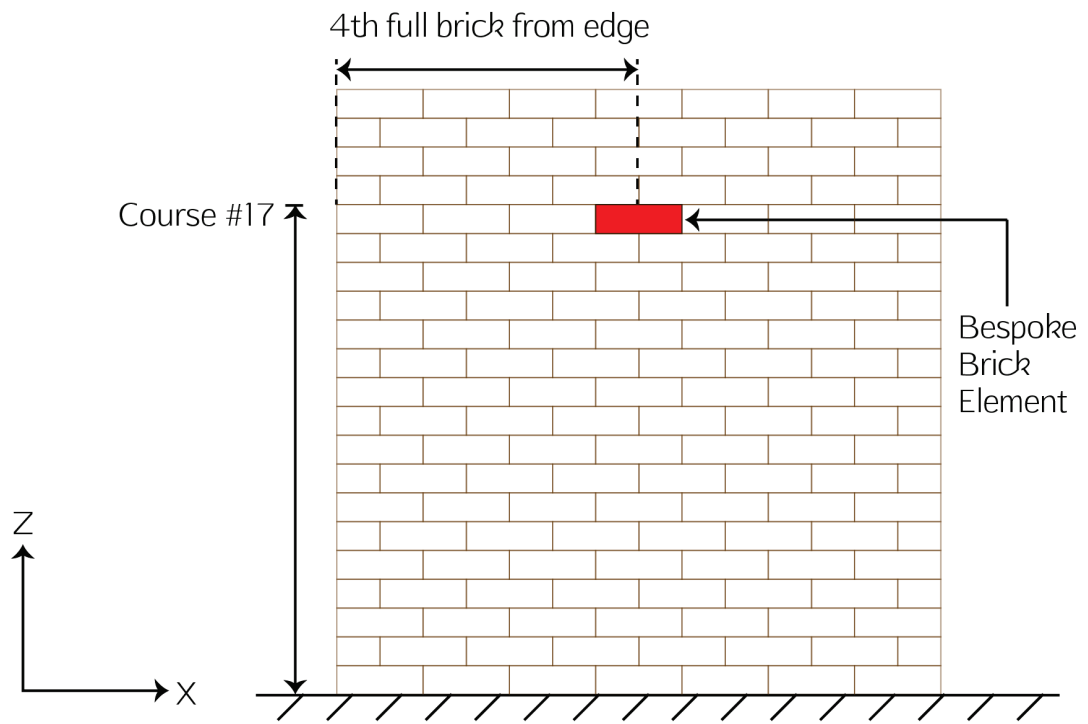


Figure 5.26: Front elevation location of accelerometer brick elements.



(a)



(b)



(c)



(d)



(e)

Figure 5.27: Model accelerometer setup: (a) Brick element; (b) Accelerometer and data storage device; (c) Magnified view of embedded housing. (d) Element mortared into masonry; (e) Completed structure with housing element.

The prototype structure was instrumented with a standard concrete block which implemented a cylindrical plastic insert to house the accelerometer and data acquisition device. The installation of this cylinder was achieved by carefully boring out some of the original block material as shown in figure 5.28a. The final mass of this insert was designed to match the mass removed, thus ensuring the total mass and geometry of the block matched the other blocks within the prototype structure. This block was built into the masonry structure via the application of mortar to enable the capture of dynamic motion at the assigned block location as shown in figure 5.28b. By capturing a dynamic record of equivalent brick and block motion throughout the blast event, this trial aimed to compare model and prototype results for acceleration, velocity and displacement. In doing so, it would be possible to ascertain whether the elements were subject to an equal acceleration as predicted by the first principle scaling methodology in chapter 4.



(a)



(b)



(c)



(d)

Figure 5.28: Prototype accelerometer setup: (a) Block element; (b) Element mortared into masonry; (c) Completed structure with housing element; (d) Magnified view of embedded housing.

The final aspect of this trial involved an assessment of the benefits of 3D laser scanning which was completed by the industrial sponsor, AWE's 3D Data Capture division. This enabled the generation of a virtual testing environment via the pre and post-trial scanning of the trial environment as depicted by the schematic in figure 5.29. The result of which is an interactive data set which can be utilised to enable a virtual walk-through of the Windrush Arena test site before and after the blast trial was conducted. The use case which represented the greatest potential value to this PhD was the ability analyse masonry blast trial results even if it is not safe to gain physical access to the model and prototype masonry structures due to partial or impending collapse. The need to utilise the data for this purpose did not however arise as it was possible to manually conduct all of the post-trial data quantification at the Windrush Arena test site directly after the trial. Despite this, the generation of a virtual trial environment enables future researchers to virtually revisit the trial site to conduct additional masonry fracture, deflection and debris distribution analyses.

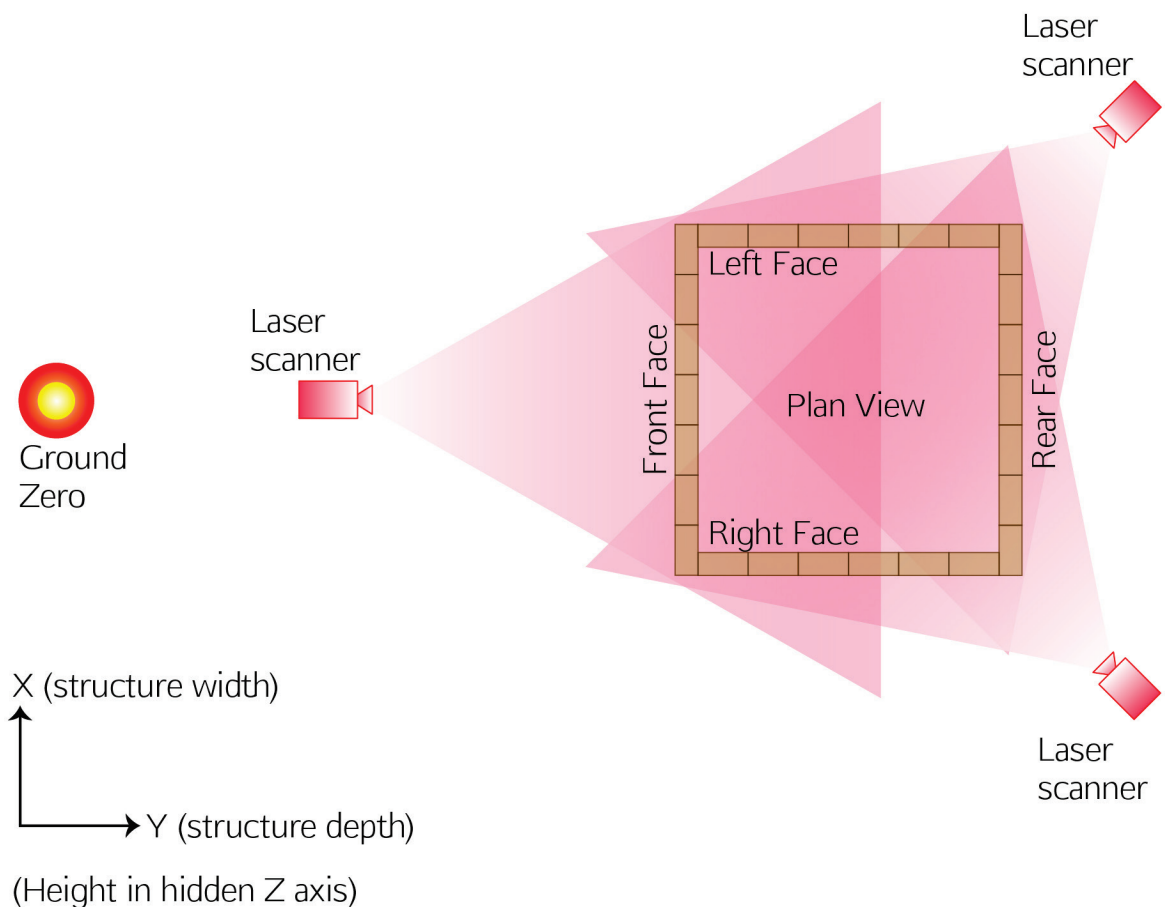


Figure 5.29: Illustrative schematic for 3D laser scanning of trial environment.

Chapter 6

Experimental Results and Scaling Analysis

6.1 Introduction

This chapter details the experimental results of a short-duration blast trial which was conducted in May 2018 via the experimental design shown in chapter 5. The analysis of these results has been divided into multiple sections with the first examining the blast environment generated within the trial via free-field and reflected blast parameters. The second section assesses the recorded levels of masonry damage with an emphasis on the comparative examination of prototype and model results to benchmark the PhD scaling methodology. The third section investigates the debris distributions generated within the blast trial to enable a cross-examination of prototype and model results and further benchmark the scaling methodology. The next section of this chapter summarises accelerometer brick element results alongside 3D laser scanning before a final summary is presented in the final section. This chapter therefore completes the second objective of this PhD and in turn, directly addresses the principal aim of this PhD.

6.2 Blast environment

Whilst this research focus is the scaling of masonry response, it is vital to examine the blast environment to ensure the experimental blast scenario matches the design presented in chapter 5. Utilising high-speed video data it was possible to visually examine the detonation of the 100kg TNT charge deployed within this PhD's blast trial. Figure 6.1 illustrates a time-lapse progression of the detonation process from the initiation of the fireball through to its

eventual diffusion. In particular, fireball symmetry within this figure provides a qualitative indication that explosive material detonation was relatively uniform.

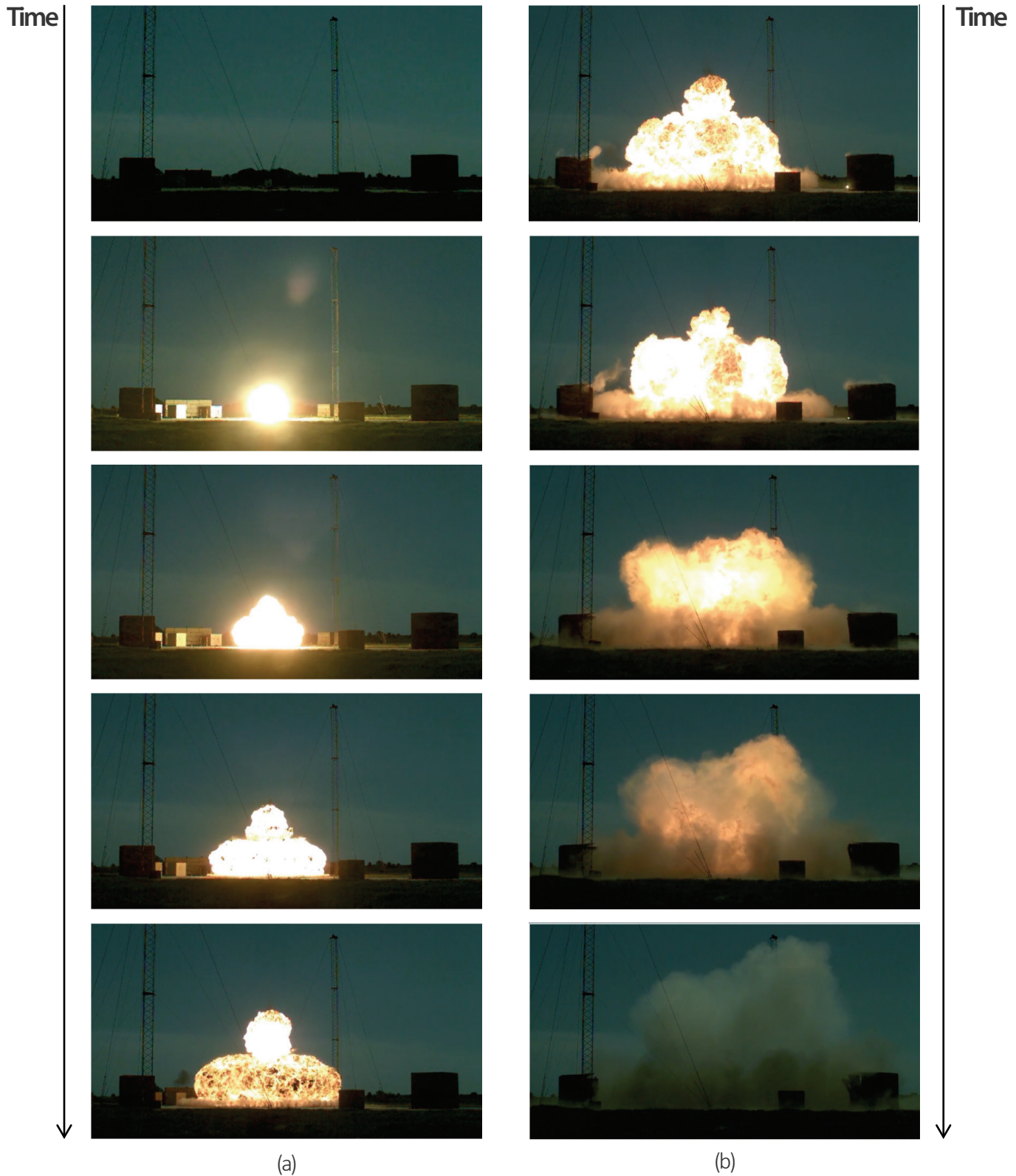


Figure 6.1: Time lapse images of 100kg TNT detonation: (a) Initial detonation and formation of fireball; (b) Fireball expansion and diffusion.

Analysis of the free-field environment was conducted by examining the blast parameters recorded by the pressure gauges deployed at the front, mid and rear stand-off distances for each prototype and model. Figure 6.2 illustrates the free-field pressure-time histories recorded by these pressure gauges. Measured values of free-field pressure, specific impulse and positive phase duration were compared with pre-trial predictions generated via the Kingery and Bulmash predictive polynomials (Kingery and Bulmash, 1984). These predictive expressions were derived from the experimental analysis of spherical burst data to enable calculation of various blast parameters. Table 6.1 details the recorded values for each of the ten free-field gauges utilised within the trial. The original trial design incorporated twelve gauges to provide redundancy for the 14.25m and 20m stand-off distances, however two pressure gauges were non-functional on the day of the trial.

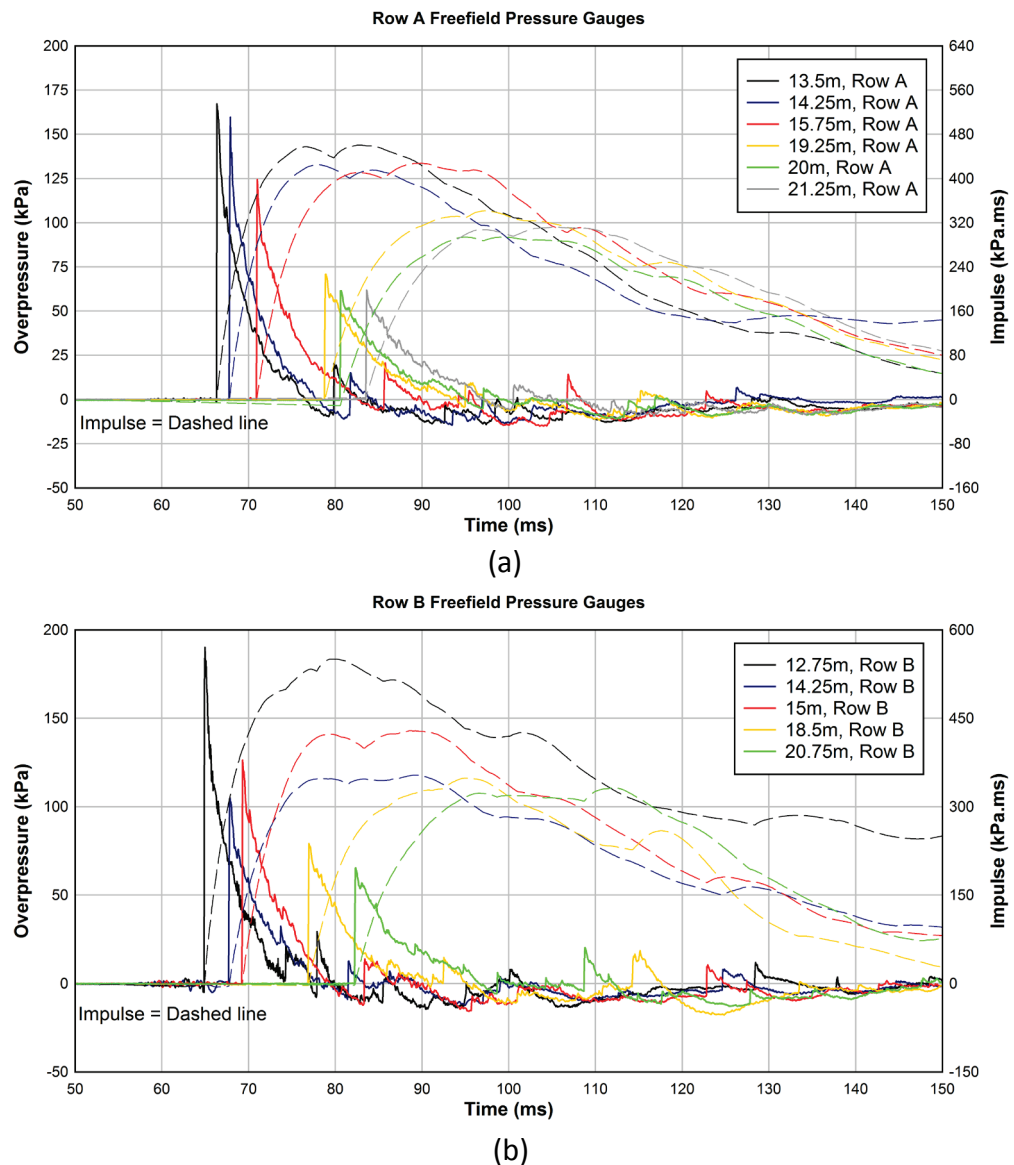


Figure 6.2: Free-field overpressure and impulse results: (a) Row A gauges; (b) Row B gauges.

Table 6.1: Measured free-field blast parameters for short-duration trial.

Stand-off (m)	P_s (kPa)	i_s (kPa-ms)	t^+ (ms)
12.75	190.2	550.4	9.25
13.5	170.8	460.3	10.15
14.25	169.8	424.6	9.9
15	133.4	428.8	11.05
15.75	128.4	427.5	11.55
18.5	80.2	348.6	13.8
19.25	72.6	341.4	15.1
20	71.3	293.8	14.25
20.75	71.8	331.9	14.25
21.25	62.7	311.9	13.4

Table 6.2 compares recorded values with pre-trial predictions where it can be seen that mean peak overpressure was +34% greater than expected with a maximum difference of +54% at the 14.25m stand-off. Mean positive phase duration can also be seen to differ with an average difference of -18%. The combination of increased pressure and reduced pressure duration can be shown to produce a cancellation effect to produce a mean impulse which is +6% larger than predicted. Of the three measured blast parameters, impulse can be shown to be the most critical for this research due to the representation of a masonry structure as an inertial resistance to the momentum of the blast wave, which is an equality for impulse. A mean difference of +6% therefore represents an acceptable level of variation when implementing a $\pm 10\%$ acceptability criteria.

Table 6.2: Measured free-field parameters versus 100kg spherical charge predictions.

Stand-off (m)	P_s	i_s	t^+
12.75	+36%	+18%	-23%
13.5	+38%	+4%	-21%
14.25	+54%	+1%	-26%
15	+35%	+6%	-21%
15.75	+43%	+11%	-20%
18.5	+23%	+4%	-13%
19.25	+20%	+5%	-7%
20	+26%	-6%	-14%
20.75	+36%	+9%	-15%
21.25	+24%	+5%	-21%
Average	+34%	+6%	-18%

Despite the relative agreement demonstrated for impulse, it is evident that the experimental blast environment diverged from the trial design. These differences can be traced to the utilisation of a physically cylindrical charge as shown in figure 6.3. An investigation of the literature relating to physical charge shape demonstrates that cylinders with aspect ratios greater than 1:1 can result in blast wave asymmetry in the axial direction in tandem with varying degrees of blast amplification in the radial direction (Simoens and Lefebvre, 2015). The cylindrical charge utilised by this trial had an aspect ratio 1:1.6 to produce moderate charge shape effects, the result of which is evident in table 6.2.



Figure 6.3: Cylindrical charge shape utilised within short-duration blast trial.

In terms of blast wave asymmetry and its potential influence on masonry structural loading, the generation of an AUTODYN CFD model enabled a visual assessment of blast wave formation for a 100kg TNT charge with a 1:1.6 cylindrical charge shape. Figure 6.4 provides a time-lapse view of this CFD model where it can be seen that the effect of asymmetry is most pronounced in the vertical axial direction over 0-15m, after which the wave resembles a typical hemispherical blast. In contrast, the radial propagation of the blast flow towards the structures resembles a hemispherical blast for the duration of the event, indicating that the potential effect of asymmetry on masonry loading was negligible in this trial. It is therefore clear that the dominant effect of the cylindrical charge shape was the amplification of peak overpressure and the reduction of positive phase duration as shown in table 6.1. The net result of which is cancellation effect to produce a mean difference of +6% for impulse over the range of stand-off distances.

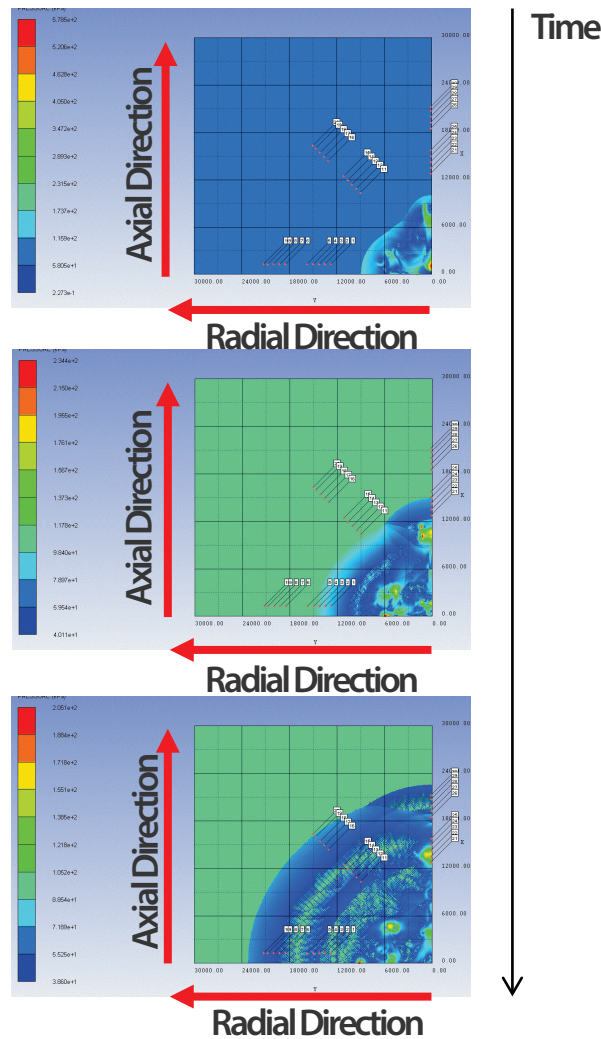


Figure 6.4: CFD simulation of blast wave formation for 100kg TNT charge with 1:1.6 cylinder.

In an attempt to quantify the effect of the cylindrical charge, a trial and error study was conducted to determine whether increased TNT mass would produce similar blast parameters to those measured in the trial. A range of parameters were investigated via the same predictive polynomial expressions used to generate the design blast parameters. As a result, it can be shown in table 6.3 that a 134kg TNT charge with a physically spherical shape represents a better level of agreement with experimental pressure results as shown by mean difference of +10% compared to +34% in table 6.2. This charge mass also produces a mean pressure duration difference of -20%, representing a similar result to 100kg TNT in table 6.2. The combination of pressure and positive phase duration produces a mean impulse difference of -12% compared with +6% for the 100kg TNT comparison. As a whole, the data set for 134kg of spherical formed TNT can be shown to more closely replicate the measured trial environment. This is however counterbalanced by a mean impulse value which exceeds the $\pm 10\%$ acceptability criteria by -2%.

Table 6.3: Measured free-field parameters versus 134kg spherical charge predictions.

Stand-off (m)	P_s	i_s	t^+
12.75	+10%	-2%	-21%
13.5	+12%	-14%	-19%
14.25	+25%	-16%	-27%
15	+10%	-11%	-22%
15.75	+17%	-8%	-22%
18.5	+2%	-14%	-17%
19.25	-1%	-12%	-11%
20	+5%	-22%	-18%
20.75	+14%	-9%	-19%
21.25	+4%	-13%	-25%
Average	+10%	-12%	-20%

This trial also fielded two non-responding timber structures to measure blast wave interaction with geometries that replicate the model masonry structures, the results of which were intended to benchmark CFD models in chapter 7. The performance of these trial elements was variable with the structure at 14.25m incurring significant damage to the frontal face, resulting in the ejection of pressure gauges from their apertures as shown in figure 6.5a-b. The structure located at 20m survived the application of blast loading with no visible damage. In spite of this, one of the front panel pressure gauges was forced out of the aperture as shown in figure 6.6a-b.



Figure 6.5: Post-trial images of timber structure, 14.25m: (a) Frontal damage; (b) Magnified view of damage.

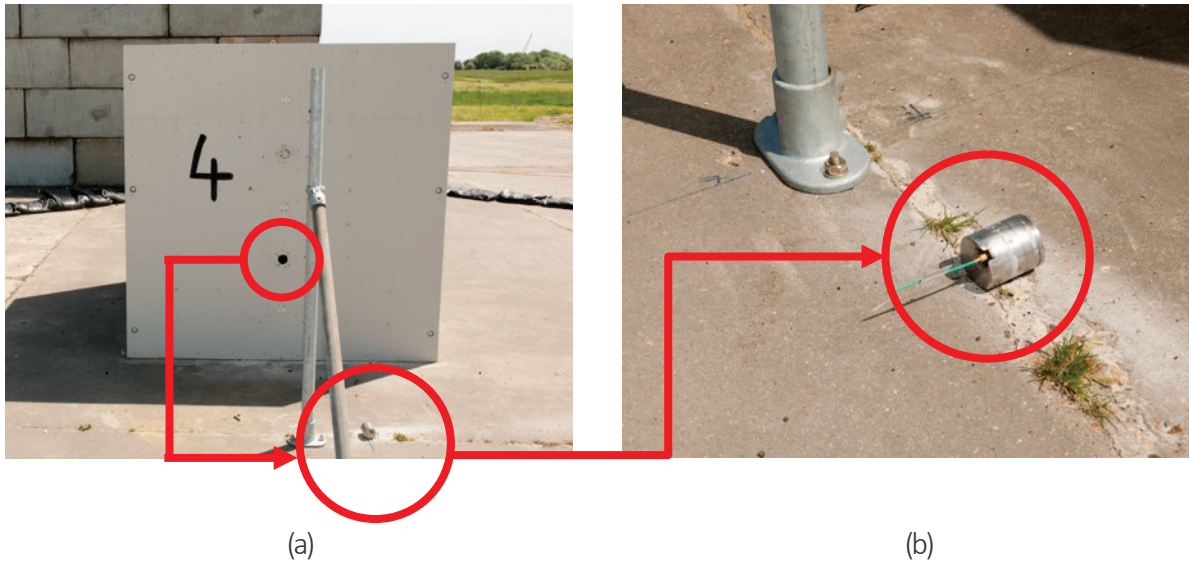


Figure 6.6: Post-trial images of timber structure, 20m: (a) Ejected pressure gauge ; (b) Magnified view of pressure gauge.

Focussing initially on the side-on pressure gauges located at 14.25m and 20m as shown in figure 6.7, it was possible to perform a comparison with measured free-field values. By definition, side-on pressure should be an equality for free-field pressure at the same stand-off, yet it can be seen in table 6.4 that the 14.25m value for side-on pressure was -32% lower than the free-field with the 20m pressure value being -11% lower than the free-field value. The side-on values for positive phase duration can be seen to exceed the free-field by +18% at 14.25m and +17% at 20m. The net effect of these side-on parameter pairs are impulse differences of -25% at 14.25m and +7% at 20m. The level of variance identified within the side-on data indicates potential circumferential asymmetry within the blast wave alongside the potential influence of structural damage at 14.25m. This research is not however able to assess blast asymmetry due to two gauge malfunctions which prevented the deployment of multiple pressure gauges at 14.25m and 20m stand-off distances. Of the two data sets, the wider range of stand-off distances monitored in the free-field enabled an overall trend assessment of blast parameter values with increasing stand-off in table 6.1. As a result, it can be seen that the steady reduction in pressure and impulse in the free-field resembles typical radial blast wave behaviour. In contrast, the level of variability exhibited for side-on data does not represent typical radial blast behaviour, demonstrating a greater level of uncertainty which limits the viability of this data for CFD benchmarking in chapter 7.

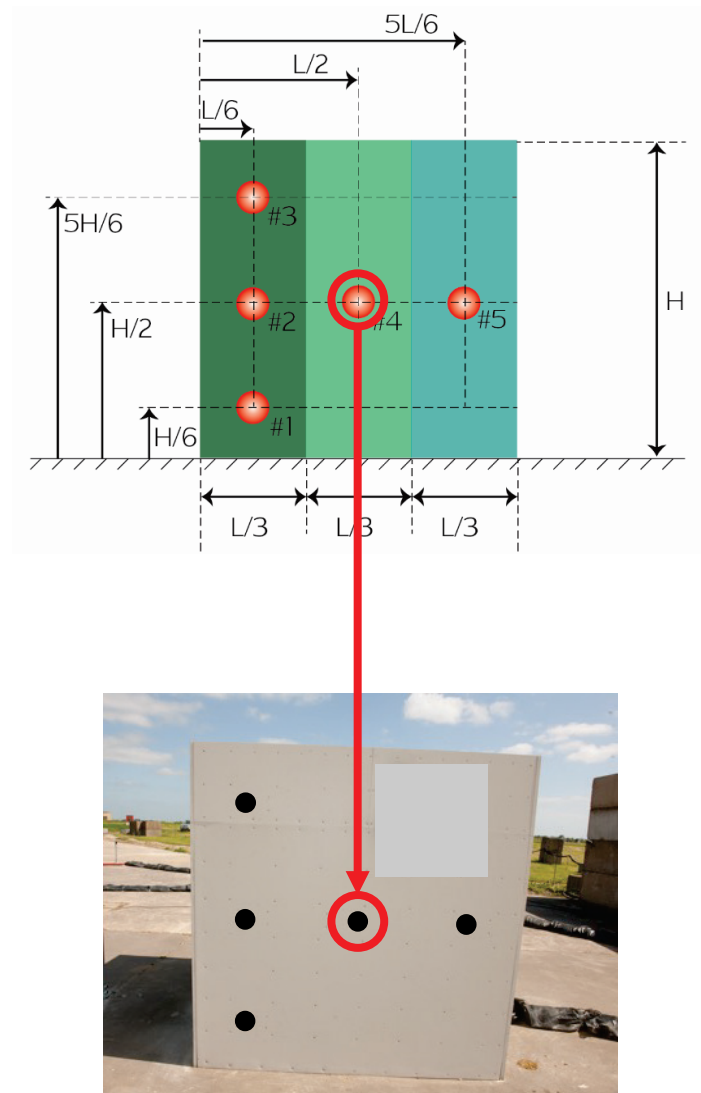


Figure 6.7: Side-on pressure gauge for non-responding structure.

Table 6.4: Measured side-on blast parameters versus measured free-field parameters.

Stand-off (m)	P_s	i_s	t^+
14.25	-32%	-25%	+18%
20	-11%	+7%	+17%

An examination of reflected pressure data was constrained by the ejection of gauges from the front faces of both timber structures. It was however possible to measure peak reflected at the central gauge position for each stand-off as shown in table 6.5. Note that the stand-off distances in table 6.5 represent the front structure distance rather than the mid structure distance referenced in table 6.4. The ability to compare measured values for reflected pressure for a finite structural geometry to predictions typically requires a CFD solution to model the effect of blast wave clearing. Chapter 7 will examine reflected blast

pressures in more detail where it will be shown that the values measured within this trial can be closely matched by CFD solutions for a 134kg TNT equivalent blast scenario to integrate the effect of blast amplification due to the cylindrical charge shape.

Table 6.5: Measured reflected blast parameters.

Stand-off (m)	P_r	I_r	t^+
13.5	481.3	985	6
19.25	184.07	469	7.55

6.3 Scaling of masonry damage

As detailed in chapter 3, masonry damage can be qualitatively characterised via cracking and degrees of potential structural collapse. This section will assess the ability of this PhD's scaling methodology to accurately replicate masonry damage at 14.25m and 20m stand-off distances for 100kg TNT. This will be accomplished by examining the two pairs of prototype and model structures individually, beginning with the 14.25m stand-off distance and followed by the 20m stand-off distance.

6.3.1 14.25m stand-off

Figures 6.8a-b provide a time-lapse illustration of prototype and model structure response at 14.25m stand-off obtained via high-speed video captured from a front-side position as detailed in chapter 5. Due to the cross-site perspective captured at these positions, the evolution of the explosive fireball leads to the obstruction and over-exposure of these high-speed video files before the prototype and model structures develop their full response to blast loading. As a result, it was not possible to determine frontal cracking or potential collapse modes from these data files.

In contrast, figures 6.9-10 provide time-lapse accounts of prototype and model response with a rear-side camera perspective that was not obscured by the explosive fireball. Figure 6.9a illustrates blast wave arrival and the completion of blast clearing at the rear as shown by the blast vortices. Further inspection of this figure also shows that the roof enclosure was subject to substantial compressive forces, resulting in permanent deflection of the steel cross-support frame. Finally, this figure demonstrates the onset of side panel

cracking which precedes the sequential process of deflection and partial panel collapse shown in figure 6.9b. Examination of figure 6.9b indicates partial collapse of the side panel versus survival of the rear panel. It can also be seen that a section of the side panel remains attached to the 'built in' corner with the rear panel. Whilst it is not possible to observe the cracking and collapse mode, it can also be inferred from figure 6.9b that the prototype's front panel was subject to sufficient blast impulse to produce collapse. The combination of side panel and front panel collapse finally results in the generation of a debris distribution.

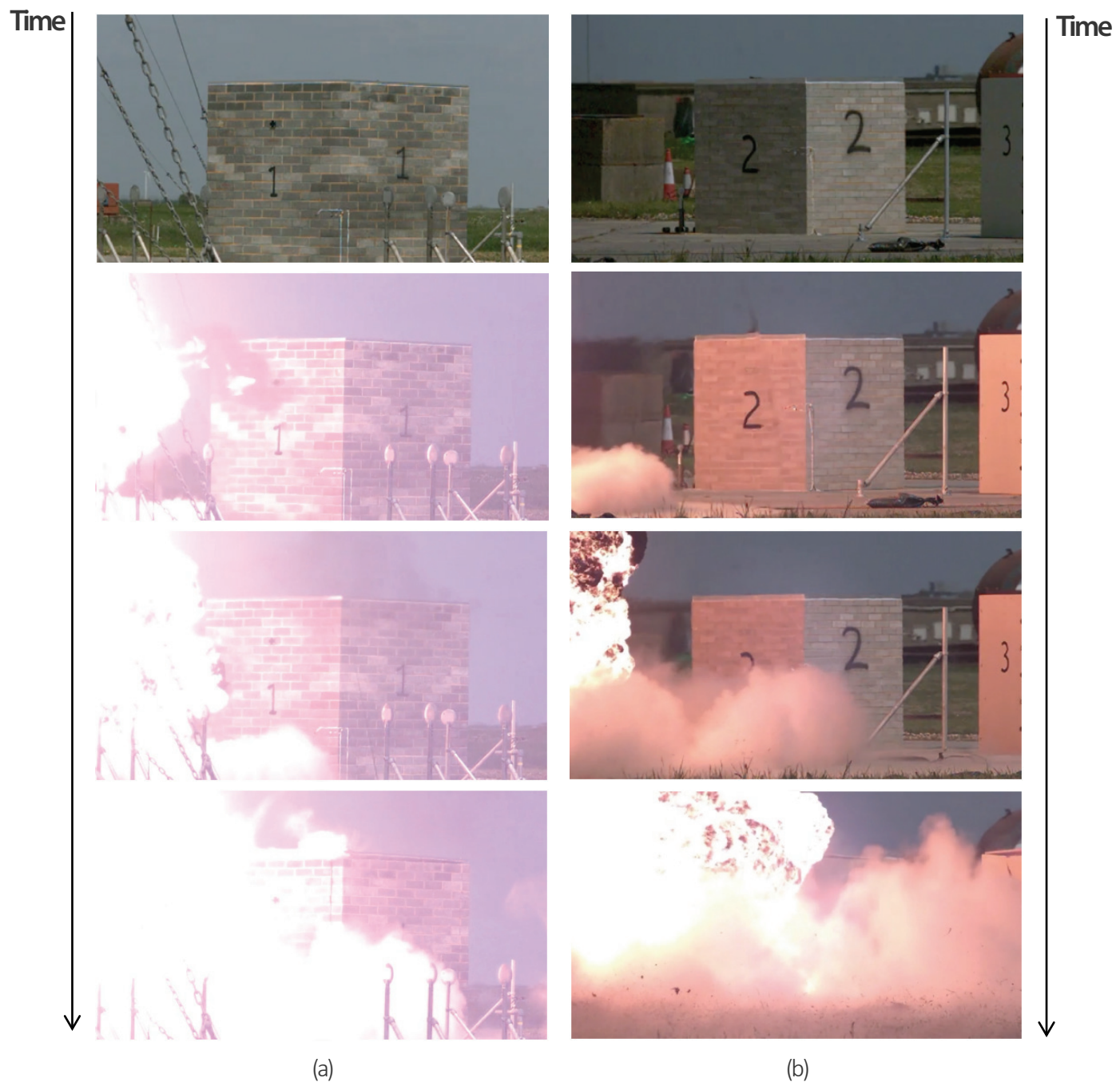


Figure 6.8: Post-trial time-lapse images of masonry response at 14.25m, front-side perspective: (a) Prototype; (b) Model.

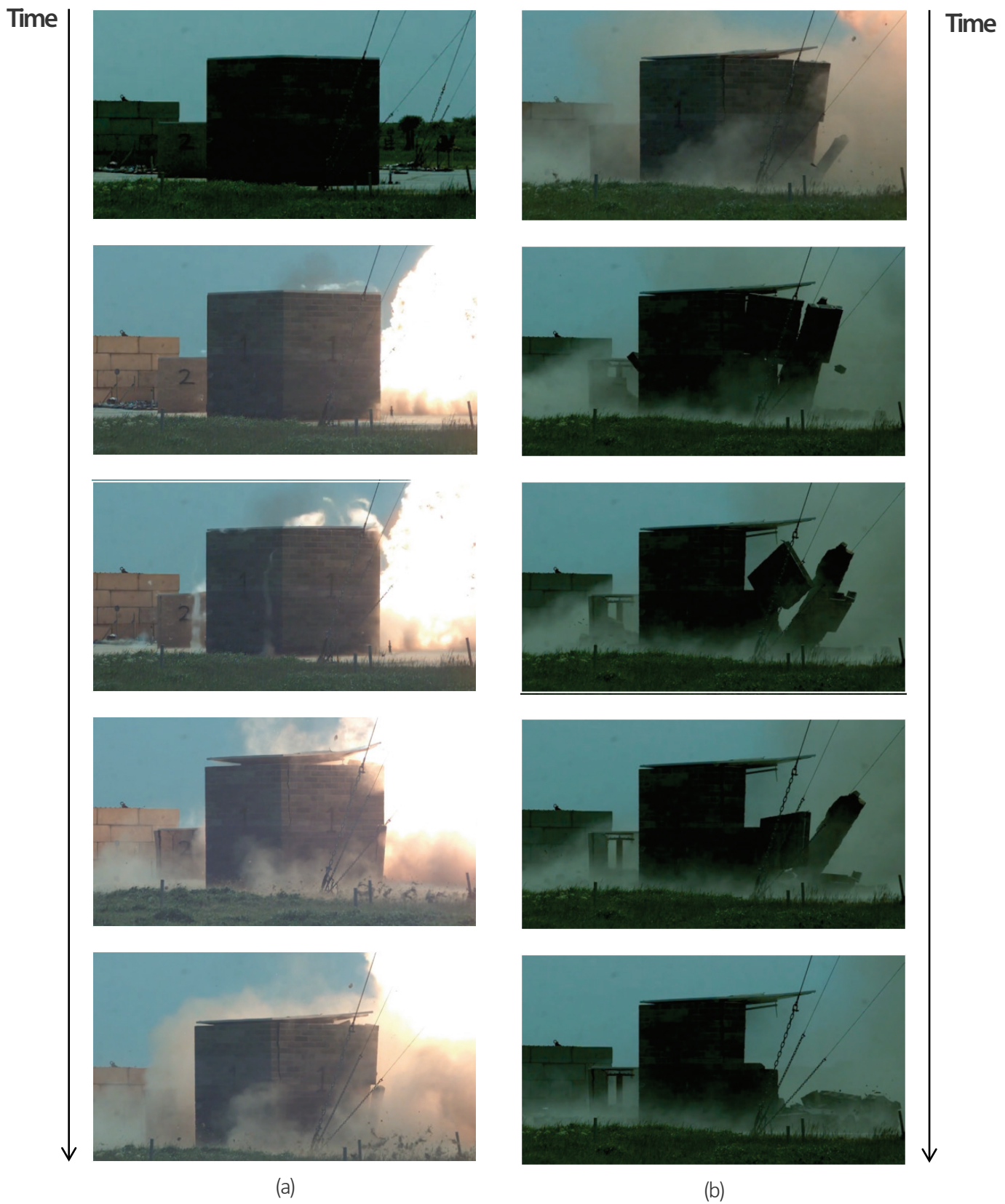


Figure 6.9: Post-trial time-lapse images of prototype response at 14.25m, rear-side perspective:
(a) Blast arrival and crack initiation; (b) Partial collapse.

Figure 6.10a shows the process of blast arrival and blast clearing at the model structure via the generation of blast vortices which are also visible for the non-responding structure at the same stand-off. Examination of this figure also shows that the roof enclosure withstood the application of compressive forces to a better degree than the prototype counterpart, resulting in less permanent deflection of the steel support frame. Finally, figure 6.10a illustrates the process of side panel cracking at both of its 'built in' corners in tandem with a vertical crack near to the panel centre. Figure 6.10b shows the sequential stages of side panel deflection and partial panel collapse alongside the development of a vertical crack at the centre of the rear panel. It can also be seen that the rear panel in tandem with an adjacent section of the side panel do not collapse. As seen in figure 6.9b, it is not possible to observe the cracking and collapse mode of the front panel, but it can be inferred from figure 6.10b that the model's front panel collapsed, contributing to the debris distribution.

Figure 6.11 provides a comparative view of the final damage incurred by the prototype and model structures at 14.25m. For the purposes of this research, full-thickness masonry cracks will be referred to as primary cracks to represent an ultimate limit state (ULS) whilst superficial cracking will be referred to as secondary cracks to represent a serviceability limit state (SLS). Examination of figures 6.11a and 6.11b shows that the front panel fully collapsed for both the prototype and model structures. Figure 6.11a shows that the prototype rear panel develops a primary crack down the full height of the panel at the central position. Figure 6.11b demonstrates that the rear panel of the model structure also develops a primary vertical crack at the centre of the panel before developing a branch towards the base of the panel. These differences in crack shape for the prototype and model structure can be linked to the discussion of panel wall crack patterns in chapter 3 where it was shown that edge fixity can influence the cracking of laterally loaded panel walls. As a result, it appears likely that the prototype structure was built with fully restrained corners and limited fixity at the interface with the ground surface. In contrast, the model structure appears to have utilised the same fully restrained corners in tandem with greater fixity at the ground surface, resulting in a divergence in crack shape. This divergence in construction indicates that similar variations may also have been present for the front and side panel walls of the masonry structures. The influence of which was examined in chapter 3 where it was shown that edge restraint can greatly affect each of the sequential phases of masonry response.

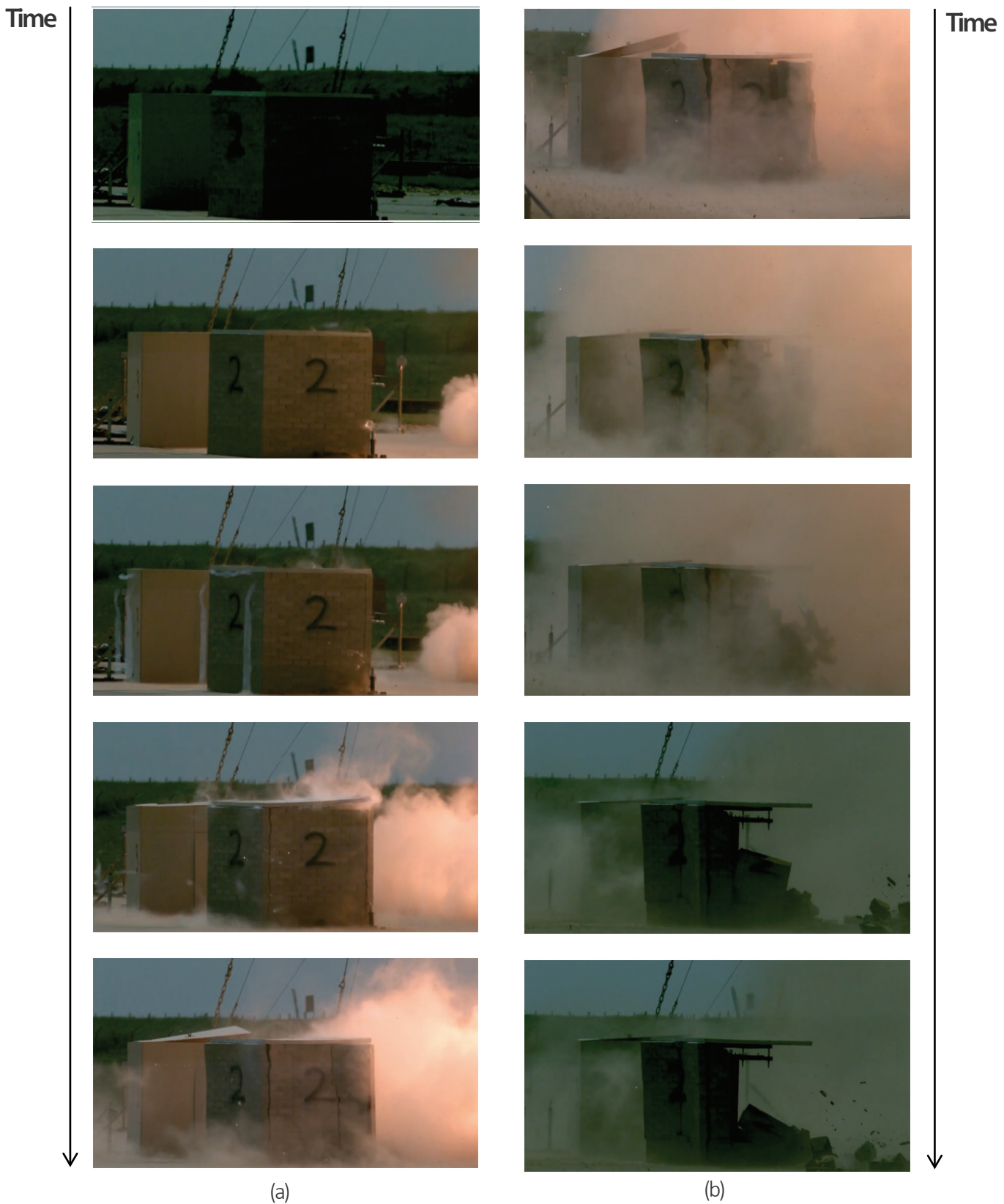
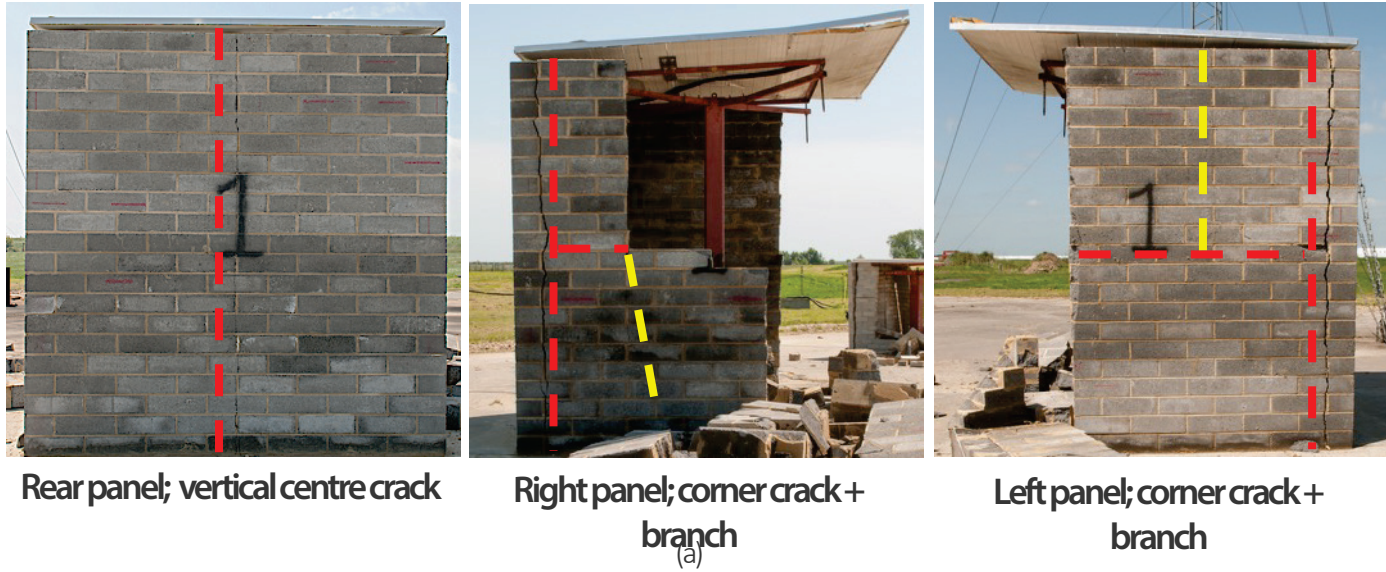


Figure 6.10: Post-trial time-lapse images of model response at 14.25m, rear-side perspective: (a) Blast arrival and crack initiation; (b) Partial collapse.

Prototype: 5x Primary Cracks & 2x Secondary cracks



Model: 5x Primary Cracks & 2x Secondary cracks

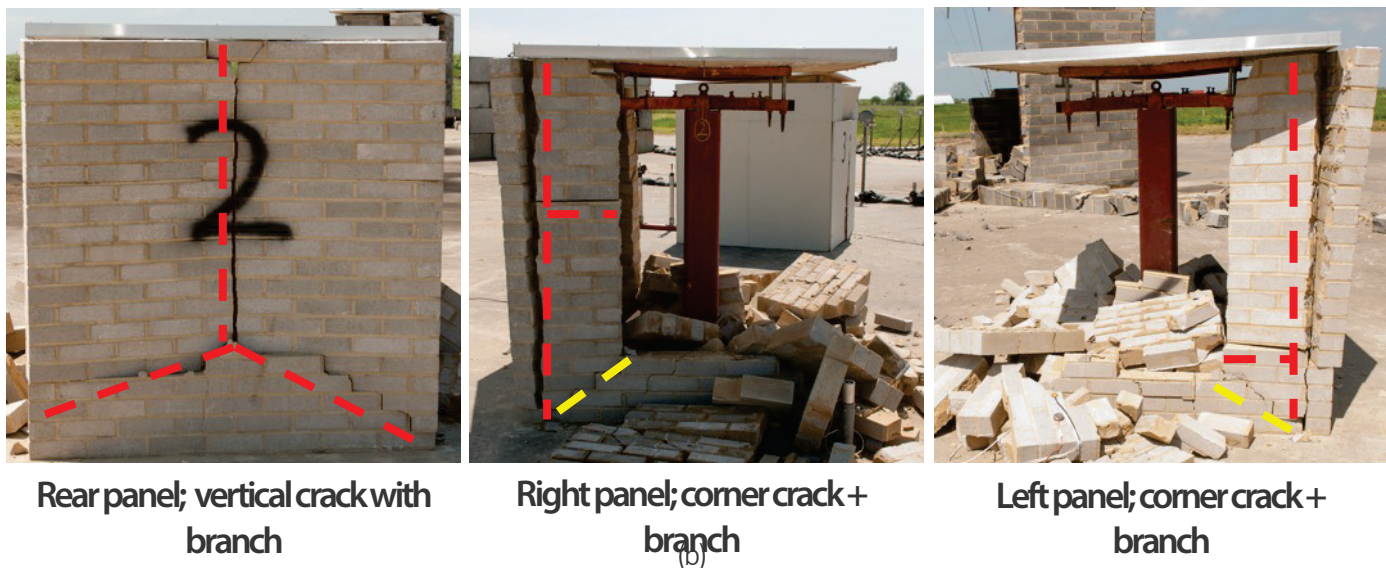


Figure 6.11: Post trial images of masonry damage at 14.25m: (a) Prototype; (b) Model.

Figure 6.11a shows that the prototype structure's right panel partially collapsed whilst the in-tact masonry developed a primary vertical crack with a horizontal branch at the fully restrained corner. This section of masonry also developed a secondary crack with an oblique path from the horizontal primary branch to the base of the structure. Figure 6.11b demonstrates that the model structure's right panel collapsed to a greater degree than the prototype. This panel also developed a primary vertical crack with a horizontal branch at the 'built in' corner. It can also be seen that the right panel developed a secondary crack with a differing oblique path to the prototype. Further inspection of figure 6.11a shows that the left

panel partially collapsed to a lesser degree than the right panel. This panel also developed a primary vertical crack with a horizontal branch at the full restrained corner, representing a similar damage state to the right panel. It can also be seen that the left panel developed a secondary crack with a vertical orientation at the centre line of the remaining panel. Figure 6.11b shows that the left panel of the model structure partially collapsed to a similar extent as the right panel. It can also be seen that the left panel developed a primary vertical crack with a horizontal branch at the corner, representing a similar overall damage state to the right panel. Figure 6.11b also shows that the left panel developed a secondary crack with an oblique path which matches the right panel.

The assessment of masonry damage for the prototype and model structures at 14.25m has demonstrated a number of important similarities for the defining elements of damage state. Firstly, both the prototype and model structures were subject to full front panel collapse and partial collapse of both the side panels. This indicates that the model was able to qualitatively replicate the potential collapse mode of the prototype structure. Secondly, both the prototype and model structure developed a total of five primary cracks and two secondary cracks. In particular, both the prototype and model structures developed a vertical primary crack at the rear panel with no actual collapse. Similarly, both the prototype and model structures developed primary vertical cracks within the side panels at the fully restrained corner interface with the rear panel. These primary cracks were also found to develop horizontal branching for both the prototype and model structures. Lastly, both the prototype and model structures developed one secondary crack with varying paths at each of the side panels. As a collective, these cracking behaviours demonstrate that the model structure was able to replicate the quantity and location of the prototype's primary cracks alongside the quantity of secondary cracks.

The areas of divergence between prototype and model damage state results were found to be the variance in rear panel crack shape and the extent of side panel collapse. Specifically, the former can be linked to variable construction of the ground surface mortar bed for the prototype and model structures, indicating that an equivalent level of fixity would likely remove this difference. Variable levels of side panel collapse may also have been due to a lower degree of edge fixity for the prototype structure, the result of which was a slightly conservative model collapse result versus the prototype. It can therefore be shown that the

model structure was able to accurately replicate the fundamental damage response characteristics of the prototype structure at 14.25m with 100kg TNT with a slightly conservative collapse prediction for the side panels.

6.3.2 20m stand-off

Representing a larger stand-off distance, masonry response at 20m was expected to represent partial cracking and minimal collapse when compared with the 14.25m response results. Figures 6.12a-b demonstrate a time-lapse of prototype and model masonry structure response at 20m stand-off. These images were captured from high-speed video data for the front and side panel walls. As discussed in the 14.25m stand-off results, these camera positions were partially obstructed by the evolution of the explosive fireball but in this instance, it was possible to partially determine front panel cracking modes. Closer inspection of figure 6.12a shows that the prototype's front panel developed a central vertical crack in conjunction with two vertical cracks at the fully restrained corners. It can also be seen that the prototype roof enclosure was subject to significant compressive forces, resulting in the permanent deflection of the steel support frame and the polyurethane roof panels. Examination of figure 6.12b shows that the model structure also developed a central vertical crack in the front panel in conjunction with vertical cracks at the built-in corners.

Figures 6.13-14 provide time-lapse accounts of prototype and model response with a rear-side camera perspective that was not obscured by the explosive fireball. Figure 6.13a illustrates blast wave arrival as indicated by the illuminated light source. This was the only instance that the light source provided a visual marker within high-speed video data as the other implementations were obscured by the fireball or they did not activate. Figure 6.13a also shows the completion of blast clearing at the rear of the structure as indicated by the blast vortices. Finally, this figure demonstrates the onset of side panel cracking which precedes the sequential process of partial panel collapse shown in figure 6.13b. Examination of figure 6.13b indicates partial collapse of the side panel versus survival of the rear panel. It can also be inferred from figure 6.13b that the prototype's front panel did not collapse, indicating that debris distribution was generated solely by side panel collapse. Figure 6.14a shows the process of blast arrival at the model structure. Examination of this figure also shows that the roof enclosure was subject to minimal deflection versus the prototype. Figure

6.14b illustrates the onset of vertical side panel cracks and the complete survival of each panel. As a result, there was no debris generated for the model structure.

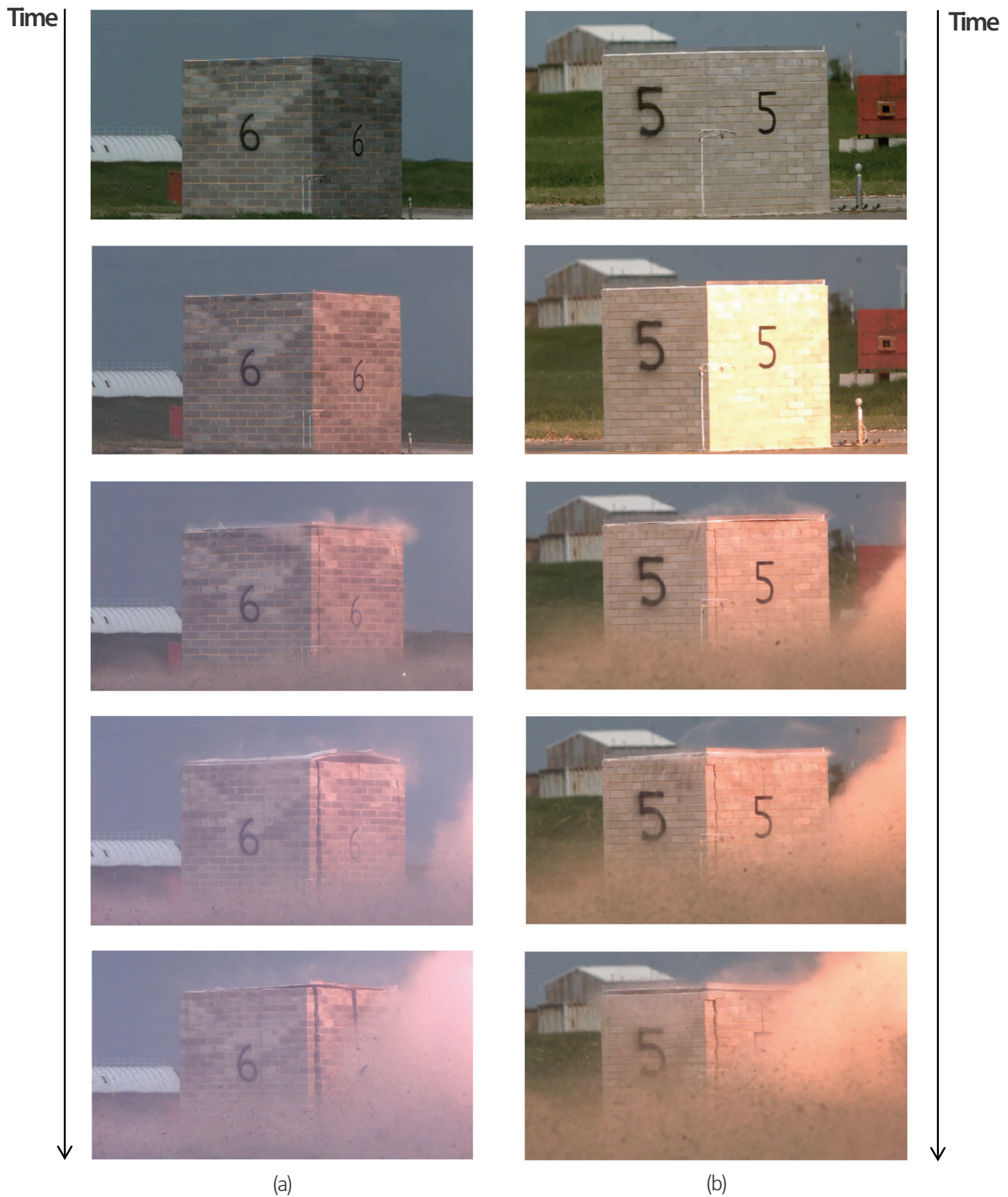


Figure 6.12: Post-trial time-lapse images of masonry response at 20m, front-side perspective: (a) Prototype; (b) Model.

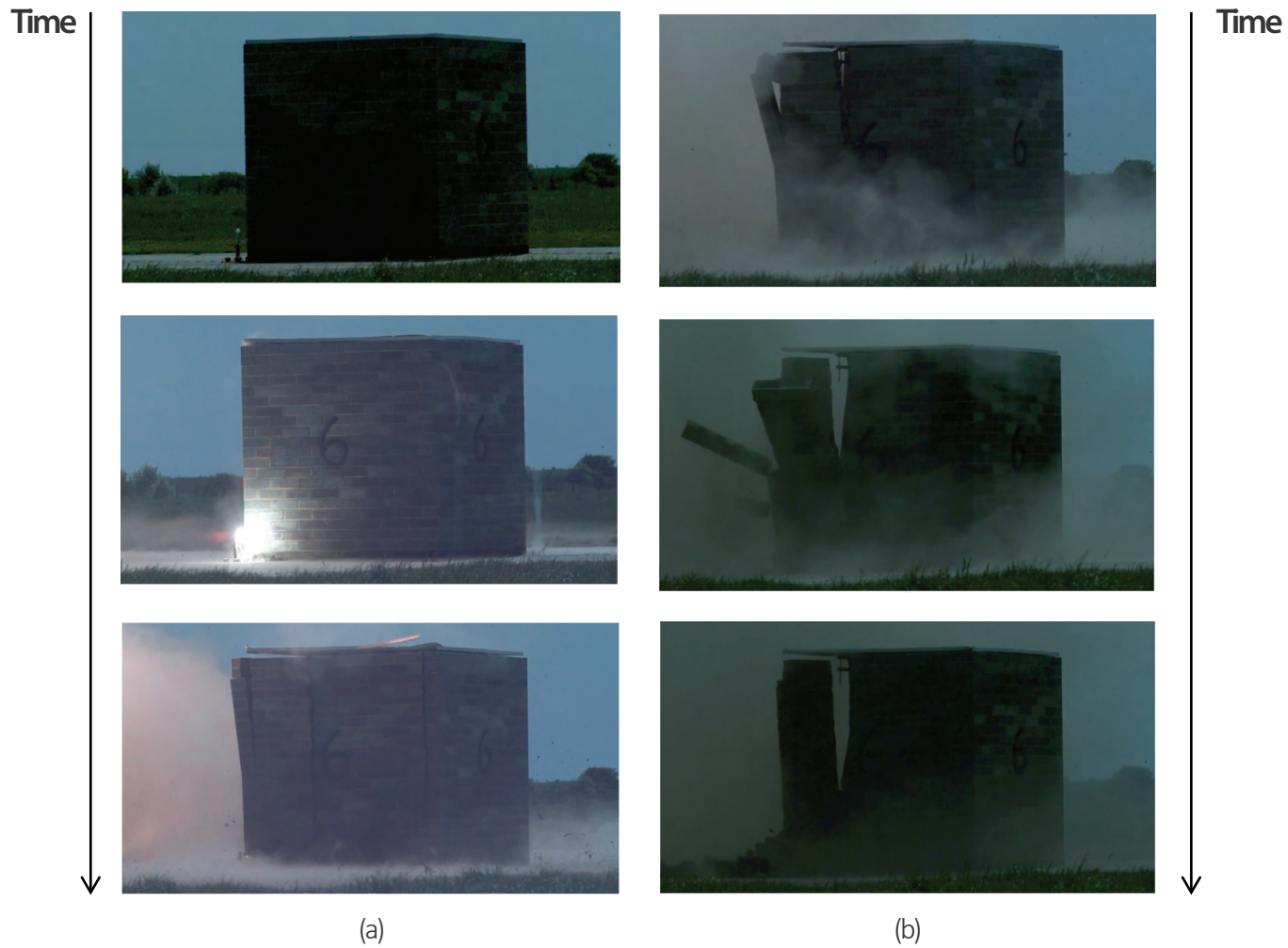


Figure 6.13: Post-trial time-lapse images of prototype response at 20m, rear-side perspective: (a) Blast arrival and crack initiation; (b) Partial collapse.

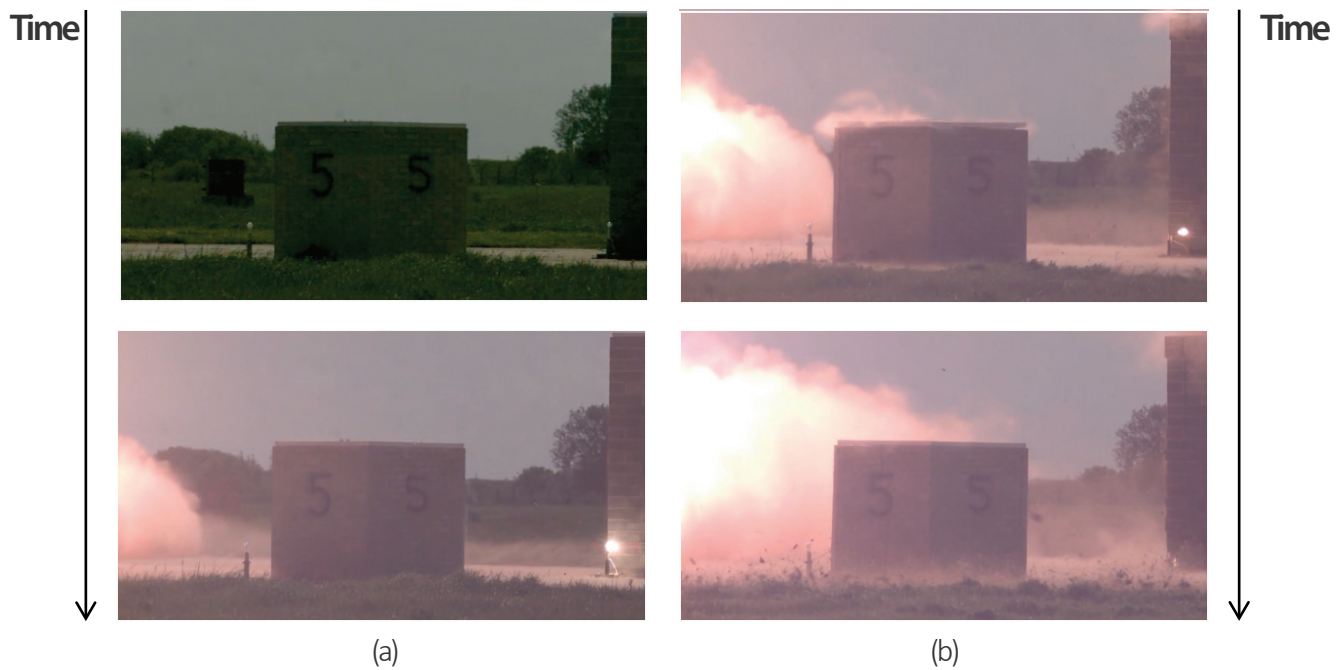


Figure 6.14: Post-trial time-lapse images of model response at 20m, rear-side perspective: (a) Blast arrival; (b) Side panel crack initiation.

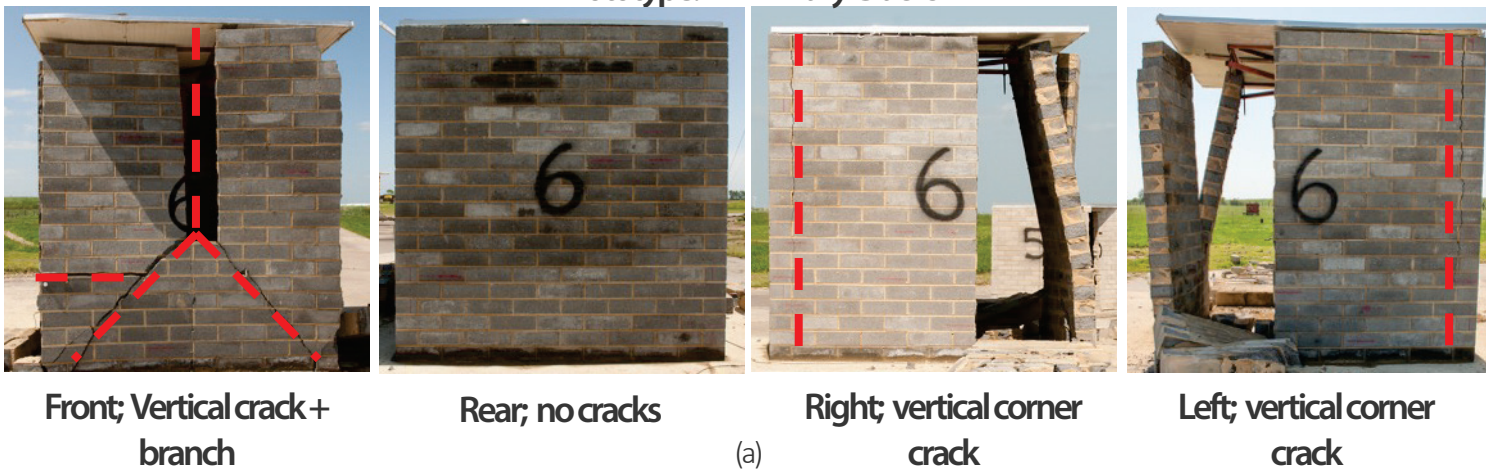
Figure 6.15 provides a comparative view of the final damage state of the prototype and model structures at 20m. This comparison will also refer to full-thickness cracks as primary cracks to represent an ultimate limit state (ULS). Examination of figures 6.15a and 6.15b shows that the front panel deflected but did not collapse for both the prototype and model structures. Figure 6.15a shows that the prototype front panel developed a primary crack down the full height of the panel centre before developing a branch towards the base of the panel to simulate an inverted Y shape. This primary crack is also linked to a smaller primary crack with a horizontal orientation. Figure 6.15b illustrates that the model structure's front panel also developed a primary vertical crack at the central position which then branches towards the base to form an inverted Y shape. This figure also shows that the main primary crack is linked to a smaller primary crack with a vertical orientation. Both of the inverted Y shape crack modes seen in figure 6.15 replicate the rear panel crack mode for the model structure at 14.25m. Thereby indicating that the prototype and model structures at 20m possessed equal levels of edge to the model at 14.25m via 'built in' corners and full fixity at the ground interface.

Further examination of front panel wall response in figure 6.15a reveals that the prototype front panel deflected inwards to a greater extent than the model structure before being halted by the roof enclosure frame. The prototype roof structure was also found to incur a far greater degree of permanent deflection than the model counterpart. The result of which was an upward tilt which prevented frictional contact with the side panel walls, thereby permitting partial collapse. Inspection of figure 6.15b shows that the model structure's front panel wall deflection was also halted by contact with the roof enclosure, indicating a similar response to the prototype. Despite this, closer visual inspection of the model roof enclosure revealed that the polyurethane panel had also 'pinned' the side masonry panels in place, preventing additional outward deflection and the subsequent side panel collapse observed with the prototype.

Closer inspection of figure 6.15a shows that the prototype rear panel did not develop any cracks whilst figure 6.15b demonstrates that the model structure's rear panel developed two primary vertical cracks at the fully restrained corner locations. Figure 6.15a shows that the prototype's right and left panel walls partially collapsed whilst the in-tact masonry panels developed primary vertical cracks at the fully restrained corners. Figure 6.15b shows that the

model structure's right and left side panels developed vertical primary cracks near the centre of the panels, representing similar locations to the cracks which preceded partial collapse of the prototype structure's right and left panels. By grouping primary corner cracks with primary side panel crack locations, it can be shown that the model replicated the prototype's onset of cracking at the restrained corners, albeit with a variance in corner edge location. Similarly, the location of side panel primary cracks can be shown to replicate the prototype up until the instant that the prototype panels collapsed. As discussed above, there is a high likelihood that the model's side panel walls would have also collapsed if the roof enclosure had not produced a frictional interlocking to prevent additional deflections.

Prototype: 4x Primary Cracks



Model: 6x Primary Cracks

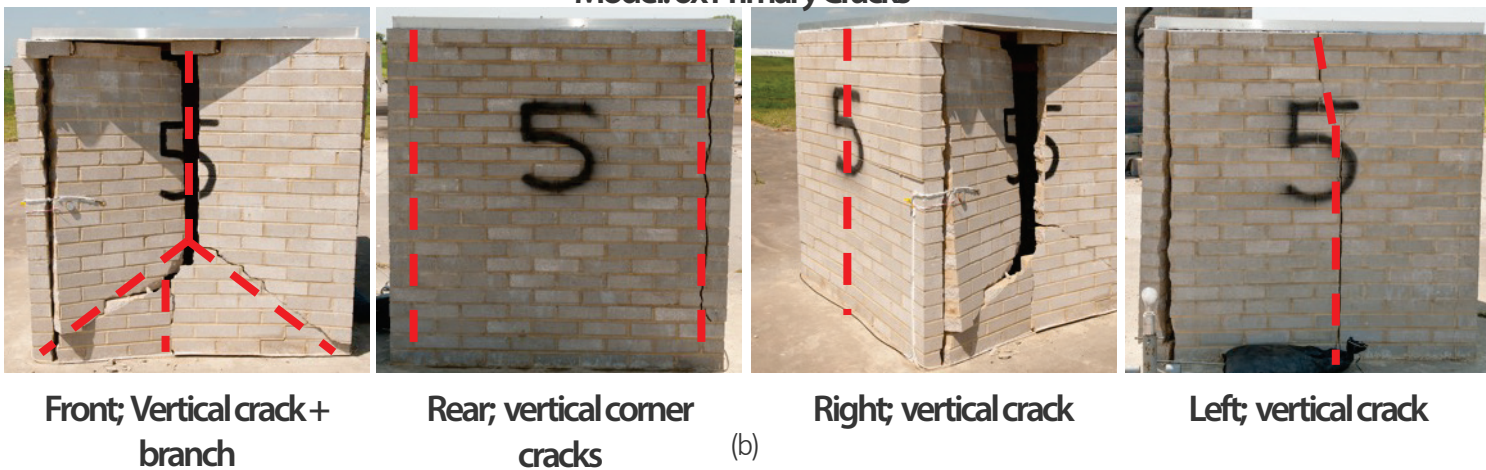


Figure 6.15: Post trial images of masonry damage at 20m: (a) Prototype; (b) Model.

The assessment of masonry damage for the prototype and model structures at 20m has demonstrated a number of important similarities and differences for the defining elements of damage state. Firstly, both the prototype and model structures were subject to

front panel deflection without the onset of collapse due to roof enclosure interactions. In contrast, the prototype structure incurred partial side panel collapse whilst the model structure was likely prevented from replicating this behaviour by frictional forces exerted by the roof enclosure. This indicates that the model was able to qualitatively replicate the frontal deflection mode of the prototype structure with the potential for side panel collapse if the roof enclosure design could be modified to more closely replicate the prototype's response. Alternatively, if the prototype roof enclosure could be strengthened to replicate the model enclosure, the side panel collapse observed for the prototype may also be prevented to demonstrate an alternative but equal level of collapse correlation with the model. The modification of roof enclosure stiffness represents an area of future work if additional trials were design to assess scaled masonry response without the effect of a load transmitting roof structure.

Secondly, both the prototype and model structure developed the same primary cracking behaviour at the front panel wall, demonstrating the ability to replicate the cracking mode with equivalent levels of edge restraint. This further demonstrates that the differing rear panel crack modes identified for the 14.25m stand-off were most likely due to varying ground edge fixity. The quantity of primary cracks were found to differ with four observed for the prototype and six for the model. This can also be linked to the influence of the roof enclosure on potential side panel collapse and as such, if this behaviour was modelled, the quantity of primary cracks in the model would likely have matched the four for the prototype. The location of side and rear panel primary cracks within prototype and model results were found to differ in absolute location but represent the same exertion of cracking stress at the 'built in' corners of the side to rear panels. As a result, it can be shown that both instances represent the same level of damage and the subsequent exceedance of the ultimate limit state at the rear corner locations. Thus indicating the need to rebuild the structure due to masonry unit splitting and the subsequent generation of an unsupported panel wall. As a collective, the cracking behaviours demonstrate that the model structure was able to potentially replicate both the quantity and structural location of primary cracks of the prototype structure.

It can therefore be shown that the model structure was potentially able to replicate the fundamental damage response characteristics of the prototype structure at 20m with

100kg TNT albeit with an underestimation of partial collapse prediction for the side panel walls due to experimental differences in roof enclosure response to blast loading.

6.4 Scaling of debris distributions

This section will assess the ability of this PhD's scaling methodology to accurately represent masonry debris distributions at a 14.25m stand-off distance for 100kg TNT by cross-examining prototype and model structure debris data from the blast trial. As a result of the different side panel damage levels for the prototype and model structures at 20m stand-off, it was not possible to assess debris spread for this scaled pair of structures.

6.4.1 14.25m stand-off

As discussed in section 6.3.1, both the prototype and model structures at 14.25m stand-off were subject to similar levels of partial collapse of the right and left panels in tandem with the complete collapse of the front panels. Figure 6.16a illustrates the resultant debris distribution for the prototype structure where it can be qualitatively observed that the masonry rubble occupies the internal plan area of the structure in conjunction with the zones to the left, right and front of the structure. The survival of the rear panel resulted in no debris behind the structure's original location. Figure 6.16b demonstrates the model structure debris distribution where it can also be qualitatively indicated that the rubble is situated within the internal plan area of the structure in tandem with zones to the left, right and front of the original structure location. This figure also shows that there was no debris scatter in the area behind the original structure due to the survival of the rear panel wall.

In order to quantitatively assess the scaling methodology's ability to replicate the prototype structure's debris distribution it was necessary to implement a mixture of debris collection methodologies as detailed in chapter 5. Model debris data was subsequently captured via the manual weighing and positional logging of masonry rubble with mass values being assigned to a spatial grid location in the local X and Y axes. The grid resolution utilised for this analysis was 50% of the structural width and 50% of the structural length, thus providing a non-dimensionalised grid for comparisons with prototype results. The enormous masses associated with prototype rubble required an alternative method via the manual

counting of blocks. Block quantities were then assigned to a spatial grid location within the local X and Y axes where the grid resolution matched the model.



(a)



(b)

Figure 6.16: Post trial images of masonry debris at 14.25m: (a) Prototype; (b) Model.

Figure 6.17 illustrates the distribution of model debris in the terms of absolute mass and the percentage of the original structural mass. Closer examination of which demonstrates that 52.7% of the original structural mass was subject to collapse before forming the resultant

debris spread. It can also be seen that the debris within the internal plan area represents 25.8% of the original structural mass. Figure 6.18 shows the debris distribution of the prototype structure in terms of number of blocks, percentage of total blocks and inferred percentage of total mass. This inference builds upon the benchmarking conducted in chapter 5 where it was shown that the percentage of total bricks within the model debris closely matched the percentage of total mass. The accuracy of this methodology links to the fact that the majority of the mortar remained attached to brick and block units, indicating that brick and block spread is highly representative of total debris spread. Examination of figure 6.18 shows that 43.3% of the original structure mass collapsed to generate the rubble spread with 14.9% being found within the internal plan area.

An initial comparison of model and prototype debris indicates that the percentage of total mass was approximately +10% greater within the model data where it can also be seen that this net difference closely matches the difference for internal debris spread. This can be shown to result from the additional levels of side panel collapse observed with the model structure in section 6.3.1. As a result, the moderately conservative model damage prediction resulted in a moderately conservative debris spread. Figure 6.19 illustrates a comparison of prototype and model debris for each grid location to yield a net difference grid. The examination of which shows a mean difference of $\pm 2.0\%$ with a maximum disparity of +5.3% and minimum of -4.5% due to the additional side panel collapse observed with the model structure.

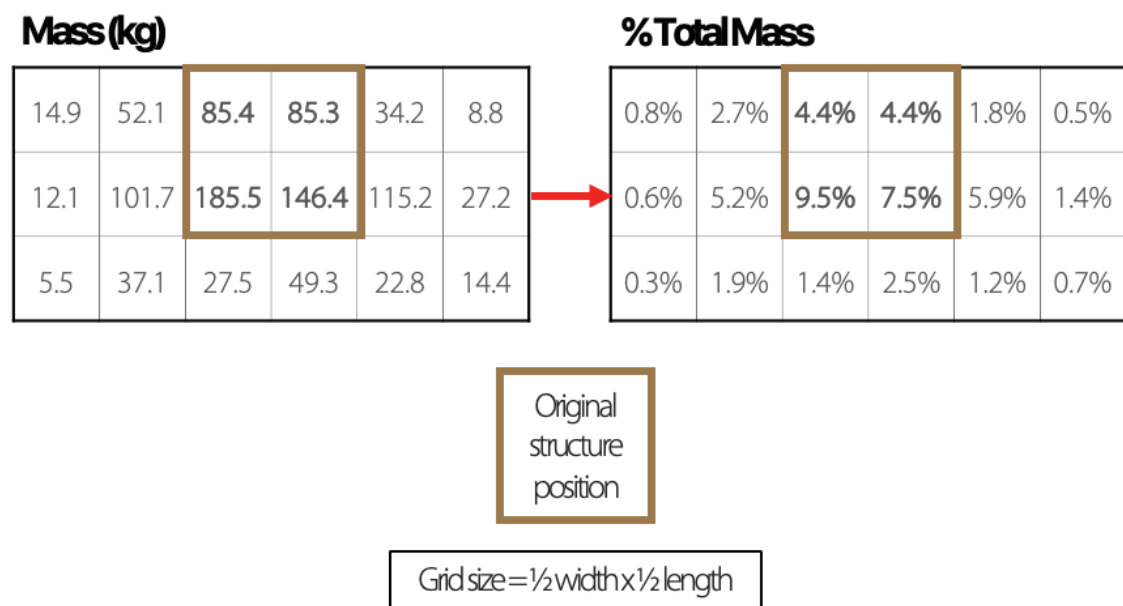


Figure 6.17: Spatial distribution of model masonry debris at 14.25m.

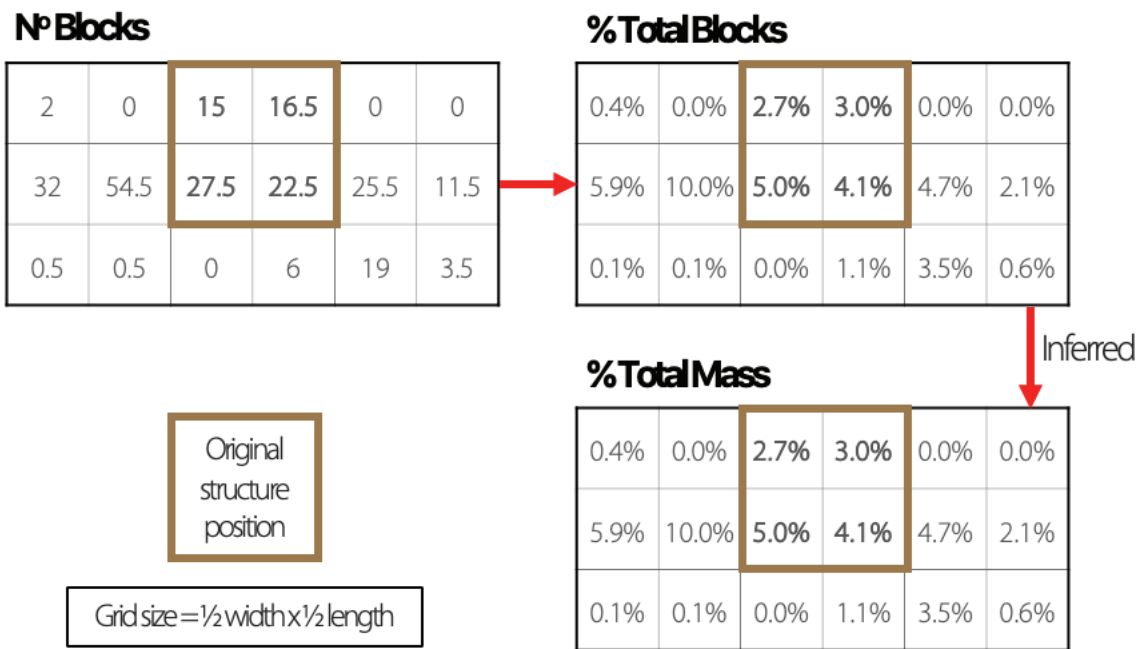


Figure 6.18: Spatial distribution of prototype masonry debris at 14.25m.

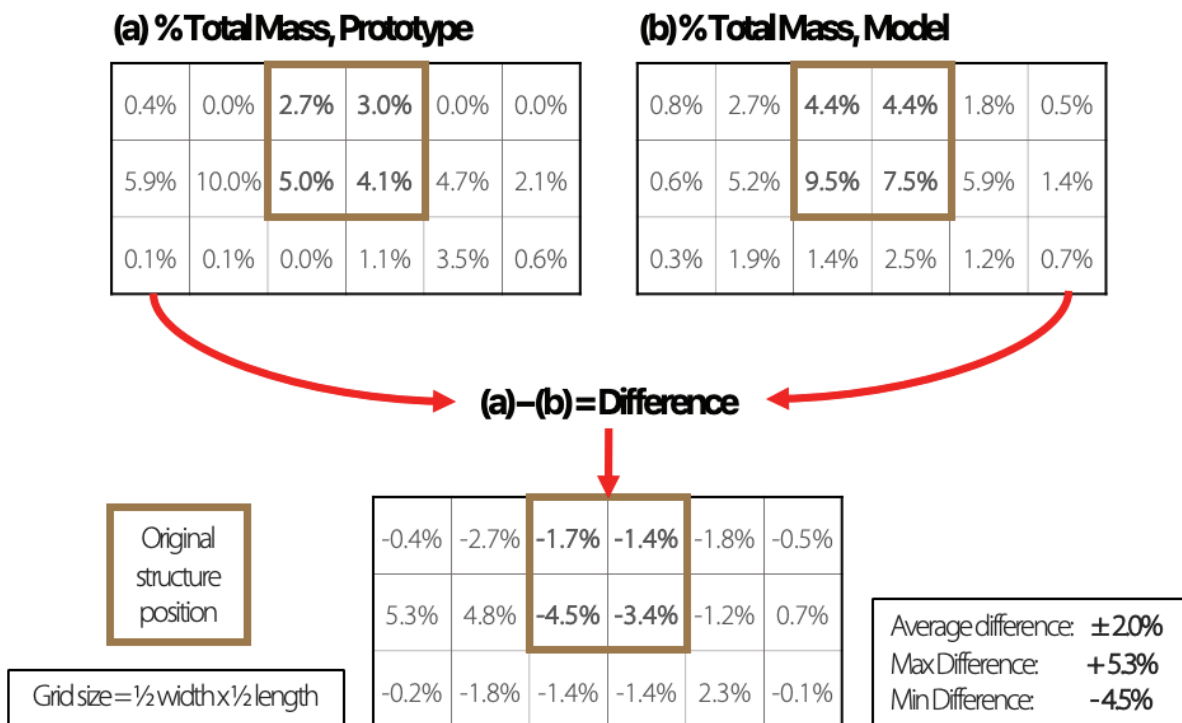


Figure 6.19: Comparison of prototype and model debris distributions at 14.25m.

The quantitative assessment of the scaling methodology's ability to accurately characterise masonry debris distributions requires the definition of a quantitative metric in the form of an acceptability criteria. Based on the potential level of ground bed fixity variation identified for the rear panel of the model and prototype structures at 14.25m, it can also be assumed that additional degrees of variation in ground bed restraint may have also been present at the side and front panels. This variation can thus be shown to exert a substantial influence on the sequential process of cracking, deflection and potential collapse to generate the debris distribution as detailed in chapter 3. With no available methods to quantify the extent of these potential differences or their potential influence on the damage and debris data, this research will adopt an acceptability criteria of $\pm 10\%$ for prototype and model debris comparisons. It can therefore be seen that the model debris results have met this condition in the global sense when referencing the +9.4% difference in percentage of total structural mass for the entire debris spread. The mean, maximum and minimum differences calculated in figure 6.19 also fall within these bounds, indicating that the acceptability criteria was met on a local basis within the non-dimensional grid.

Whilst the presentation of masonry debris data within a grid represents a simple but effective methodology for analysing the spatial distribution of masonry rubble, the level of resolution provided by this method may not be suitable for fast-running engineering algorithms (FREA) which aim to rapidly determine the extent and effect of masonry debris. These FREAs therefore require high-level masonry debris information to determine risk and hazard for both the nearby population and emergency service response teams which will be sent to assist after an explosive event. Figures 6.20 and 6.21 therefore illustrate an alternative representation of masonry debris results via non-dimensional quadrants where the radius is equal to the structural width multiplied by 1.5. Further examination of these two figures shows that masonry debris can be expressed as a percentage of the original structural mass within four quadrants where each monitors two adjacent sides of the structure. These figures also provide additional information relating to the internal debris distribution for the structure as shown by the percentage values in parentheses. By distilling debris data into an assessment which emphasises larger quadrants, these diagrams represent a data resolution which can be used by FREAs including those developed by this research's industrial sponsor, AWE, to rapidly assess the risk posed by masonry debris for by explosive events of any size.

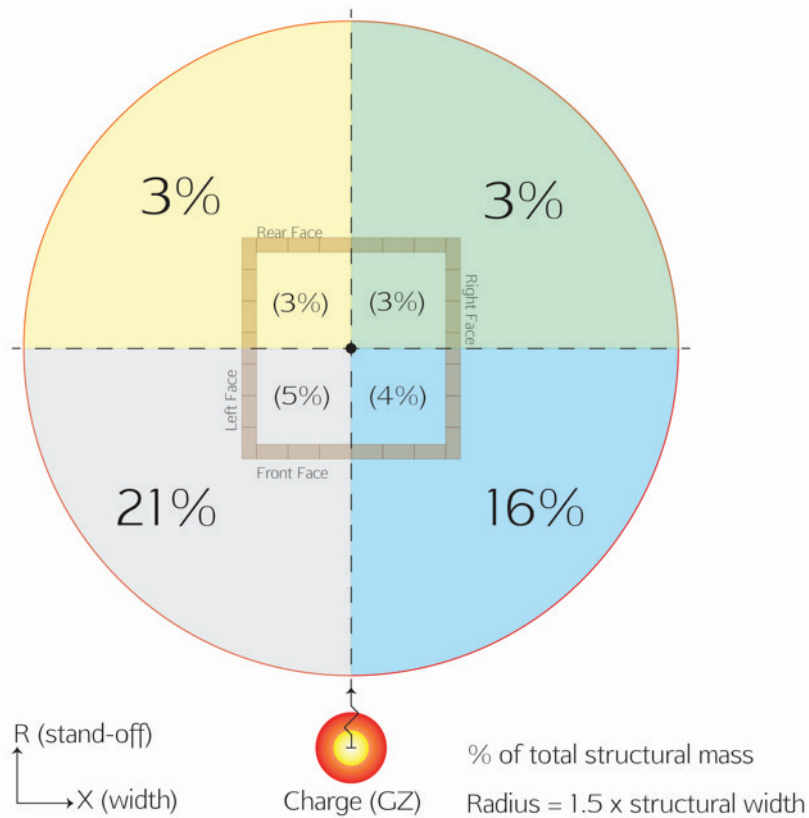


Figure 6.20: Prototype debris distribution in non-dimensional quadrants at 14.25m stand-off.

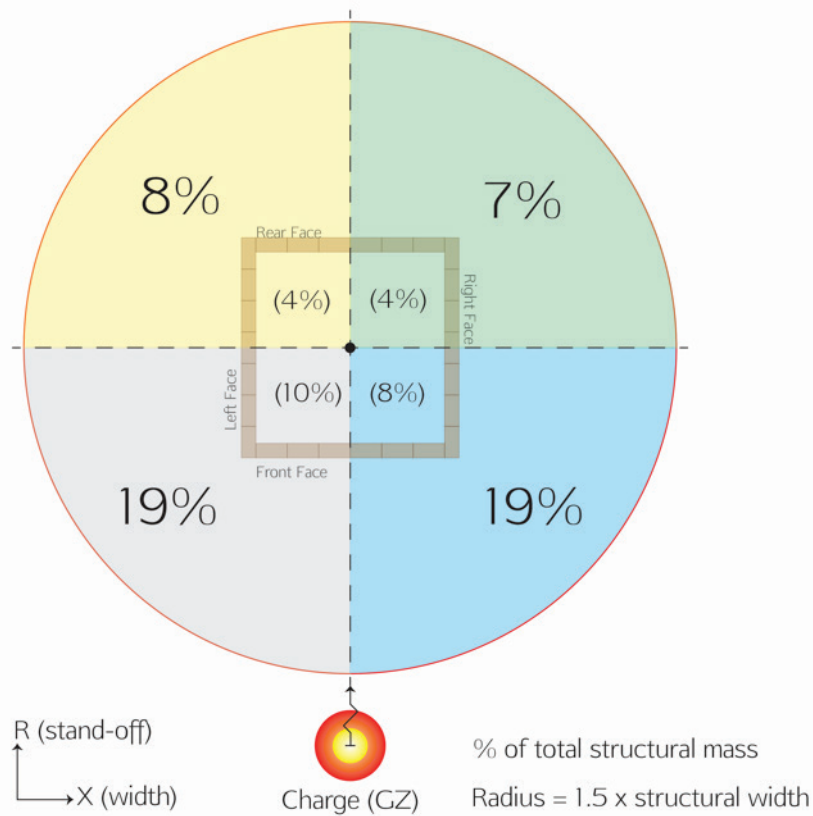


Figure 6.21: Model debris distribution in non-dimensional quadrants at 14.25m stand-off.

Additional examination of figures 6.20 and 6.21 indicates that the ~9% increase in total debris mass for the model structure is encapsulated within the rear two quadrants as demonstrated by differences of ~4% and ~5%. It can also be seen that the majority of debris mass is located within the front two quadrants in figures 6.20 and 6.21 as shown by net percentage values of 37% and 38% respectively to produce a negligible difference of 1%. The internal structure area within both charts represents a significant magnification in quadrant resolution where it can be seen that the net difference of ~9% mass percentage is almost entirely characterised by the internal debris spread within these two figures. Whilst the interpretation of risk and hazard posed by the debris data presented within figures 6.20 and 6.21 is beyond the scope of this research, these diagrams indicate that model debris closely replicates the prototype data for the front two quadrants with a moderately conservative estimate of total debris in both the rear two quadrants and the internal structure area. Future work which assesses the scaling methodology for a wider range of masonry structures and blast scenarios will be necessary to determine whether this conservative prediction is representative of the scaling methodology's overall performance as one data point cannot be utilised to extrapolate a trend. Furthermore, it can also be shown that potential variations in ground bed fixity may also introduce variations in the sequential phases of masonry response for both prototype and model masonry structures. As a result, it will be important to emphasise the need to control this parameter as closely as possible in future blast trials.

6.5 Accelerometers and laser scans

6.5.1 Accelerometer performance

This section assesses the accelerometer brick and block elements implemented at 14.25m stand-off within the front panel wall of the prototype and model masonry structures as detailed in chapter 5. Figure 6.22a illustrates that the model accelerometer element was subject to early ejection from the front masonry panel, preventing the capture of and comparison of meaningful data for dynamic motion of an equivalent brick element with the prototype. The rapid release of the model accelerometer housing was likely due to an insufficient level of adhesion between the steel element's outer surface and the mortar used within the masonry panel. As a result, this element was rapidly forced out of the front panel

wall at a much faster rate than the actual structural response. Examination of the prototype accelerometer indicated that this element was also unable to capture meaningful data related to equivalent block response. Unlike the model structure, this accelerometer was installed within an actual prototype block, removing the model issue of insufficient mortar adhesion. Despite this, the prototype element was subject to unit splitting at the centre line during masonry deflection as shown in figure 6.22b. As a result, the cylindrical insert which housed the accelerometer was released to prevent the measurement and comparison of equivalent dynamic motion with the model. Based on the performance of the model and prototype elements it can be shown that the methodology for measuring equivalent dynamic brick and block motion requires modification. In particular, the re-location of accelerometer housing units to an off-centre position should mitigate any potential unit splitting. It will also be necessary to manufacture an alternative model element housing which can be sufficiently adhered to mortar and thus accurately replicate model unit behaviour.

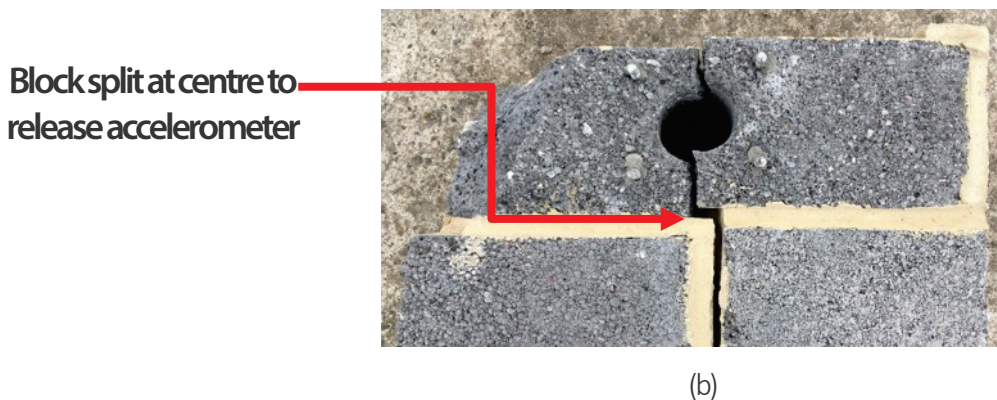
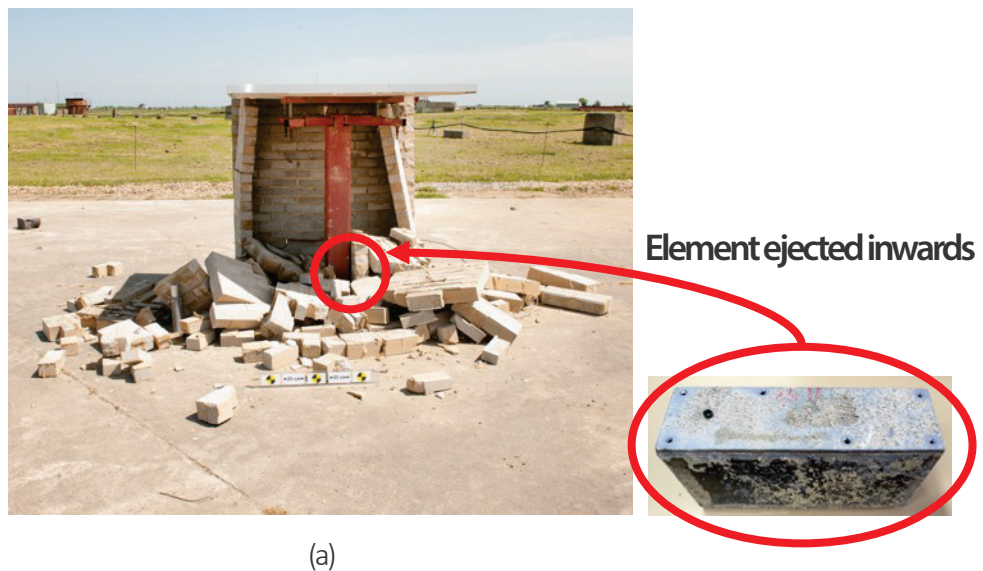


Figure 6.22: Accelerometer performance at 14.25m stand-off: (a) Model element ejected quickly; (b) Prototype element split at centre.

6.5.2 3D data capture

This final section briefly discusses the 3D data capture results produced via laser scanning of the post-trial environment as detailed in chapter 5. The implementation of this methodology was linked to potential safety hazards associated with masonry response when structures are in a state of impending collapse, thus presenting unacceptable levels of risk. As a result, figures 6.23-26 illustrates the high-fidelity and highly accurate virtual post-trial blast environment generated by these scans for each of the prototype and model structures fielded within the trial. Closer examination of these figures also demonstrates the ability to generate debris scatter perspectives which are not easily achieved with a traditional photographer. In the case of this blast trial, it was however possible to manually conduct the full range of qualitative and quantitative masonry analyses. In spite of this, the 3D data set generated via laser scanning provides a future capability to virtually re-visit the actual post-trial environment, enabling additional analyses alongside potential comparisons with future masonry scaling trials which also implement 3D laser scanning.

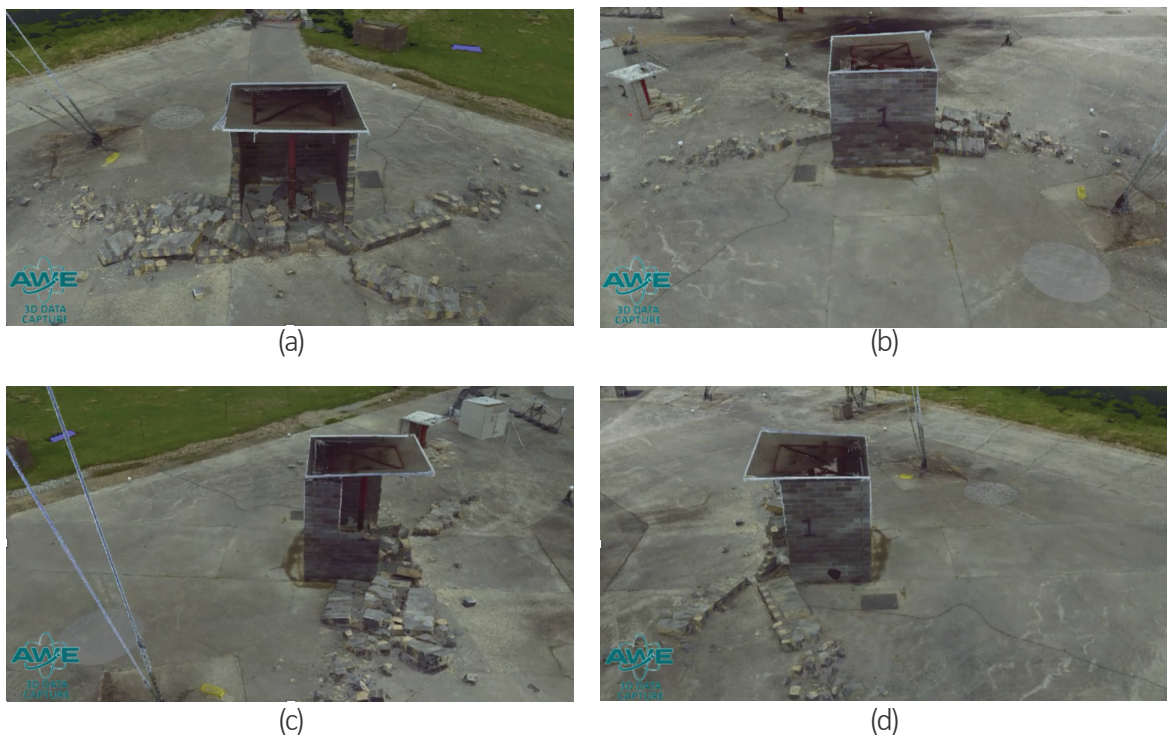


Figure 6.23: Post-trial images of 3D data capture for prototype masonry structure at 14.25m: (a) Front panel; (b) Rear Panel; (c) Right panel; (d) Left panel.

Scaling Masonry Structures in a Blast Environment

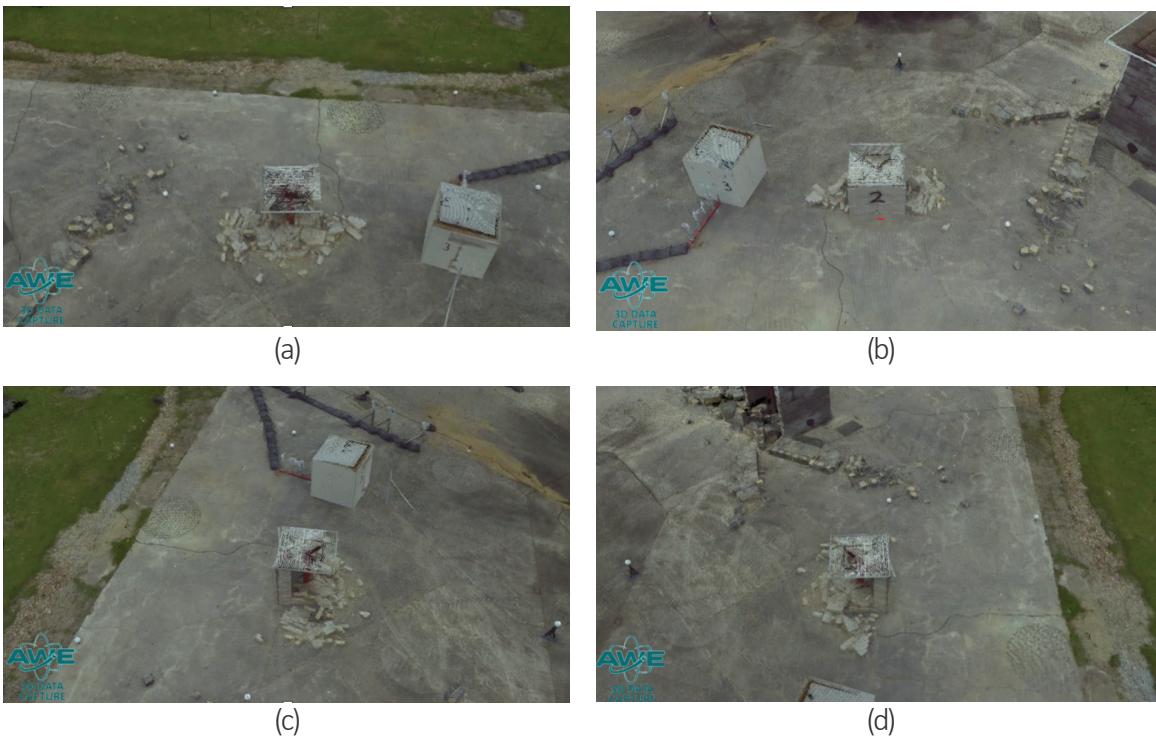


Figure 6.24: Post-trial images of 3D data capture for model masonry structure at 14.25m: (a) Front panel; (b) Rear Panel; (c) Right panel; (d) Left panel.

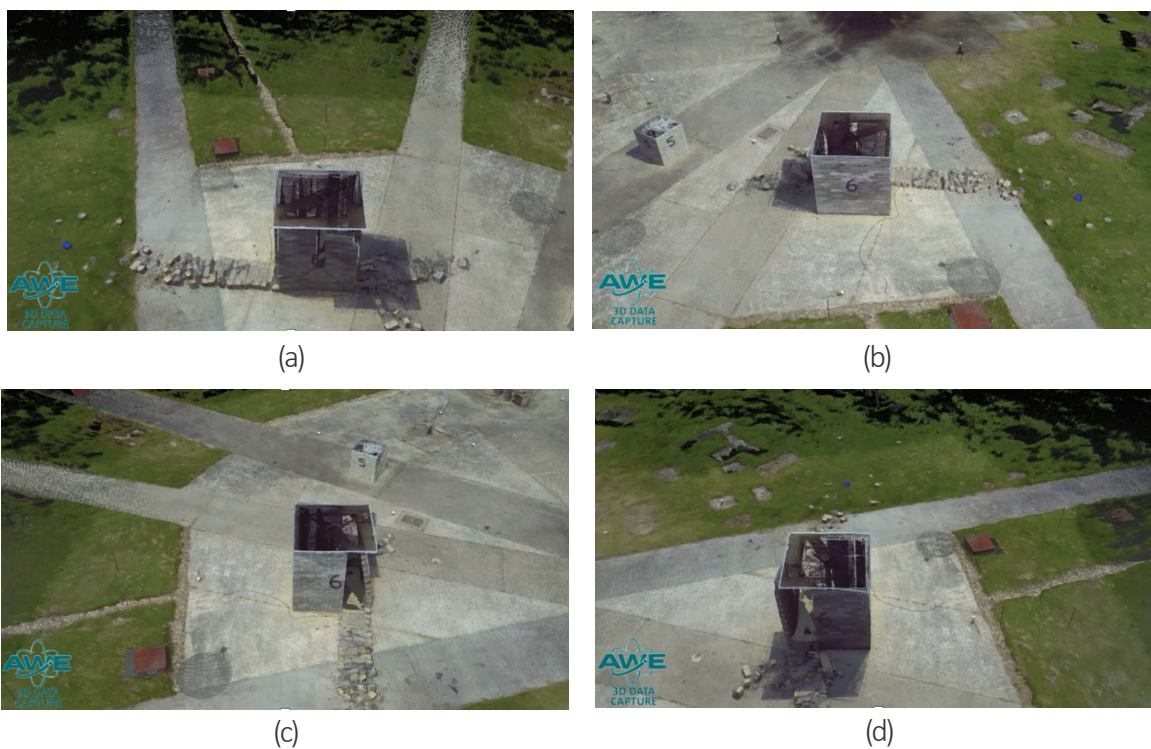


Figure 6.25: Post-trial images of 3D data capture for prototype masonry structure at 20m: (a) Front panel; (b) Rear Panel; (c) Right panel; (d) Left panel.

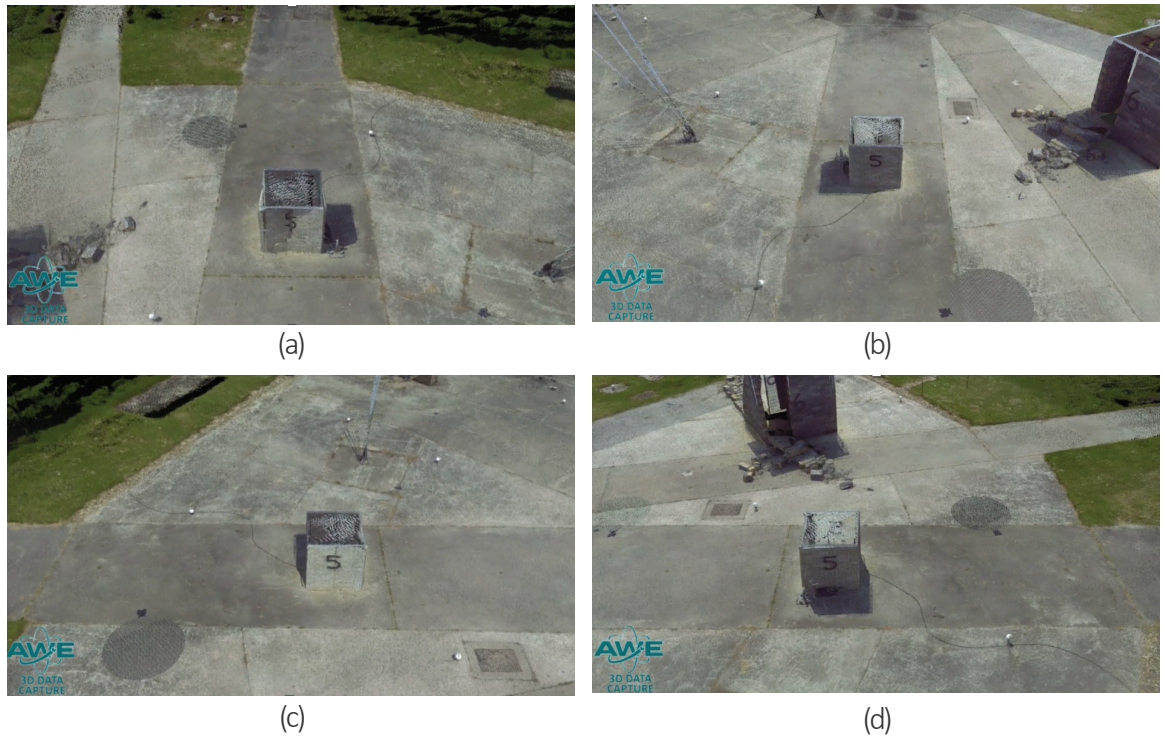


Figure 6.26: Post-trial images of 3D data capture for model masonry structure at 20m: (a) Front panel; (b) Rear Panel; (c) Right panel; (d) Left panel.

6.6 Summary

This chapter has analysed a range of data captured from the short-duration blast trial designed in chapter 5 with the aim of benchmarking this PhD's scaling methodology, thus fulfilling the second research objective and addressing this PhD's principal aim. This commenced with an analysis of the experimental blast environment where results revealed an amplification in blast pressure due to the utilisation of a physically cylindrical explosive charge. It was also shown that this charge shape would likely have produced a degree of blast wave asymmetry in the axial direction which decreased with increasing stand-off, finally resulting in a typical hemispherical blast wave. Of these two characteristics, it was found that blast pressure amplification was dominant with a mean increase of +34% versus pre-trial predictions. Despite this, it was shown that a mean difference in blast impulse of +6% met the acceptability criteria of $\pm 10\%$, indicating that the blast momentum applied to the structures closely matched the trial design. This was essential given the scaling methodology's dependence on the representation of a masonry structure as an inertial resistance to blast

loading. As a means of quantifying blast modification due to charge shape, an additional investigation found mean pressure, duration and impulse differences of +10%, -20% and -12% for a physically spherical 134kg TNT charge. Thus indicating that this charge configuration more closely replicated the measured trial environment

This section also assessed the performance of two non-responding timber structures to measure blast wave interaction with model structure geometries. The front face of the structure at 14.25m was found to be significantly damaged whilst the structure at 20m incurred no visible damage. An initial focus was placed on side-on pressure gauges to perform a comparison with measured free-field values. This analysis showed a -32% and -11% difference for the 14.25m and 20m values of side-on pressure versus the free-field. In contrast, side-on values for positive phase duration were found to exceed the free-field by +18% and +17% at 14.25m and 20m. The net effect of which was found to be impulse differences of -25% and +7% for side-on gauges at 14.25m and 20m stand-off distances. The level of variability found within the side-on data set was shown to contrast with free-field data which showed a steady reduction in pressure and impulse with increased stand-off to represent typical radial blast behaviour. As a result, it was found that the non-responding structure data set may not provide a meaningful additional benchmark for CFD models produced in chapter 7.

This chapter then assessed the performance of the scaling methodology for characterising masonry damage state for the two prototype and model structure pairs at 14.25m and 20m stand-offs. Examination of qualitative data captured at 14.25m showed that the prototype and model structures were both subject to full front panel collapse, partial side panel collapse and rear panel survival. Thus indicating that the model was able to qualitatively replicate the prototype structure's collapse mode. It was also shown that the prototype and model structure both developed five primary cracks with a vertical crack at the rear panel and vertical branching cracks at the fully restrained corners of the side and rear panel. As a result, it was found that the model structure was able to replicate the quantity and location of the prototype's primary cracks along with the quantity of superficial secondary cracks. Key areas of divergence identified for masonry damage at 14.25m included variance in rear panel crack shape and the extent of side panel collapse. Based on the discussion of edge restraint and its influence on masonry response in chapter 3, it was shown that variable levels of ground bed

fixity for the prototype and model rear panels was likely the cause of varying crack modes. Based on this evidence, it was also inferred that variable levels of side panel collapse may have been influenced by a lower degree of edge fixity for the prototype.

Analysis of masonry damage data for the prototype and model structures at 20m stand-off showed a number of important similarities and differences. Prototype and model structures were both found to produce front panel deflection without collapse due to interactions with the roof enclosures. The prototype was found to incur partial side panel collapse versus survival for the model which was likely due to the side panels being pinned in place by frictional forces from the roof enclosure. As a result, it was found that the model was able to qualitatively replicate prototype frontal deflection mode with the potential for equivalent side panel response if the prototype and model roof enclosure designs were modified to represent equal structural stiffness and blast response. Additional analysis of the prototype and model damage state demonstrated equal primary cracking behaviour at the front panel wall, highlighting the ability to replicate cracking mode with equivalent levels of edge restraint. The quantity of primary cracks were found to differ as a result of varying levels of side panel collapse. As a result, it was found that the frictional pinning forces exerted by the roof enclosure restricted potential side panel collapse of the model, resulting in two additional primary cracks versus the prototype. The location of side and rear panel primary cracks were found to differ in absolute location for the prototype and model structures. Closer inspection however indicated that the equivalent effect of cracking at a 'built in' corner represented the same level of damage where the exceedance of an ultimate limit state at these locations produced masonry unit splitting and the need for a re-build. It was therefore found that the model was potentially able to replicate the full set of damage response characteristics of the prototype at 20m based on similarities in actual masonry response and essential differences in the roof enclosure blast response which influenced sequential masonry response.

The next section of this chapter focussed on the quantitative analysis of masonry debris distributions for the prototype and masonry structures at 14.25m, thus enabling the assessment of the scaling methodology performance with this masonry response mode. Spatial distributions of debris were generated within a non-dimensional grid with a resolution equal to 50% of the structural width and 50% the structural length. The examination of these

data sets revealed that the model structure debris field represented ~53% of the original structural mass versus ~43% for the prototype. It was also found that the model structure produced an internal spread of debris equal to ~26% of the total structural mass versus ~15% for the prototype. Net differences of ~10% were found to meet an acceptability criteria of $\pm 10\%$ which was determined by considering the potential influence of varying ground restraint on sequential masonry response, culminating in the generation of debris. These differences in debris production were shown to be the result of additional side panel collapse for the model which was linked to potential variance in ground bed fixity for the prototype and model at 14.25m. As a result, it was shown that the moderately conservative model damage prediction yielded a moderately conservative debris spread. Additional examination of the spatial distribution data for prototype and model debris revealed a mean grid cell difference of $\pm 2\%$ which also met the acceptability criteria of $\pm 10\%$.

The final aspect of the debris analysis section included an alternative presentation of debris data via diagrams which showed debris as a percentage of original structural mass within four non-dimensional quadrants that each monitor two adjacent structural sides. These diagrams also provided a magnified view of the internal debris distribution. By distilling debris data into a high-level assessment, these diagrams illustrated the ability to represent detailed debris results at a resolution which is suited to fast-running engineering algorithms which rapidly assess risk and hazard due to masonry debris. These diagrams also enabled this section to succinctly compare prototype and model debris data where it was shown that the increase in total debris mass for the model was encapsulated within the rear two quadrants. Thus resulting in a moderately conservative prediction of total mass in these zones. It was also found that the vast majority of debris mass was located within the front two quadrants for both the prototype and model with a minor difference of 1% for the data sets. This showed that the model results closely replicate the global distribution of debris within the two pairs of non-dimensional quadrants.

The last section of this chapter briefly discussed the accelerometer brick elements implemented at 14.25m stand-off. It was shown that the model accelerometer element was unable to capture meaningful data for dynamic motion of an equivalent brick element due to early element ejection within the front panel. This was likely due to insufficient adhesion of the steel element's coarse outer material with the structural mortar, thus enabling the

element to be rapidly forced out of the front panel wall at a much faster rate than the actual structural response. The prototype element was also unable to capture meaningful data related to equivalent block response as a result of unit splitting at the centre line during masonry deflection. As a result, the cylindrical insert which housed the accelerometer was released, preventing the measurement of dynamic motion. The lack of data for brick and block motion during the blast trial prevented this section from conducting a comparison of masonry unit acceleration with the aim of benchmarking the predicted behaviour outlined in chapter 4. This final section also briefly discussed 3D data capture results produced via laser scanning. It was shown that the fidelity of the data set produced a high-resolution and highly accurate virtual post-trial blast environment. This therefore provides an ability to re-visit the actual post-trial environment, enabling additional analyses and comparisons with future trials which also implement masonry scaling alongside 3D laser scanning of the post-trial debris.

Chapter 7

Computational Modelling: Short-duration Blast

7.1 Introduction

This chapter details the design and benchmarking of CFD models of blast flow and AEM models of masonry response against data captured during the short-duration blast trial detailed in chapters 5-6. This chapter commences with a brief comparative study of the blast flow solvers, Air3D and AUTODYN to determine the most suitable package for modelling the blast wave generated during the blast trial. This comparison focusses on the relative benefits of accurately modelling the detonation of a cylindrically shaped charge versus an increased TNT charge mass of 134kg TNT as discussed in chapter 6. Satisfactory benchmarking of CFD models precedes the remap of solutions to enable the configuration of blast loading within AEM models of masonry response. This chapter therefore aims to assess the predictive capabilities of these two computational methods to provide an interpolative methodology when it is not possible to conduct experimental trials. As a result, this chapter aims to fulfil the third objective of this research whilst addressing the secondary aim of this PhD.

7.2 CFD Modelling

CFD analyses were conducted to replicate the short-duration blast environment detailed in chapter 6. Each simulation was conducted using a dedicated quad-core Intel i7-3840QM 2.8GHz system with 16GB RAM. The utilisation of CFD to simulate dynamic blast environments is an essential component of this research's computational methodology as this

data can be re-mapped into an AEM model of masonry response, thereby configuring the loading component of these simulations.

7.2.1 Comparative study

As a means of determining the most suitable CFD package for modelling the short-duration blast flow produced during this PhD's blast trial, an initial study was undertaken to compare two of the most widely used CFD solvers available to model blast, namely Air3D and AUTODYN. This aimed to assess the relative accuracy of these CFD packages versus free-field and reflected blast data which was experimentally recorded during the blast trial as detailed in chapter 6.

Air3D models were initially designed as 1D simulations within which gauges were located at the front, mid and rear stand-off distances associated with each of the prototype and model masonry structures deployed in this research's short-duration blast trial. The AUTODYN solution required an initial 2D model to implement 100kg TNT with a physically cylindrical charge shape with 1:1.6 aspect ratio, thus enabling the model to simulate blast wave asymmetry in the axial direction as shown in figure 7.1. In contrast, the Air3D model implemented 134kg TNT with a physically spherical charge shape with an emphasis on modelling the effect of blast amplification. Each of the CFD models utilised an optimised simulation by implementing Air3D's 1D domain expression shown in equation 2.22 within chapter 2. Each of the CFD model domains were designed to run from 0m to the maximum stand-off distance plus 20m. Thus providing a buffer between the final pressure gauge and the upper domain boundary to limit the likelihood of boundary perturbations interfering with the solution results. This analysis focussed initially on the free-field accuracy of these two potential approaches with the results detailed in tables 7.1 and 7.2. The examination of which shows that each of mean differences between CFD and experimental data were lower with Air3D and the detonation of 134kg TNT with a spherical charge. In particular, mean differences of +3% and -6% for pressure and impulse compared to mean increases of +14% and +18% with AUTODYN indicates the Air3D solution more closely replicates the blast pressure and momentum produced with this PhD's trial. As discussed in chapters 4 and 6, accurate modelling of impulse is an essential aspect of this research due to the representation of masonry structures as an inertial resistance to blast loading.

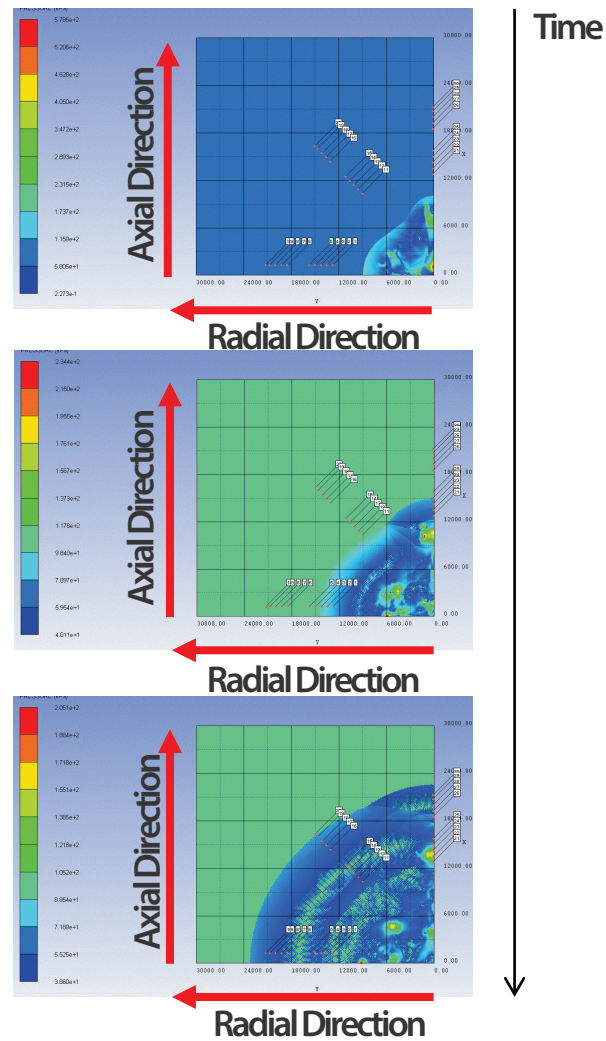


Figure 7.1: AUTODYN simulation of blast wave formation for cylindrical TNT charge.

Table 7.1: AUTODYN free-field blast parameters versus short-duration blast trial results.

Stand-off (m)	P_s	i_s	t^+
12.75	+8%	+26%	+18%
13.5	+12%	+16%	+20%
14.25	+23%	+14%	+13%
15	+14%	+18%	+17%
15.75	+20%	+22%	+16%
18.5	+10%	+17%	+13%
19.25	+9%	+18%	+19%
20	+14%	+8%	+13%
20.75	+21%	+21%	+11%
21.25	+14%	+18%	+4%
Average	+14%	+18%	+14%

Table 7.2: Air3D free-field blast parameters versus short-duration blast trial results.

Stand-off (m)	P_s	i_s	t^+
12.75	+2%	+2%	+10%
13.5	+4%	-4%	+13%
14.25	+15%	-8%	+5%
15	+3%	-3%	+11%
15.75	+9%	+	+10%
18.5	-4%	-8%	+13%
19.25	-6%	-6%	+18%
20	-1%	-10%	+11%
20.75	+7%	-2%	+8%
21.25	-2%	-7%	+5%
Average	+3%	-6%	+10%

Expanding upon this free-field analysis, solutions were re-mapped to produce 3D simulations of masonry structural interaction with the blast wave, thus enabling a comparison of CFD and experimental values of reflected pressure. This analysis does not integrate side-on pressure due to the high degree of uncertainty and result variability identified within this data set in chapter 6. Both the Air3D and AUTODYN models utilised an infinitely rigid structural geometry to represent the model timber structures via the definition of unavailable cells as detailed in chapter 2. The AUTODYN implementation of 3D blast flow required the re-mapping of an axially symmetrical 2D cylindrical domain into a cuboid domain with a cartesian co-ordinate system, thus introducing potential re-map error when transferring from cylindrical cell elements to standard cubic cells. The AUTODYN 3D simulation was limited to a maximum of 22 million cells with 100mm 3D cell size, representing a coarse step from ~5mm in 1D and thus introducing further potential re-map error. The simulation produced in Air3D implemented the same domain mesh shape for 1D, 2D and 3D to limit any potential re-map errors associated with varying mesh shapes. The Air3D simulation utilised a 25mm 2D cell size followed by a 50mm cell size in 3D to produce a total of 220 million cells and a finer step grading when re-mapping from 1D to 2D to 3D. Despite a ten-fold increase in total cell count, Air3D run-times for 3D blast flow solutions were orders of magnitude lower than the AUTODYN equivalents. Each of the AUTODYN and Air3D models implemented a reflected pressure gauge located at the centre point of the front panel to replicate the experimental gauge position as shown in figure 7.2.

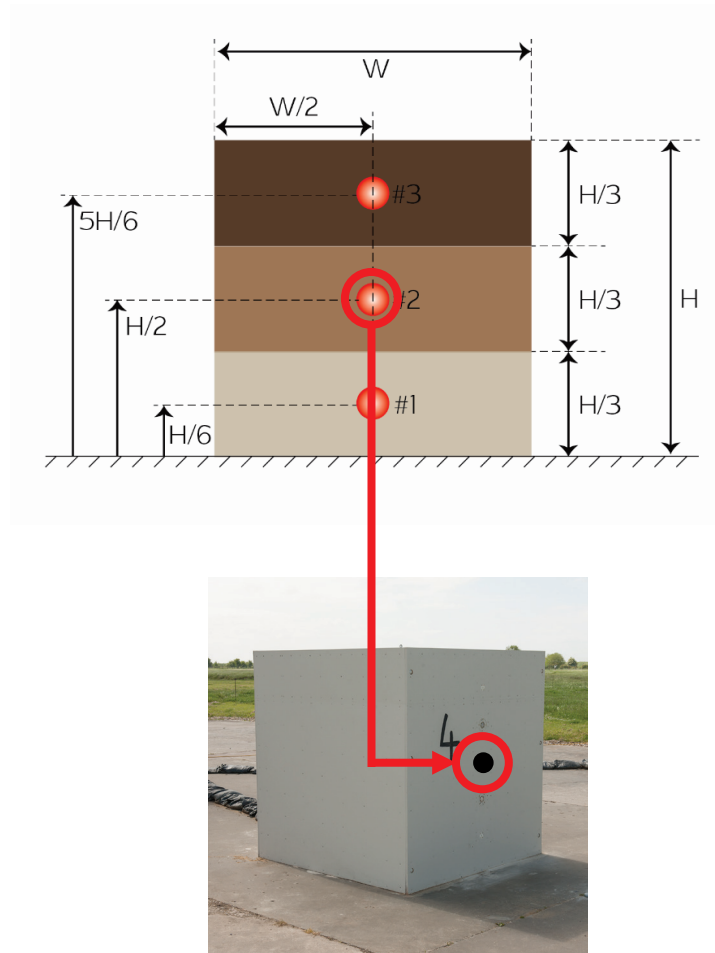


Figure 7.2: Reflected gauge location determined by experimental trial.

The reflected data captured at these computational gauge points is compared to experimental measurements in tables 7.3 and 7.4. Examination of which demonstrates that the mean differences between CFD and experimental data for reflected blast parameters were lower overall with Air3D and the detonation of 134kg TNT with a spherical charge. This is underlined by negligible differences of -1% and -4% for pressure and differences of -2% and -17% for impulse. In contrast, AUTODYN was found to produce larger differences in both pressure and impulse with three of the four data points exceeding $\pm 20\%$ variance. The combination of these reflected results with the free-field solutions shown in table 7.1 and 7.2 indicates that the Air3D solution more closely replicates the free-field and reflected blast environments produced with this research's trial. In particular, closer levels of agreement for free-field and reflected impulse infers a higher likelihood of reproducing the blast momentum transmitted to the masonry structures within the blast trial. This is an essential component of this research due to the representation of masonry structures as an inertial resistance to

blast. Based on these results, the remainder of this PhD's short-duration blast flow modelling will be conducted with Air3D with the detonation of 134kg TNT to account for blast amplification due to the physically cylindrical charge shape.

Table 7.3: AUTODYN reflected blast parameters versus short-duration blast trial results.

Stand-off (m)	P_r	I_r	t^+
13.5	+6%	-36%	+26%
19.25	+23%	+20%	+11%

Table 7.4: Air3D reflected blast parameters versus short-duration blast trial results.

Stand-off (m)	P_r	I_r	t^+
13.5	-1%	-2%	+11%
19.25	-4%	-17%	-5%

7.2.2 Air3D methodology

Building upon the comparative study, Air3D models were produced to detonate 134kg of spherically shaped TNT to simulate the reflected and free-field blast environments produced in short-duration blast trial detailed in chapters 5 and 6. Each of the models utilised an optimised 1D simulation from 0m in the X direction up to 1m to represent the height of burst designed for the trials. These 1D simulations were subsequently re-mapped into 2D then 3D with the domain extents and cell size detailed in tables 7.5 and 7.6.

Table 7.5: CFD domain parameters for 134kg TNT at 20m stand-off.

	X Direction (m)	Y Direction (m)	Z Direction (m)	Cell Size (mm)	Cell Limit Per Dimension	Cell Count in X (Max)	Total Cells	Run Time (mins)
1D	0-1	-	-	4.6	-	-	218	15
2D	0-18	0-10	-	25	3800	720	288×10^3	
3D	10-30	0-15	0-10	100	3000	200	3×10^6	

Table 7.6: CFD domain parameters for 134kg TNT at 14.25m stand-off.

	X Direction (m)	Y Direction (m)	Z Direction (m)	Cell Size (mm)	Cell Limit Per Dimension	Cell Count in X (Max)	Total Cells	Run Time (mins)
1D	0-1	-	-	4.6	-	-	218	32
2D	0-12	0-10	-	25	3800	480	192×10^3	
3D	4-24	0-15	0-10	100	3000	200	3×10^6	

The visual interpretation of the CFD domain extents can be seen in figure 7.3a for the X-Y plane in 2D. Figure 7.3b illustrates the domain visualisation for the X-Y plane in 3D whilst figure 7.3c details the X-Z plane in 3D. Closer inspection of tables 7.5-6 shows that each of models was designed to not exceed 2D or 3D cell limits which are outlined in Appendix A.

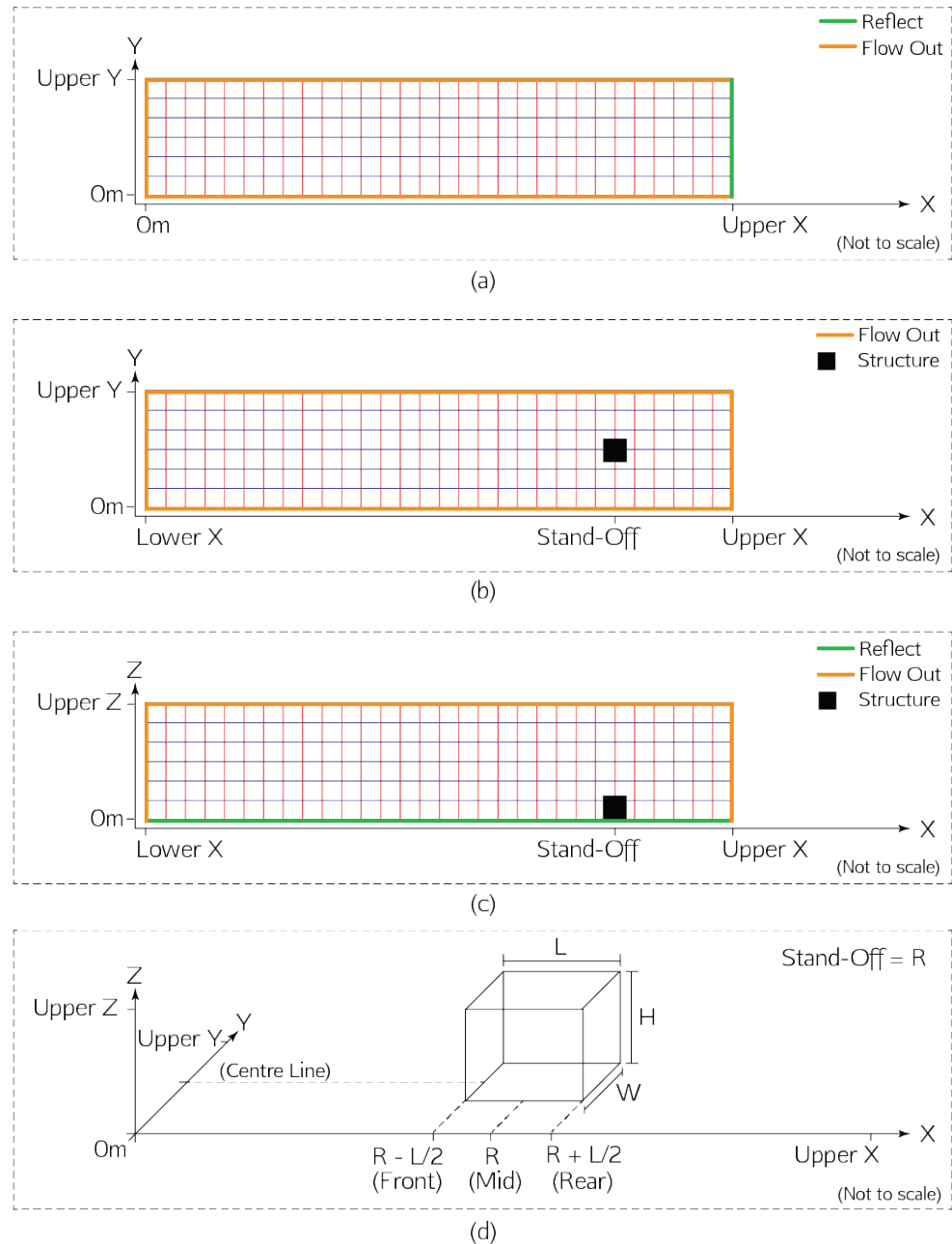


Figure 7.3: CFD domain schematic in 2D-3D, short-duration blast trial: (a) X-Y plane 2D; (b) X-Y plane 3D; (c) X-Z plane 3D; (d) Masonry cuboid proportions and location.

Further inspection of figure 7.3a shows that the upper X boundary was set to reflect in order to trigger a 3D re-map once the blast wave reaches the upper boundary limit in 2D. Figure 7.3b shows that each of the boundaries were set to flow out in the X-Y plane whilst the ground surface was set to reflect in the X-Z plane as shown in figure 7.3c. Figures 7.3b and 7.3c illustrate the spatial positioning of non-responding cuboid structures where the centroid was located at the desired stand-off distance. This approach represents the trial design detailed in chapter 5 which aims to apply an average pressure gradient to the model and prototype structures. This is further demonstrated in figure 7.3d where the front, mid and rear stand-off locations have been identified. Closer examination of figure 7.3d also shows that the cuboid structures were located along the centre line in the Y direction, resulting in a centrally positioned cuboid within the 3D domain.

Pressure gauges were located throughout CFD model domains to measure free-field and reflected parameters. Reflected pressure gauges were located at multiple sites on the front, side and rear face of the cuboid structure. In particular, the front face was instrumented with one reflected pressure gauge at the centroid position to replicate the functioning gauge from the blast trial as shown in figure 7.4. Whilst the initial trial design in chapter 5 aimed to assess variation in reflected pressure over the height of a structure, the utilisation of fewer pressure regions is particularly useful for reducing simulation complexity within the AEM models of masonry response. Future work may investigate additional pressure regions on the front face in tandem with a re-designed experimental non-responding structure, thus enabling additional CFD benchmarking. Figure 7.5 shows that the side face was instrumented with three vertical regions, each representing $\frac{1}{3}$ of the area over the structural length with one gauge located at the centre of each region. Due to the symmetrical nature of the structures with side-on loading, the pressure record from these gauges can be seen to represent both sides of the structure. Figure 7.6 demonstrates that the rear face was divided into three equal regions with each accounting for $\frac{1}{3}$ of the area over the height of the structure. Gauges were subsequently located at the centre of each area, enabling the pressure to be monitored as it clears over the structure and down the rear face. Free-field pressure was monitored at the stand-off distance of interest in each model with the gauges located $\sim 5\text{m}$ away from the lower and upper X boundaries at a 1.5m height to prevent reflections or interference from the blast wave's progression over the structure.

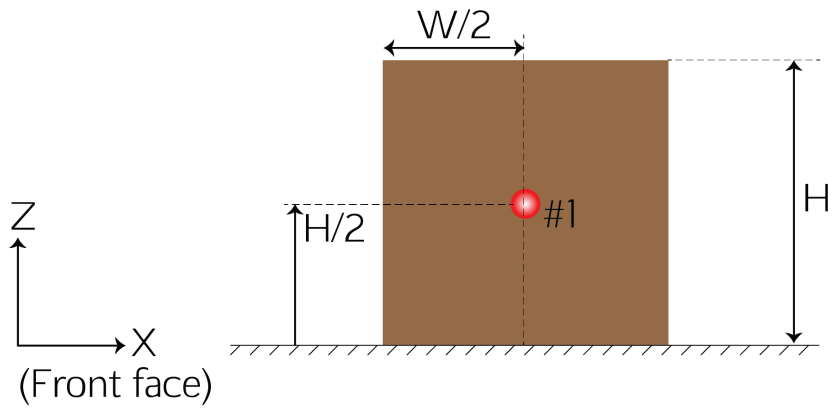


Figure 7.4: CFD reflected gauge schematic, front face.

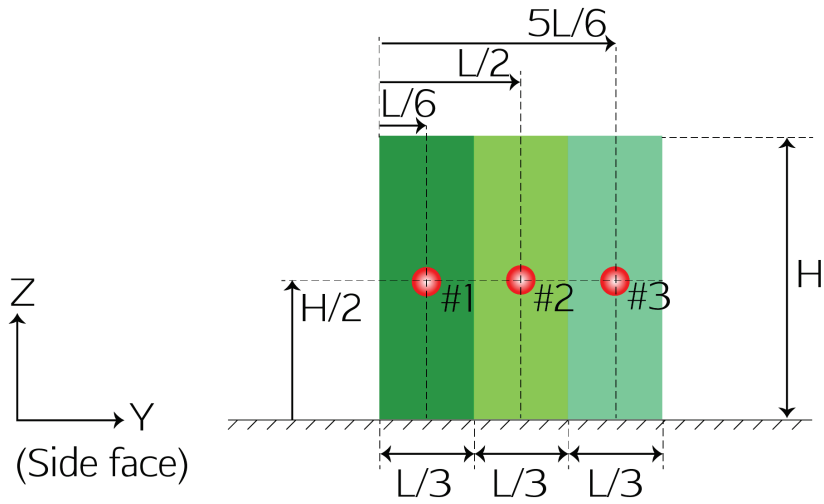


Figure 7.5: CFD gauge schematic, side face.

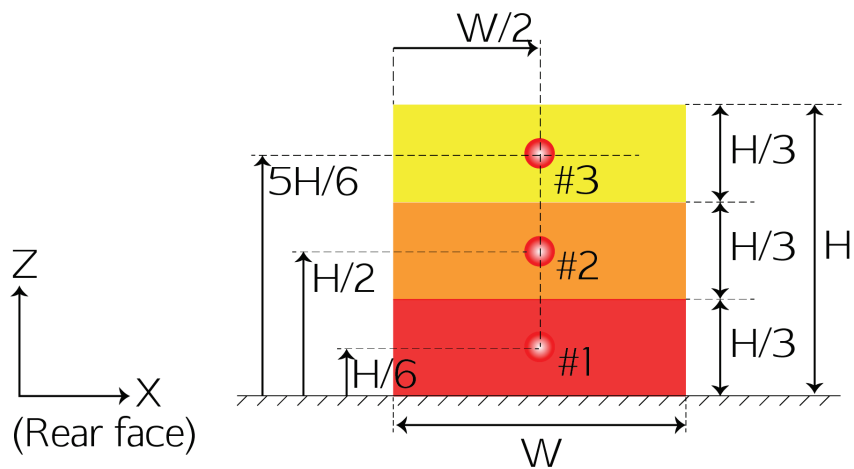


Figure 7.6: CFD gauge schematic, rear face.

7.2.3 Air3D solution analysis

CFD solutions were benchmarked against experimental blast parameters for the free-field stand-off distances detailed in chapter 5. An acceptability criteria of $\pm 10\%$ was used within the free-field to demonstrate accurate modelling of blast flow throughout the CFD domain. Thus inferring that blast interaction with non-responding structural geometries is also representative, enabling solution remaps to configure AEM blast loading.

Tables 7.7 and 7.8 compare CFD solutions of the free-field environment at relative stand-off distances for prototype and model structures against measured values from chapter 6. Inspection of table 7.7 for the prototype shows CFD values for free-field overpressure to be within $\pm 10\%$ of the experimental values for 14.25m and 20m stand-offs with the exception of the mid location at 14.25m. Closer inspection of which indicates that the CFD model represents an idealised decay in overpressure with increased radial distance whilst the experimental value indicates a slight variance in expected pressure when compared with the front and rear values for the 14.25m stand-off reference. It can therefore be assumed that the trial result may have been due to gauge error or blast wave asymmetry, the random effects of which cannot be modelled via CFD. Examination of CFD results for impulse can be seen to be within $\pm 10\%$ of the experimentally measured values across the stand-off range. Analysis of positive phase duration shows CFD values to range from -2% to -13% below experimental values, indicating an accuracy level which slightly exceeds $\pm 10\%$. It can be shown however that Air3D tends underestimate the positive phase duration as detailed by the comparative study in Appendix A.

Examination of table 7.8 for the model structure shows CFD results for free-field overpressure to be within $\pm 10\%$ of experimental values for the range of stand-offs with the exception of the mid location at 14.25m. This discrepancy was discussed in relation to table 7.7 above and as such, the same reasoning can be applied. Analysis of CFD results for impulse can be shown to be within $\pm 10\%$ of the experimental values for each stand-off location whilst positive phase results represent differences ranging from -5% to -13% below experimental values. As a result, these duration results slightly exceed the acceptability criteria of $\pm 10\%$. These positive phase results correlate with those for the prototype where it was shown that Air3D tends to underestimate positive phase as expanded upon in Appendix A.

Table 7.7: CFD free-field blast results for prototype structures at 14.25m and 20m.

		Prototype Structural Geometries					
		w = 134kg, r = 20m			w = 134kg, r = 14.25m		
		P _s (kPa)	i _s (kPa.ms)	t ⁺ (ms)	P _s (kPa)	i _s (kPa.ms)	t ⁺ (ms)
Front (R – 1.5m)	CFD	83.6	375	12.0	186.5	542	8.3
	Trial	80.2	349	13.8	190.2	550	9.3
	+/- %	4%	8%	-13%	-2%	-2%	-10%
Mid (R)	CFD	71.7	323	12.8	144.8	460	9.4
	Trial	71.3	294	14.3	169.8	425	9.9
	+/- %	1%	10%	-11%	-15%	8%	-5%
Rear (R + 1.5m)	CFD	63.9	333	12.8	116.5	427	10.4
	Trial	62.7	312	13.4	128.4	428	11.6
	+/- %	2%	7%	-5%	-9%	0%	-10%

Table 7.8: CFD free-field blast results for model structures at 14.25m and 20m.

		Model Structural Geometries					
		w = 134kg, r = 20m			w = 134kg, r = 14.25m		
		P _s (kPa)	i _s (kPa.ms)	t ⁺ (ms)	P _s (kPa)	i _s (kPa.ms)	t ⁺ (ms)
Front (R - 0.75m)	CFD	77.3	363	12.4	163.5	479	8.8
	Trial	72.6	341	15.1	170.8	460	10.2
	+/- %	6%	6%	-18%	-4%	4%	-13%
Mid (R)	CFD	71.7	323	12.8	144.8	460	9.4
	Trial	71.3	294	14.3	169.8	425	9.9
	+/- %	1%	10%	-11%	-15%	8%	-5%
Rear (R + 0.75m)	CFD	66.8	340	13.1	129.4	443	9.9
	Trial	71.8	332	14.3	133.4	429	11.1
	+/- %	-7%	2%	-8%	-3%	3%	-11%

As discussed in chapter 4, the scaling approach adopted within this thesis relies upon the representation of masonry as an inertial resistance to blast loading, indicating an emphasis on the accurate delivery of blast energy. This infers that an acceptability level of $\pm 10\%$ for impulse in both the prototype and model CFD solutions across the stand-off range represents an acceptable level of accuracy for modelling the transmission of blast energy to masonry. Based on these accuracy levels for CFD in the free-field environment, it was possible to remap reflected pressure data into AEM models of masonry response, thus configuring blast loading. Figures 7.17-7.10 illustrate the reflected pressure and impulse utilised to configure AEM loading for each gauge location detailed in section 7.2.2. An additional consideration for blast load configuration within AEM was the potential application of a

pressure gradient to each of the structures. This behaviour is illustrated in figures 7.11-7.14 via time-lapse progressions of CFD blast wave interaction with masonry structural geometries. Closer inspection of these figures shows that the notional blast thickness is relatively small versus the masonry structures. As a result, it can be seen that a pressure gradient is applied to the structures as the wave progresses over and beyond the structures, indicating agreement with the predicted behaviour detailed in chapter 5.

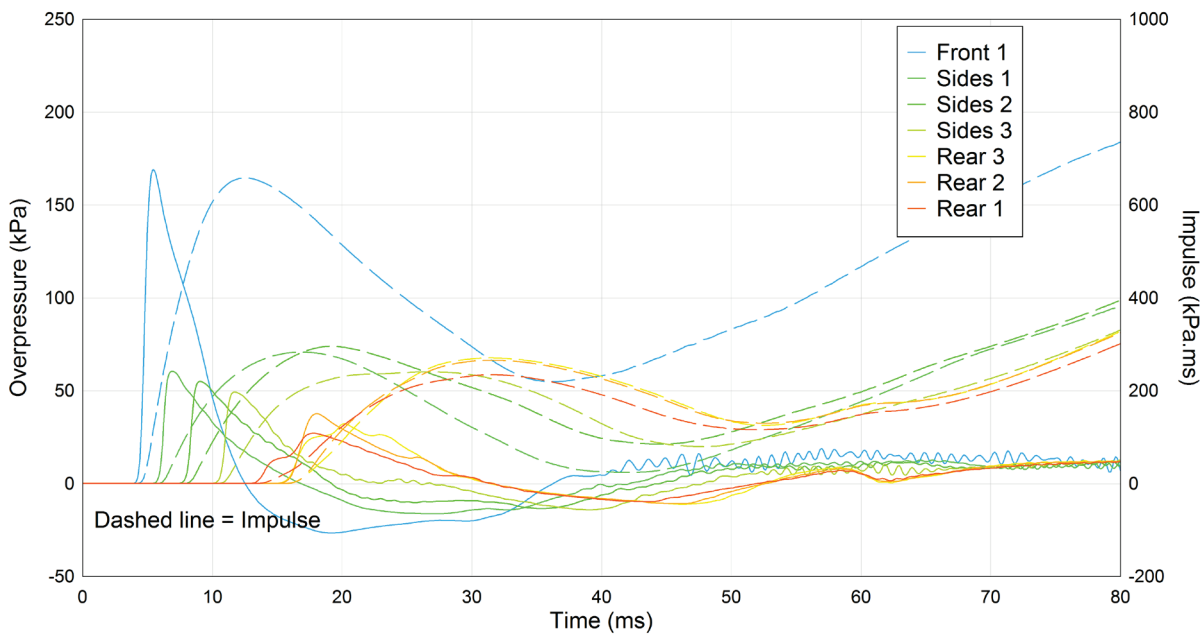


Figure 7.7: CFD reflected pressure and impulse, prototype at 20m.

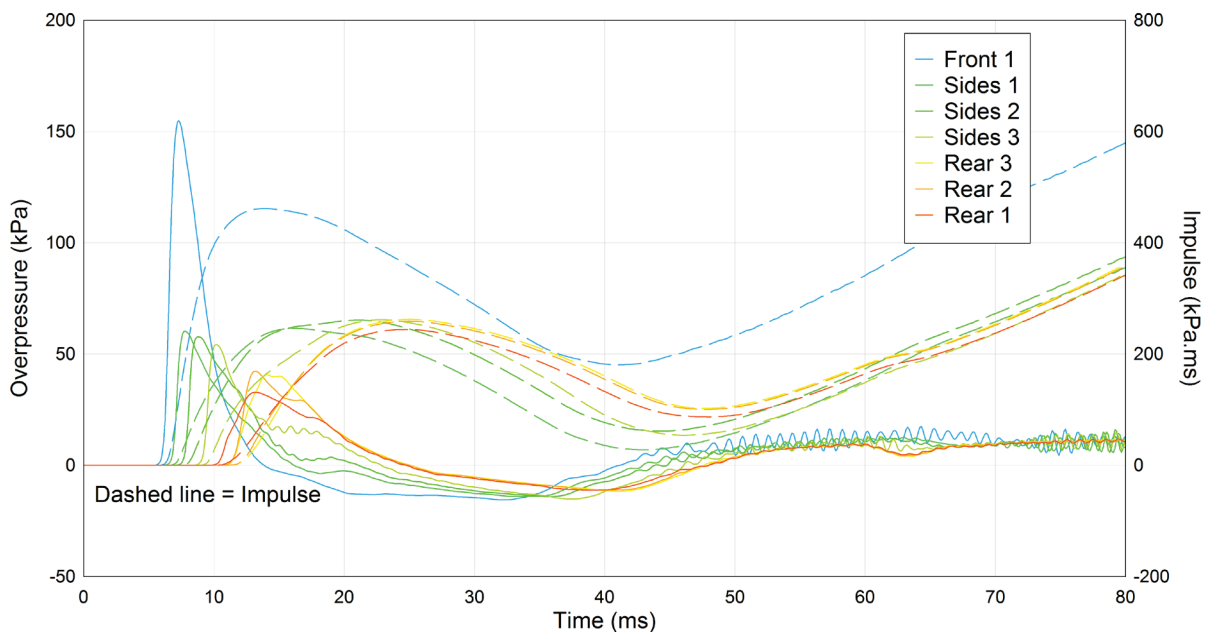


Figure 7.8: CFD reflected pressure and impulse, model at 20m.

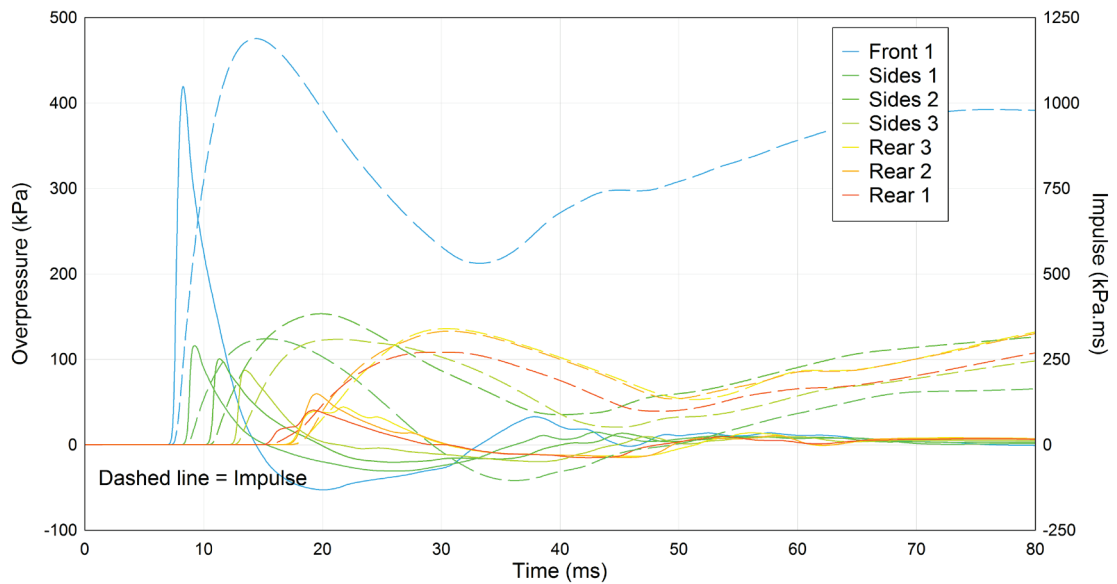


Figure 7.9: CFD reflected pressure and impulse, prototype at 14.25m.

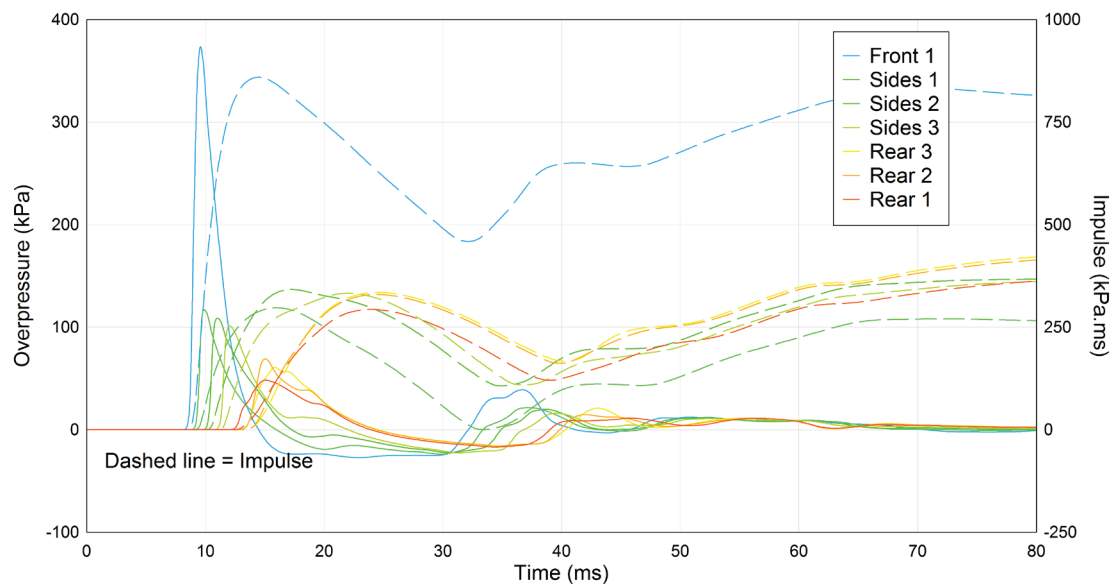


Figure 7.10: CFD reflected pressure and impulse, model at 14.25m.

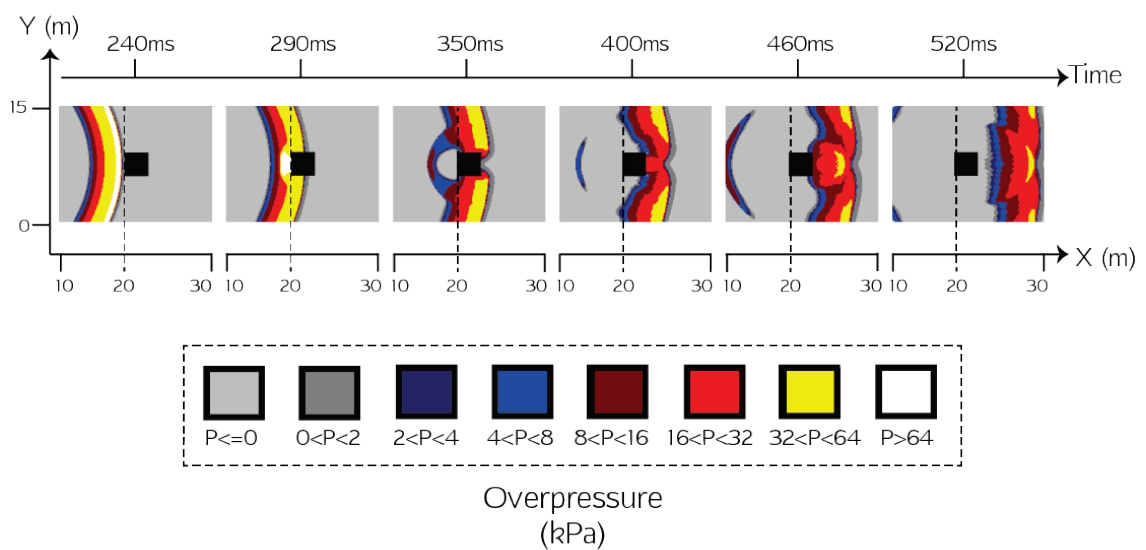


Figure 7.11: Time-lapse of CFD blast wave progression around prototype at 20m.

Scaling Masonry Structures in a Blast Environment

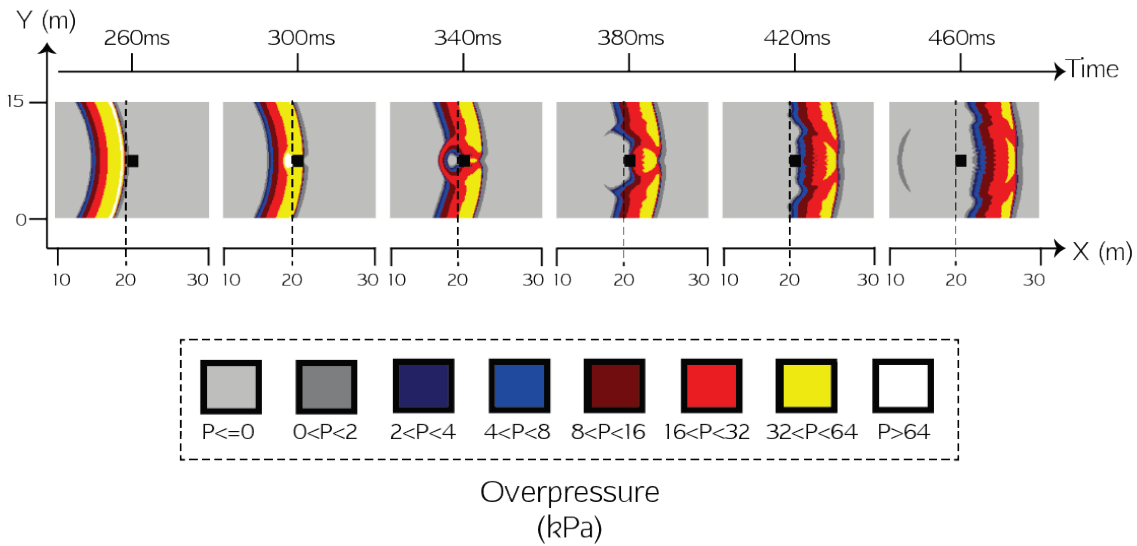


Figure 7.12: Time-lapse of CFD blast wave progression around model at 20m.

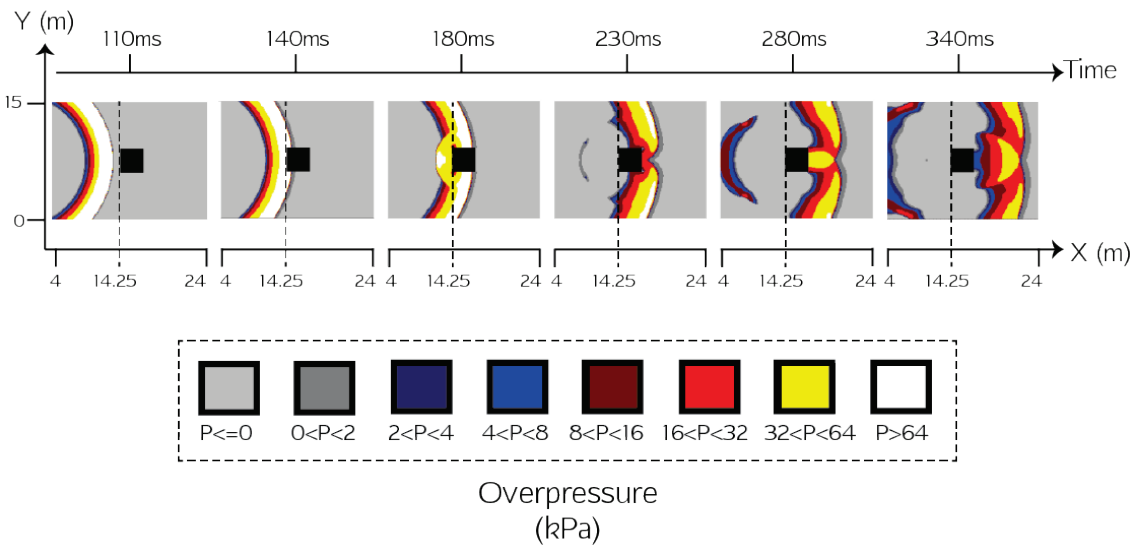


Figure 7.13: Time-lapse of CFD blast wave progression around prototype at 14.25m.

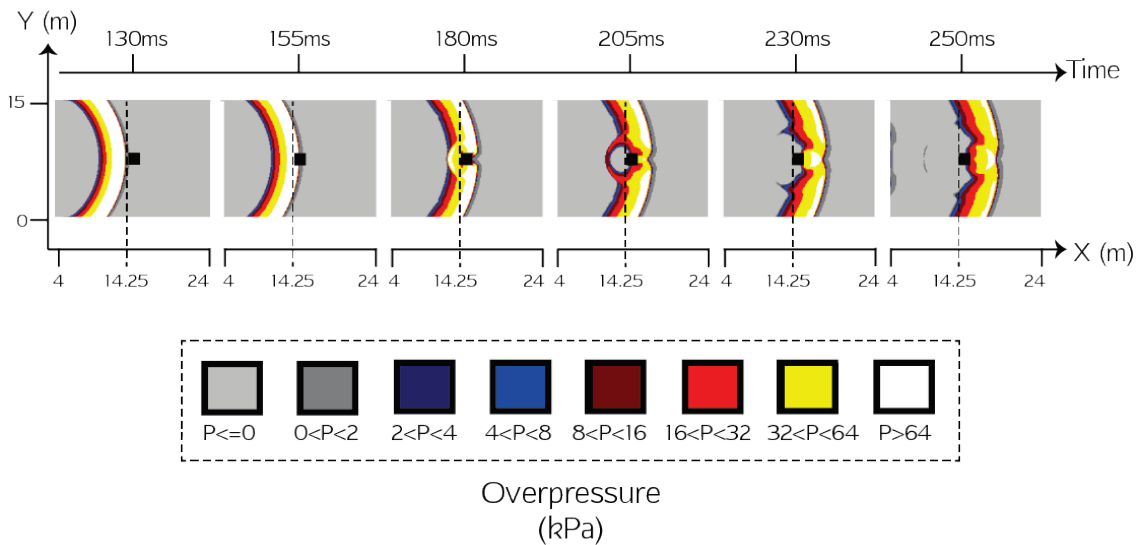


Figure 7.14: Time-lapse of CFD blast wave progression around model at 14.25m.

7.3 AEM Modelling

AEM analyses were conducted and benchmarked against masonry response data for the prototype and model structures fielded within the short-duration blast trial detailed in chapter 6. Each AEM simulation was conducted with the explicit Lagrangian solver, Extreme Loading for Structures (ELS) via a dedicated quad-core Intel i7-3840QM 2.8GHz system with 16GB RAM. The utilisation of AEM to model masonry response to blast loading is an essential part of this research's computational methodology as it can potentially provide a predictive capacity when experimental trials are either not available or not feasible, thus fulfilling the third objective and secondary aim of this PhD.

7.3.1 AEM methodology

AEM solutions were produced to analyse model and prototype masonry response to short-duration blast utilising the structural design detailed in chapter 5 as shown in figures 7.15 and 7.16. Closer examination of figure 7.16 illustrates the implementation of half-thickness stretcher bond construction within these AEM simulations to fully replicate the masonry construction of the experimental trial. The blast loading of each AEM simulation was configured by re-mapping reflected pressure data produced by the CFD models which were detailed in section 7.2.3. Specifically, this was implemented by dividing the masonry structures into loading regions which match the zones demonstrated in section 7.2.2. The individual pressure histories were subsequently mapped onto each structural surface zone to configure the application of dynamic blast loading. Each AEM model utilised the material parameters shown in table 7.9 where the physical properties replicate those observed for the experimental trial. The separation strain coefficient within table 7.9 represents the default ELS setting for the onset of concrete cracking due to tensile strain. Table 7.10 details the element quantities for each AEM simulation in conjunction with solution time frames. Closer inspection of table 7.10 shows that the 20m stand-off models utilised 1 second problem times versus 4 seconds for the 14.25m models to reflect the reduced degree of masonry response at the greater stand-off.

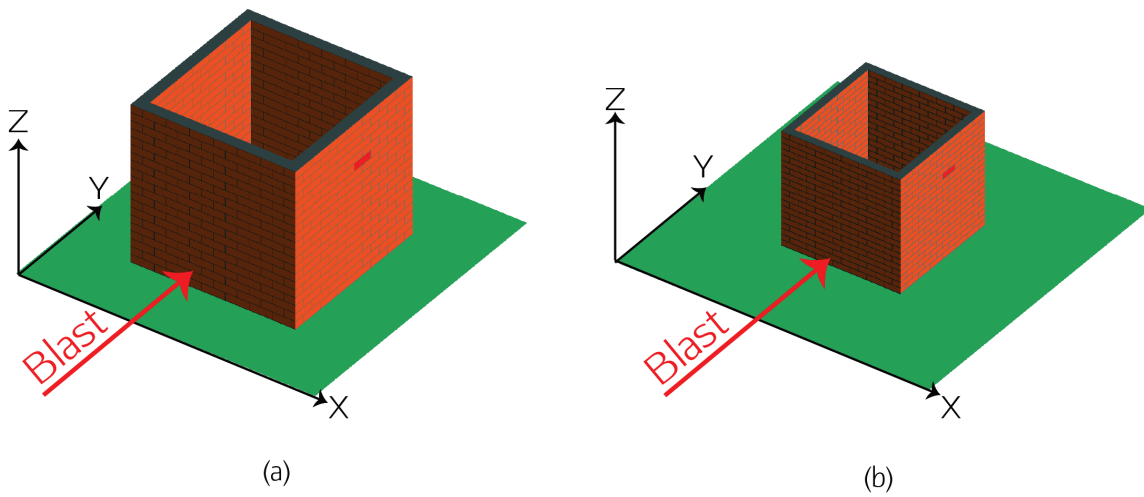


Figure 7.15: ELS masonry structure visualisation in isometric view: (a) Prototype; (b) Model.

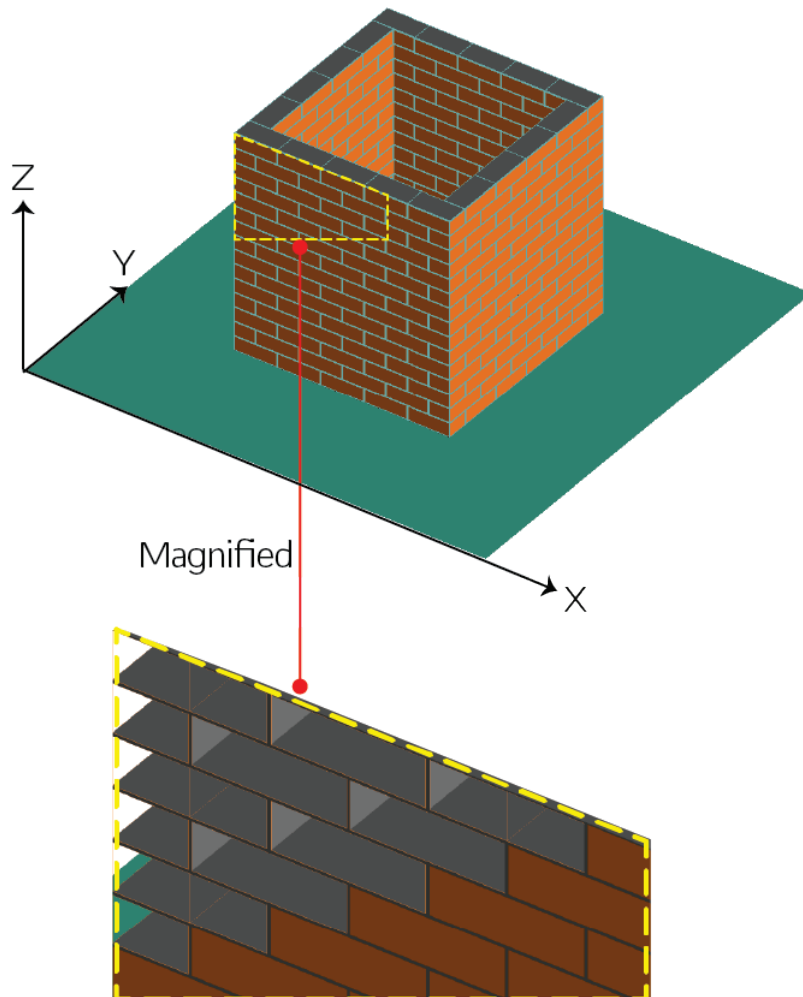


Figure 7.16: ELS masonry structure stretcher bond visualisation with magnified view.

Table 7.9: ELS material parameters for masonry simulations.

	Density (kg/m ³)	Young's Modulus (GPa)	Shear Modulus (GPa)	Compressive Strength (MPa)	Tensile Strength (MPa)	Separation Strain
Concrete Blocks	1055	26.2	10.5	7.3	0.73	0.1
Concrete Bricks	2060	26.2	10.5	20	2	0.1
Mortar	1720	26.2	10.5	4	0.6	0.1

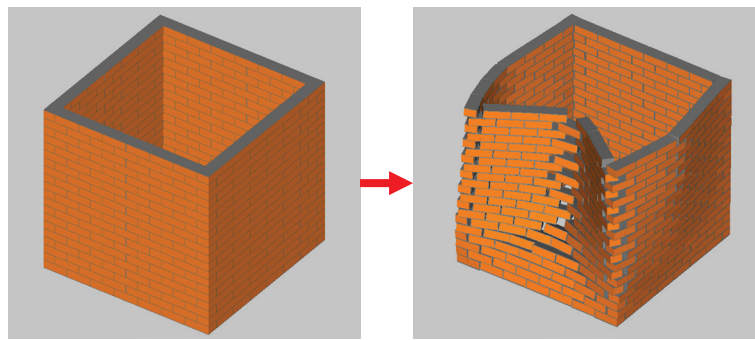
Table 7.10: ELS solution parameters for masonry simulations.

		Whole Brick Elements	Half Brick Elements	Mortar Elements	Problem Time (s)	Solution Step (μ s)
20m stand-off	Prototype	400	240	2080	1	100
	Model	400	240	2080	1	100
14.25m stand-off	Prototype	400	240	2080	4	100
	Model	400	240	2080	4	100

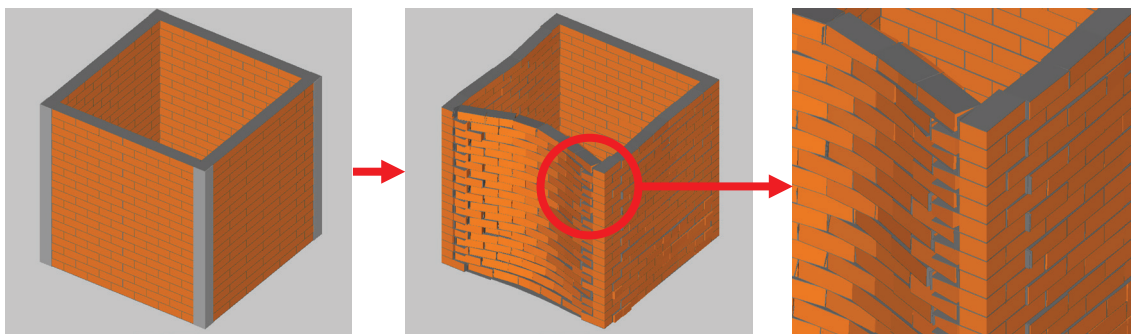
7.3.2 AEM solution analysis

As discussed throughout this research, the sequential phases of masonry response can each be shown to exert an influence over the next stage, with the first being represented by the onset of cracking as a function of blast loading and structural edge restraint. It was therefore essential to ensure that AEM simulations of masonry response to short-duration blast loading correctly replicated the phases of response observed within the PhD blast trial.

An initial examination of AEM's ability to accurately represent edge restraint and in particular, the implementation of fully restrained corners indicated that the composite stiffness achieved via real-world masonry construction was not natively modelled within AEM. As a result, it was found that the 'built in' corners would not represent a moment resisting connection, but rather the individual elements were found to pivot in position as shown in figure 7.17a. As a result, it could be seen that the moment resistance which led to the initiation of masonry unit splitting and the sequential stages of cracking, deflection and collapse was not being modelled. In order to remove this limitation, a non-responding steel-frame structure was modelled at each of the corners to simulate a fully-restrained edge, moment-resisting edge condition as illustrated in figure 7.17b.



(a)



(b)

Figure 7.17: AEM modelling of masonry corners: (a) Unrestrained; (b) Restrained.

The implementation of restrained corners provided the necessary moment resistance at these structural locations to enable the onset of masonry unit splitting after the application of blast loading. This is shown in figures 7.18a-d where it can be seen that the front panel corners of the prototype and model structures at 14.25m exhibited vertical primary cracking behaviour. Despite this initial agreement, additional examination of these AEM solutions indicated that the energy required to initiate masonry unit splitting at the front corners resulted in a damping effect on the degree of front panel deflection for both the prototype and model at 14.25m stand-off. The result of which was limited additional cracking at the centre of the front panels and no front panel collapse. Examination of the side panel structures also showed zero primary cracks and minimal deflection, preventing the onset of partial collapse in both the prototype and model solutions. Inspection of the rear panels of both structures was also found to show zero primary cracks after the application of blast loading. The net effect of these cracking and deflection behaviours was a final damage state that differs from the experimentally observed results detailed in chapter 6.

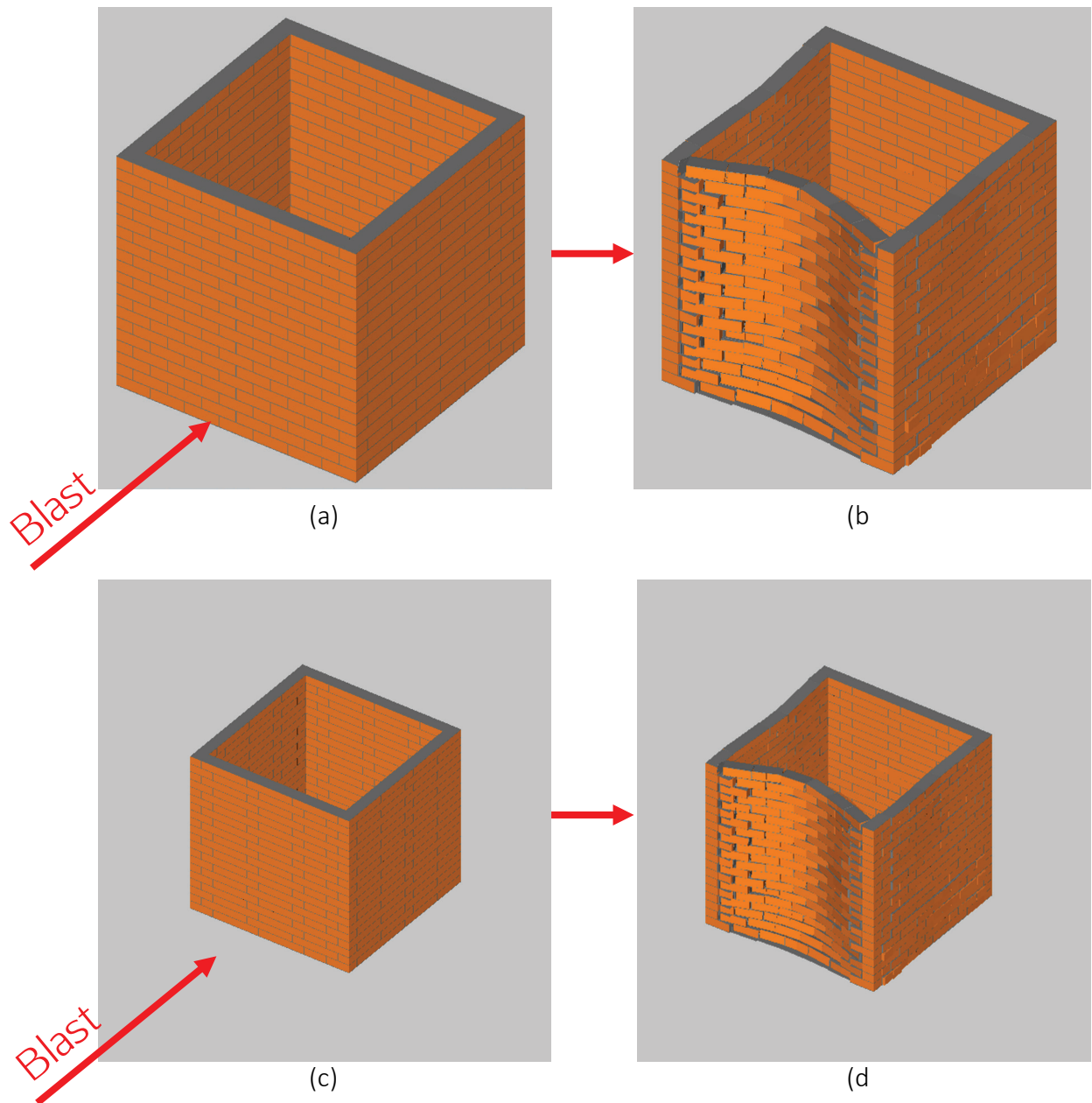


Figure 7.18: AEM masonry damage results at 14.25m stand-off: (a) Prototype initial state; (b) Prototype final damage state; (c) Model initial state; (d) Model final damage state.

AEM solutions were also produced for the lower blast impulse scenario at 20m stand-off distance by implementing the fully restrained corner conditions described above. The analysis of which showed differing global damage behaviour to the experimentally observed masonry results. This is demonstrated in figures 7.19a-d where it can be seen that both the prototype and model were not subject to any primary vertical cracking at the centre of the front panel, the front corners or at the side panels. These AEM solutions also indicate zero deflection at the front panel alongside zero deflection at the remaining structural faces.

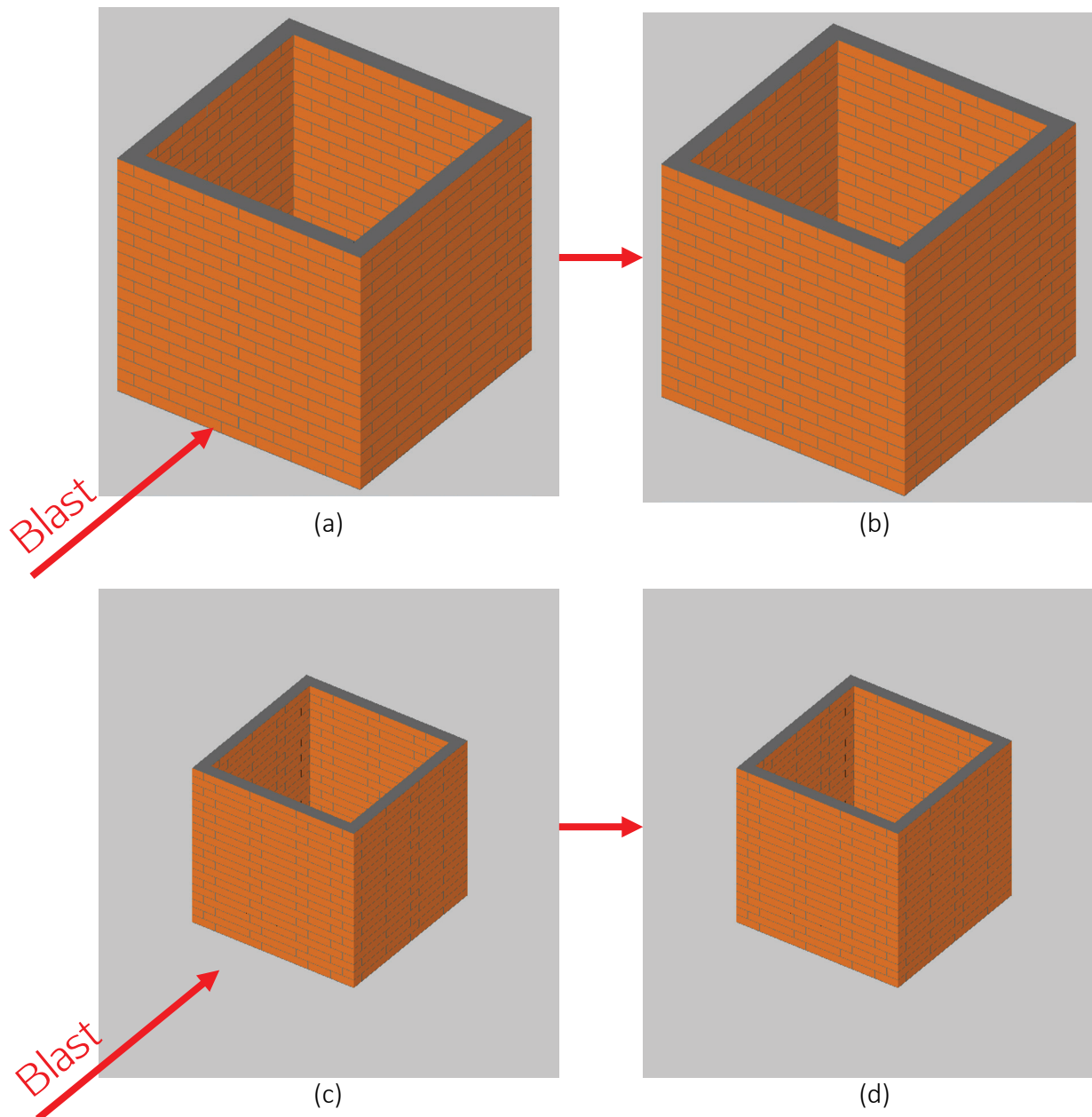


Figure 7.19: AEM masonry damage results at 20m stand-off: (a) Prototype initial state; (b) Prototype final damage state; (c) Model initial state; (d) Model final damage state.

As previously described, the ability to accurately model the individual stages of response in the correct sequence underpins the ability to model the actual response observed within a short-duration blast scenario. At present, the native modelling of fully restrained masonry corners within AEM does not accurately represent the stiffness and moment resistance associated with this edge condition. In an effort to amend this, the application of a rigid edge restraint condition was found to provide an increased degree of stiffness and moment resistance to enable the onset of primary cracking with the higher

impulse scenario at 14.25m stand-off. Despite this, it was also found to produce an unrealistic damping effect on the application of blast energy, the result of which is an underestimation of masonry damage state at both 14.25m and 20m distances.

As a result of the masonry damage state analysis, it has been found that that AEM models of masonry response to short-duration blast loading require an intermediate 'patch' which occupies the region between the two edge restraint extremes described above. This can be described as a semi-rigid edge condition which simulates moment resistance up to a predefined limit, followed by the removal of the edge restraint to limit the energy damping effect associated with the current moment resisting edge condition. An alternative methodology available in some FE packages represents the pausing of a dynamic model partway through a loading scenario to enable the re-mapping of the structure into a new simulation. This is not currently possible with ELS due to the progression of linear continuous structures into non-linear discrete geometries which underpins AEM's ability to represent masonry damage and automatic generation of debris. As a result of the damage model and the subsequent transition into a discrete phase, it can be shown that there is a commensurate increase in complexity when aiming to accurately apply blast loading to a partially damaged structure.

Consequently this research has identified that future work may be able to overcome the current limitations by investigating the development of a frangible corner restraint element which integrates a pre-defined stiffness and separation parameter. Thus enabling the AEM model to provide the required rigidity for a 'built in' corner before removing this stiffness to limit the effect of damping on the remaining aspects of masonry response. The development of this element would require a parametric study to determine whether the corner stiffness element should utilise constant material properties or if they should be varied as a function of blast energy or other essential parameters. It is therefore likely that this parametric study would require benchmarking via a survey of available literature or lab testing which is substantially more cost effective than a range of blast trials.

As a result of the aforementioned accuracy limitations for masonry damage state at both 14.25m and 20m stand-off distances, it can be shown that the result will be a sequential influence on the generation of debris fields. This was highlighted by the masonry damage

assessment discussed above which showed minimal panel deflection for both stand-off locations, thus preventing the onset of deflection and collapse. In an effort to assess the capability of AEM to generate a discrete debris distribution, additional simulations were conducted which utilised the native model of limited edge restraint, thus representing an examination of masonry response at the other end of the identified rigidity spectrum. In doing so, it was found that masonry damage was greatly amplified for prototype and model structures at both the 14.25m and 20m stand-off locations. Figures 7.20-7.22 illustrate the time-lapse progression of masonry damage for an unrestrained edge model at 14.25m stand-off. The examination of which demonstrates a pivoting action at the corner of each prototype and model structure, resulting in the prevention of the 'snapping' action which is observed with the experimental structures due to the true level of moment resistance.

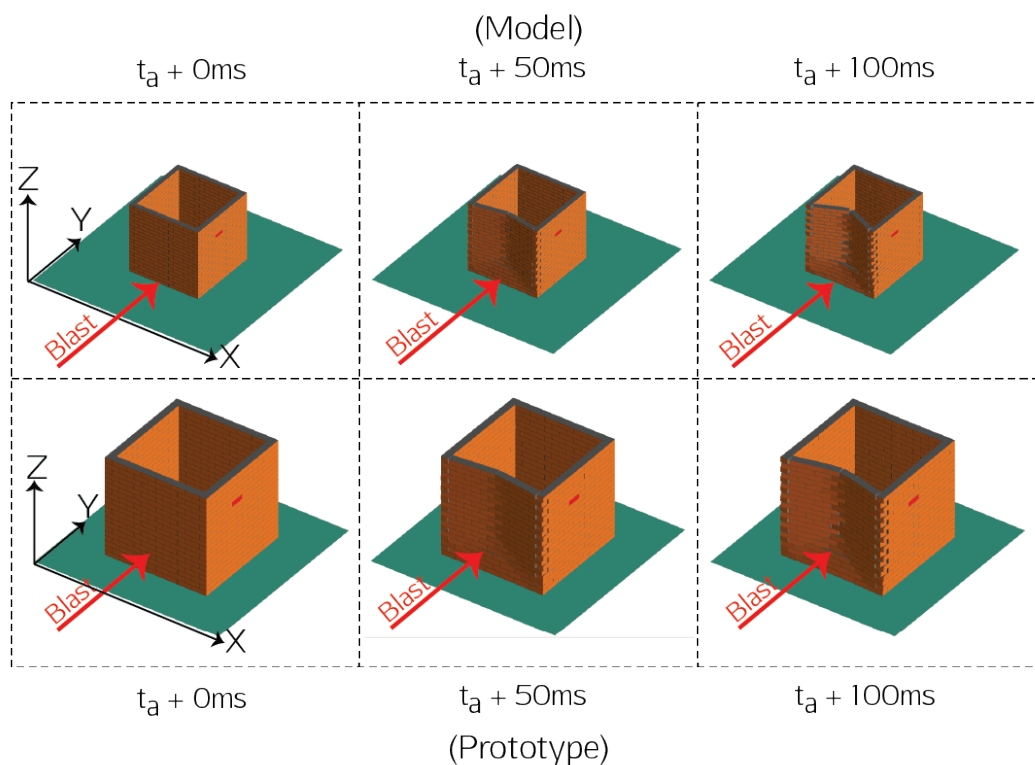


Figure 7.20: AEM masonry response, model & prototype 0-100ms, 14.25m stand-off.

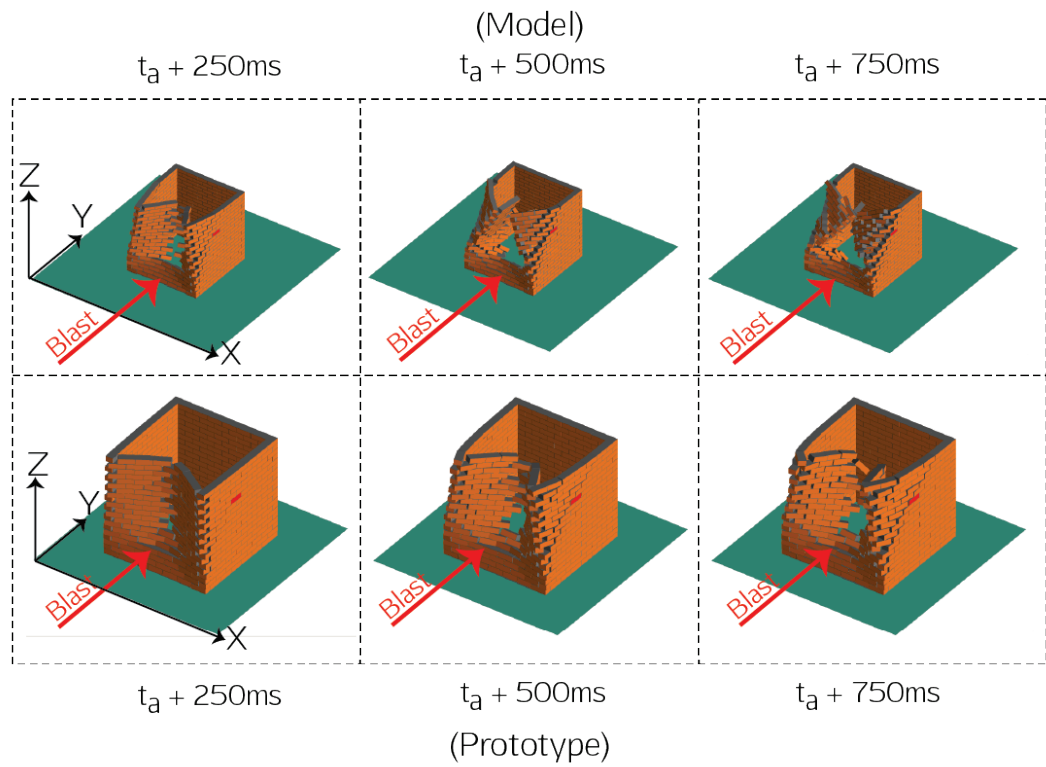


Figure 7.21: AEM masonry response, model & prototype 250-750ms, 14.25m stand-off.

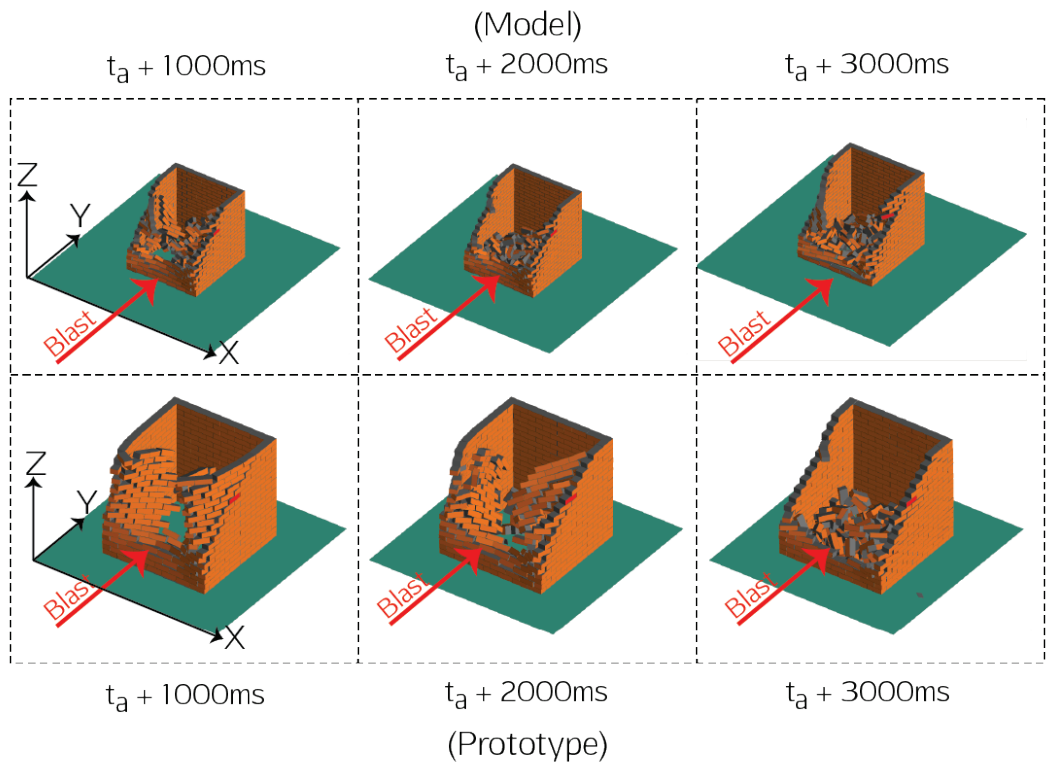


Figure 7.22: AEM masonry response, model & prototype 1000-3000ms, 14.25m stand-off.

Closer inspection of figure 7.22 indicates that the higher relative impulse scenario at 14.25m stand-off generated a debris distribution which is fully located within the internal structural area. It can also be seen that the majority of the front panel collapses in tandem with partial side panel collapse for both the prototype and model. As a result, it can be shown that the AEM model solution was able to qualitatively replicate the damage state and debris generation of the AEM prototype solution. It can also be seen that the model and prototype solutions qualitatively replicated the number and location of collapse panels seen within the experimental trial. In contrast, it can be seen that the AEM distributions of debris diverge from experimental data due to the confined, internal debris spread with the AEM results. This difference can be traced to an inaccurate representation of crack mechanisms for each of the AEM masonry corners which in turn, modified the onset of front, side and rear panel cracking. The result of which is an inaccurate characterisation of front and side panel collapse, leading to an inaccurate debris spread. These results therefore highlight the essential need to accurately model the sequential phases of response to produce a realistic debris distribution. These results also demonstrate that by implementing a reduced degree of edge restraint, AEM was able to qualitatively replicate the quantity and location of collapsed panels, inferring a capability to partially simulate damage state.

Figures 7.23-7.24 illustrate a time-lapse progression of masonry response for the lower impulse scenario at 20m stand-off distance. The examination of which demonstrates that the AEM solution produced moderate levels of front panel deflection with zero collapse for both the prototype and model structures. In this instance, it can be inferred that the pivoting action that exists between brick elements and interfacing mortar material introduces a frictional resistance which likely reduced the extent of deflection and potential collapse. Another source of potential divergence for AEM solutions and experimental results at this stand-off relates to varying degrees of experimental roof enclosure interaction as identified within chapter 6. The result of which was partial collapse for the experimental prototype and no collapse for the experimental model. It can thus be surmised that the AEM solutions qualitatively replicated the structural survival and thus partial damage state of the experimental model rather than the experimental prototype. As a result, it can be shown that both the AEM and experimental data sets for masonry damage and potential collapse were subject to uncertainty caused by unintended frictional interactions, the former due to

element pivoting and the latter due to the roof enclosure. Despite the differences identified between AEM and experimental results, it can be shown that the AEM model structure qualitatively replicated the damage response and lack of debris of the AEM prototype.

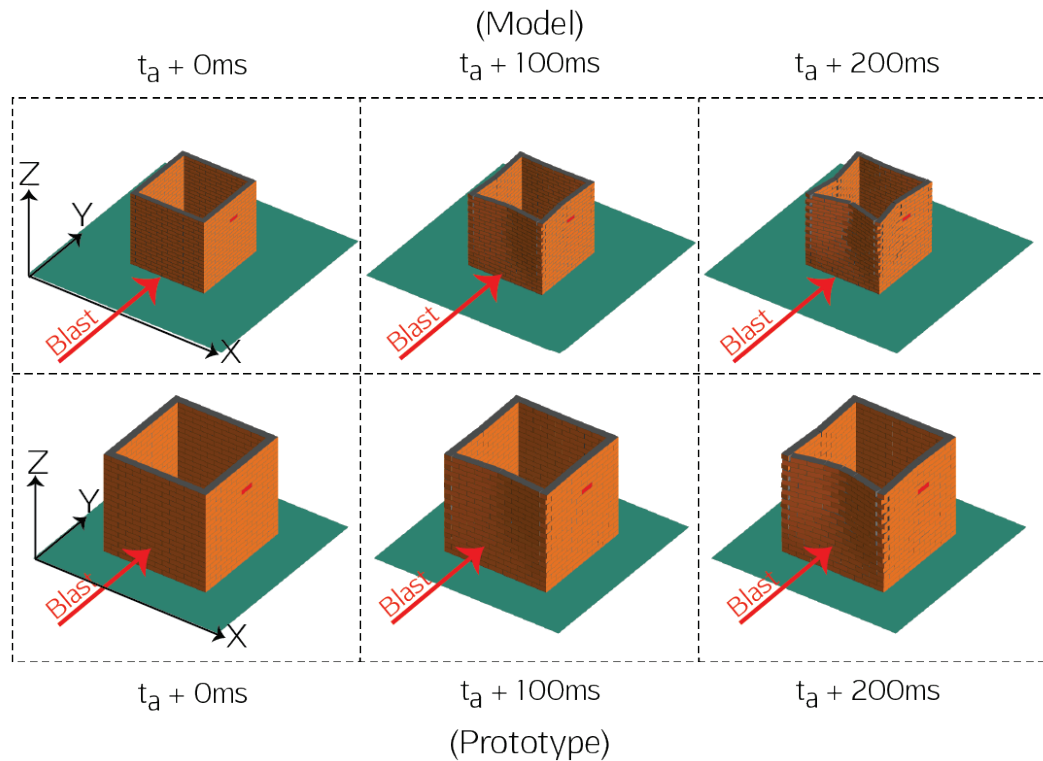


Figure 7.23: AEM masonry response, model & prototype 0-200ms, 20m stand-off.

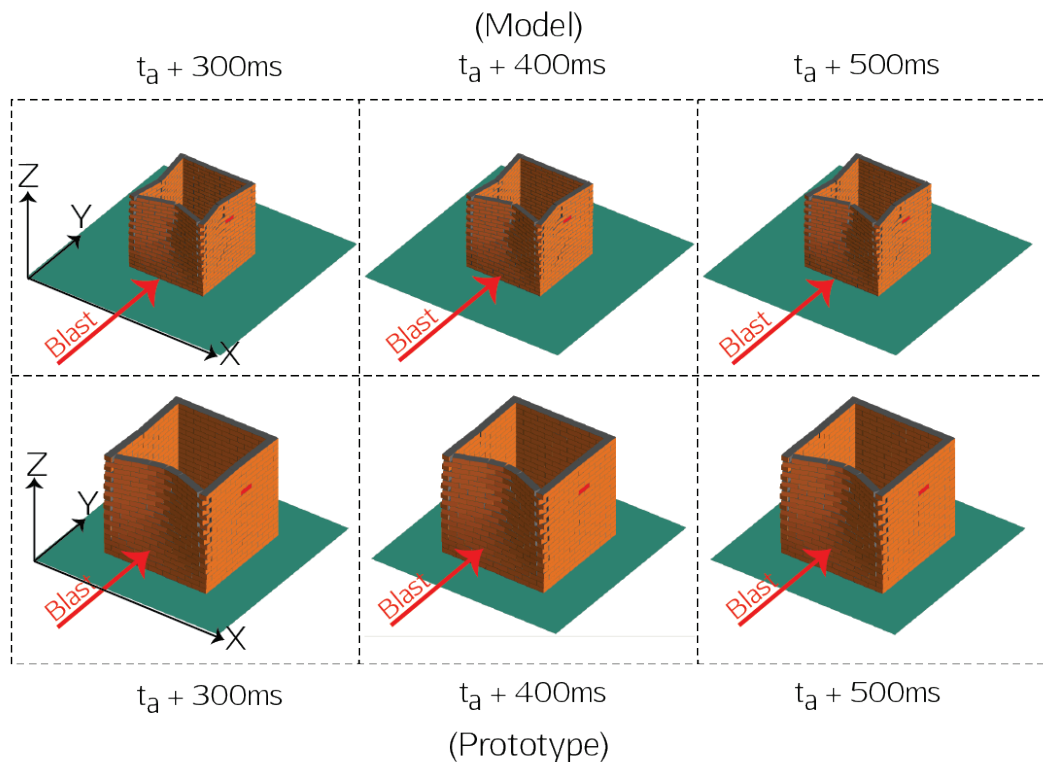


Figure 7.24: AEM masonry response, model & prototype 300-500ms, 20m stand-off.

By producing AEM solutions which implement either a fully rigid representation of the corner restraint or a native implementation which enables inter-element pivoting rather than cracking, AEM solutions have shown variable degrees of potential for the replication of experimental masonry response. Specifically, it was shown that the onset of vertical primary cracking can be modelled when implementing rigid corner restraints albeit with an over-damping effect on system response. It was also found that the implementation of native ‘flexible’ corner restraints enabled the onset of panel deflection, collapse and the subsequent generation of a debris field. It is clear therefore that the desired AEM solution represents the integration of both these response capabilities with the correct sequential application to accurately simulate experimental masonry response. This research has subsequently identified the need for future work to develop a methodology which implements a ‘semi-rigid’ and removable corner restraint with the required stiffness and moment resistance to enable cracking and subsequent phases of response whilst preventing the system from being over-damped for a short-duration blast scenario.

Despite the observed variability in AEM solution accuracy versus experimental data, AEM model solutions have been found to replicate the damage state and debris distribution of AEM prototypes. Thus inferring that the underlying physics of the scaling methodology have been correctly implemented within the AEM solutions. This analysis can therefore be extended to include the cross-examination of masonry debris generated by the model and prototype AEM solutions at the 14.25m stand-off as shown in table 7.11.

Table 7.11: AEM rubble distribution, model & prototype, 14.25m stand-off.

Structure	Wall Panel	Number of Blocks	Equivalent Mass	% Total Mass
Model	Front	110	369	21%
Prototype		101	1548	19%
			+/- %	+2%
Model	Left	37	124	7%
Prototype		31	475	6%
			+/- %	+1%
Model	Right	36	121	7%
Prototype		30	460	6%
			+/- %	+1%
Model	Sum	183	613	35%
Prototype		162	2483	31%
			+/- %	4%

Owing to the fact that the debris was confined to the internal structure areas, table 7.11 compares the relative percentages of total mass within the AEM debris field as a function of the original panel wall position. Closer examination of table 7.11 shows that the model structure debris data closely matches the prototype with minimal differences of +2%, +1% and +1% for the debris distribution from the front, left and right panels respectively. The culmination of which is a minor difference of +4% of total structural mass within the debris distribution for the model structure versus the prototype. As a means of further comparing these debris data sets, table 7.12 details the results of a Chi-square goodness of fit test for the discrete distributions shown in table 7.11. In this case, a significance level of 5% has been selected with the null hypothesis defined as “the model distribution matches the prototype distribution” whilst the alternative hypothesis has been defined as “the model distribution does not match the prototype distribution”.

Table 7.12: Chi-square goodness of fit, AEM rubble, model & prototype, 14.25m stand-off.

Chi-Square Goodness of Fit Test			
Panel	Model = Observed (O)	Prototype = Expected (E)	(O-E) ² /E
	Number of Bricks	Number of Blocks	
Front	110	101	0.80
Left	37	31	1.16
Right	36	30	1.20
		Sum = χ^2	3.16
		P-value	20.6%

Examination of table 7.12 reveals a P-value of 20.6%, which indicates that there is insufficient evidence to reject the null hypothesis. Thereby adding weight to the argument that the model debris distribution matches the prototype. As a result, it can be shown that the AEM model solution further replicates the AEM prototype when implementing this research’s scaling methodology.

7.4 Summary

This chapter has detailed the benchmarking of CFD models of blast flow and AEM models of masonry response against experimental data captured during the short-duration blast trial detailed in chapters 5-6, thus fulfilling the third objective of this research.

This chapter commences with a comparative study of Air3D and AUTODYN CFD solvers for modelling the experimental blast environment. AUTODYN implemented 100kg TNT with a physically cylindrical charge shape with 1:1.6 aspect ratio whilst Air3D implemented 134kg TNT with a physically spherical charge. Examination of free-field results showed mean differences of +3% and -6% for pressure and impulse or Air3D versus the trial compared to +14% and +18% with AUTODYN. CFD solutions were re-mapped to produce 3D models of masonry structural interaction with the blast wave. Air3D results for reflected blast data showed negligible differences of -1% and -4% for pressure and -2% and -17% for impulse when compared to trial results. AUTODYN produced larger differences in both pressure and impulse with three of the four data points exceeding $\pm 20\%$. The combination of stronger agreement for reflected and free-field solutions showed that Air3D more closely replicated the experimental blast environment via 134kg TNT with a spherical shape. In particular, the demonstrated agreement for impulse inferred a higher likelihood of reproducing the blast momentum transmitted to masonry structures within the blast trial which is essential to the scaling method's representation of masonry as an inertial blast resistance. This section then detailed the production of CFD solutions with Air3D to generate reflected blast data which could be re-mapped to configure blast loading within AEM models of masonry response. This was preceded by closer examination of free-field parameters at each stand-off location to show that the impulse results which met the acceptability criteria of $\pm 10\%$ versus the trial.

The next section of this chapter examined AEM models of masonry response to short-duration blast loading by implementing the structural design detailed in chapter 5 with the AEM solver, ELS. The blast loading of each AEM simulation was configured by re-mapping reflected pressure data produced by CFD models. An initial emphasis was placed on the accurate modelling of individual stages of masonry response in the correct sequence as this has been shown to be critical to the final damage state and debris results. Initial examinations of showed that AEM did not accurately represent the stiffness and moment resistance of fully restrained corners as individual elements were found to pivot, preventing masonry unit splitting and the sequential stages of cracking, deflection and collapse. A non-responding steel-frame was subsequently modelled to simulate the required corner stiffness and moment resisting capacity. This was found to enable masonry unit splitting at front panel corners of prototype and model structures at 14.25m stand-off. This edge condition was also

found to produce a damping effect on response which limited cracking at the front panel centre and prevented front panel collapse. This damping was also found to prevent side panel cracking, deflection and partial collapse alongside zero rear panel cracking in both the prototype and model solutions. The net effect which was a differing damage state to experimental results. Analysis of prototype and model response was also conducted at the 20m stand-off distance with fully restrained conditions. These results were found to showed zero cracking at the front panel and the side panels in tandem with zero deflection, thus indicating differing global damage behaviour to the experimental masonry results.

As a means of assessing AEM's generation of debris distributions, additional simulations were conducted to utilise the native model of limited edge restraint, resulting in amplified degrees of masonry damage for prototype and model structures at the 14.25m stand-off. As a result, a large proportion of the front panel was found to collapse in tandem with partial side panel collapse to generate an internal debris spread in both the AEM prototype and model solutions. This showed that the AEM model solution qualitatively replicated the damage and debris of the AEM prototype. It was also demonstrated that the AEM model and prototype qualitatively replicated the quantity and location of collapsed panels observed within the blast trial. It was also found that AEM solutions diverged from the experimental data as a by-product of inaccurate crack mechanisms which affected panel collapse to produce debris which was fully confined to the internal area.

Similar analyses were conducted for prototype and model structures at 20m stand-off, demonstrating an increase in front deflection with zero collapse. It was also found that the pivoting action between brick elements and interfacing mortar may have introduced frictional resistance which reduced the extent of deflection and potential collapse. Despite this, it was shown that AEM solutions qualitatively replicated the structural survival and partial damage of the experimental model versus the partial collapse of the experimental prototype, indicating variable levels of agreement. In contrast, it was shown that the AEM model structure qualitatively replicated the damage response and lack of debris of the AEM prototype, indicating similar agreement to that observed for AEM models at 14.25m. These results inferred that the underlying physics of the scaling methodology were correctly implemented within the AEM solutions. This was further demonstrated by a comparison of masonry debris for model and prototype AEM solutions at 14.25m. The results of which

showed minimal differences for the debris distribution from the front, left and right panels when expressed as relative percentages of total mass, producing a difference of +4% of total mass for the model versus the prototype. A Chi-square goodness of fit test was also produced for the discrete debris distributions with a significance level of 5%. This analysis showed a P-value of 20.6% to indicate that the model debris distribution matched the prototype.

By producing AEM solutions which implement either a fully rigid representation of the corner restraint or a 'flexible' native implementation, this chapter has found variable potential for replicating experimental masonry response. Specifically, it was found that rigid restraints enabled primary cracking albeit with an over-damping effect whilst 'flexible' corners enabled panel deflection, collapse and debris generation. It can thus be shown that the desired solution integrates both response capabilities with the correct sequential application to simulate masonry response. This research has subsequently identified the need for future work to develop a 'semi-rigid' and removable corner restraint method which provides the necessary stiffness and moment resistance to enable cracking in tandem with subsequent phases of collapse and debris without over-damping the system. In the interim, it can be shown that experimental trials are essential for the assessment of masonry response to short-duration blast loading but AEM may be able to provide a predictive capacity after further investigation into the aforementioned modification to edge restraint modelling.

Chapter 8

Long-duration Blast: Methodology and Modelling

8.1 Introduction

This chapter commences with a detailed summary of four experimental trials which have been devised in collaboration with the industrial sponsor, AWE, to benchmark the masonry scaling methodology detailed in chapter 4 for long-duration blast events at a 1:2 scaling level, thereby fulfilling the fourth and final objective of this research. These trials will be conducted in the future at the UK national blast test facility, the Air Blast Tunnel (ABT) at MOD Shoeburyness. This is one of a small number of facilities capable of simulating full-scale long-duration blast waves and their complex effects on structures. Whilst the ABT provides 4.9m and 10.2m diameter testing regions, the 33% limit on a structure's frontal area restricts the range of testable masonry structures at full-scale. These trials will aim to circumvent the ABT's spatial limitations by demonstrating the capability of this PhD's scaling methodology. This will be achieved by comparing a total of two prototype and two model structures at two overpressures regions within the ABT's 4.9m testing section. These trials were originally intended to be conducted during this PhD however the extension of an essential sponsor experimental programme produced a temporary postponement beyond the research period.

The second section of this chapter represents a computational investigation into the potential performance of the scaling methodology within a long-duration blast scenario, thus fulfilling the fourth objective of this research and addressing the secondary aim. Preliminary CFD and AEM results are presented to enable an initial examination of masonry damage and debris distribution for the experimental design detailed in this chapter. The utilisation of CFD

provides the capability to generate blast flow data which can be re-mapped to configure masonry loading within AEM. As a means of determining the most appropriate CFD solver for long-duration blast, a comparative study was produced to compare Air3D and AUTODYN. This study is presented in full within Appendix A where it can be seen that the relative accuracy of these CFD packages was compared with free-field and reflected blast data that was experimentally recorded during a series of previous long-duration blast trials (Adams et. Al, 2014). Building upon the AEM benchmarking demonstrated in chapter 7, AEM models of masonry response to long-duration blast have been initially conducted by implementing native corner restraints to enable debris generation. This approach was adopted for the preliminary investigation as debris represents the dominant masonry response feature of long-duration blast events due to the substantial impulses which greatly exceed those recorded in the short-duration trial.

8.2 Methodology

8.2.1 Trial design

A total of four trials are proposed to be conducted within the 4.9m diameter section of the ABT, adopting the global axes shown in figure 8.1. These aim to capture masonry damage and debris data for two pairs of model and prototype structures at two overpressures, enabling the experimental benchmarking of the scaling methodology derived in chapter 4. The ABT is an explosively driven shock-tube which can simulate full-scale long-duration blasts via near-planar shocks (Adams et al, 2012). The ABT's conical shape and approximate length of 200m enables the production of large pulses which are characteristic of a long-duration event.



Figure 8.1: Global axes adopted for scaled masonry trials in the ABT.

The rear of the ABT is equipped with a rarefaction wave eliminator (RWE) which reduces unwanted reflections in the upstream direction. This can be seen in figure 8.2 with the ABT's entry doors partially opened.



Figure 8.2: ABT Rarefaction Wave Eliminator (RWE).

Trials 1-4 will examine simplified residential construction via single-storey enclosed masonry structures with half-thickness stretcher bonds and cuboid dimensions as shown in table 8.1 and figures 8.3-8.4. These masonry dimensions have been modified when compared with the short-duration trial design in chapter 5 to account for the reduced spatial area within the ABT's 4.9m testing section. Non-responding roof enclosures will also be implemented within this trial however the design requires further collaboration with the Foulness Trials Group of Spurpark Ltd to integrate additional stiffness as identified within chapter 6. Design overpressures of 55kPa and 110kPa in trials 1-2 and 3-4 respectively have been selected to ensure masonry failure in each trial whilst enabling variation in maximum fragment throw distances. These peak overpressures were also intended to replicate the design of the short-duration trial in chapter 5, however the amplification in pressure observed in chapter 6 indicates that there will be a slight difference between these design pressure values. By utilising varied blast environments, these trials will aim to generate varied debris distributions over the two overpressure regions.

The generation of a blast wave with the required characteristics to produce the design overpressures will be achieved by utilising 2kg and 4kg of Cordtex (PETN) in the ABT's explosive driver section for trials 1-2 and 3-4 respectively. This will enable trials 1-2 to

simulate an air-blast with TNT equivalence of 200 tonnes at 250m stand-off whilst trials 3-4 will simulate an event with 450 tonnes of TNT at 235m stand-off. These TNT equivalences were calculated via the Kingery and Bulmash predictive polynomials (Kingery and Bulmash, 1984). Each masonry structure will be constructed with a face-on loading orientation within a removable testing section that will be positioned within the 4.9m diameter region of the ABT as shown in figures 8.5-6. These front elevation schematics also provide the pertinent clearance measurements within this section.

Table 8.1: Long-duration blast trial experimental summary.

Trial	Structure Type	Dimensions x, y, z (m)	Bond Type	Design Blast Parameters		
				Overpressure, P_s (kPa)	Impulse, i_s (kPa.ms)	Positive Phase, t^+ (ms)
1	Prototype	2.25 x 2.25 x 2.25	Stretcher	55	3980	206.9
2	Model	1.13 x 1.13 x 1.13	Stretcher			
3	Prototype	2.25 x 2.25 x 2.25	Stretcher	110	6971	220.9
4	Model	1.13 x 1.13 x 1.13	Stretcher			

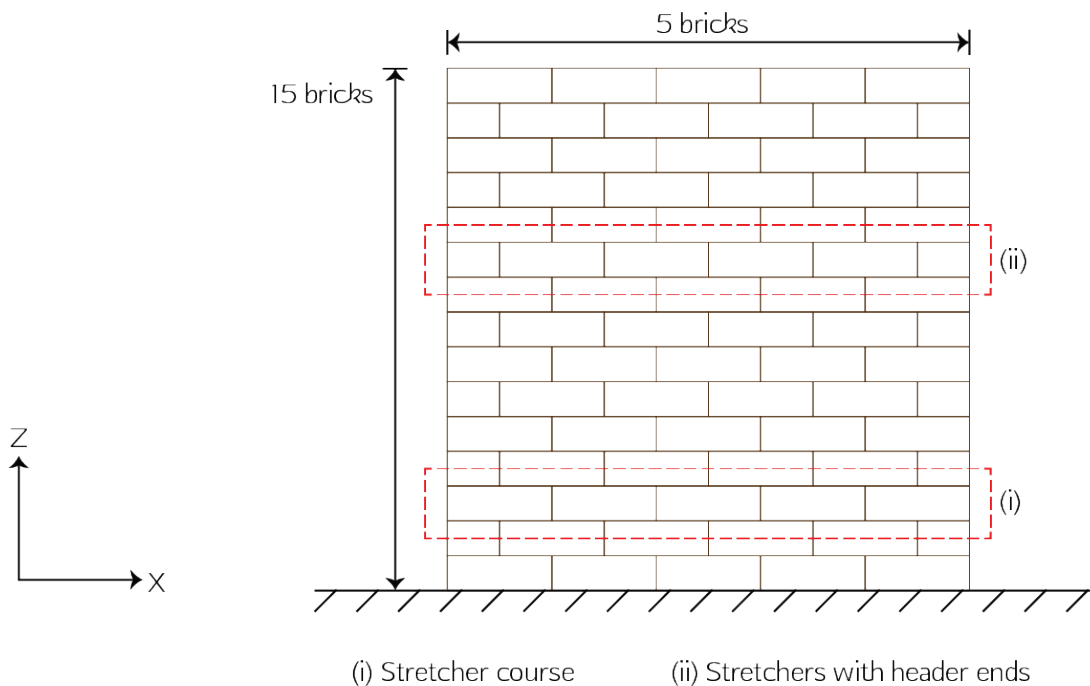


Figure 8.3: Front elevation schematic of single-storey stretcher bond courses with brick requirement in lateral and vertical directions, long-duration blast trials.

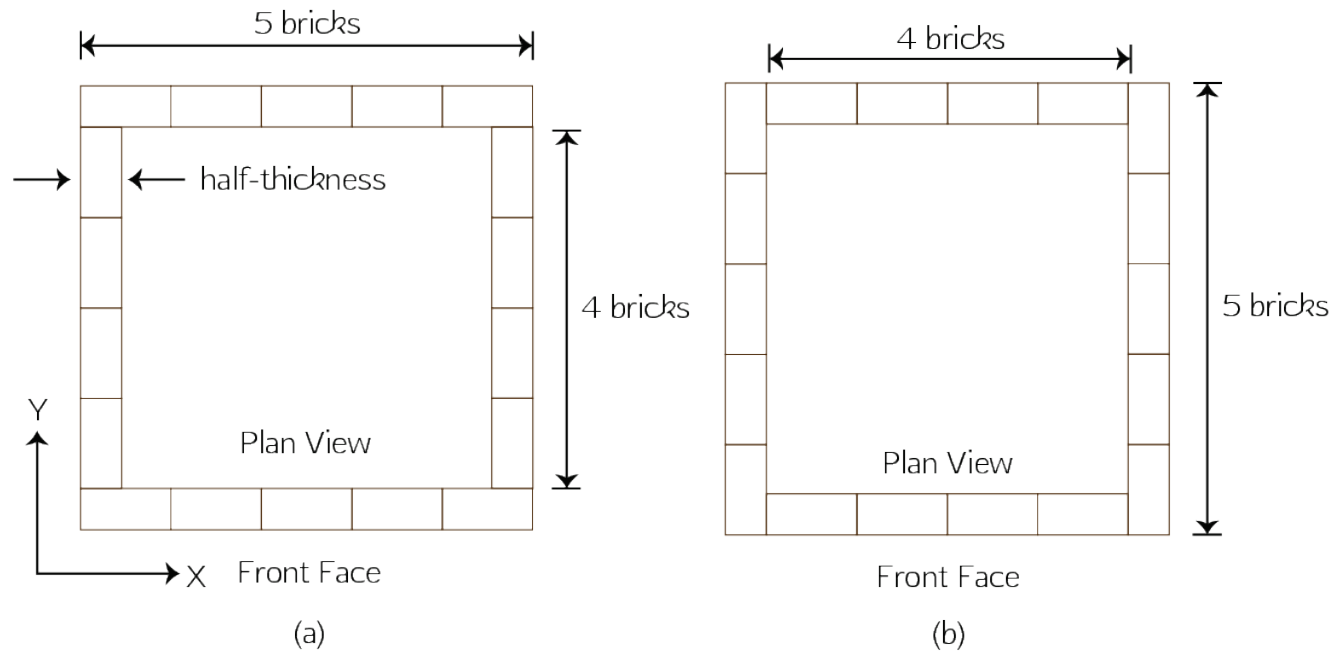


Figure 8.4: Plan schematic of stretcher bond courses, long-duration blast trials: (a) Stretcher course; (b) Header course with end blocks.

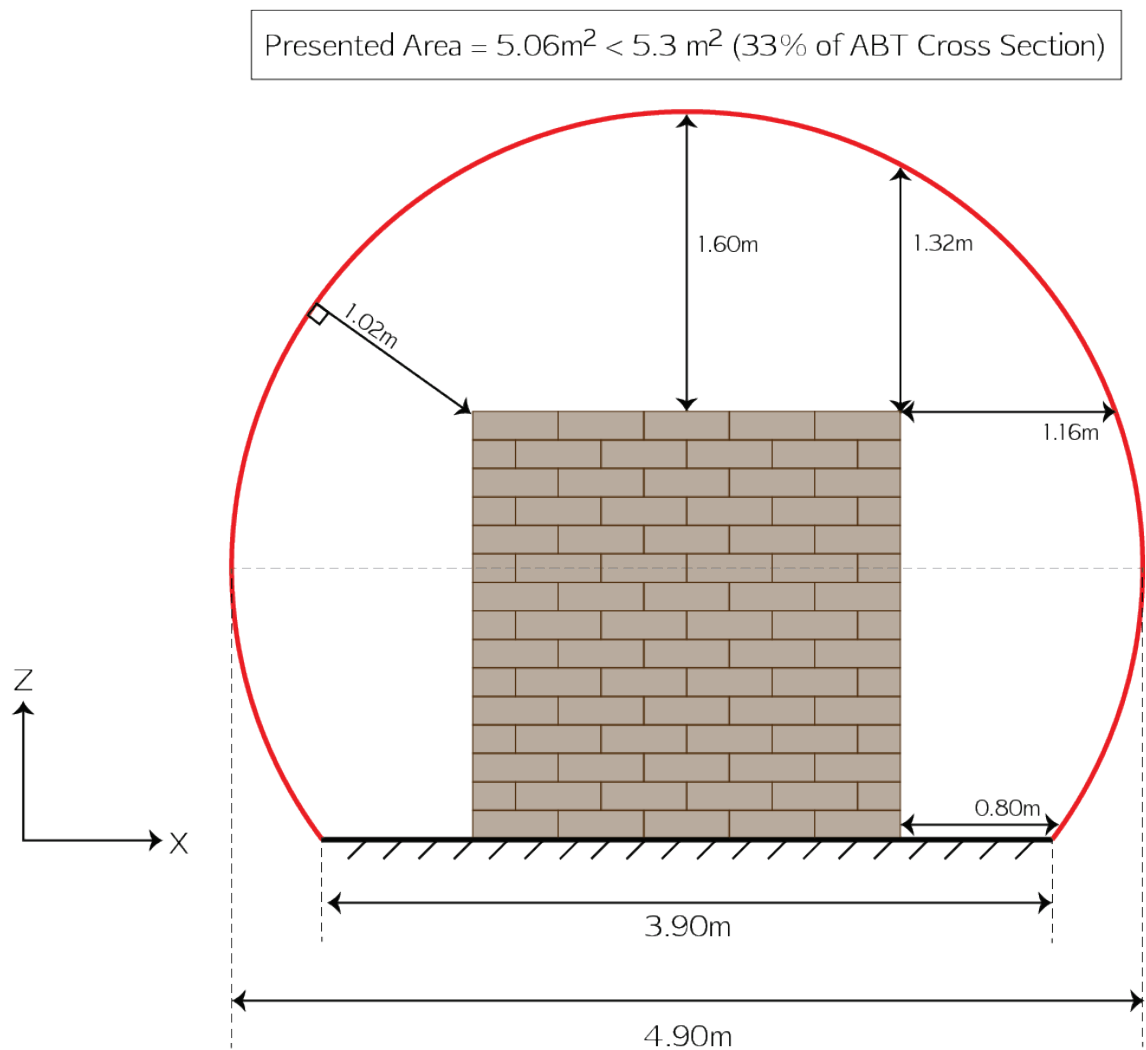


Figure 8.5: Front elevation of prototype structure within removable 4.9m section of ABT.

Presented Area = $1.32\text{m}^2 < 5.3\text{m}^2$ (33% of ABT Cross Section)

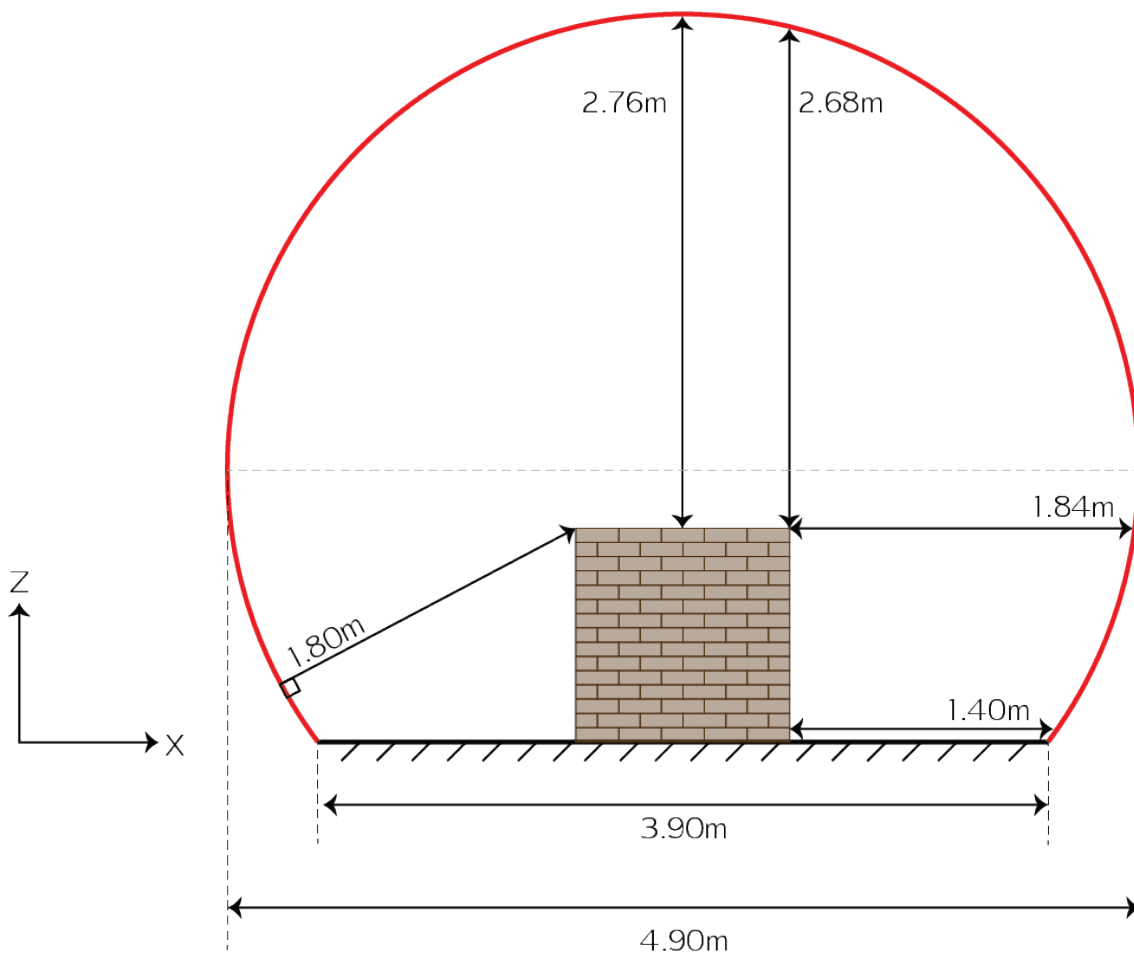


Figure 8.6: Front elevation of model structure within removable 4.9m section of ABT.

Two pairs of prototype and model masonry structures will be constructed from the same commercially available masonry units described in chapters 4-5 and as shown in figure 8.7. As a result, it is expected that prototype structures will utilise solid concrete blocks with mean block density of 1055kg/m^3 as found with the short-duration trial in chapter 5. Similarly, model structures will use solid concrete bricks with mean density of 2060kg/m^3 as detailed in chapter 5. This should produce an effective scale factor of 0.51 for density which closely matches the desired value of 0.50. Closer examination of figure 8.7 also shows that these units produce an effective scaling factor of 0.47 for the volume which is also a close match for the desired scaling factor of 0.50. As a result, the masonry units designed to be used in these four trials should meet the scaling methodology's core requirements via geometric similarity and a reciprocal scale factor for density at a 1:2 scale level.

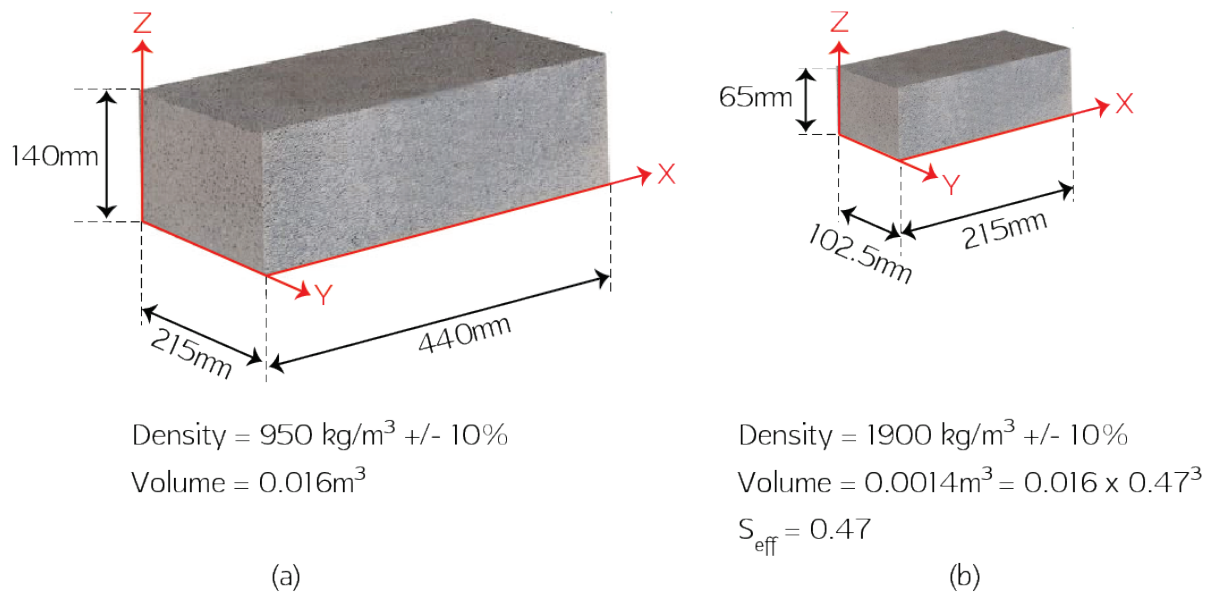


Figure 8.7: Masonry unit specifications: (a) Concrete block; (b) Concrete brick.

Each of the prototype and model structures will implement the same mortar design detailed in chapter 5 with the constituent properties of Ordinary Portland Cement, building sand and plasticizer with a combination: Cement-to-sand with plasticizer **ratio by volume of 1:5-6**. As seen in the short-duration blast trial, the physical mortar properties used within these trials cannot be determined until mortar cubes are crushed at the time of firing. These four trials will ensure that the mean curing time exceeds the minimum required value of 28 days and a range of cubes with 100mm dimensions will be cast for each new batch of mortar during the construction phase. By employing the same professional mason, it is intended that a mean crushing strength of $\sim 6 \text{ N/mm}^2$ will be achieved for these four trials to match the short-duration trial's conservative design, resulting in a type (ii) mortar versus the originally designed type (iii). It is also intended that a similar level of quality control be implemented to ensure a low standard deviation value for mean for density. This is particularly important as the scaling method depends upon a first principles representation of the masonry structure as an inertial resistance to blast loading. The physical application of mortar within each model and prototype structure will also match the short-duration trial with a constant 10mm bed thickness to meet the durability criteria specified by BS5628 (The British Standards Institution, 2005) for external walls. The decision not to scale the mortar material density or bed thickness was made after developing a proof shown in chapter 5 where It was shown that the practical approach of an industry-standard mortar bed thickness yields a mass scaling factor which matches the idealised scaling scenario for the model. This approach also mitigates

potential issues with workability, curing and crushing strength which could arise from modifying the density. Similarly, the maintenance of 10mm bed thickness will moderate the issue of bed uniformity which is more challenging with a scaled value of 5mm.

As discussed in chapter 4, this PhD's scaling methodology utilises first principles to represent a masonry structure as an inertial resistance to blast loading. It is therefore critical to determine the applied scaling factor for the composite model masonry structures as a whole. Tables 8.2-8.4 calculate the total expected masonry structure volume, mass and density for the long-duration blast trials. These values are shown alongside an idealised theoretical model at 1:2 and the expected values for the experimental prototype structure. Examination of table 8.2 shows that the volume of the experimental model structure has been scaled by an effective factor of 0.49, which closely matches the desired scale factor of 0.50. By utilising the total structural volume from table 8.2 and total mass from table 8.3 total structural density can be calculated in table 8.4. The highlighted cells in table 8.4 indicate an expected density scale factor for the experimental model structure to be 0.55. This value closely matches the ideal design value of 0.50, resulting in a 1:1.81 scaling level versus the ideal design level of 1:2.

Table 8.2: Effective scale factor of model structure volume, long-duration blast trials.

Structure type	Structure volume (m ³)	Effective Scale Factors	
Prototype	3.93	$S^3 = (\text{Volume}_{\text{model}} / \text{Volume}_{\text{prototype}})$	$S_{\text{volume}} = (S^3)^{1/3}$
Ideal Model	0.49	0.125	0.5
Model	0.47	0.119	0.49

Table 8.3: Effective scale factor of model structure mass, long-duration blast trials.

Structure type	Structure mass (kg)	Effective Scale Factors	
Prototype	4382	$S^2 = (\text{Mass}_{\text{model}} / \text{Mass}_{\text{prototype}})$	$S_{\text{mass}} = (S^2)^{1/2}$
Ideal Model	1095	0.25	0.5
Model	943	0.22	0.47

Table 8.4: Effective scale factor of model structure density, long-duration blast trials.

Structure type	Structure density (kg/m ³)	Effective Scale Factors	
Prototype	1115	$S^{-1} = (S_{\text{mass}} / S_{\text{volume}})$	$S = (S^{-1})^{-1}$
Ideal Model	2230	2	0.5
Model	2013	1.81	0.55

8.2.2 Data acquisition

The ABT blast environment will be monitored by instrumenting the 4.9m diameter section with Endevco 8510C static overpressure gauges and Kulite LQ080 dynamic pressure gauges, as shown in figures 8.8-8.9. Reflected pressure will also be measured at each panel wall via Kistler 603B1 pressure transducers which will be fixed to steel angles positioned in front of each panel wall. Each pressure device will be calibrated to sequence with the ABT detonation trigger, thereby defining an accurate blast arrival time within the pressure data. The qualitative assessment of masonry damage via cracking and degrees of collapse will be achieved by utilising a combination of high-speed video and still photography. A series of five high-speed Phantom v7.3 cameras will be deployed to monitor masonry response at 2000fps as shown in figures 8.8-8.9. These will be positioned to provide one dedicated camera per panel wall in tandem with a view in the global Y direction to provide an account of structural damage and fragment throw downstream. Each camera will be housed in armoured containers to prevent the application of blast loading directly to the camera bodies.

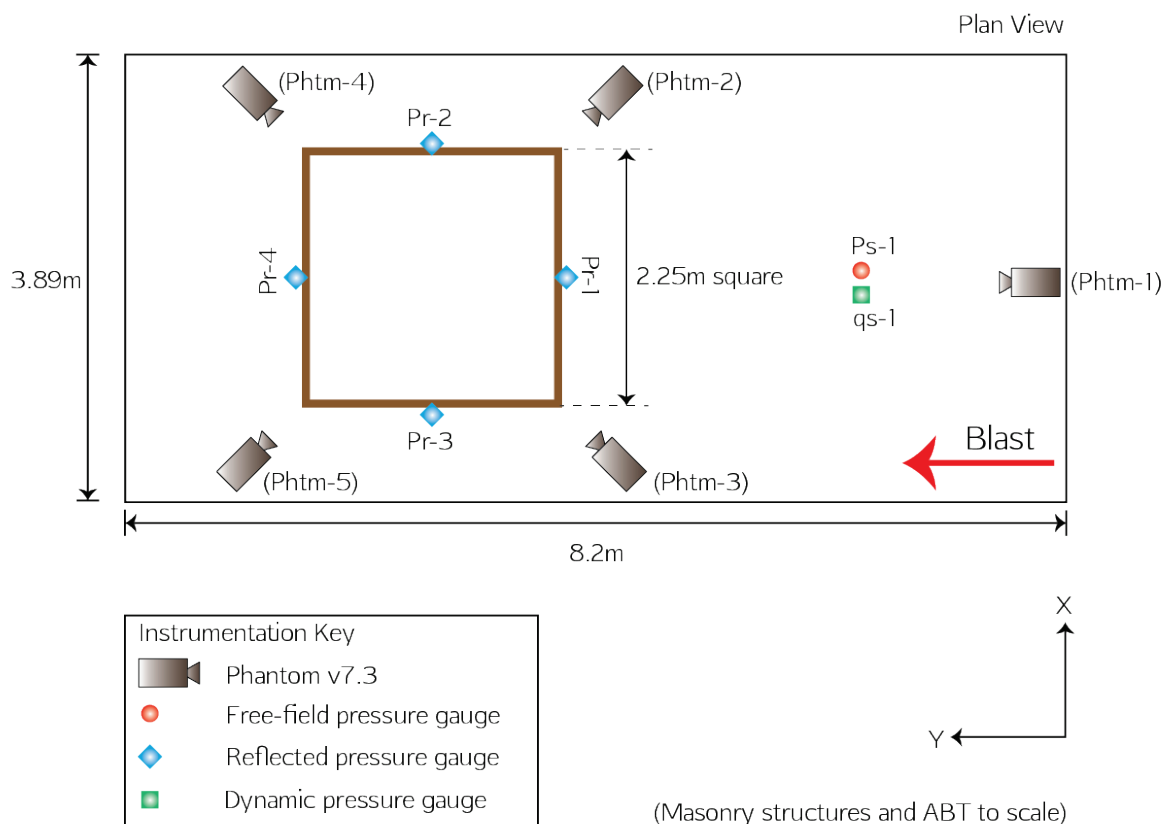


Figure 8.8: Plan schematic of prototype structure with instrumentation inside removable 4.9m section of ABT.

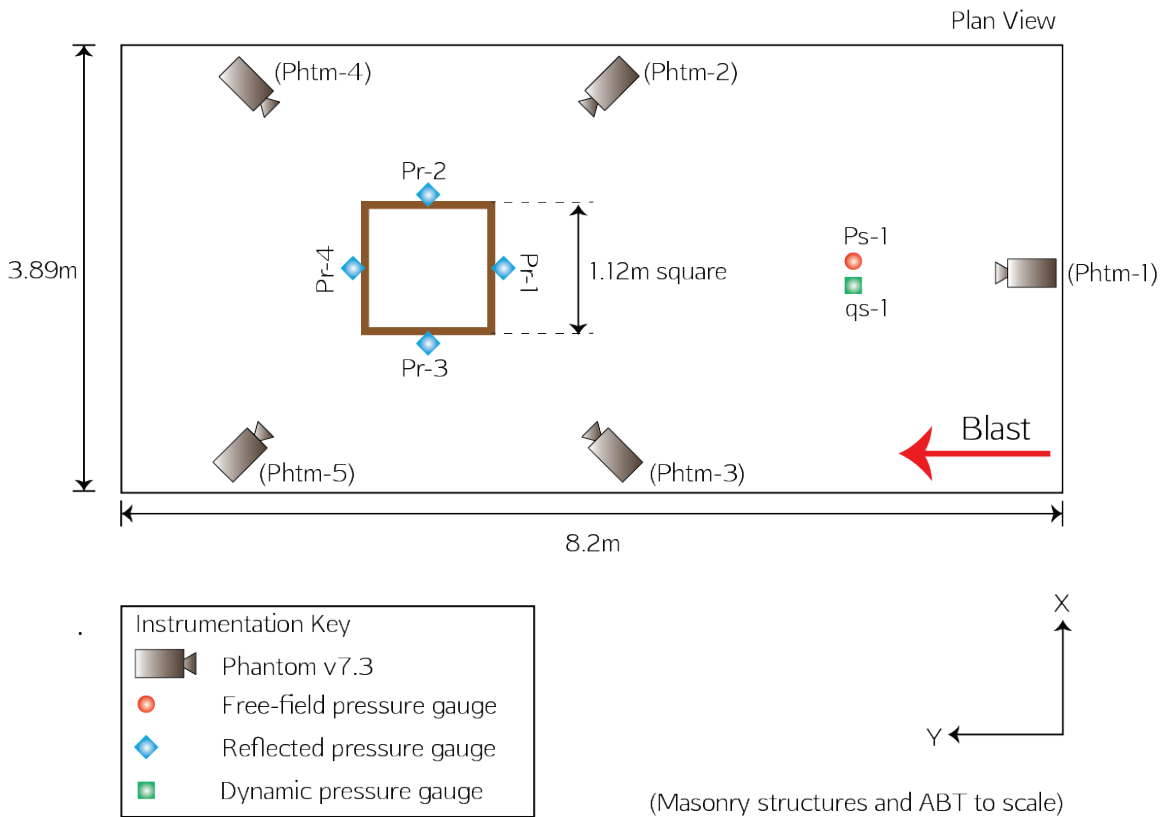


Figure 8.9: Plan schematic of model structure with instrumentation inside removable 4.9m section of ABT.

To assist the assessment of post-trial video files, pressure-triggered light sources will be deployed for each structure. These will be activated by the reflected pressure gauges shown in figures 8.8-8.9 to enable the visual identification of blast wave arrival at each structural panel within Phantom video data.

Quantitative analysis of post-trial debris distributions will be accomplished by utilising both of the methods outlined in chapter 5. As a result, manual weighing and positional logging of masonry debris will be implemented for model masonry structures, enabling debris to be represented as a percentage of total mass. These values will be assigned to a spatial grid location in the local X and Y axes with a grid resolution equal to 50% of the structural width and 50% of the structural length. The utilisation of a non-dimensional grid system will enable direct comparisons with prototype debris as demonstrated in chapter 6 for the short-duration blast trial. Prototype debris data will be generated by implementing the manual block counting procedure outlined in chapter 5 to determine the spatial distribution of blocks as a percentage of the total block quantity for the prototype. The percentage of total structural

mass can then be inferred from this data as shown demonstrated in the short-duration blast trial. The collection of model structure debris will also enable further benchmarking of the inferential procedure utilised with prototype blocks with the aim of further demonstrating its accuracy and viability as a genuine alternative to the more labour intensive method of manual weighing and logging.

These four trials will also aim to measure equivalent brick and block motion via bespoke housings for a six degree of freedom DTS-6X Pro accelerometer in the front face of the model and prototype as shown in figure 8.10.

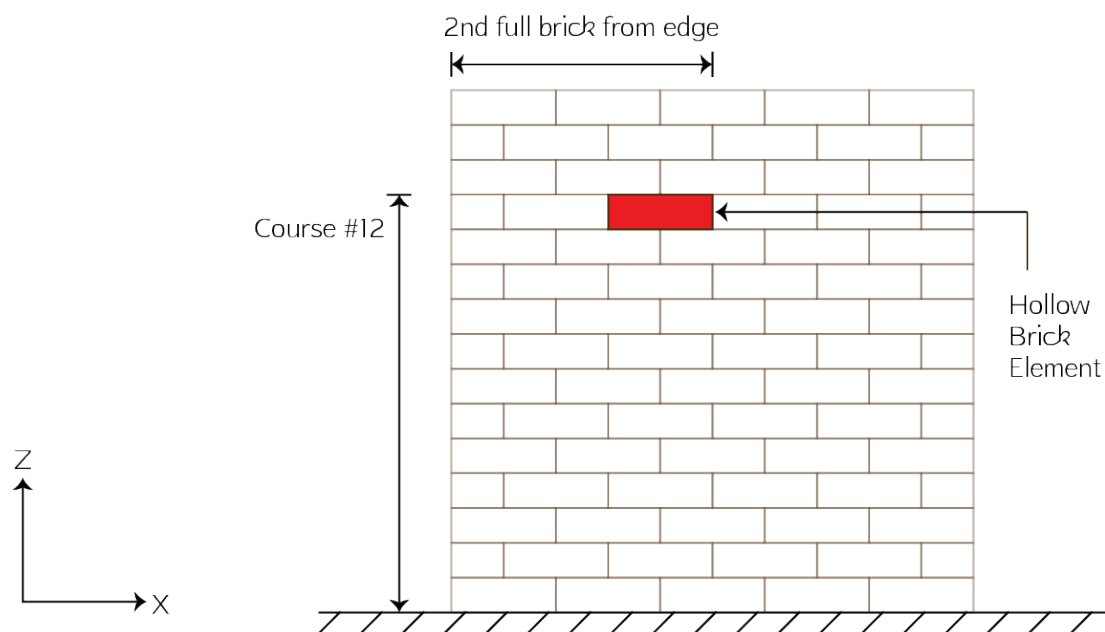


Figure 8.10: Front elevation location of accelerometer elements, long-duration blast trials.

The accelerometer housing design implemented within the short-duration blast trials will require modification as detailed in chapter 6 via further consultation with the Foulness Trials Group of Spurpark Ltd once a trial date has been set. Two fundamental requirements for the design of a new element include the maintenance of geometric similarity and the replication of mean mass for a concrete brick or block unit. Thus ensuring the geometry and density requirements of the scaling method were maintained. One immediate alteration to the methodology implemented within the short-duration trial is the re-positioning of the accelerometer element to an off-centre location as demonstrated in figure 8.10. In doing so, it should be possible to mitigate the influence of vertical primary cracking on the dynamic record of brick motion as found in chapter 6. By capturing a dynamic record of equivalent

brick and block motion throughout the long-duration blast events, these trials will aim to compare model and prototype results for acceleration, velocity and displacement. The results of which will demonstrate whether the elements were subject to an equal acceleration as predicted by the first principle scaling methodology in chapter 4.

8.3 Modelling

As a means of producing preliminary insight into the long-duration blast trials designed within this chapter, a series of CFD and AEM analyses were conducted to investigate potential masonry response and scaling methodology performance. Each of the computational models were conducted using a dedicated quad-core Intel i7-3840QM 2.8GHz system with 16GB RAM.

8.3.1 CFD comparative study overview

In order to determine the most appropriate solver for long-duration blast scenarios, a comparative study was conducted to assess two of the most widely used CFD solvers available to model blast, Air3D and AUTODYN. This study is presented in full within Appendix A where it aimed to assess the relative accuracy of these CFD packages versus free-field and reflected blast data which was experimentally recorded during a series of previously conducted long-duration blast trials (Adams et. Al, 2014).

CFD models were produced within Air3D and AUTODYN to replicate the free-field and reflected blast environments recorded in the 10.2m diameter section of the UK Air Blast Tunnel (ABT) during a series of previously conducted long-duration blast trials. 1D simulations revealed similar levels of overall accuracy within $\pm 10\%$ of the trial results in the free-field. Air3D was found however to possess a significant speed advantage in terms of solution execution versus AUTODYN in both serial and parallel modes. 2D simulations were subsequently produced to model the reflected blast environment of the ABT trials. CFD results were once again found to represent comparable levels of overall accuracy within $\pm 15\%$ of the trial data. The utilisation of a 2D analysis to model blast clearing around a finite structure may however limit the ability to improve this accuracy level. Computational

overhead comparisons in 2D revealed Air3D to possess a significant speed advantage versus AUTODYN whilst also utilising a lower overall memory overhead per cell.

Additional 2D analyses were also conducted with a finer mesh resolution to reveal similar levels of overall accuracy within $\pm 22\%$ of reflected trial data. The design phase of the 2D simulations revealed a 3800 cell count limitation for Air3D in any axis direction, restricting the ability to conduct finer resolution models at large stand-offs. 3D models were subsequently produced with the minimum Z domain extents to closely replicate a 2D analysis. These solutions produced comparable levels of accuracy with the AUTODYN models in true 2D. Analysis of the computational overhead results also showed a significant speed advantage for Air3D whilst AUTODYN was found to introduce the potential of CPU overheating when utilising parallel execution for long run times. Finally, additional attempts to utilise a finer mesh resolution in the pseudo 2D state for Air3D revealed a cell count limitation of 3000 in each direction in 3D. This demonstrated the need to calculate the notional blast wave thickness to ensure adequate space to accommodate a blast wave on either side of a structure in 3D model. Indicating a limit on the finest possible cell size as a function of blast wave thickness

8.3.2 Air3D methodology

Building upon the comparative study, CFD analyses were conducted with Air3D to model the reflected and free-field blast environments expected within the long-duration blast trials detailed in section 8.2. Each of the models utilised an optimised 1D simulation from 0m in the X direction up to the desired stand-off distance minus 10m as detailed in tables 8.5-8.6. These 1D simulations were then re-mapped into 3D with the domain extents and cell size shown in tables 8.5-8.6. The visual interpretation of these domain extents can be seen in figure 8.11a for the X-Y plane and in figure 8.11b for the X-Z plane. Further inspection of figure 8.11a shows each of the boundaries to be set as flow-out whilst figure 8.11b indicates that ground surface was set to reflect in the X-Z plane. Figures 8.11a-b also show the positioning of a non-responding cuboid structure with the front-face located at the desired stand-off. These cuboids were designed to represent the model and prototype masonry structural dimensions detailed in section 8.2. It can be seen in figure 8.11c that the cuboids were positioned on the centre line in the Y direction, resulting in a centrally positioned

structure within the domain. Tables 8.5-8.6 also show that each model was designed to not exceed the 3D cell limits outlined in Appendix A.

Table 8.5: CFD domain parameters for 200t at 250m stand-off.

	X Direction (m)	Y Direction (m)	Z Direction (m)	Cell Size (mm)	Cell Limit Per Dimension	Cell Count in X (Max)	Total Cells	Run Time (mins)
1D	0-240	-	-	58.5	-	-	4103	132
3D	150-340	0-30	0-30	250	3000	760	10.9×10^6	

Table 8.6: CFD domain parameters for 450t at 235m stand-off.

	X Direction (m)	Y Direction (m)	Z Direction (m)	Cell Size (mm)	Cell Limit Per Dimension	Cell Count in X (Max)	Total Cells	Run Time (mins)
1D	0-225	-	-	96.5	-	-	2331	278
3D	105-345	0-30	0-30	250	3000	960	13.8×10^6	

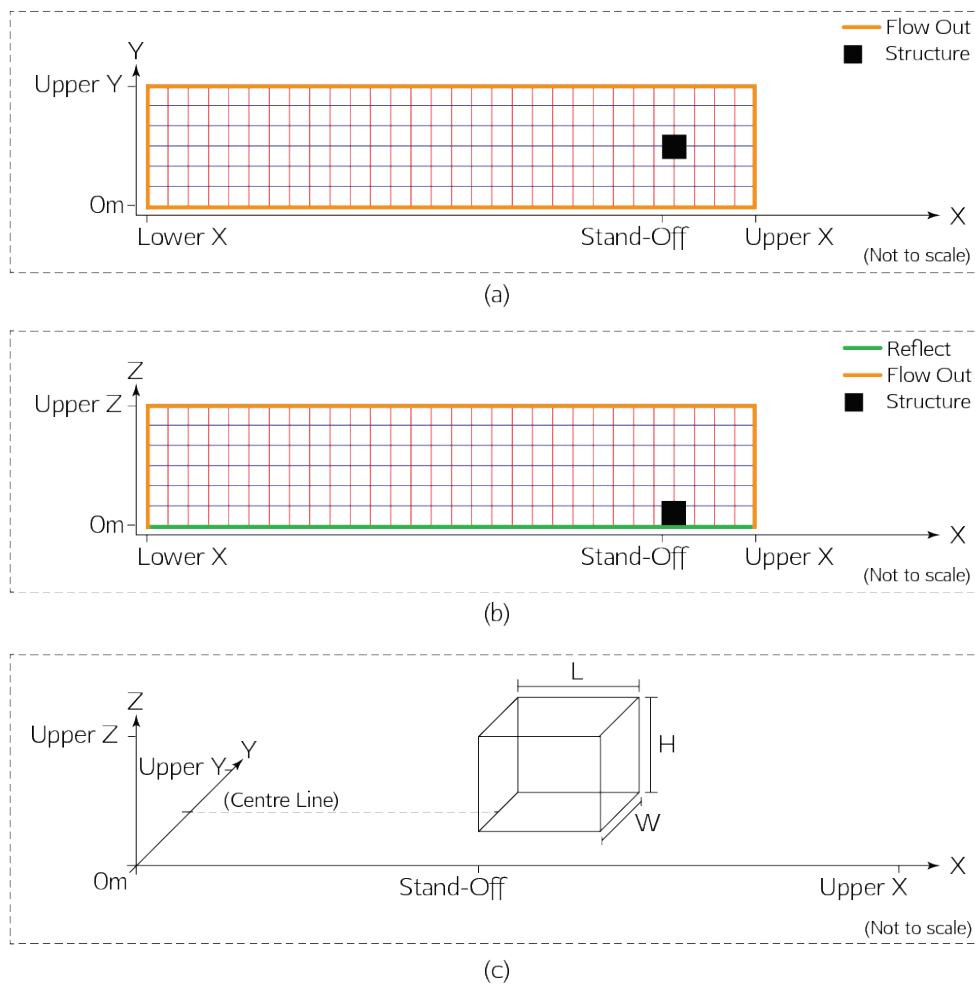


Figure 8.11: CFD domain schematic in 3D, ABT: (a) X-Y Plane; (b) X-Z Plane; (c) Masonry cuboid proportions and location.

In order to analyse the blast flow within these CFD models, pressure gauges were located throughout the domain to measure free-field and reflected parameters. Reflected pressure gauges were located at multiple sites on the front, side and rear face of the cuboid structure. In particular, the front face was instrumented with two reflected pressure gauges down the centre line of the structure as shown in figure 8.12. Initially, three gauges were located at equidistant intervals within regions which represented $1/3$ of the frontal area. The analysis of pressure time history results from these gauges was found to demonstrate approximately equal results for the gauges located over the bottom $2/3$ of the area. All further models were therefore designed to utilise one gauge in bottom $2/3$ region with a second gauge located in the centre of the top $1/3$ of the area. The utilisation of fewer monitoring regions is particularly useful for reducing simulation complexity within the AEM models of masonry response.

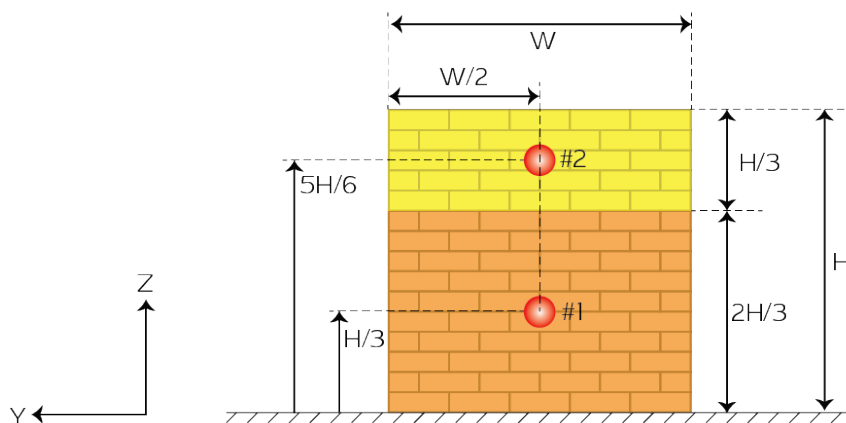


Figure 8.12: CFD reflected gauge schematic, front face.

Figure 8.13 shows that the side face was instrumented with three vertical regions, each representing $1/3$ of the area with one gauge located at the centre of each region. Due to the symmetrical nature of the structures with face-on loading, the pressure record from these gauges can be seen to represent both sides of the structure. Figure 8.14 demonstrates that the rear face was divided into three equal regions with each accounting for $1/3$ of the area. Reflected gauges were subsequently located at the centre of each area, enabling the pressure to be monitored as it clears over the structure and down the rear face. Free-field pressure was monitored at the stand-off distance of interest in each model with the gauges located

~5m away from the lower and upper X boundaries at a 1.5m height to prevent reflections or interference from the blast wave's progression over the structure.

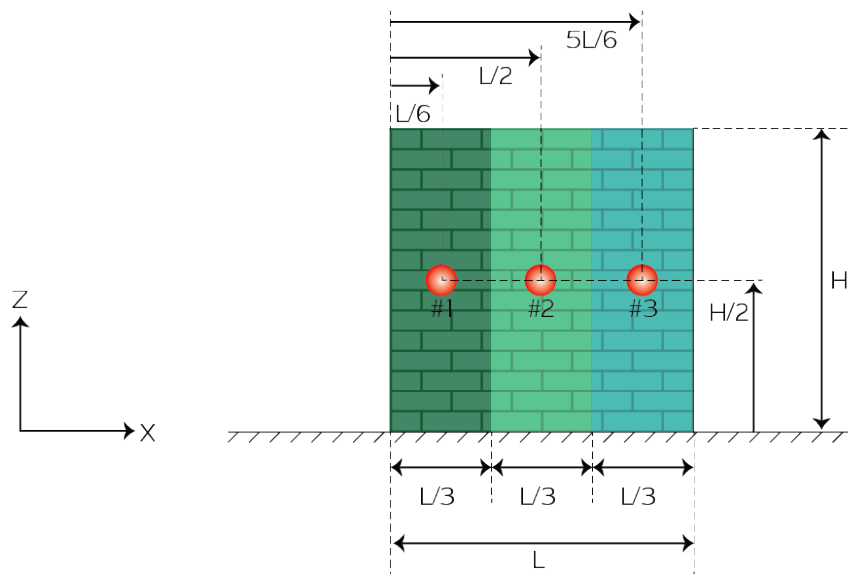


Figure 8.13: CFD reflected gauge schematic, side face.

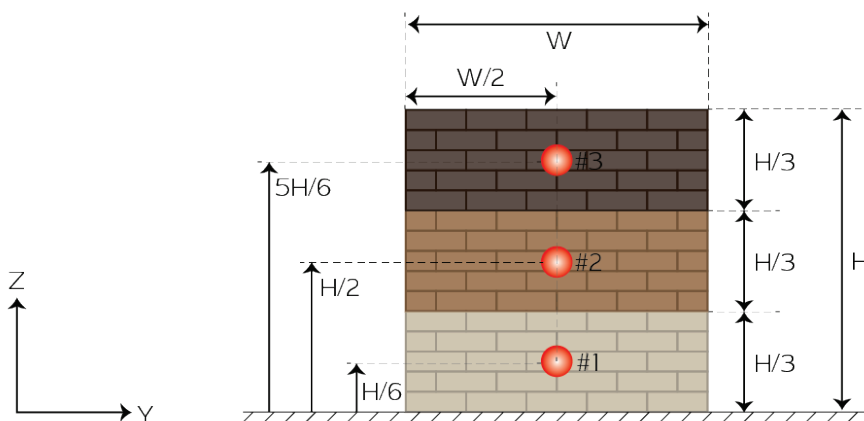


Figure 8.14: CFD reflected gauge schematic, rear face.

8.3.3 Air3D solution analysis

Table 8.7 compares CFD solutions for the free-field blast environment to predicted values from section 8.2. Initial inspection shows the CFD results for free-field overpressure to be +6% and +7% larger than the predicted values in the trials 1-2 and 3-4 respectively, representing accuracy within $\pm 10\%$. Similarly, CFD results for specific impulse can be seen to be +9% and +10% greater than the predicted values in trials 1-2 and 3-4 respectively, further

demonstrating accuracy within $\pm 10\%$ of the trial design. Examination of the positive phase duration results reveals the CFD results to be -12% and -13% below the predicted values in trials 1-2 and 3-4. The net effect of these pressure and duration results is an impulse error within $\pm 10\%$ for trials 1-2 and 3-4.

As demonstrated in chapter 4, the scaling approach adopted within this research relies upon first principles and the inertial resistance of masonry to blast energy. An acceptability level of $\pm 10\%$ for impulse indicates that the CFD solutions represents an acceptable level of accuracy for modelling the effect of the blast force when compared to the predicted values. Figures 8.15-8.16 illustrate the free-field overpressure and specific impulse time histories produced by the CFD solutions for trials 1-2 and trials 3-4. Closer inspection of these reveals a close approximation of the idealised Friedlander shape shown in chapter 2, which indicates that the blast wave was properly produced throughout the 3D domain.

Table 8.7: CFD free-field blast results for long-duration blast trials.

	ABT Trials 1-2			ABT Trials 3-4		
	$w = 200t, r = 250m$			$w = 450t, r = 235m$		
	P_s (kPa)	i_s (kPa.ms)	t^+ (ms)	P_s (kPa)	i_s (kPa.ms)	t^+ (ms)
CFD	60.7	4320	182.2	118.1	7687	192.6
Prediction	57.3	3980	206.9	110.4	6971	220.9
+/- %	6%	9%	-12%	7%	10%	-13%

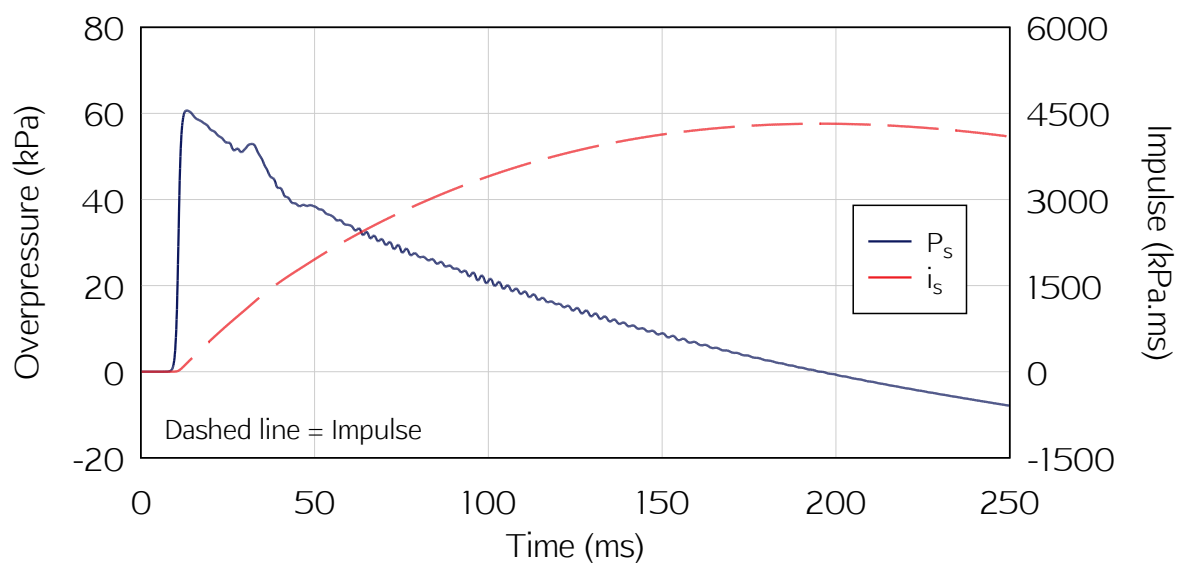


Figure 8.15: CFD free-field overpressure and specific impulse, ABT Trials 1-2.

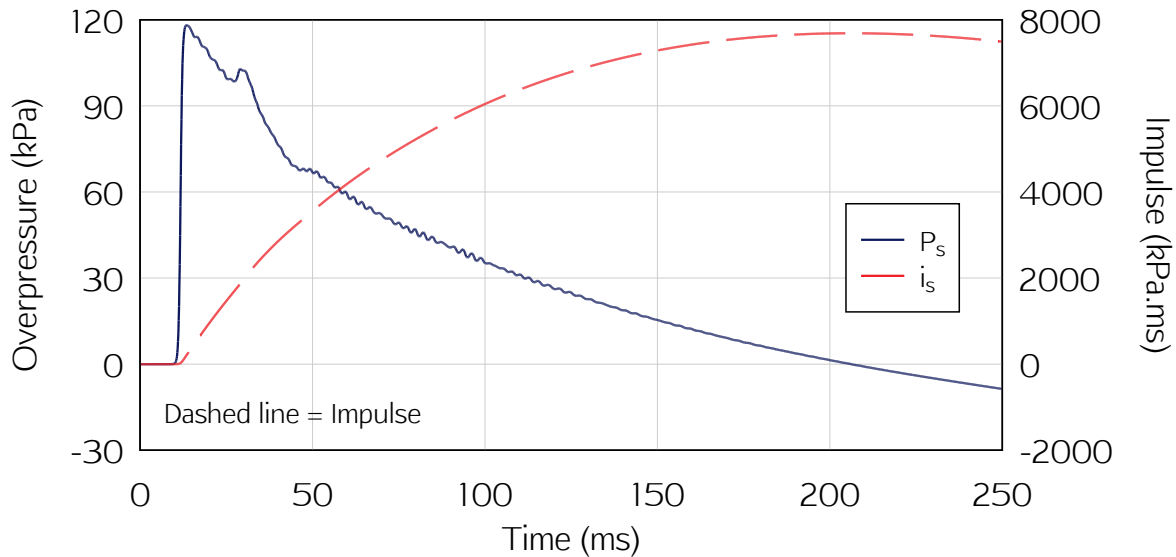


Figure 8.16: CFD free-field overpressure and specific impulse, ABT Trials 3-4.

Based on the levels of CFD accuracy demonstrated for the free-field environment, it was possible to utilise the reflected pressure records to configure blast loading for the AEM models of masonry response in section 8.3.4. Figures 8.17-8.20 show the reflected pressure and impulse results for each gauge location detailed in section 8.3.2 for ABT trials 1-4. Figures 8.21-9.24 illustrate a time-lapse progression of the blast wave interaction with the masonry structures in ABT trials 1-4 where it is evident that the notional blast thickness dwarfs the structural dimensions which contrasts with the pressure gradient observed within the short-duration blast models in chapter 7.

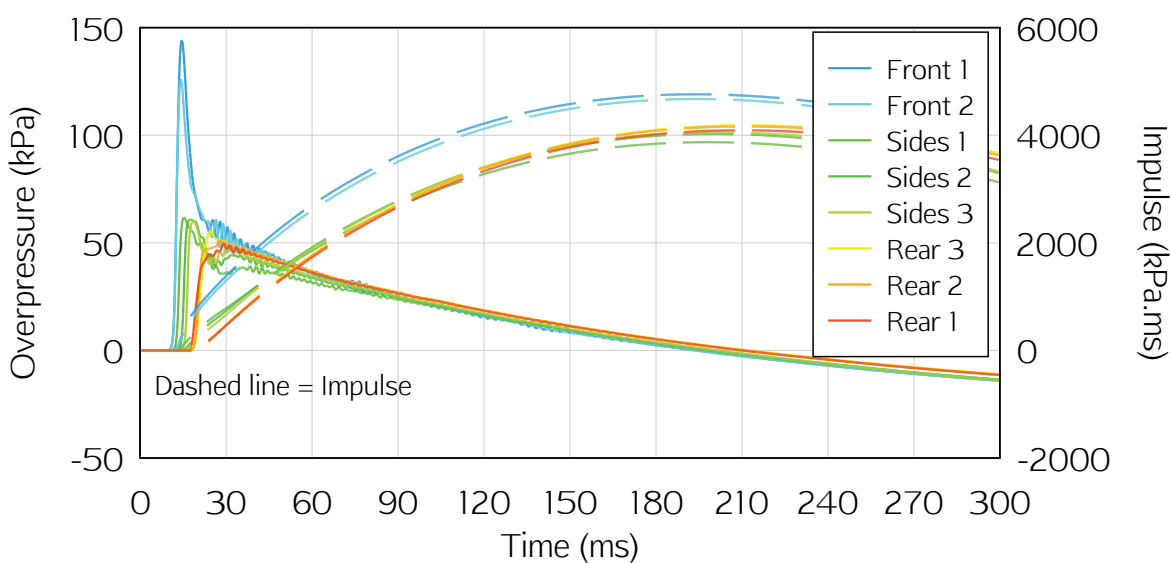


Figure 8.17: CFD reflected pressure and impulse, prototype structure in ABT Trial 1.

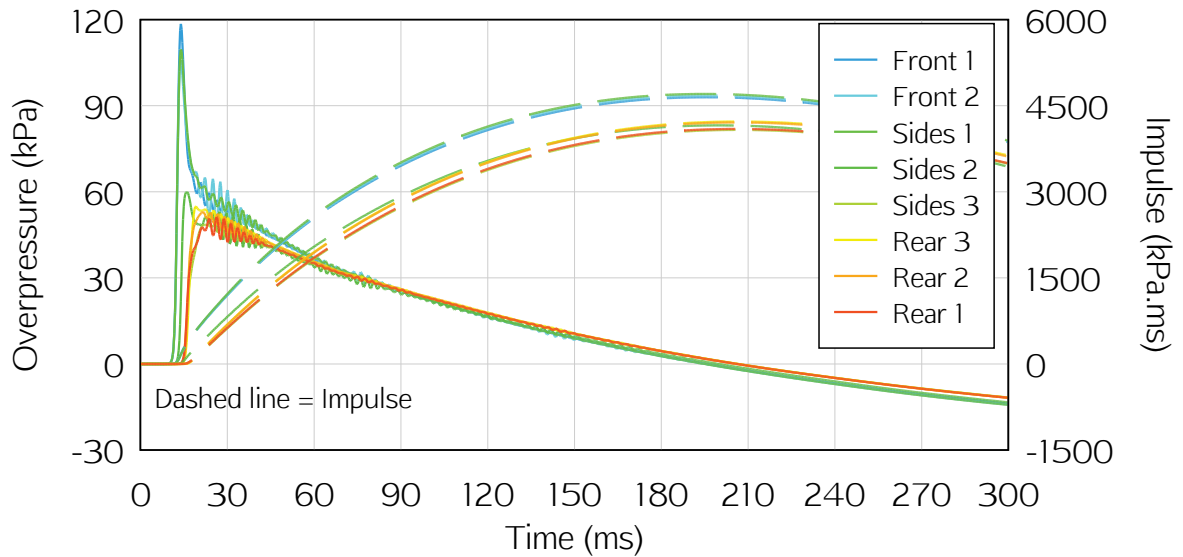


Figure 8.18: CFD reflected pressure and impulse, model structure in ABT Trial 2.

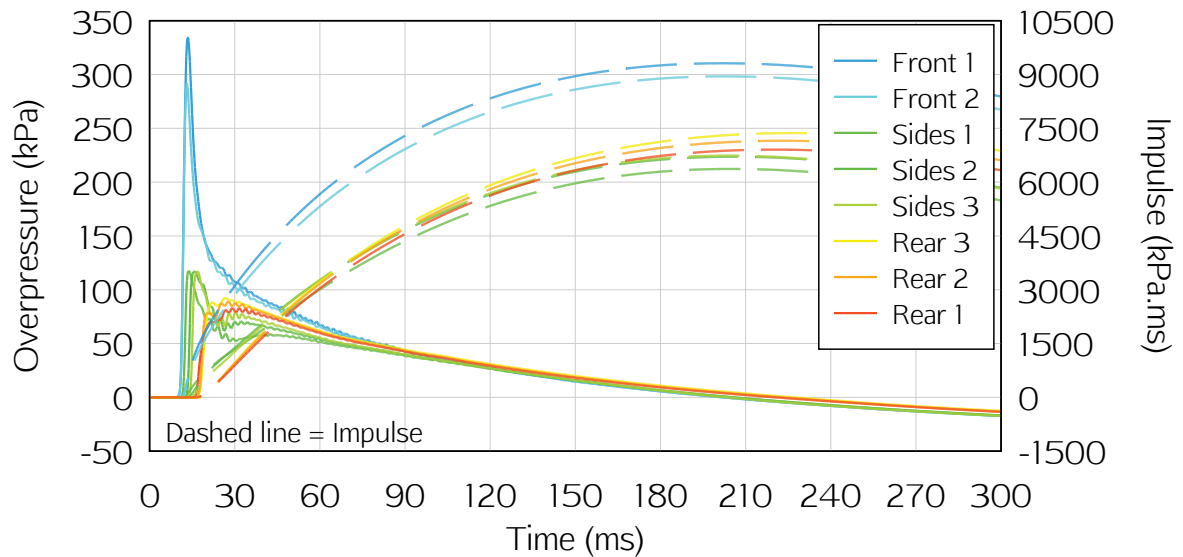


Figure 8.19: CFD reflected pressure and impulse, prototype structure in ABT Trial 3.

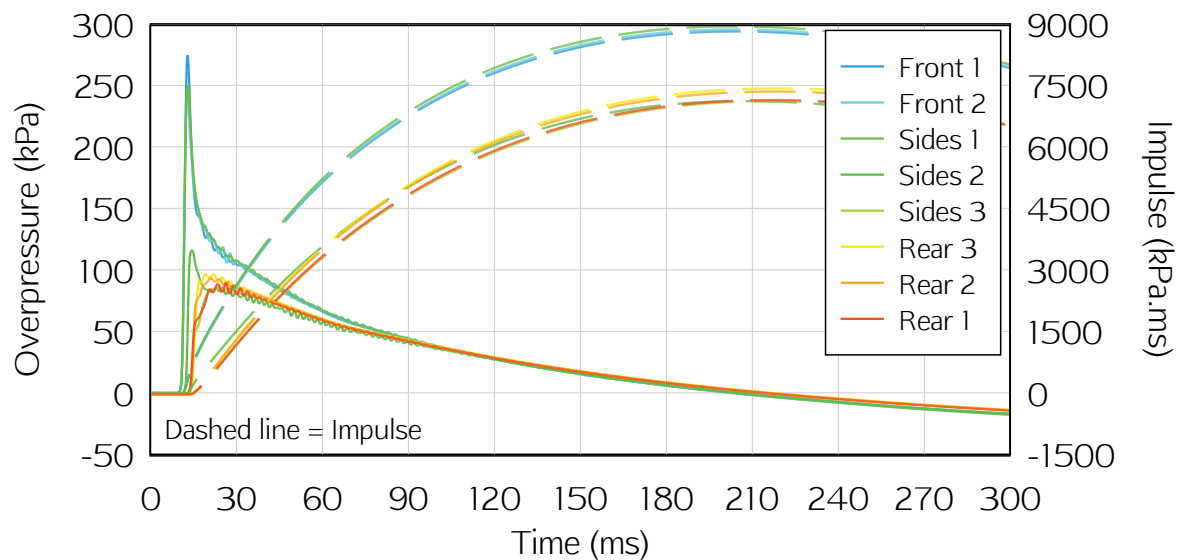


Figure 8.20: CFD reflected pressure and impulse, model structure in ABT Trial 4.

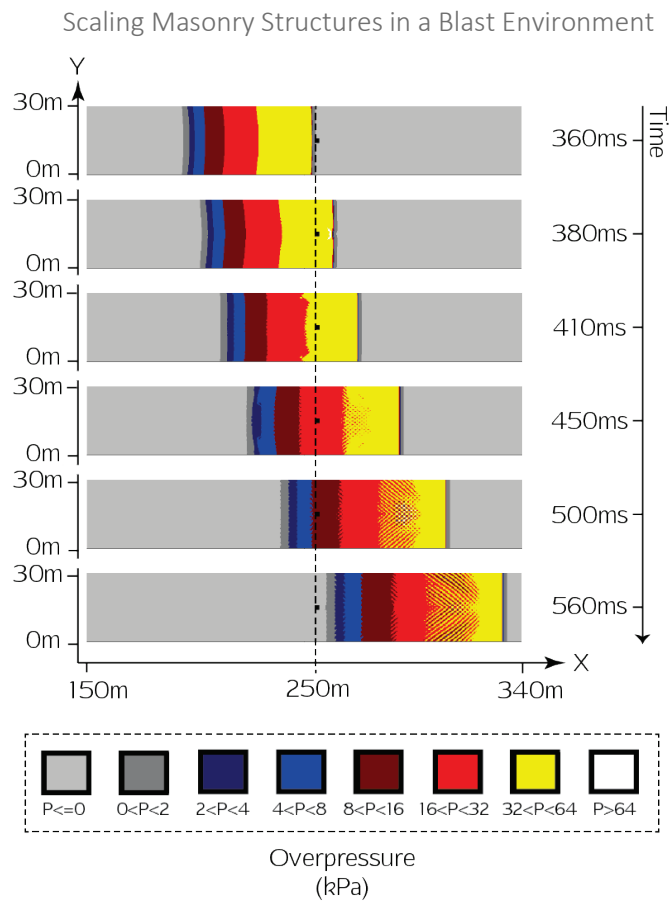


Figure 8.21: Time-lapse visualisation of CFD blast wave progression around prototype structure in ABT Trial 1.

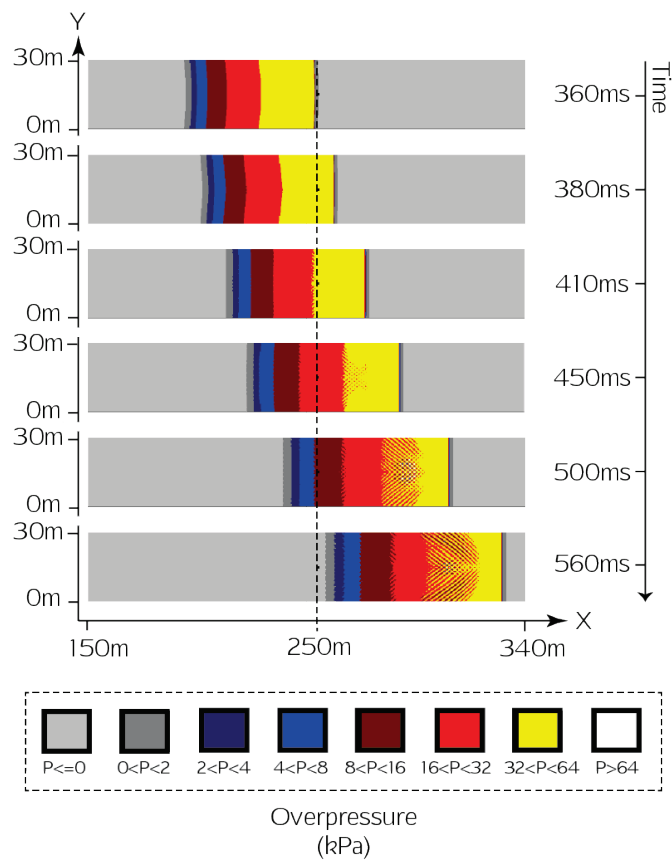


Figure 8.22: Time-lapse visualisation of CFD blast wave progression around model structure in ABT Trial 2.

Scaling Masonry Structures in a Blast Environment

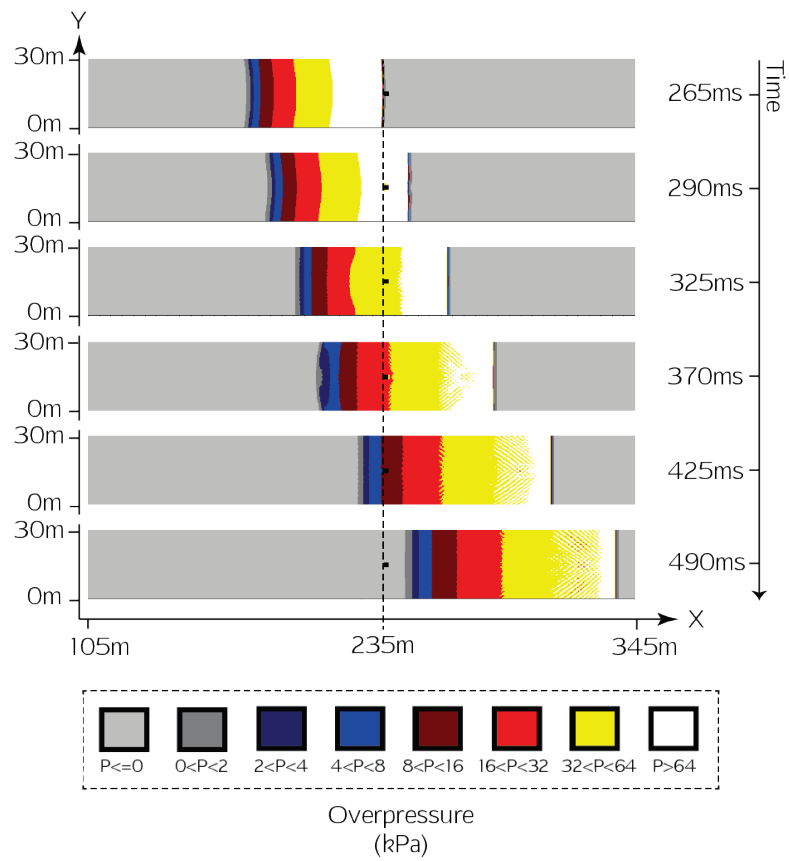


Figure 8.23: Time-lapse visualisation of CFD blast wave progression around prototype structure in ABT Trial 3.

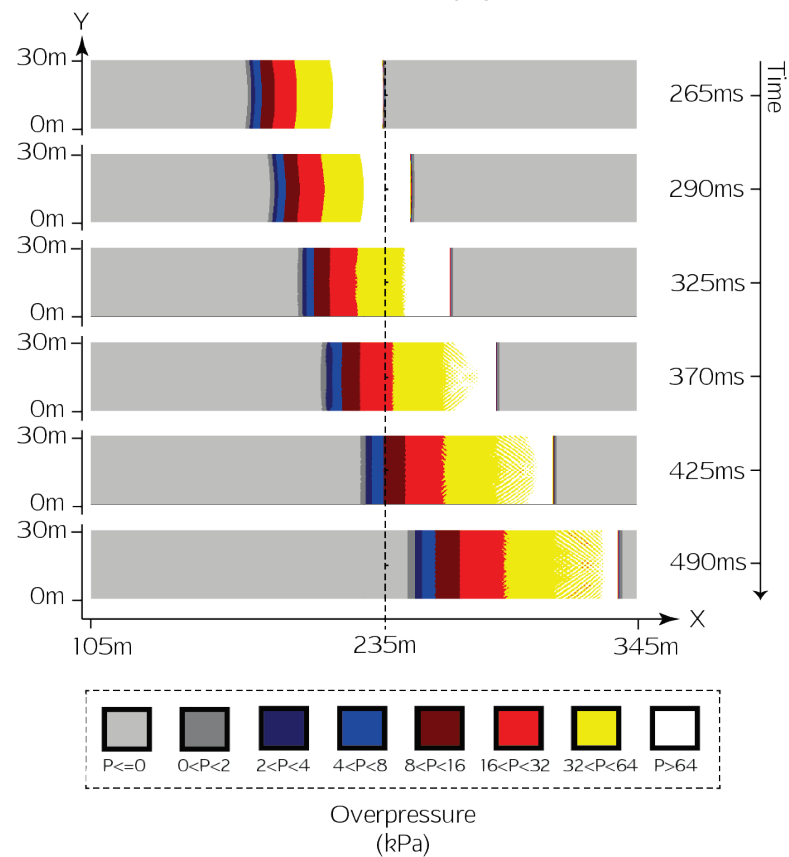


Figure 8.24: Time-lapse visualisation of CFD blast wave progression around model structure in ABT Trial 4.

8.3.4 AEM methodology

A series of AEM models were produced to analyse model and prototype masonry response to long-duration blast utilising the experimental details in section 8.2. Each AEM model used the material parameters shown in table 8.8 where the physical properties replicate those expected for the experimental trial design in section 8.2. The separation strain coefficient within table 8.8 represents the default ELS setting for the onset of concrete cracking due to tensile strain and thus replicates the short-duration blast models discussed in chapter 7. Figure 8.25 demonstrates the implementation of half-thickness stretcher bond construction within these AEM simulations to fully replicate the masonry construction of the experimental trial. Each of these solutions were produced using the native representation of a 'flexible' corner structure as detailed in chapter 7. This method was chosen for these preliminary models to enable the generation of debris as this is the dominant feature of long-duration blast loading of masonry structures. It may also be shown that the substantial impulse associated with these events mitigates the importance of accurate cracking due to the rapid rate of collapse in conjunction with significantly large fragment throw distances versus short-duration behaviours observed in chapter 6. Blast loading was configured for the model and prototype simulations by re-mapping reflected pressure data produced within the CFD models detailed in section 8.3.3. This was accomplished by dividing the masonry structures into loading regions which match the areas demonstrated in section 8.3.2. Table 8.9 details the element quantities and solution parameters for each of these models.

Table 8.8: ELS material parameters for ABT Trial simulations.

	Density (kg/m ³)	Young's Modulus (GPa)	Shear Modulus (GPa)	Compressive Strength (MPa)	Tensile Strength (MPa)	Separation Strain
Concrete Blocks	1055	26.2	10.5	7.3	0.73	0.1
Concrete Bricks	2060	26.2	10.5	20	2	0.1
Mortar	1720	26.2	10.5	4	0.6	0.1

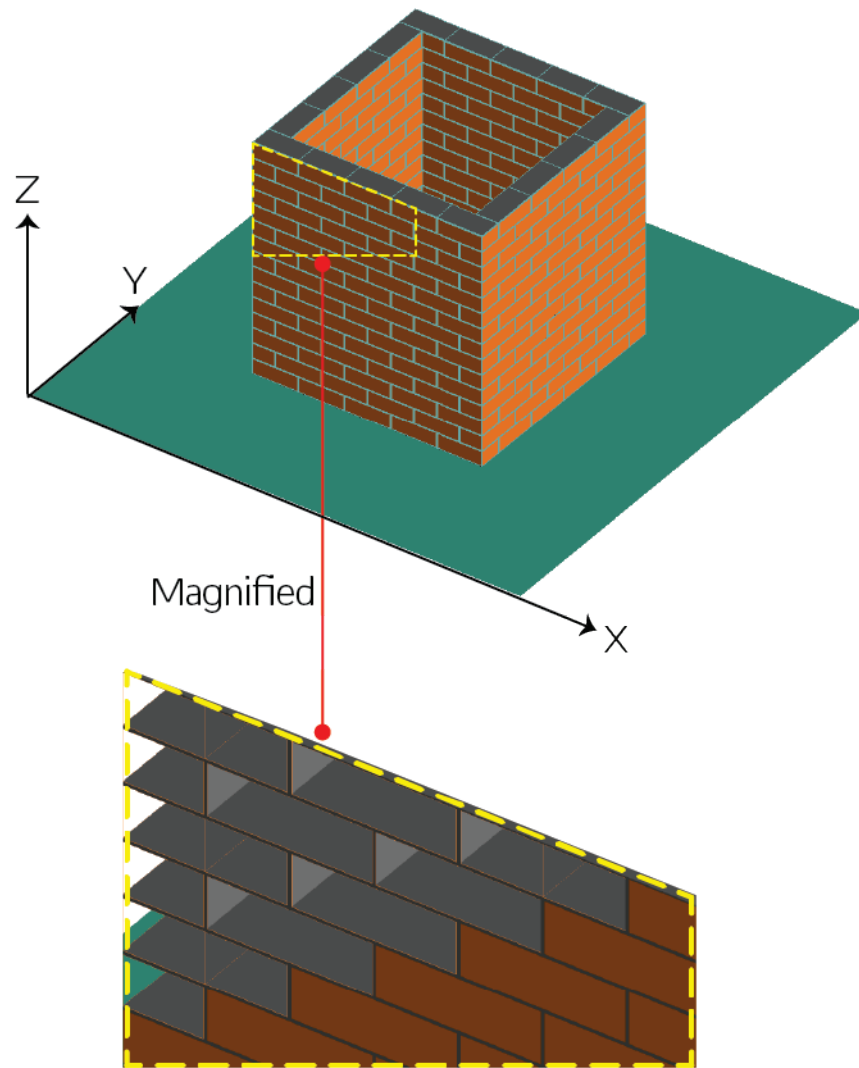


Figure 8.25: ELS masonry structure stretcher bond visualisation with magnified view.

Table 8.9: ELS solution parameters for ABT Trial simulations.

		Whole Brick Elements	Half Brick Elements	Mortar Elements	Problem Time (s)	Solution Step (μ s)
ABT Unconstrained, 55kPa	Prototype	180	180	1080	2.5	100
	Model	180	180	1080	2.5	100
ABT Unconstrained, 110kPa	Prototype	180	180	1080	2.5	100
	Model	180	180	1080	2.5	100
ABT Constrained, 55kPa	Prototype	180	180	1080	2.5	100
	Model	180	180	1080	2.5	100
ABT Constrained, 110kPa	Prototype	180	180	1080	2.5	100
	Model	180	180	1080	2.5	100

8.3.5 AEM analysis - ABT Trials 1-2

Figures 8.26-8.27 show a time-lapse progression of blast wave interaction with AEM masonry structures for ABT trials 1-2 for 55kPa. It can be seen that both the model and prototype structures fail via a symmetrical 'squashing' action. This is not likely to be representative of experimentally observed results due to the AEM implementation of flexible corner restraints which allow pivoting versus cracking. In spite of this, it can be observed that the model structure replicates the failure mode of the prototype, inferring that the scaling method has been correctly applied within these solutions.

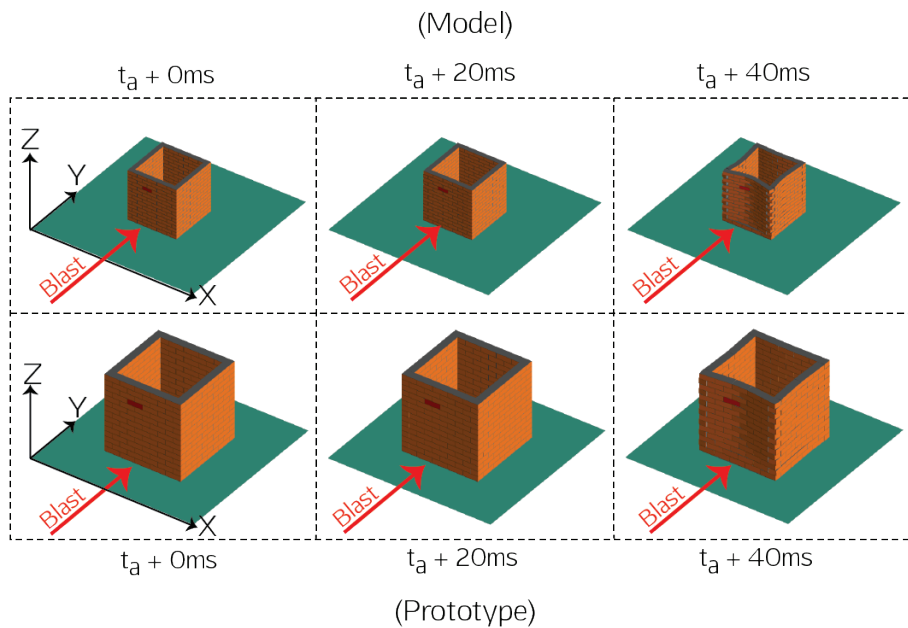


Figure 8.26: AEM masonry response, model & prototype 0-40ms, ABT Trials 1-2.

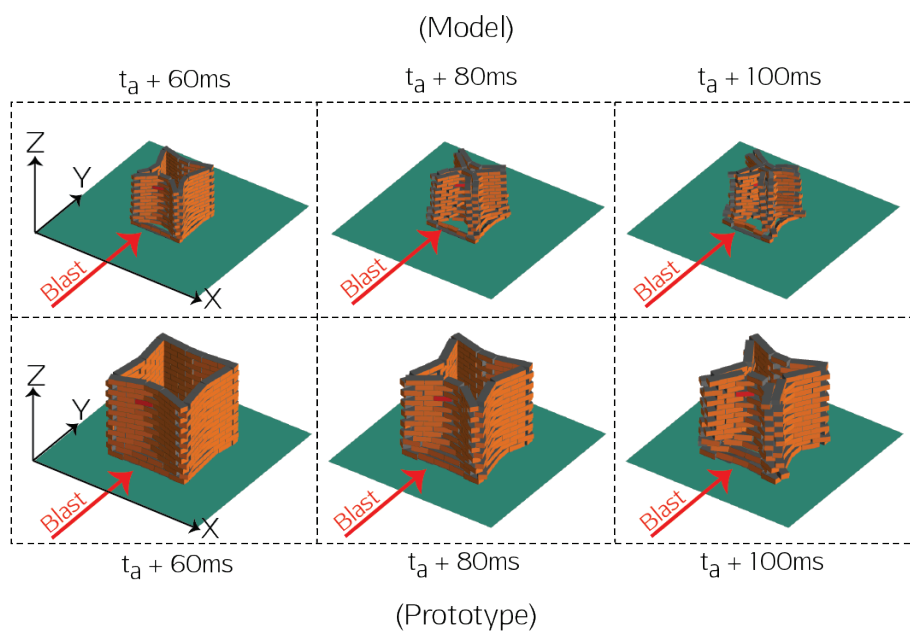


Figure 8.27: AEM masonry response, model & prototype 60-100ms, ABT Trials 1-2.

Figures 8.28.-8.29 illustrate the spatial rubble distribution for the prototype and model structures in ABT trials 1-2. It can be immediately observed that the fragment throw distances greatly exceed those recorded for the short-duration blast trial at 55kPa. Total mass in the ABT's global X and Y axes has been logged within a grid which utilises a bin resolution of 0.5m x 0.5m. Initial inspection of these distributions indicates that the model produced a narrower distribution in the X axis whereas the Y direction results correlate more closely with the prototype in terms of the maximum positive Y position.

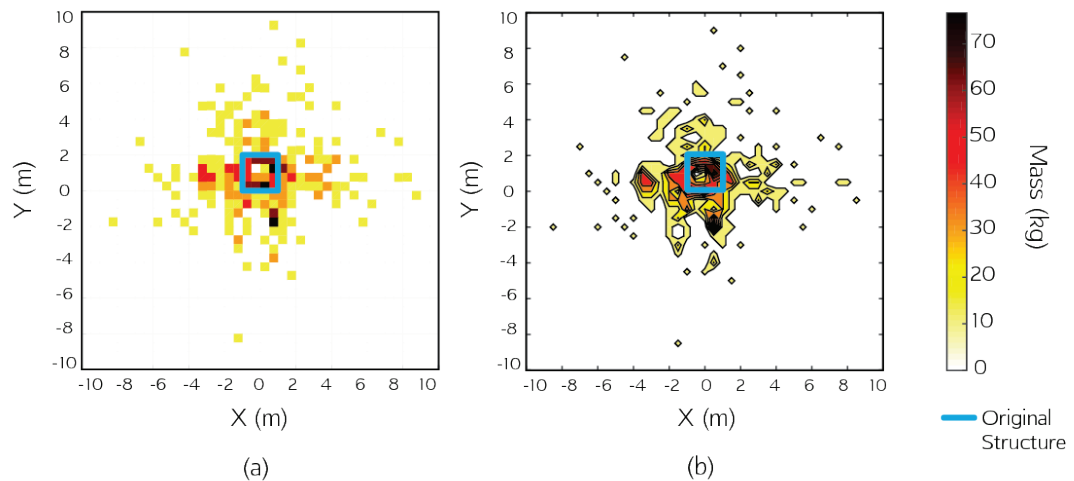


Figure 8.28: AEM masonry rubble distribution, absolute mass of Prototype, ABT Trial 1: (a) 0.5m x 0.5m bins; (b) Smoothed contours.

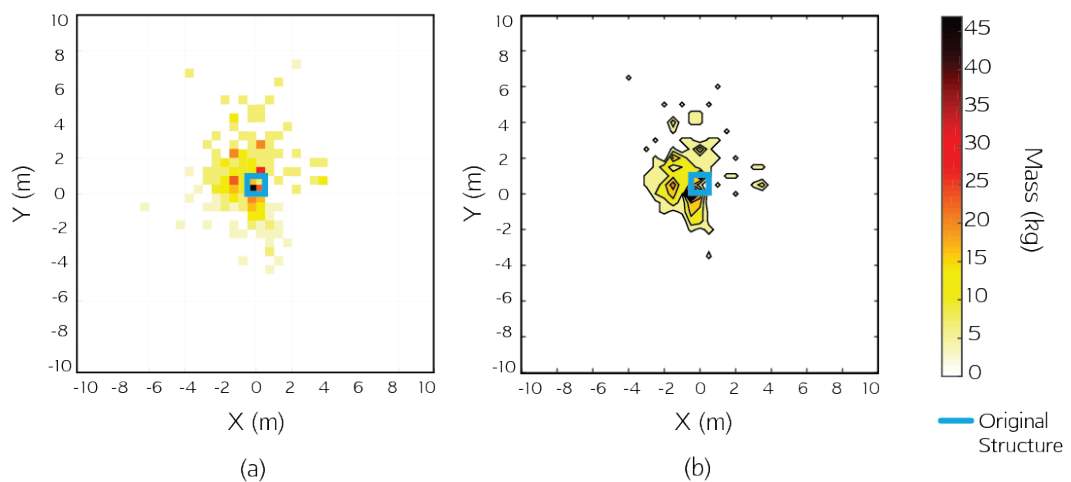


Figure 8.29: AEM masonry rubble distribution, absolute mass of Model, ABT Trial 2: (a) 0.5m x 0.5m bins; (b) Smoothed contours.

Figures 8.30-8.31 extend these results by plotting the percentage of total mass in the ABT's global X and Y axes with the same binning resolution. The inspection of which confirms

that the model produced a narrower X distribution whilst more closely matching the Y distribution. Due to the much larger spread of masonry debris, this section will assess the X and Y distribution data as separate components. In doing so, figure 8.32 can be seen to overlay prototype and model histogram results for percentage of total mass in the X direction. The inspection of which shows that the model distribution is more tightly concentrated between -1.5m and +1m than the prototype. Interestingly, both the model and prototype results demonstrate visual properties which align with a normal distribution. This is shown in figures 8.33-8.34 for the prototype and model distributions respectively where the two sets of histogram data are plotted with a theoretical Gaussian fit.

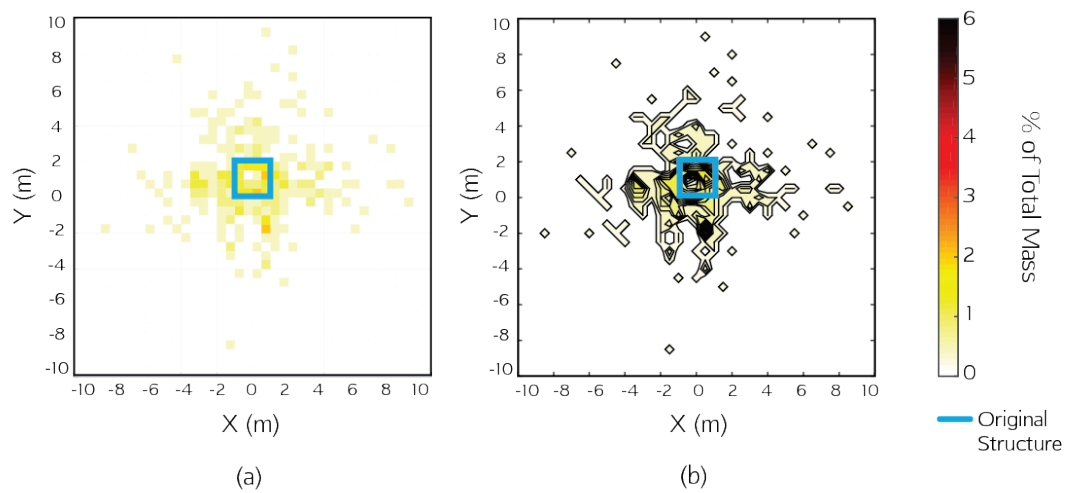


Figure 8.30: AEM masonry rubble distribution, % of total mass of Prototype, ABT Trial 1: (a) 0.5m x 0.5m bins; (b) Smoothed contours.

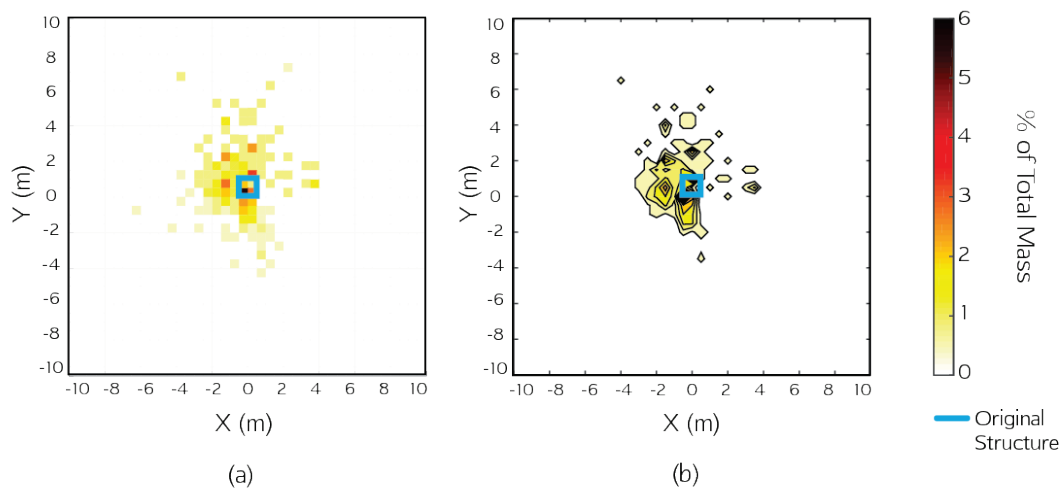


Figure 8.31: AEM masonry rubble distribution, % of total mass of Model, ABT Trial 2: (a) 0.5m x 0.5m bins; (b) Smoothed contours.

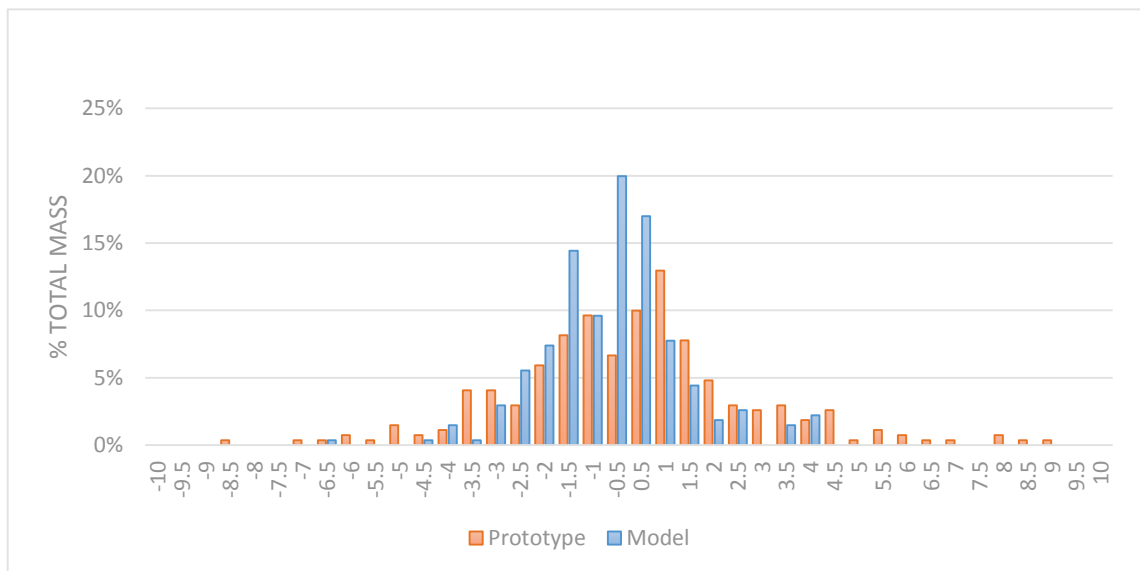


Figure 8.32: AEM masonry rubble distribution histogram in X, % of total mass of model & prototype, ABT Trials 1-2.

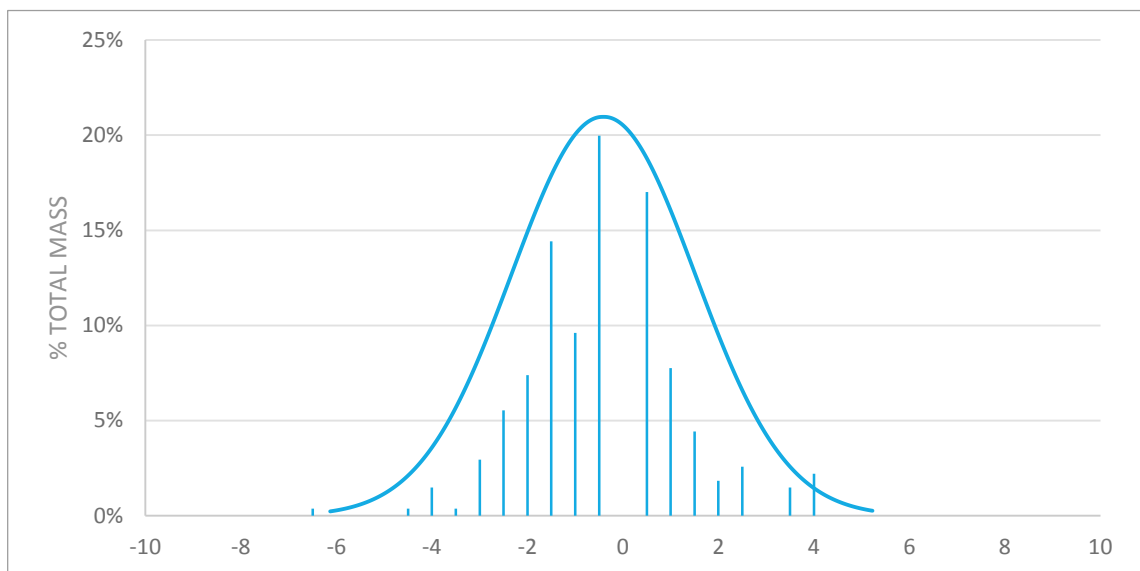


Figure 8.33: AEM masonry rubble distribution histogram in X, % of total mass of model with theoretical Gaussian fit, ABT Trial 2.

Table 8.10 compares theoretical Gaussian fit results for percentage of total mass within ± 1 and ± 2 standard deviations of the mean to binned data for the model and prototype X distributions. An examination of which shows the ideal fit to under-predict within ± 1 standard deviation by -9% and -18% for the prototype and model. Further examination shows the ideal fit to closely match prototype and model results within ± 2 standard deviations with minor differences of -1% and -2%. Figure 8.35 overlays two Gaussian fits for the model and prototype results where it can be seen that the model distribution is condensed with the mean shifted slightly in the negative X direction versus the prototype. These differences in the

X distribution width indicate that a transformation function may be required to enable the model to represent the prototype at 55kPa. The future development of which may enable a Gaussian prediction function to be derived and thus provide a broader data set of debris distribution data in tandem with experimental and computational methods.

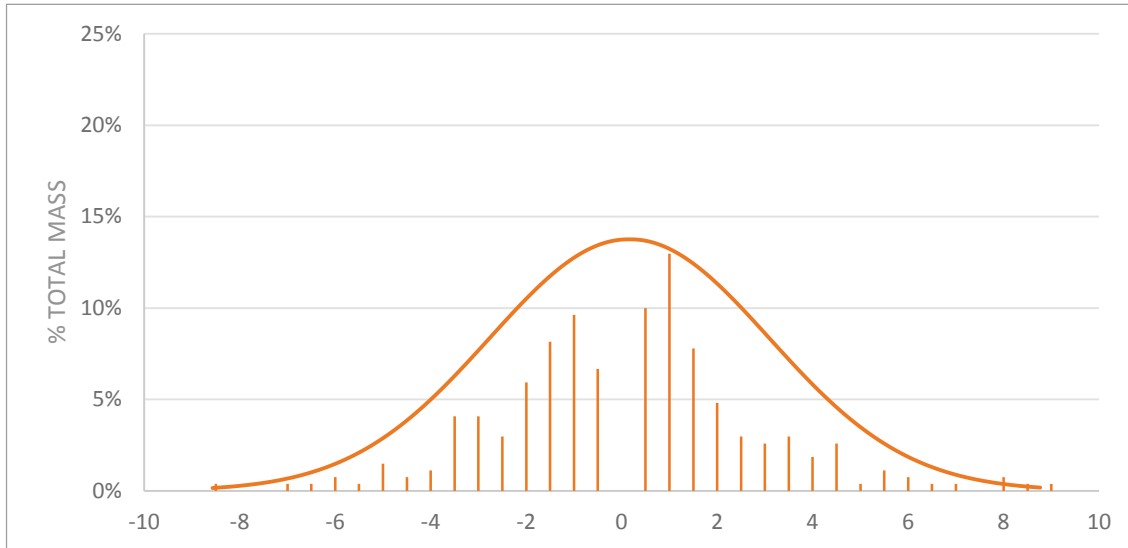


Figure 8.34: AEM masonry rubble distribution histogram in X, % of total mass of prototype with theoretical Gaussian fit, ABT Trial 1.

Table 8.10: Statistical fit of AEM rubble distribution in X, model and prototype, ABT Trials 1-2.

	Mean +/- 1* σ (% Total Mass)			Mean +/- 2* σ (% Total Mass)		
	Ideal	Binned Equivalent	+/-	Ideal	Binned Equivalent	+/-
Prototype	68%	77%	-9%	95%	96%	-1%
Model	68%	86%	-18%	95%	97%	-2%

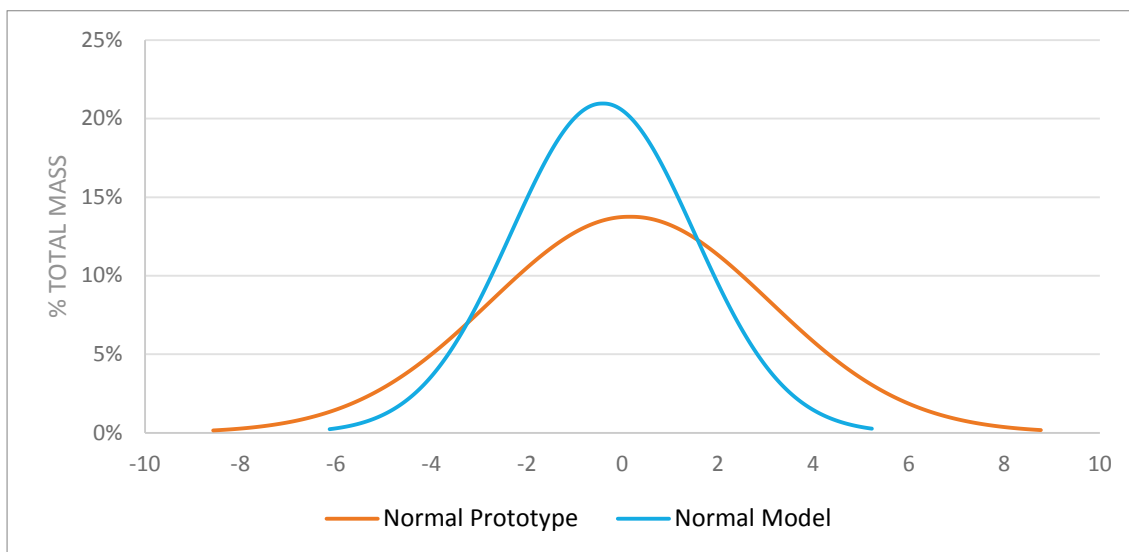


Figure 8.35: Gaussian fit of AEM masonry rubble distribution histogram in X, % of total mass, model & prototype, ABT Trials 1-2.

Figure 8.36 focusses on the debris distribution in the Y direction by presenting an overlay of prototype and model histogram results for percentage of total mass in Y. An inspection of this figure shows that the model distribution closely matches the prototype results with both data sets demonstrating visual properties which align with a normal distribution. This is shown in figures 8.37-8.38 for the prototype and model distributions respectively where the histogram data has been plotted with a theoretical Gaussian fit.

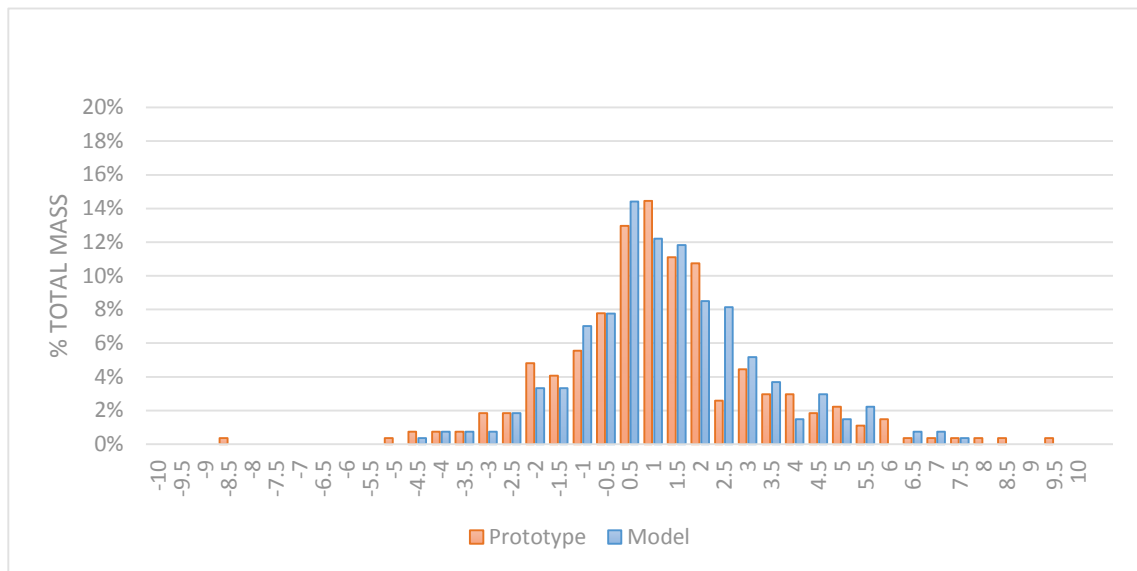


Figure 8.36: AEM masonry rubble distribution histogram in Y, % of total mass of model & prototype, ABT Trials 1-2.

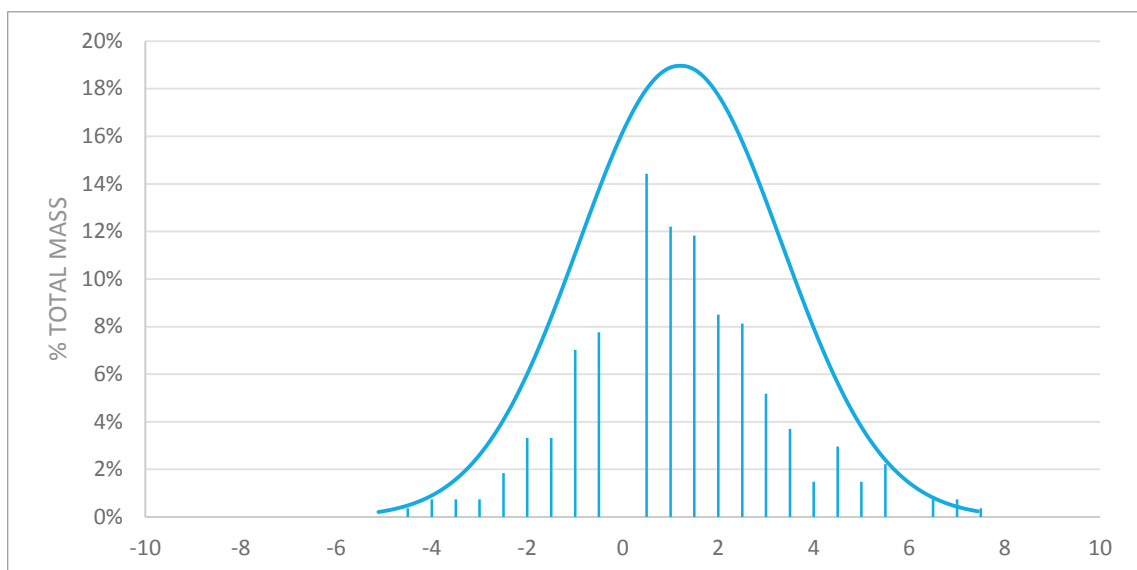


Figure 8.37: AEM masonry rubble distribution histogram in Y, % of total mass of prototype with theoretical Gaussian fit, ABT Trial 1.

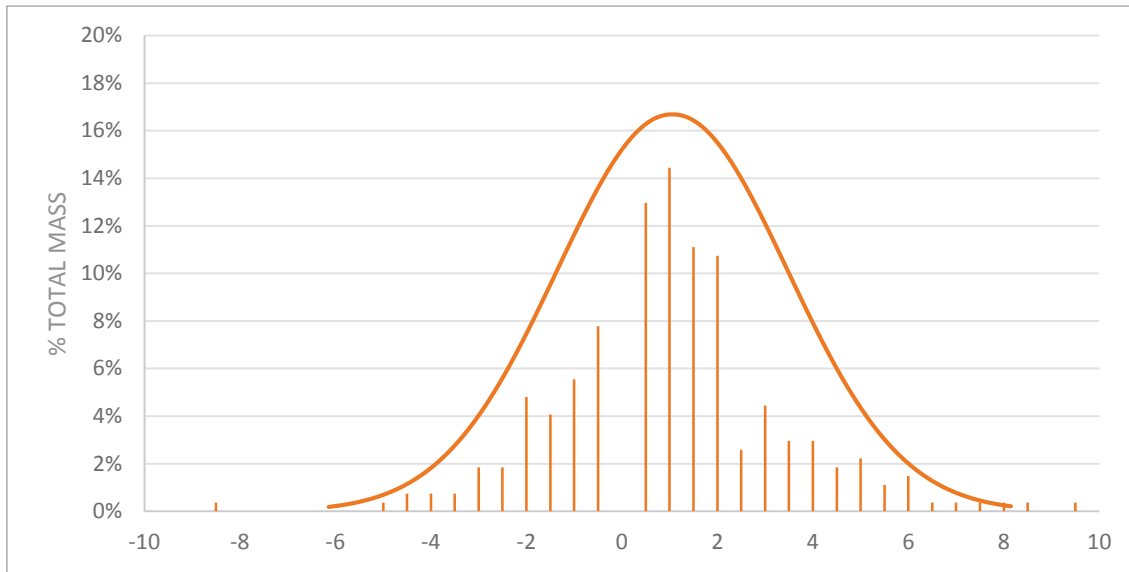


Figure 8.38: AEM masonry rubble distribution histogram in Y, % of total mass of model with theoretical Gaussian fit, ABT Trial 2.

Table 8.11 compares the theoretical Gaussian fit results for percentage of total mass within ± 1 and ± 2 standard deviations of the mean to the binned data for the model and prototype Y distributions. Initial inspection of this table shows that ideal fit under-predicts the percentage of total mass within ± 1 standard deviation by -9% and -11% for the prototype and model respectively. Further examination of table 8.11 shows the ideal fit to closely match the prototype and model results within ± 2 standard deviations with negligible differences of -1%. Figure 8.39 overlays two Gaussian fits for the model and prototype results where it can be seen that the model distribution closely replicates the prototype albeit with a slightly greater concentration of mass at the mean location versus the prototype. This indicates that the model Y distribution data may provide a direct representation of prototype data, thus inferring that the derivation of a Gaussian prediction function may enable a broader set of debris data to be developed in tandem with experimental and computational methods.

Table 8.11: Statistical fit of AEM rubble distribution in Y, model and prototype, ABT Trials 1-2.

	Mean +/- 1* σ (% Total Mass)			Mean +/- 2* σ (% Total Mass)		
	Ideal	Binned Equivalent	+/-	Ideal	Binned Equivalent	+/-
Prototype	68%	77%	-9%	95%	96%	-1%
Model	68%	79%	-11%	95%	96%	-1%

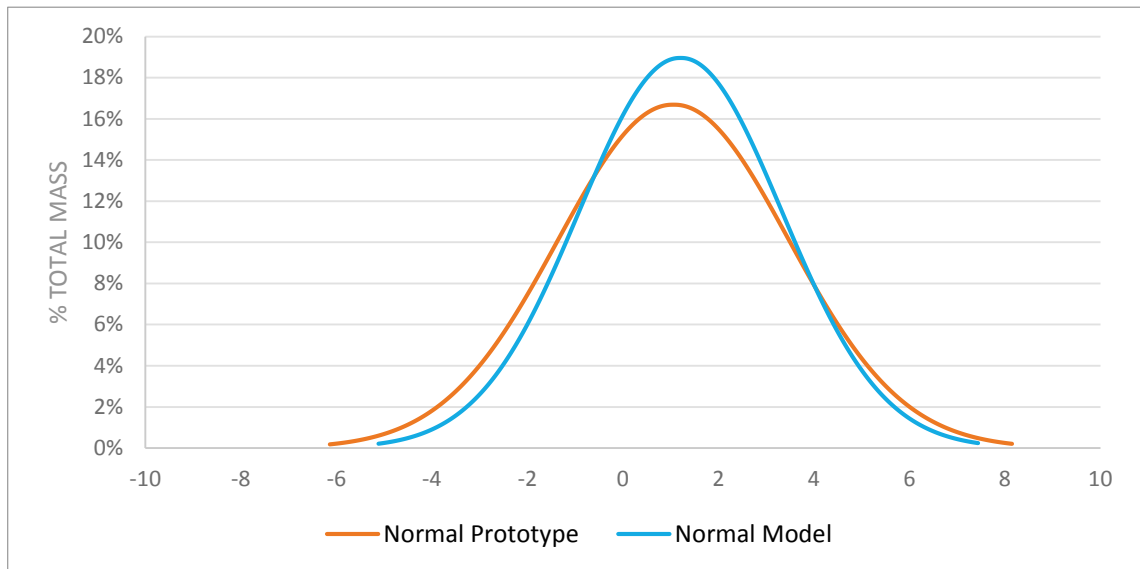


Figure 8.39: Gaussian fit of AEM masonry rubble distribution histogram in Y, % of total mass, model & prototype, ABT Trials 1-2.

8.3.6 AEM analysis - ABT Trials 3-4

Figures 8.40-8.41 show a time-lapse progression of blast wave interaction with AEM masonry structures for ABT trials 3-4 at 110kPa. It can be seen that both the model and prototype structures fail via the same symmetrical ‘squashing’ action identified in section 8.3.5. As a result, it can be inferred that this failure mode is not likely to be representative of experimentally observed results due to the flexible corner restraints modelled within these solutions. Despite this, it can be seen that the model structure replicates the failure mode of the prototype, indicating an accurate application of scaling within these solutions.

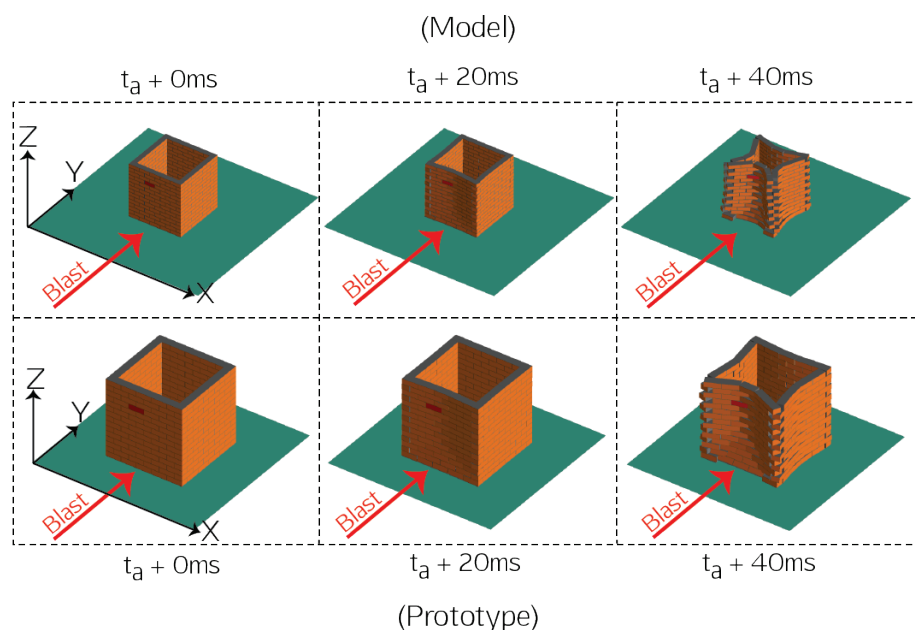


Figure 8.40: AEM masonry response, model & prototype 0-40ms, ABT Trials 3-4.

Scaling Masonry Structures in a Blast Environment (Model)

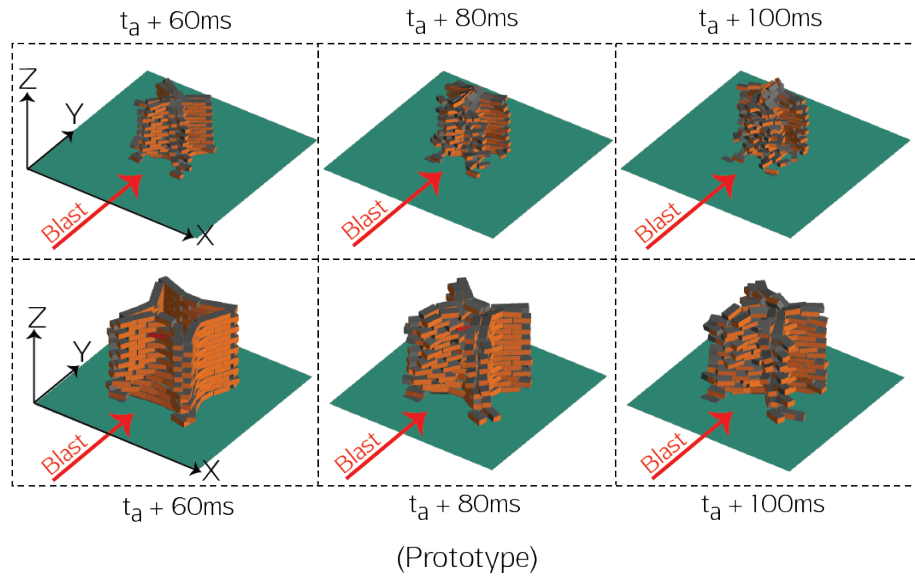


Figure 8.41: AEM masonry response, model & prototype 60-100ms, ABT Trials 3-4.

Figures 8.42-8.43 present the total mass debris distribution of the prototype and model structures in ABT trials 3-4. It can be clearly seen that the fragment throw distances greatly exceed those recorded for the short-duration blast trial at 110kPa as well as those for the ABT trial prediction at 55kPa. These figures utilise the ABT's global X and Y axes with a bin resolution of 0.5m x 0.5m. Initial examination indicates that the model produced a slightly narrower distribution width in the X axis whilst the Y distribution results correlate more closely with the prototype results.

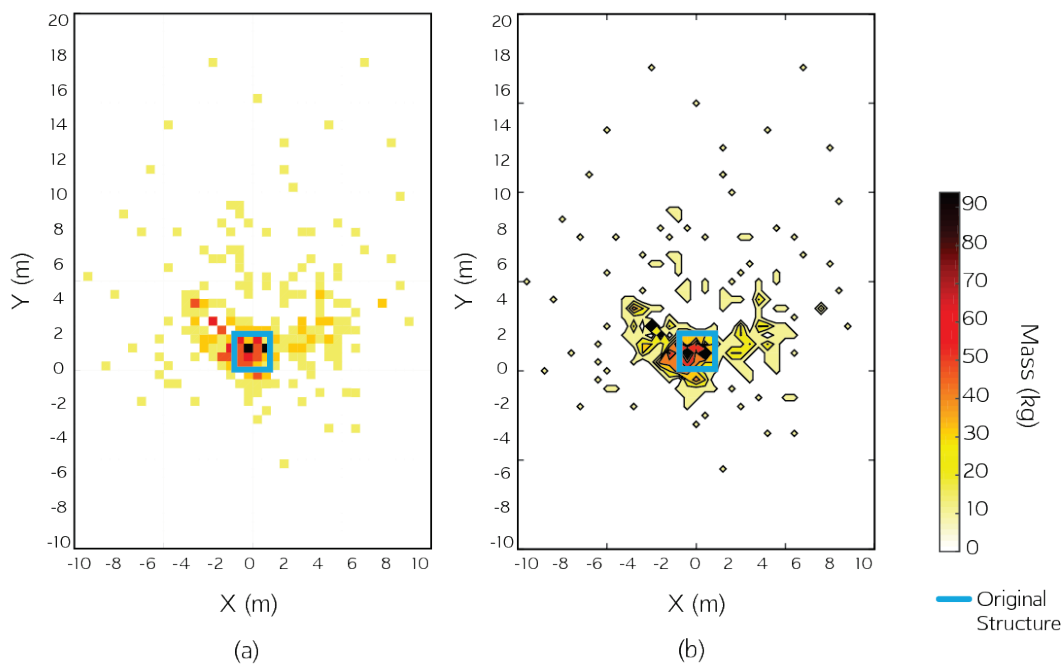


Figure 8.42: AEM masonry rubble distribution, absolute mass of Prototype, ABT Trial 3: (a) 0.5m x 0.5m bins; (b) Smoothed contours.

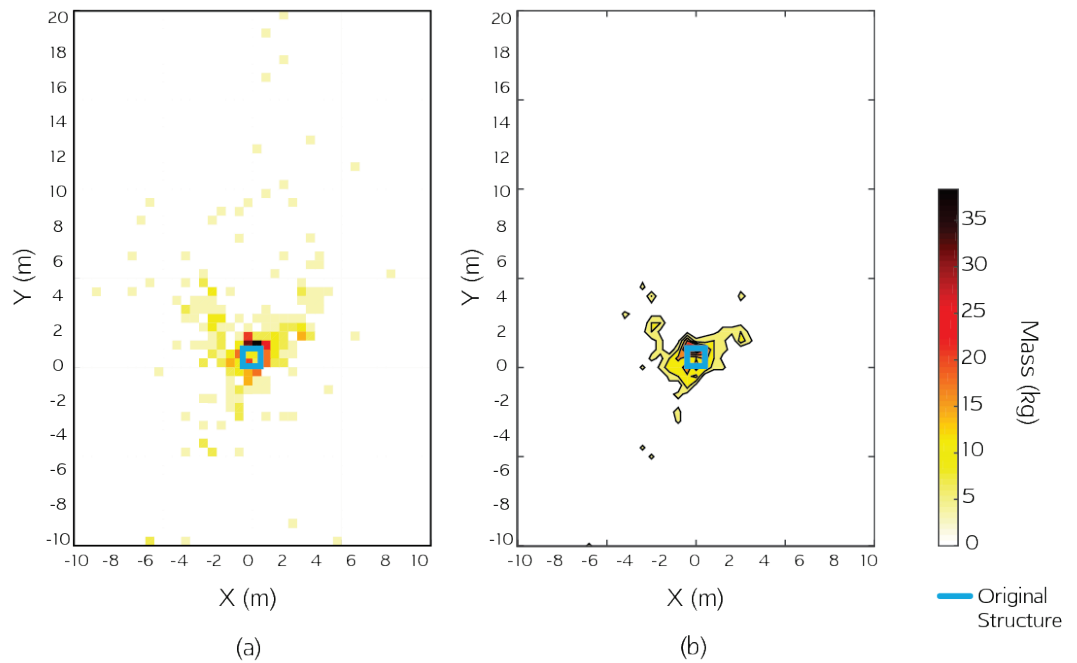


Figure 8.43: AEM masonry rubble distribution, absolute mass of Model, ABT Trial 4: (a) 0.5m x 0.5m bins; (b) Smoothed contours.

Figures 8.44-8.45 plot the percentage of total mass in the ABT's global X and Y axes with the same 0.5m x 0.5m bin resolution. The examination of which further confirms that the model solution produced a narrower X distribution whilst the Y can be seen to more closely correlate with the prototype distribution in the same direction.

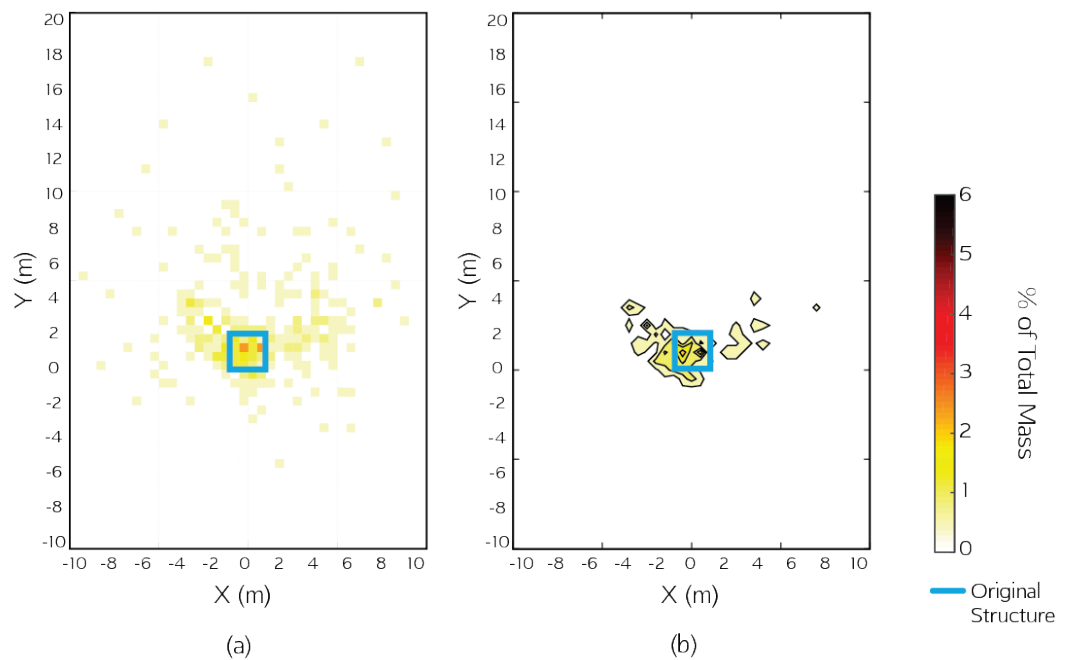


Figure 8.44: AEM masonry rubble distribution, % of total mass of Prototype, ABT Trial 3: (a) 0.5m x 0.5m bins; (b) Smoothed contours.

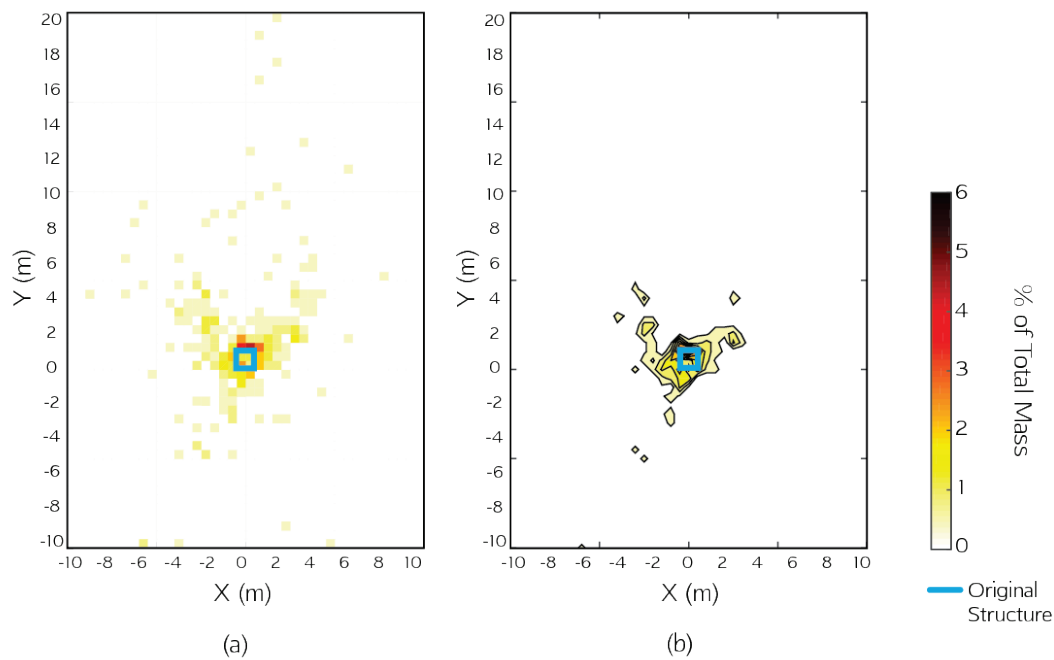


Figure 8.45: AEM masonry rubble distribution, % of total mass of Model, ABT Trial 4: (a) 0.5m x 0.5m bins; (b) Smoothed contours.

Due to the large spread of masonry debris, this section will also assess the X and Y distribution data as separate components. Figure 8.46 overlays prototype and model histogram results for percentage of total mass in the X direction where it can be seen that the model distribution is more tightly concentrated than the prototype. It can also be seen that both sets of data also demonstrate the visual properties of a normal distribution. This is shown in figures 8.47-8.48 for the prototype and model distributions where the histogram data has been plotted with a theoretical Gaussian fit. Table 8.12 compares theoretical Gaussian fit results for percentage of total mass within ± 1 and ± 2 standard deviations of the mean to the distribution data of the model and prototype. Initial inspection of table 8.12 shows that the ideal fit under-predicts the percentage of total mass within ± 1 standard deviation by -6% and -11% for the prototype and model results. Further examination shows that the ideal fit closely matches the prototype and model results within ± 2 standard deviations with a difference of -1% for both data sets. Figure 8.49 overlays two Gaussian curves for the model and prototype results. Examination of this figure demonstrates that the model distribution is condensed with the mean shifted slightly in the negative X direction versus the prototype. These results therefore indicate a transformation function may also be required for the model X distribution as seen with the 55kPa debris results. The development of which would enable the model data to directly represent a prototype equivalent.

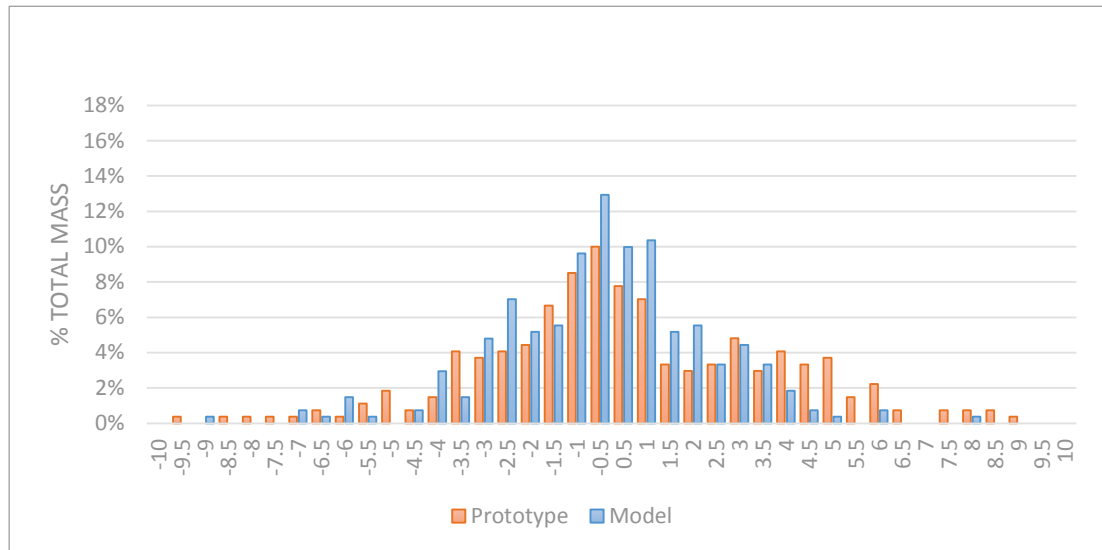


Figure 8.46: AEM masonry rubble distribution histogram in X, % of total mass of model & prototype, ABT Trials 3-4.

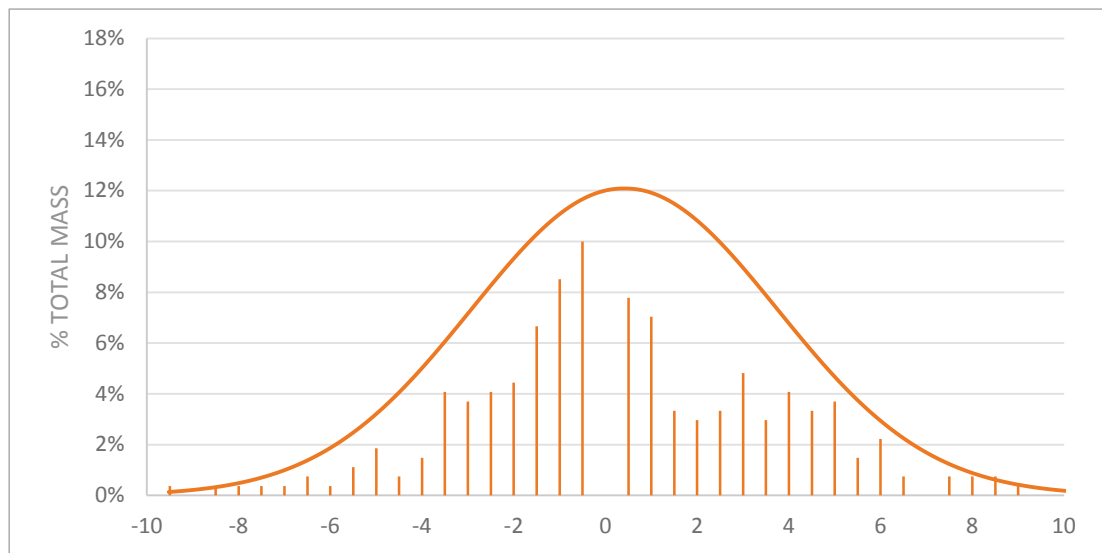


Figure 8.47: AEM masonry rubble distribution histogram in X, % of total mass of prototype with theoretical Gaussian fit, ABT Trial 3.

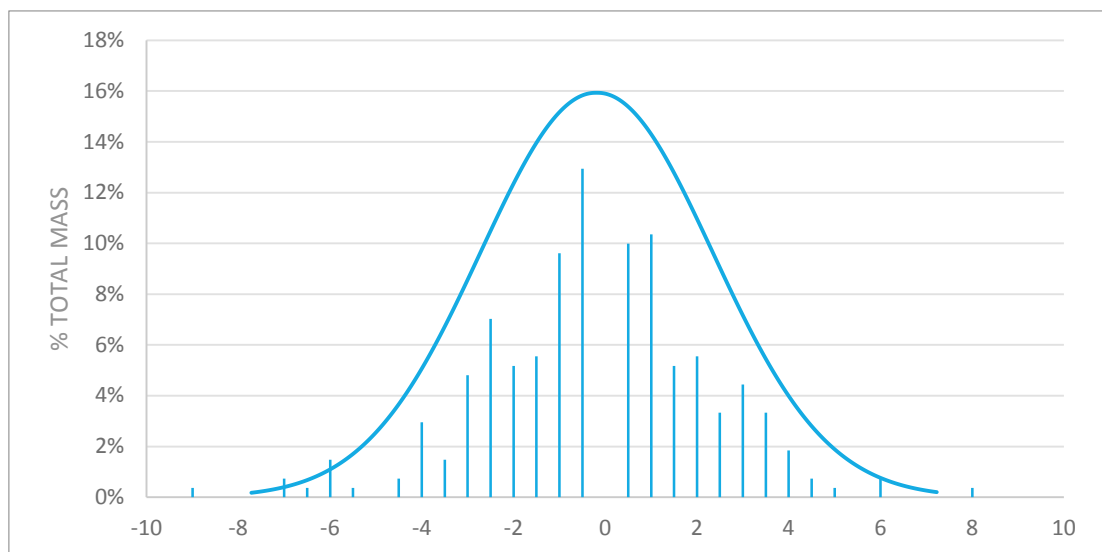


Figure 8.48: AEM masonry rubble distribution histogram in X, % of total mass of model with theoretical Gaussian fit, ABT Trial 4.

Table 8.12: Statistical fit of AEM rubble distribution in X, model and prototype, ABT Trials 3-4.

	Mean +/- 1* σ (% Total Mass)			Mean +/- 2* σ (% Total Mass)		
	Ideal	Binned Equivalent	+/-	Ideal	Binned Equivalent	+/-
Prototype	68%	74%	-6%	95%	96%	-1%
Model	68%	79%	-11%	95%	96%	-1%

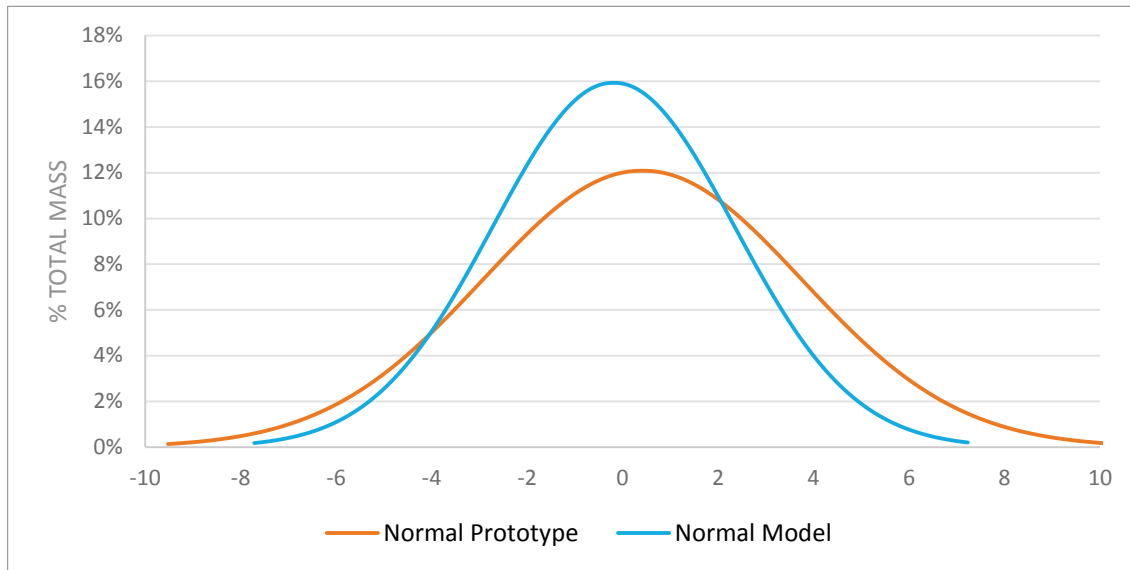


Figure 8.49: Gaussian fit of AEM masonry rubble distribution histogram in X, % of total mass, model & prototype, ABT Trials 3-4.

Figure 8.50 focusses on the debris in the Y direction where it can be seen to overlay the prototype and model histogram results for percentage of total mass in Y direction. Examination of this figure shows that the model distribution closely matches the prototype results albeit with a slightly larger concentration of mass at the mean location. Much like the previously examined results, these model and prototype results in the Y direction also demonstrate the visual properties of a normal distribution. This is shown in figures 8.51-8.52 for the prototype and model distributions respectively where the histogram data has been plotted with a theoretical Gaussian fit. In contrast to ABT Trials 1-2, this data is more positively skewed due to the increase in overpressure and impulse. Indicating that the Gaussian fit would require extending beyond +/- 3 standard deviations to account for extreme positive Y locations.

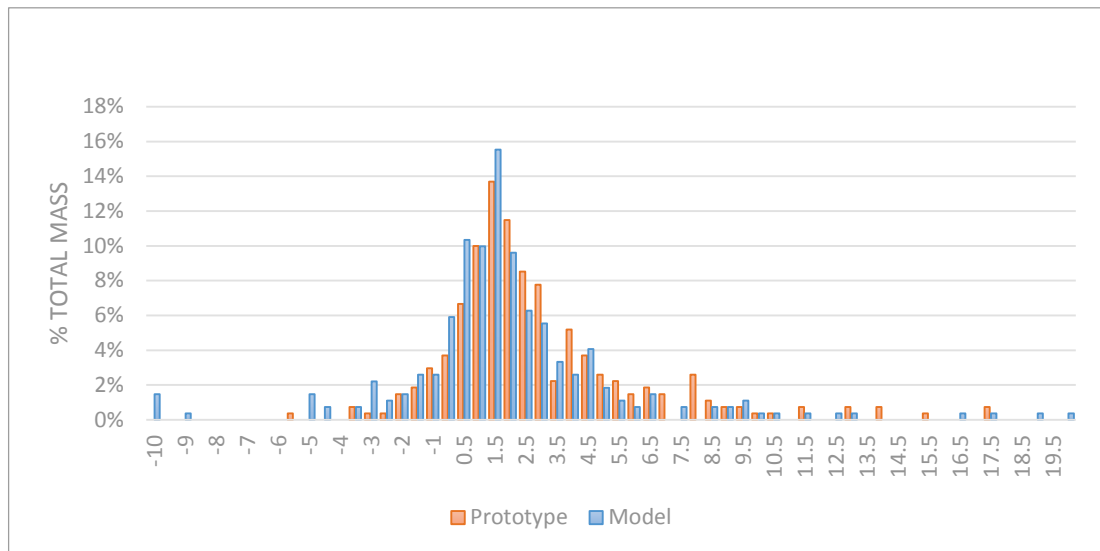


Figure 8.50: AEM masonry rubble distribution histogram in Y, % of total mass of model & prototype, ABT Trials 3-4.

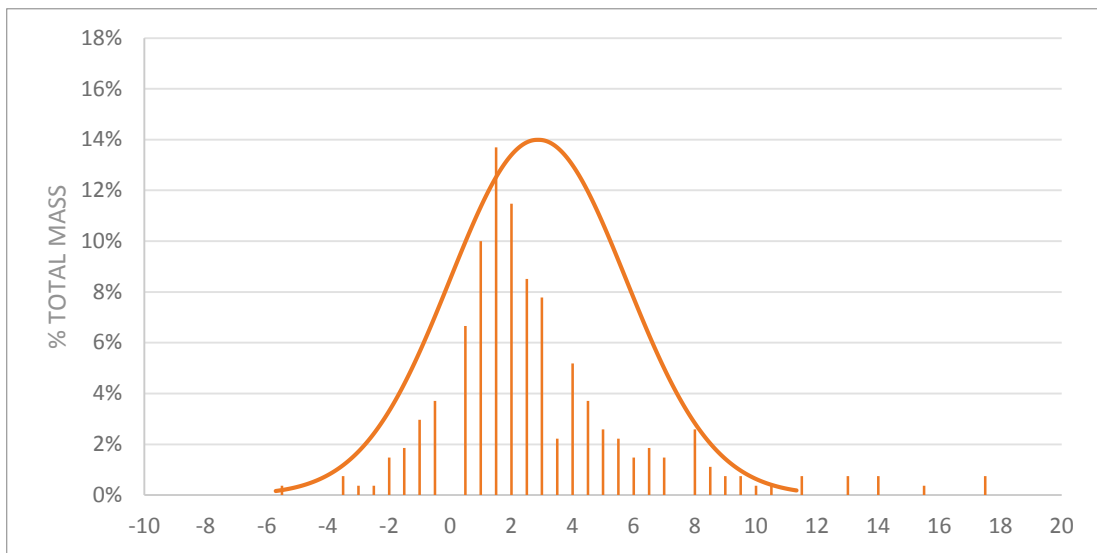


Figure 8.51: AEM masonry rubble distribution histogram in Y, % of total mass of prototype with theoretical Gaussian fit, ABT Trial 3.

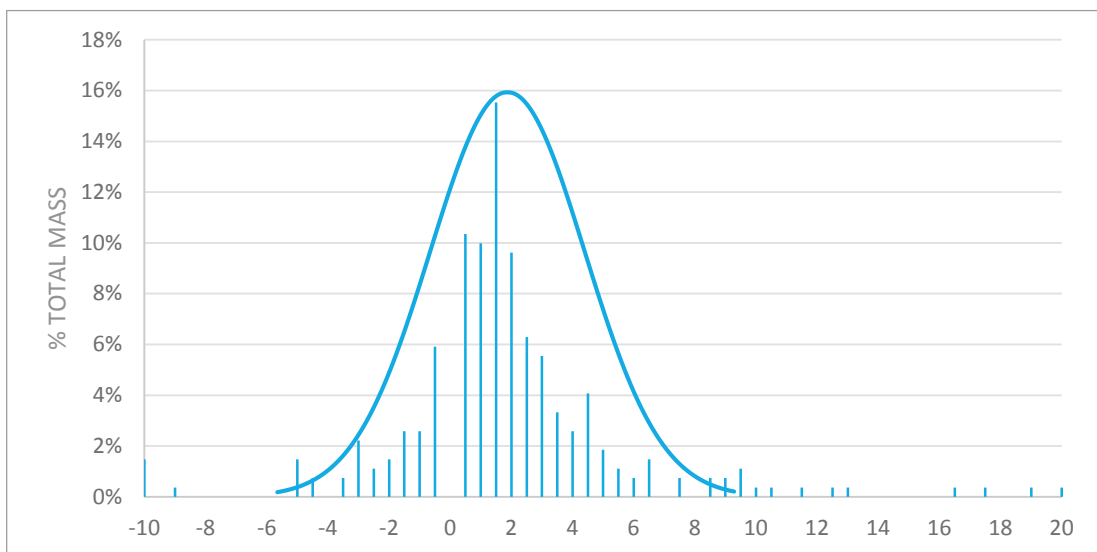


Figure 8.52: AEM masonry rubble distribution histogram in Y, % of total mass of model with theoretical Gaussian fit, ABT Trial 4.

Table 8.13 compares the theoretical Gaussian fit results for percentage of total mass within ± 1 and ± 2 standard deviations of the mean to the binned data from the model and the prototype Y distributions. Initial inspection of this table shows the ideal fit to under-predict the percentage of total mass within ± 1 standard deviation by -11% and -8% for the prototype and model results. Further examination of table 8.13 shows that the ideal fit closely matches the prototype and model results within ± 2 standard deviations with differences of +1% and +6% respectively. Figure 8.53 overlays two Gaussian curves for the model and prototype results where it can be seen that the model distribution reasonably replicates the prototype albeit with a slightly greater concentration of mass at the mean location which is moderately shifted in the negative Y direction. This indicates that the model Y distribution data may be able to provide representation of prototype data after the application of an offset function. As a result, it may be possible to derive a Gaussian prediction function which enables debris data to be modelled in tandem with experimental and computational methods

Table 8.13: Statistical fit of AEM rubble distribution in Y, model and prototype, ABT Trials 3-4.

	Mean +/- 1* σ (% Total Mass)			Mean +/- 2* σ (% Total Mass)		
	Ideal	Binned Equivalent	+/-	Ideal	Binned Equivalent	+/-
Prototype	68%	79%	-11%	95%	94%	+1%
Model	68%	76%	-8%	95%	89%	+6%

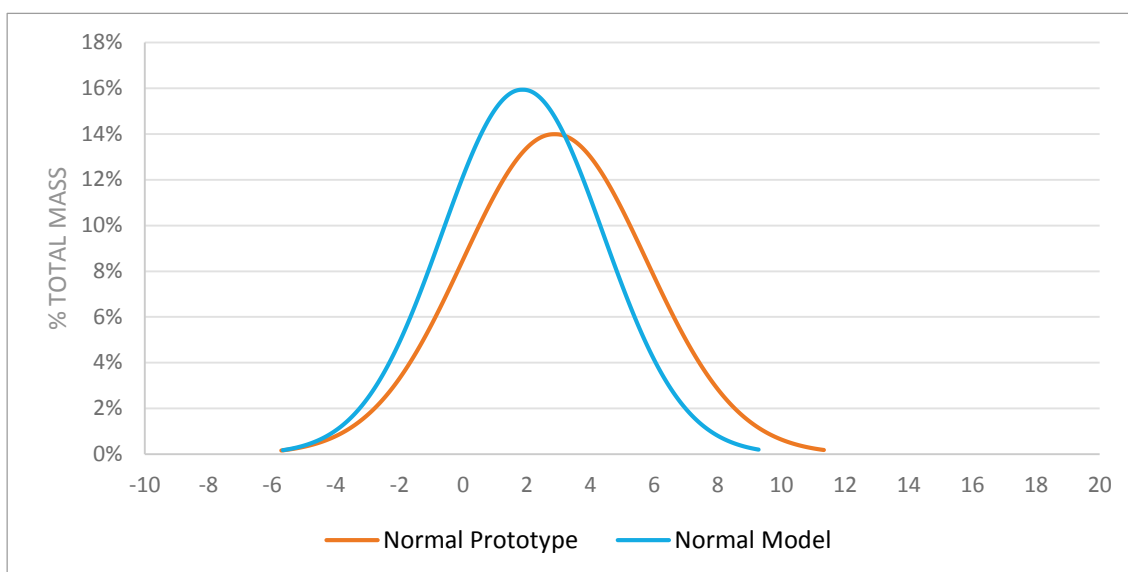


Figure 8.53: Gaussian fit of AEM masonry rubble distribution histogram in Y, % of total mass, model & prototype, ABT Trials 3-4.

8.4 Summary

This chapter has detailed the design of four long-duration blast trials which aim to benchmark the PhD scaling methodology within a high-impulse blast scenario. These trial designs were followed by a preliminary analysis of the expected trial results by producing CFD models of blast flow and AEM models of masonry response to fulfil the fourth objective of this research.

This chapter then presented the results of a comparative study to determine the relative accuracy of Air3D or AUTODYN solutions of long-duration blast flow when benchmarked against existing trial data. This showed comparable levels of accuracy with both solvers producing results within $\pm 10\%$ of experimental free-field results. Despite this, Air3D solutions were found to produce far quicker run times and as such, this software was utilised for the remainder of this chapter's CFD modelling. Air3D solutions were subsequently produced to replicate the free-field and reflected blast environments expected within the ABT trials. Examination of free-field blast parameters versus predictions revealed impulse error levels of $\pm 10\%$ for the ABT models, indicating that the models met the acceptability criteria of $\pm 10\%$. This enabled the re-mapping of reflected pressure data to configure blast loading within AEM models of masonry response.

AEM models of masonry response were produced with ELS to investigate the performance of this PhD's scaling methodology for characterising masonry damage and debris within a high-impulse blast scenario. Examination of masonry damage with ABT trials 1-4 demonstrated a 'squashing' failure mode in each case for both the model and prototype AEM structures, indicating that the model structures were able to qualitatively replicate the damage incurred by the prototypes. It was also found that this failure mode may not be representative of that expected within the experimental trials due to the lack of stiffness and moment resistance at the structural corners which prevented cracking. Despite this potential variance, it can also be shown that the substantial increase in impulse for long-duration blast scenarios infers a high likelihood of total structural failure and the sizable projection of masonry fragments downstream. As a result, data captured from the future conducting of long-duration trials may be found to demonstrate that the influence of cracking is less dominant when the structure is subject to far greater translational forces. Inferring that the

hierarchy of masonry response modes identified within the short-duration blast trials may not exert the same influence over debris generation in a long-duration blast.

Analysis of the debris distributions in ABT trials 1-2 with 55kPa overpressure revealed the model results to have a condensed X distribution width versus the prototype. In contrast, the model's Y distribution was found to closely match the prototype. Interestingly, each of the AEM rubble distributions could be seen to exhibit Gaussian curve characteristics in both the X and Y directions. This enabled theoretical curves to be fitted to each data set with reasonable levels of agreement in the ranges of ± 1 and ± 2 standard deviations of the mean for percentage of total mass. Examination of the debris distributions in ABT trials 3-4 with 110kPa overpressure showed the model to have a condensed X distribution width versus the prototype whilst the model's Y distribution was found to closely match the prototype. Thus inferring that the model data may be able to directly represent the prototype distribution in the Y axis. In contrast to ABT trials 1-2 at 55kPa, the model and prototype Y distributions in ABT trials 3-4 demonstrated a greater positive skew due to increased overpressure and impulse. Despite this, both the model and prototype distributions in the X and Y directions were found to possess Gaussian characteristics, enabling theoretical curve fits with similar levels of agreement to that seen in ABT trials 1-2.

Based on the demonstrated levels of debris agreement for the model and prototype in the Y direction, it was found that the model Y distribution may be able to directly represent the prototype. It was also found that future development of transformation functions for model debris distributions in the X direction may be required to match prototype debris data in this orientation. As a result, the future development of Gaussian prediction functions may enable the generation of masonry debris data in tandem with experimental and computational methods.

Chapter 9

Conclusions and Future Work

9.1 Conclusions

The primary aim of this thesis has been to develop and experimentally benchmark a new scaling methodology for unreinforced masonry structures subject to blast loading by utilising the aerodynamic concepts of dynamic similitude. In doing so, this research sought to demonstrate that reduced-scale (model) structures can be built using commercially available materials at a 1:2 scaling level and subsequently deployed within smaller, cost-effective blast trials to characterise the damage (cracking and collapse) and debris distribution of full-scale (prototype) structures. This research has also assessed CFD and AEM models of blast flow propagation and masonry response via experimental benchmarking to determine the viability of these techniques to provide a predictive capacity when experimental trials are unavailable.

In chapter 2, a review of the relevant background theory relating to this research was presented including blast wave formation, the interaction of blast waves with structures and fundamental blast parameters. This chapter also included a summary of the essential concepts for the computational methods of CFD and AEM. In chapter 3, the concepts of masonry structural response to blast loading alongside a synopsis of fundamental scaling theory was presented to complete the foundation of knowledge relating to this research.

Chapter 3 also reviewed the limited number of studies into experimental scaling of unreinforced masonry in a blast environment where existing research was found to utilise geometric similarity via full-scale materials to enable computational benchmarking and the

demonstration of a future predictive capacity. The use of full-scale materials was however found to be insufficient due to an inequality between prototype and model ratios of lateral to vertical forcing. This chapter also reviewed tangentially related research where the majority referred to seismic loading with varying versions of geometric scaling. Somewhat surprisingly, a large section of the reviewed research combined experiments with computational benchmarking, but none were found to demonstrate the accuracy of the scaling method via comparisons with prototype data.

Chapter 4 presented the development of a new scaling methodology for unreinforced masonry structures subject to blast loading, thus addressing the principal aim of this PhD and fulfilling the first research objective. It was shown that geometric similarity underpins a successful structural scaling approach, but the reliance upon construction via full-scale materials is insufficient due to an inequality in the ratio of lateral to vertical blast forcing. The result of which was shown to be an underestimation of vertical forcing for a model, producing an overestimation of damage and debris spread. Additional numerical derivations were thus produced by adopting a rule-set defined by dynamic similitude's core requirements of equal lateral to vertical force ratios for geometrically similar structures. As a result, it was shown that a model structure's density could be scaled via a reciprocal scale factor to introduce a pseudo-increase in structural inertia. The combination of which with geometric similarity was found to satisfy the core requirements of dynamic similitude. The scaling methodology developed within this research was thus summarised as the implementation of geometric similarity in tandem with a reciprocal scale factor for model structure density. This methodology implements a first principles approach where a masonry structure is represented as an inertial resistance to blast loading. Chapter 4 also identified two commercially available masonry construction units which adhere to the geometric and density scaling requirements at a 1:2 scale level, thus enabling an improvement over the limited accuracy of geometric scaling with prototype materials without the need to manufacture bespoke, scaled materials.

Chapters 5-6 detailed the experimental component of this thesis, thus fulfilling the principal aim and the second objective of this research. One short-duration blast trial was conducted in May 2018 with the aim of benchmarking the masonry scaling methodology derived in chapter 4. This trial deployed two pairs of prototype and model structures which

were constructed from commercially available materials at a 1:2 scaling level to enable the capture of masonry damage and debris distribution data with 100kg TNT equivalence. Chapter 6 commenced with an analysis of the experimental blast environment where results showed an amplification in blast pressure alongside a degree of blast wave asymmetry due to a physically cylindrical explosive charge. Of the two characteristics, blast pressure amplification was found to be dominant with a mean increase of +34% versus pre-trial pressure predictions. A smaller mean difference of +6% for impulse was shown to meet the trial design acceptability criteria of $\pm 10\%$, indicating accurate application of blast momentum to the masonry structures. This was essential given the representation of masonry as an inertial resistance to blast loading. It was also found that the measured trial environment closely replicated that from a physically spherical 134kg TNT charge

Chapter 6 then assessed the scaling methodology's ability to characterise masonry damage. Qualitative data captured at 14.25m showed that the prototype and model structures were both subject to full front panel collapse, partial side panel collapse and rear panel survival. Thus indicating that the model qualitatively replicated the prototype's collapse mode. The prototype and model structure both developed five primary cracks with vertical cracks at the rear panel and the fully restrained rear corners. Thus indicating that the model replicated the quantity and location of the prototype's primary cracks. Rear panel crack shape was found to differ slightly between prototype and model results alongside the extent of side panel collapse. Variable levels of ground bed fixity for the prototype and model was identified as the cause of these differences.

Masonry damage at 20m stand-off showed that the prototype and model both produced front panel deflection without collapse due to roof enclosure interactions. The prototype incurred partial side collapse which was not replicated by the model due to side panels being restrained by the roof enclosure. Thus indicating that the model replicated the prototype's frontal deflection with the potential for equivalent collapse if the roof enclosure designs represented equal stiffness and blast response. Further analysis demonstrated equal primary cracking behaviour at the prototype and model front panel, highlighting the ability to replicate cracking with equivalent levels of edge restraint. Primary crack quantities were found to differ due to differing side panel collapse. Side and rear primary crack locations were found at the 'built in' corners in both the model and prototype to represent equal damage via

the exceedance of an ultimate limit state. The model thus demonstrated the potential to replicate the full damage characteristics of the prototype at 20m based on actual similarities and potentially controllable differences via modified roof enclosure design.

Chapter 6 also examined quantitative masonry debris data at 14.25m where spatial distributions were generated in a non-dimensional grid for the prototype and model. This analysis showed net differences of $\sim 10\%$ for the total generation of debris which was encapsulated within the internal area distribution. These results met an acceptability criteria of $\pm 10\%$ which integrated the potential influence of varying edge restraint on sequential masonry response and debris generation. Examination of these differences revealed the cause to be additional side panel collapse and variance in ground bed fixity at 14.25m. A slightly conservative model damage prediction was thus found to yield a slightly conservative model debris spread. Additional debris analyses were conducted via high-level diagrams which showed percentage of original structural mass in non-dimensional quadrants at a resolution suited for fast-running engineering algorithms. The net increase in model debris was found to be confined to the rear quadrants whilst the front two quadrants represented the majority of the prototype and model debris, indicating that the model replicated the global debris distribution of the prototype.

Chapter 7 detailed the benchmarking of CFD and AEM models against experimental data from chapter 6 to assess the predictive capability of these methods, fulfilling the secondary PhD aim and the third research objective. This chapter commenced with a comparative study of Air3D and AUTODYN CFD solvers for modelling blast flow. AUTODYN models implemented 100kg TNT with a physically cylindrical charge shape whilst Air3D implemented 134kg TNT with a physically spherical charge. Stronger agreement for reflected and free-field solutions showed that Air3D more closely replicated the experimental environment via 134kg TNT with a spherical shape. Closer examination of Air3D free-field parameters at each experimental stand-off showed that impulse results met an acceptability criteria of $\pm 10\%$ versus the trial which was essential to this research's representation of masonry as an inertial blast resistance. Air3D solutions subsequently generated reflected blast data which was re-mapped to configure blast loading in AEM models. Chapter 7 also benchmarked AEM models of masonry response against experimental data from chapter 6. Initial analyses showed that AEM did not natively represent the stiffness and moment

resistance of fully restrained corners, preventing primary crack behaviour and its influence on sequential stages of deflection and collapse. A non-responding frame was modelled to provide the required corner stiffness and moment resistance which enabled masonry unit splitting at the prototype and model front corners at 14.25m stand-off. This edge condition also produced a damping effect on response via limited front panel cracking and zero side and rear panel cracking. The result of which was zero degrees of front panel or side panel collapse in the prototype and model solutions, producing a different damage state to the experimental results. AEM prototype and model response was also simulated at 20m with fully restrained corners where results showed zero cracking or deflection at each panel wall, indicating different global damage to the experimental results.

AEM's generation of debris was assessed by implementing a default representation of limited corner restraint. Additional masonry damage was observed at 14.25m stand-off in both the prototype and model solutions with equal levels of front panel and partial side panel collapse which produced internal debris fields. The AEM model solution was thus found to qualitatively replicate the damage and debris of the AEM prototype. AEM model and prototype data was also shown to qualitatively replicate the quantity and location of collapsed panels observed experimentally albeit with a confined debris spread due to inaccurate crack mechanisms which affected collapse and the generation of debris. Similar analyses at 20m stand-off demonstrated an increase in front panel deflection with zero collapse for the AEM prototype and model. AEM solutions were thus shown to qualitatively replicate the structural survival and partial damage of the experimental model versus partial collapse of the experimental prototype. Despite this variance, the AEM model solution was found to qualitatively replicate the AEM prototype. AEM models were thus shown to replicate the damage state of AEM prototypes at both stand-off distances, inferring that the underlying physics of the scaling methodology was correctly implemented within AEM's force-displacement methodology. Analysis of AEM debris for model and prototype solutions at 14.25m showed close agreement when assessing the relative percentages of total mass from the front, left and right panels with a mean difference of +4% for the model. A Chi-square goodness of fit test for the discrete debris distributions further confirmed this with a P-value of 20.6% versus a significance level of 5%.

The examination of AEM solutions via fully rigid or default 'flexible' corner restraints demonstrated variable potential for replicating experimental masonry response. Rigid restraints enabled primary cracking with an over-damping effect whilst 'flexible' corners enabled deflection, collapse and debris generation. The desired solution would thus integrate both capabilities with the correct sequential application to simulate masonry response. This may be achievable via the development of a 'semi-rigid' and removable corner restraint which provides the required stiffness and moment resistance to enable cracking without over-damping response to enable debris generation. AEM may thus be able to provide a predictive capacity after further investigation into this semi-rigid edge restraint.

Chapter 8 detailed four future experimental trials which have been designed to benchmark the masonry scaling methodology detailed in chapter 4 for long-duration blast events at a 1:2 scaling level, thereby fulfilling the fourth and final objective of this PhD. These trials will be conducted in the future at the UK national blast test facility, the Air Blast Tunnel (ABT) at MOD Shoeburyness with two pairs of prototype and model structures at two overpressures. Chapter 8 also presented a computational investigation into scaling performance within a long-duration blast scenario via preliminary CFD and AEM models. This began with the results of a comparative study which benchmarked Air3D and AUTODYN solutions against existing long-duration blast trial data. Whilst results showed comparable levels of accuracy within $\pm 10\%$ of free-field results, Air3D was found to produce quicker run times and as such was utilised for the remainder of this chapter's CFD modelling. Air3D solutions were subsequently produced to model the expected free-field and reflected blast trial environments and free-field impulse was shown to be within $\pm 10\%$ of predictions which met the acceptability criteria of $\pm 10\%$. Reflected pressure data was thus re-mapped to configure blast loading within AEM models.

Building upon the benchmarking in chapter 7, AEM models of masonry response to long-duration blast were conducted by implementing flexible corner restraints to enable debris generation. This was shown to be acceptable as the substantial increase in impulse within a long-duration blast infers a high likelihood of total failure and sizable projection of masonry fragments. As a result, long-duration trials may demonstrate that the influence of cracking is less dominant when the structure is subject to greater translational forces, inferring that the hierarchy of masonry response modes identified for short-duration blast

events may not influence debris generation in a long-duration blast. Examination of masonry damage for each AEM prototype and model structure at each stand-off demonstrated a 'squashing' failure mode, indicating that the models qualitatively replicated the prototype damage. These initial failure modes may not represent the expected experimental results due to insufficient corner stiffness to enable cracking.

Analysis of AEM debris at 55kPa showed the model distribution to be condensed in the X direction versus the prototype whilst the Y distribution closely matched the prototype. Each of the AEM debris distributions were found to exhibit Gaussian curve characteristics in the X and Y directions which enabled theoretical curve fits. Reasonable agreement was subsequently found within ± 1 and ± 2 standard deviations for percentage of total mass. Similar analyses were conducted for AEM debris at 110kPa where results showed the model X distribution to be condensed versus the prototype whilst the Y distribution closely matched the prototype. The model and prototype Y distributions at 110kPa both showed greater positive skew due to increased overpressure and impulse. Despite this, both the model and prototype distributions in X and Y showed Gaussian characteristics, enabling theoretical curve fits to demonstrate similar levels of agreement to the 55kPa results. Based on demonstrated levels of debris agreement, it was found that the model Y distributions may be able to directly represent the prototype whilst the development of transformation functions for model debris in the X direction may be required to match the prototype in X. As a result, it was found that future development of Gaussian prediction functions may be possible for debris in X and Y to enable the generation of masonry debris data in tandem with experimental and computational methods.

In summary, the principal aim of this PhD has been to develop and experimentally benchmark a new scaling methodology for unreinforced masonry structures subject to blast loading by utilising the concepts of dynamic similitude. This has been achieved via numerical derivation and a short-duration blast trial which revealed the following:

- Masonry structural scaling can be achieved by implementing a geometric similarity and a reciprocal scale factor for model structural density, enabling construction via commercially available materials at a 1:2 scale level.

- Qualitative characterisation of masonry damage via primary crack mode, crack location and subsequent degrees of collapse can be modelled by implementing the PhD scaling methodology at 1:2 for short-duration blast loading at two overpressure regions.
- Quantitative representation of spatial masonry debris distributions can be modelled within $\pm 10\%$ by implementing the PhD scaling methodology at 1:2 for short-duration blast loading at one overpressure region.

The secondary aim of this research was to examine and benchmark AEM and CFD models of masonry response and blast wave propagation, thus determining whether these methods can provide a future predictive capacity. This was achieved by producing comparative analyses with experimental data to reveal the following:

- CFD models were found to closely replicate free-field and reflected blast environments from the short-duration blast trial, enabling blast load re-maps into AEM simulations.
- AEM model and prototype solutions for masonry damage and debris showed close agreement, indicating that AEM correctly modelled the underlying physics of the PhD scaling methodology.
- AEM models of masonry damage state showed the need to develop a semi-rigid corner restraint condition to enable primary cracking and representative collapse modes which is fundamental to accurate debris generation in a short-duration blast.

Finally, this PhD's findings will enable the industrial sponsor, AWE, to conduct future scaled masonry trials at the national blast test facility, MOD Shoeburyness. The results of which will refine their fast running engineering algorithms (FREA) for predicting masonry response to blast.

9.2 Future work

This PhD has emphasised the simplest form masonry construction and as such, future work can investigate the scaling of masonry with a range structural bonds which integrate full-thickness construction, cavity wall designs and the inclusion of window apertures. This work can also integrate load-bearing roof structures. Each of these elements will likely

introduce additional masonry response complexities that will affect the damage and debris phases of masonry response. The quantification of which will enable future work to assess the performance of the scaling methodology with a wider spectrum of masonry designs.

Future investigations into short-duration blast loading will need to ensure that the masonry construction phase implements an equal degree of mortar bed fixity between the structure and the ground surface. This is essential to ensure equal cracking modes for prototype and model structures which can in turn influence the potential deflection, collapse and generation of debris for a masonry structure.

The non-responding timber elements fielded within chapters 5-6 highlighted the need for a more robust structural material to enable the measurement of reflected pressure. Future work could therefore include the modification of these structures to enable their implementation within short and long-duration blast environments, thus yielding valuable data relating to blast wave interaction.

The performance of this PhD's scaling methodology has not yet been examined in a high-impulse, long-duration blast scenario. The range of trials outlined in chapter 8 provide the ability to do this whilst building upon the experimental findings of the short-duration blast trial detailed in chapters 5-6. The conducting of these long-duration blast trials will enable damage state and debris distribution analyses to be conducted in the same manner demonstrated in chapter 6, thus assessing the applicability of the scaling approach for a range of blast environments.

As detailed in chapter 6, the short-duration blast trial conducted within this PhD identified the need to re-design accelerometer housing elements in order to capture meaningful data relating to equivalent brick and block motion. Based on the performance of the model element within chapter 6, it was shown that the manufacture of an alternative housing will require a modified outer surface material to enable sufficient adhesion to structural mortar and thus enable the element to be embedded within the masonry panel and replicate model unit behaviour.

Examination of the short-duration trial results also indicated the need to re-design the non-responding roof enclosure frames to prevent unwanted interactions with the masonry

structures. In particular, it will be essential to ensure that the structural stiffness of these enclosures is matched for prototype and model structures at the same overpressure region.

The successful completion of long-duration blast trials will also enable CFD and AEM models in chapter 8 to be compared with experimental data. Specifically, blast environment data from each of the trials will enable CFD models to be experimentally benchmarked further demonstrating the suitability of CFD for configuring masonry loading within AEM. This will also enable further investigations into the predictive capacity of CFD.

As discussed in chapter 7, this research has identified the need for future AEM work to develop a 'semi-rigid' and removable corner restraint methodology which provides the necessary stiffness and moment resistance to enable cracking in tandem with subsequent phases of collapse and debris without over-damping the system. The requirement for this methodology may be confined to short-duration blast events where the structural response time is far longer than the pressure duration. The development of this element will likely require a parametric study to determine whether the semi-rigid element should utilise constant material properties or varied properties as a function of blast energy or other parameters. This parametric study will also likely require benchmarking via a survey of available literature or lab testing. An additional area of future work could also include an investigation into other finite element techniques which enable the modelling of masonry damage and debris, thus enabling a comparative study with AEM solutions with the aim of selecting the most appropriate software for providing a predictive capacity.

10. References

- Adams, J.; Rose, T.A.; Garforth, R.; Evans, G.; Tate, J. 2012. Simulating explosive events in the Air Blast Tunnel, MABS 2012 conference, Bourges France, Crown Copyright.
- Adams, L.J.; Clubley, S.K.; Johns, R.V. (2014). Archived long-duration blast trial summary. AWE Aldermaston, United Kingdom, the Atomic Weapons Establishment.
- Anderson, J.D. (2002). Modern compressible flow with historical perspective, New York, USA, McGraw Hill.
- Anderson, J.D. (1995). Computational fluid dynamics – the basics with applications, New York, USA, McGraw Hill.
- Applied Research Associates Inc. (2004) 'A.T.-Blast, version 2.2'. Albuquerque, NM USA: Applied Research Associates, Inc. Available at: <http://www.ara.com/products/AT-blast.htm>.
- Applied Science International LLC. (2010). Applied Science International, Durham, North Carolina. ELS Theoretical Manual, 2010
- Arya, C. (2009). Design of Structural Elements, London, England, Spon Press.
- Baker, W. E. (1973). Explosions in air: University of Texas Press Austin.
- Brick.org.uk, UK. (2019). The design of laterally loaded walls, viewed: 05 September 2018, <<http://www.brick.org.uk/admin/resources/s-design-of-laterally-loaded-walls.pdf>>.
- Brode, H. L. (1954). Numerical solutions of spherical blast waves. Santa Monica, California, U.S.A: The RAND Corporation.

Chopra, A. (2000). Dynamics of structures: theory and applications to earthquake engineering, New York: Prentice-Hall.

CNN, USA. (2019). A week after the Easter Sunday attacks, Sri Lank is no closer to understanding or healing, viewed: 01 May 2019, <<http://edition.cnn.com/2019/04/27/asia/sri-lanka-easter-sunday-attacks-intl/index.html>>.

Cormie, D., Mays, G., Smith, P. (2009). Blast effects on buildings, London, England, ICE Publishing.

Dennis S, Baylot J, Woodson S. Response of 1/4-scale Concrete Masonry Unit (CMU) walls to blast. Journal of Engineering Mechanics 2002;128(2):134–42.

Dolan, P. J. (1972). Capabilities of Nuclear Weapons, Washington, DC, Defence Nuclear Agency.

Drougkas, A; Roca, P.; Molins, C. (2016). Experimental analysis and detailed micro-modeling of masonry walls subjected to in-plane shear. Engineering Failure Analysis, 95(2019), 82-95.

Foulness Island. (2019). Foulness Island. [online] Available at: <https://www.google.co.uk/maps/place/Foulness+Island,+Southend-on-Sea> [Accessed 1 May 2019].

Friedlander, F. G. (1946). The diffraction of sound pulse. I. Diffraction by a semi-infinite plate. Proceedings of the Royal Society of London A: Mathematical, Physical and Engineering Sciences, 186, 322-344.

Glasstone, S., Dolan, P.J. (1977). The effects of nuclear weapons, Washington, DC, United States Department of Defence and Energy Research and Development Administration.

Gohel, V., Patel, P. V. and Joshi, D. (2013) 'Analysis of frame using applied element method (AEM)', Procedia Engineering, 51(NUiCONE 2012), pp. 176–183.

Health and Safety Executive (2009) Buncefield Explosion Mechanism Phase 1, vols 1 and 2. Health and Safety Executive Research Report RR718.

Herbert, D.M; Gardner, D.R.; Harbottle, M.; Hughes, T.G. (2014). Uniform lateral load capacity of small-scale masonry wall panels. Materials and Structures, 47(2014), 805-818.

- Hirsch, C. (2007). Numerical computation of internal and external flows, Oxford, England, Butterworth-Heinemann.
- Iqbal, J. (2014) Containment evaluation against ground shock in rock. ICE Proceedings Structures and Buildings, 167, (SB2).
- Keys, R. (2016) Experimental & Computational Structural Analysis of Masonry Panels Subject to Long Duration Blast Loading. PhD Thesis. University of Southampton.
- Keys, R., Clubley, S. (2017). Establishing a predictive method for blast induced masonry debris distribution using experimental and numerical methods. Engineering Failure Analysis, 82(2017), 82-91.
- Kingery, C. N. and Bulmash, G. (1984). Air blast Parameters from TNT Spherical Air Burst and Hemispherical Surface Burst. Maryland, U.S.A: Ballistic Research Laboratory.
- Laefer, D.F; Hong, L.T.; Erkal, A.; Long, J.H.; Cording, E.J. (2011). Manufacturing, assembly, and testing of scaled, historic masonry for one-gravity, pseudo-static soil-structure experiments. Construction and Building Materials, 25(2011), 4362-4373.
- Larcher, M. (2008). Pressure-time functions for the description of air blast waves. Technical Report JRC46829, Joint Research Centre, Ispra, Italy, 2008.
- Lupoae, M. and Bucur, C. (2009) 'Building demolition - positive aspect of progressive collapse', MTA Review, XIX(4), pp. 399–408
- McKenzie, W.M.C. (2001). Design of Structural Masonry, New York, USA, Palgrave.
- Meguro. K. and Tagel-Din. H. (2000). Applied element method for structural analysis: theory and application for linear materials. Structural Eng./Earthquake Eng., International Journal of the Japan Society of Civil Engineers (JSCE), Vol. 17, No. 1, 21-35, April 2000
- Milani, G; Rossi, M.; Calderini, C; Lagomarsino, S. (2016). Tilting plane tests on a small-scale masonry cross vault: Experimental results and numerical simulations through a heterogeneous approach. Engineering Structures, 123(2016), 300-312.

Mohammed, A; Hughes, T.G.; Mustapha, A. (2011). The effect of scale on the structural behaviour of masonry under compression. *Construction and Building Materials*, 25(2011), 303-307.

Pelfrene, J., Kuntsche, J., Van Dam, S., Van Paepegem, W. and Schneider, J. (2016) 'Critical assessment of the post-breakage performance of blast loaded laminated glazing: Experiments and simulations', *International Journal of Impact Engineering*, 88, pp. 61–71.

Petry, S; Beyer, K. (2012) Testing unreinforced masonry structures at reduced scale. *Proceedings of the 15th World Conference on Earthquake Engineering*, Paper Number 1391.

Rankine, W. J. M. (1870). On the thermodynamic theory of waves of finite longitudinal disturbance. *Proceedings of the Royal Society of London*, 18, 80-83.

Rose, T A. (2001) an approach to the evaluation of blast loads on finite and semi-infinite structures. PhD thesis, Engineering Systems Department, Cranfield University, Royal Military College of Science, 2001

Rose, T A (2011) *Air3d Computational Fluid Dynamics Hydrocode*, Version 9, Cranfield University.

Simoens, B; Lefebvre, M. (2015). Influence of the Shape of an Explosive Charge: Quantification of the Modification of the Pressure Field. *Central European Journal of Energetic Materials*, 12(2015), 195-213.

Smith, P. D., Hetherington, J. G. (1994). *Blast and ballistic loading of structures*, Oxford, UK: Butterworth-Heinemann.

Szyska, M; Jerzy, J.; Ochsendorf, J.A. (2018). Out-of-plane behavior of two-layered free-standing masonry walls: Analytical solutions and small-scale tests. *Engineering Structures* 163(2018), 11-24.

Tagel-Din. H. (2009). High Fidelity Modelling of Building Collapse with Realistic Visualization of Resulting Damage and Debris Using the Applied Element Method. (HDTRA1-09-P-0006). North Carolina. Applied Science International. September 2009

The British Standards Institution. BS5628-1:2005 - Code of Practice for the use of Masonry, 2005.

The Telegraph, UK. (2015). Death toll rises after huge fire in Tianjin, viewed: 01 Sept 2016, <<http://www.telegraph.co.uk/news/picturegalleries/worldnews/11800173/Explosions-and-fireball-in-Tianjin-China-in-pictures.html>>.

Tomažević, M., Velechovsky, T. (1992) Some aspects of testing small-scale masonry building models on simple earthquake simulators. Earthquake engineering & structural dynamics, 21(1992), 945-963.

UFC 3-340-02. Structures to resist the effects of accidental explosions. United States of America: Department of Defense; 2008

Appendix A

CFD Sensitivity Study: Long-duration Blast

A.1 CFD Modelling

The secondary aim of this PhD is to assess the capability of computational methods to provide a predictive capacity for assessing masonry response to blast loading. CFD modelling represents vital component of this as it enables the simulation of dynamic blast environments and the generation of blast loading data for masonry structures. This data can be re-mapped into an AEM model of masonry response, thereby configuring the loading component of these simulations.

As a means of determining the most appropriate solver for long-duration blast scenarios, a comparative study was conducted to assess two of the most widely used CFD solvers available to model blast, namely Air3D and AUTODYN. This aimed to assess the relative accuracy of these CFD packages versus free-field and reflected blast data which was experimentally recorded during a series of previously conducted long-duration blast trials (Adams et. Al, 2014).

A.1.1 Comparative study overview

CFD analyses were conducted with Air3D and AUTODYN to model the free-field and reflected blast environments recorded in the 10.2m section of the UK Air Blast Tunnel (ABT) at MOD Shoeburyness during a series of eighteen long-duration blast trials conducted in 2014

(Adams et. Al, 2014). These trials utilised 0.55kg of helically wound Cordtex (PETN) in the ABT's explosive driver section to generate a blast event with TNT equivalence of 15 tonnes at 250m stand-off. The utilisation of pressure instrumentation in both the free-field and on a heavily armoured test structure subsequently yielded valuable pressure data to assess the accuracy of this study's CFD solutions. As a product of the careful execution of these trials, each was able to produce a blast environment of ~14kPa peak free-field overpressure with ~110ms duration within the acceptability criterion of +/- 10%. This enables the mean trial results to be utilised when assessing CFD solutions. These trials represent an ideal first phase of benchmarking as the structural dimensions of the test structure represent the upper limit of that which can be fielded in the ABT. This can therefore be utilised to represent a rigid masonry structure with equivalent dimensions.

CFD models were initially designed as 1D simulations within which gauges were located at 250m stand-offs to assess the accuracy of the free-field solutions. These solutions were subsequently re-mapped into 2D models where the cell size was varied to further assess and compare the accuracy of the CFD solutions. Each of these 2D models simulated the trial test structures by utilising non-responding void cells with equal dimensions. These were positioned on the y-axis centre line in 2D, enabling the solutions to utilise planar symmetry during the 1D re-map phase. Reflected pressure data was subsequently obtained by monitoring the rigid test-cubicle front surface in each model. Each CFD simulation was conducted using a dedicated quad-core Intel i7-3840QM 2.8GHz system with 16GB RAM.

A.1.2 Air3D vs AUTODYN

Initial attention focussed upon 1D models in the free-field by utilising the domain extents, cell size and solution duration shown in table A.1. AUTODYN models were executed utilising serial and parallel computing methodologies to assess the run-time advantages of maximising the available CPU cores. A cell size of 30mm was selected for both CFD solvers after calculating the most efficient cell size in Air3D with equation 5.4. Closer inspection of table A.1 reveals that Air3D required a larger 1D domain to ensure a 1000ms solution duration. This was not necessary in AUTODYN as the upper boundary was set to flow-out, enabling a smaller domain buffer in front of the 250m stand-off location.

Table A.1: CFD parameters for 1D solutions.

	1D Domain Extents (m)	Cell Size (mm)	Problem Duration (ms)
Air3D	0-415	30	1000
AUTODYN Serial	0-300	30	1000
AUTODYN Parallel	0-300	30	1000

Table A.2 compares Air3D and AUTODYN solutions for the free-field blast environment to mean results from the previously conducted experimental trials. Initial analysis shows the Air3D and AUTODYN results for free-field overpressure to be +5% and +8% larger than the trial result respectively, representing comparable levels of accuracy within +/- 10% of the trials. Similarly, the Air3D and AUTODYN results for positive phase duration can be seen to be -7% and -6% lower than the trial result respectively, further demonstrating comparable accuracy within +/- 10% of the trial data. Examination of the specific impulse results reveals the Air3D result to be -12% below the trial result versus -16% with the AUTODYN result. Figure A.1 compares an experimental pressure-time history with the CFD results where it can be seen that the CFD solutions have closely matched the exponential decay rate of the trial result up to the point of a smaller second peak at ~100ms on the time axis for the trial data. This is likely the root of the minor impulse discrepancies and may be the result of a small blast reflection from the rear doors of the ABT.

Table A.2: CFD free-field blast environment results versus ABT trials.

	P_s (kPa)	t^+ (ms)	i_s (kPa-ms)
Air3D	14.5	101	686
AUTODYN	14.9	102	661
Trial ($\bar{X}$)	13.8	109	788

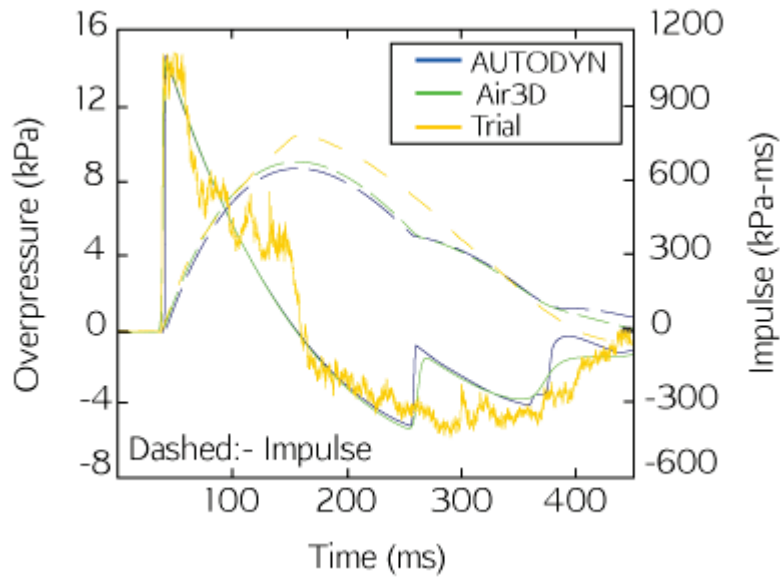


Figure A.1: CFD free-field overpressure & impulse comparison with ABT trial.

Examination of table A.3 reveals the computational overhead of each CFD solution with the final column detailing the individual run times. It is evident that the Air3D time of 4 minutes is substantially lower with the AUTODYN serial and parallel models requiring 4000% and 2775% increases to execute respectively. Closer inspection also demonstrates a substantially lower memory requirement for Air3D with the AUTODYN serial and parallel models requiring a 7400% and 5550% increase in memory respectively. Thereby demonstrating a significantly lower memory overhead per cell in Air3D despite this solution requiring 3849 more cells to accommodate the larger domain. Table A.2 also demonstrates a run time reduction of 30% when utilising AUTODYN with parallel execution versus serial. The parallel model can also be seen to require 25% less memory than the serial solution, indicating the increased efficiency to be a result of the sizable jump in CPU utilisation from 20% to 85% over all 4 of the CPU cores.

Table A.3: Computational overhead summary for CFD solutions in 1D.

	Cells	Memory (MB)	CPU (%)	Run Time (mins)
Air3D	13.849E+03	4	19	4
AUTODYN Serial	10.0E+03	300	20	164
AUTODYN Parallel	10.0E+03	226	85	115

Table A.4 details the domain extents, cell size and solution durations utilised to conduct 2D models of the reflected blast environment. Having demonstrated an appreciable gain in run-time efficiency in table A.3, all proceeding AUTODYN models were executed in parallel. Closer inspection of table A.4 reveals that AUTODYN required a 10m larger 1D domain in order to prevent boundary interference within the re-map file. This was not necessary in Air3D as the 1D solution automatically re-maps at the instant when the blast wave reaches the upper boundary. Each of the models was configured to simulate a 700ms total run time to fully accommodate the positive phase of the reflected blast wave.

The 2D component of the AUTODYN model utilised a graded mesh in an attempt to optimise the run time efficiency. This is illustrated in figure A.2a where it can be seen that the centre line of the Y axis utilised 10m of 100mm cells with the lower X axis utilising 70m of 100mm cells. In contrast, figure A.2b shows that the Air3D model utilised 100mm cells throughout the domain. Further inspection of figures A.2a-b also reveals that each of the models utilised flow out boundaries at the upper X, lower Y and upper Y positions. These were selected to permit planar symmetry in each solution, thereby enabling a planar blast wave to interact with the test-structures as seen in figures A.3 and A.4. It can also be seen in figures A.2a-c that the test-structures were positioned on the centre line of the Y-axis to simulate the location utilised within the trials.

Table A.4: CFD parameters for 2D solutions with 100mm cell size.

	Domain Extents		Cell Size (mm)	Model Duration (ms)
	X Direction (m)	Y Direction (m)		
Air3D - 1D	0-240	-	30	515
Air3D - 2D	0-300	0-30	100	185
AUTODYN Parallel - 1D	0-250	-	30	515
AUTODYN Parallel - 2D	0-300	0-30	100 (graded)	185

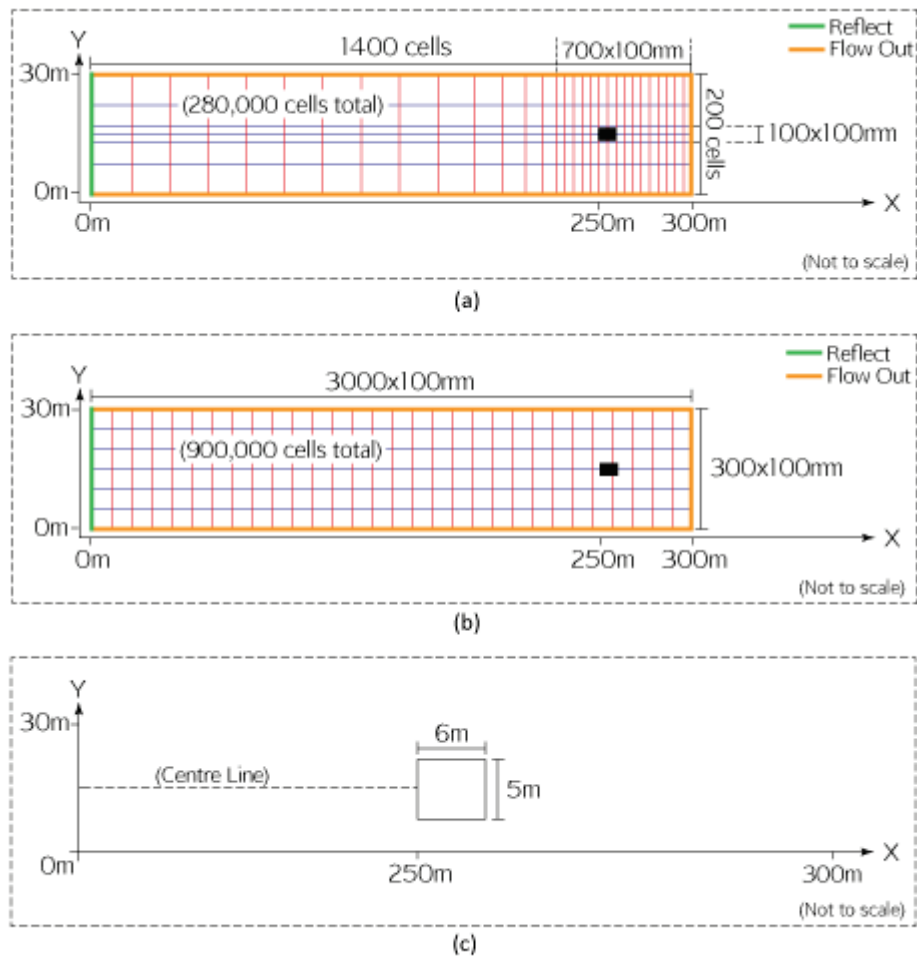


Figure A.2: CFD domain discretization schematic in 2D at 100mm: (a) AUTODYN X-Y; (b) Air3D X-Y; (c) Test structure dimensions.

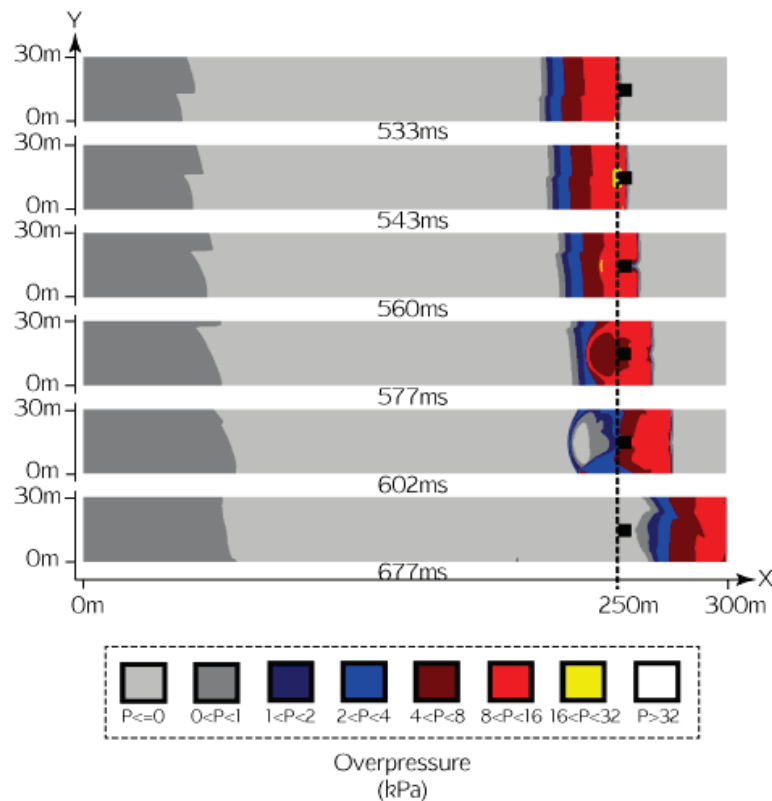


Figure A.3: Air3D simulation of blast-structure interaction in 2D.

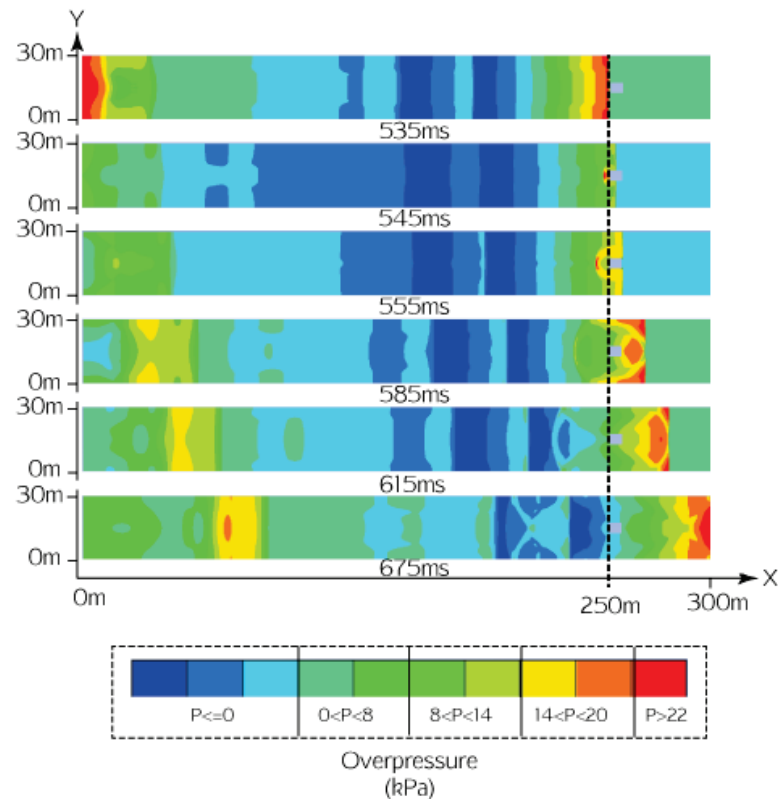


Figure A.4: AUTODYN simulation of blast-structure interaction in 2D.

Table A.5 compares Air3D and AUTODYN solutions for the reflected blast environment to mean trial results. Initial inspection shows the Air3D result for reflected overpressure to be +2% greater than the trial result versus +11% in AUTODYN. Examination of the positive phase duration results reveals Air3D and AUTODYN to be -15% and -13% shorter than the trials respectively. Further analysis reveals the Air3D result for specific impulse to be -8% below the trial result versus +0.3% in AUTODYN.

Table A.5: CFD reflected blast environment results, 100mm cell size versus ABT trials.

	P_r (kPa)	t^+ (ms)	i_r (kPa-ms)
Air3D	31.2	98.0	823
AUTODYN	34.2	101	897
Trial ($\bar{X}$)	30.7	116	894

Figure A.5 compares an experimental pressure-time history with the CFD results where it can be seen that the CFD solutions appear to decay at a slower rate than the trial result. It is possible that the utilisation of a 2D analysis may introduce minor differences in the effect of blast wave clearing which was shown in chapter 2 to be a function of the blast

environment and the structural geometry. This indicates that the planar analysis conducted in 2D may affect the accuracy of the blast clearing solutions in CFD as they do not consider the finite height of the test structures. It must be noted however that the CFD results demonstrate comparable levels of accuracy with both sets of data lying within $\pm 15\%$ of the trial results.

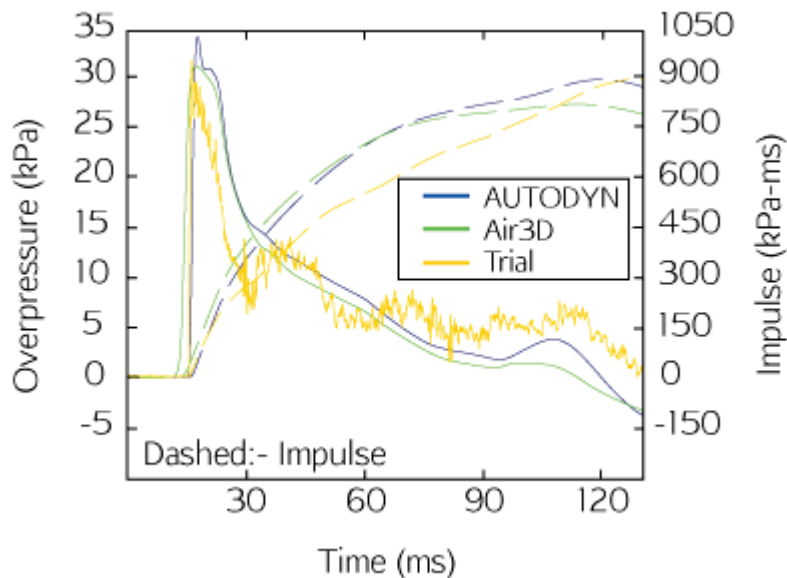


Figure A.5: CFD reflected overpressure & impulse at 100mm versus ABT trial.

Examination of table A.6 reveals the computational overhead of the two dimensional CFD solutions where it is evident that the Air3D run time of 40 minutes is once again lower than the AUTODYN result of 148 minutes. This disparity is however reduced versus the 1D results where AUTODYN required a 2775% increase versus +270% in 2D. Further analysis also demonstrates a substantially lower memory requirement in Air3D with the AUTODYN model requiring a 1510% increase in memory. This underscores a substantially lower memory overhead per cell in Air3D despite the solution requiring $\sim 620 \times 10^3$ more cells to accommodate the inability to grade the mesh. Table A.5 also shows a higher CPU utilisation for the AUTODYN model with an average value of 85% versus 20% in 1D and 60% in 2D for Air3D.

Table A.6: Computational overhead summary for CFD solutions in 2D, 100mm cell size.

	Cells	Memory (MB)	CPU (%)	Run Time (mins)
Air3D - 1D	8.0E+03	3.1	19	1
Air3D - 2D	900.0E+03	33	60	39
Total				40
AUTODYN Parallel - 1D	8.333E+03	221	85	47
AUTODYN Parallel - 2D	280.0E+03	360	85	101
Total				148

As a means of quantifying the influence of cell size on 2D solution accuracy, additional simulations were designed to utilise 50mm cells. Unusually however, this process uncovered a cell count limitation within Air3D as demonstrated in table A.7. This demonstrates a 3800 cell limit in either the X or Y directions, beyond which the solver will encounter a stack overflow error. This was verified on two additional systems with the first utilising a quad-core Intel i5-3470 3.2GHz with 8GB RAM and the second with a dual-core Intel i5-3320M with 8GB RAM. This cell threshold was established by initially determining the domain extent limit for 100mm, 50mm and 25mm cell sizes by trial and error. It was then possible to determine that each model was restricted to a total of 3800 cells in either direction before encountering the error, inferring a maximum number of cells in 2D simulation of 14.44×10^6 or 3800^2 . This limit indicates that the finest cell size available for a 300m domain in the X direction within Air3D was approximately 80mm.

Table A.7: Air3D domain extent limits in 2D.

Cell Size (mm)	10.0	20.0	30.0	40.0	50.0	60.0	70.0	80.0	90.0	100.0
Extent Limit (m)	38.0	76	114	152	190	228	266	304	342	380
Number of Cells	3800	3800	3800	3800	3800	3800	3800	3800	3800	3800

In order to circumvent the 2D cell count limitation with Air3D, a 3D model was designed to represent a 'pseudo 2D' scenario. This was specified to run in 3D with the minimum possible domain extents in the Z direction, namely twice the cell size. Thereby simulating an exceptionally thin 3D domain which is approximately analogous to the plane

utilised in 2D. As a by-product of utilising a 3D solution, it was also possible to specify a smaller X domain in absolute terms as the lower X boundary could be specified above 0m. This is demonstrated in table A.8 where the X domain extents were set to cover 190-300m, representing 110m in total versus the 300m required in 2D. The remaining parameters were chosen to replicate the 100mm cell size models shown in table A.4.

Table A.8: CFD parameters for 2D solutions with 50mm cell size.

	Domain Extents		Cell Size (mm)	Model Duration (ms)
	X Direction (m)	Y Direction (m)		
Air3D - 1D	0-240	-	30	515
Air3D - 2D	190-300	0-30	50	185
AUTODYN Parallel - 1D	0-250	-	30	515
AUTODYN Parallel - 2D	0-300	0-30	50 (graded)	185

Table A.8 also shows that the 2D AUTODYN model utilised a graded mesh to optimise run time efficiency. This is illustrated in figure A.6a where it can be seen that the centre line of the Y axis utilised 10m of 50mm cells and that the lower X axis utilised 70m of 50mm cells. In contrast, figure A.6b demonstrates that the Air3D setup utilised 50mm cells throughout the domain. Figure A.6a also reveals that the AUTODYN model utilised flow out boundaries at the upper X, lower Y and upper Y positions to simulate a planar wave. In contrast, the Air3D model in figure A.6b was set to flow-out at each boundary to account for the removal of the reflecting ground surface which would have been at 0m in the X direction. It can also be seen in figures A.6c-d that the Z domain extent was set to twice the cell size in the Air3D model. Consequently, the test structure was also set to span the full extent of Z domain, thereby replicating a planar object with infinite height as represented in 2D. The Air3D gauge monitoring location was subsequently positioned at the mid-height of the Z domain directly in front of the test structure in an attempt to minimise interference from the lower and upper Z boundaries.

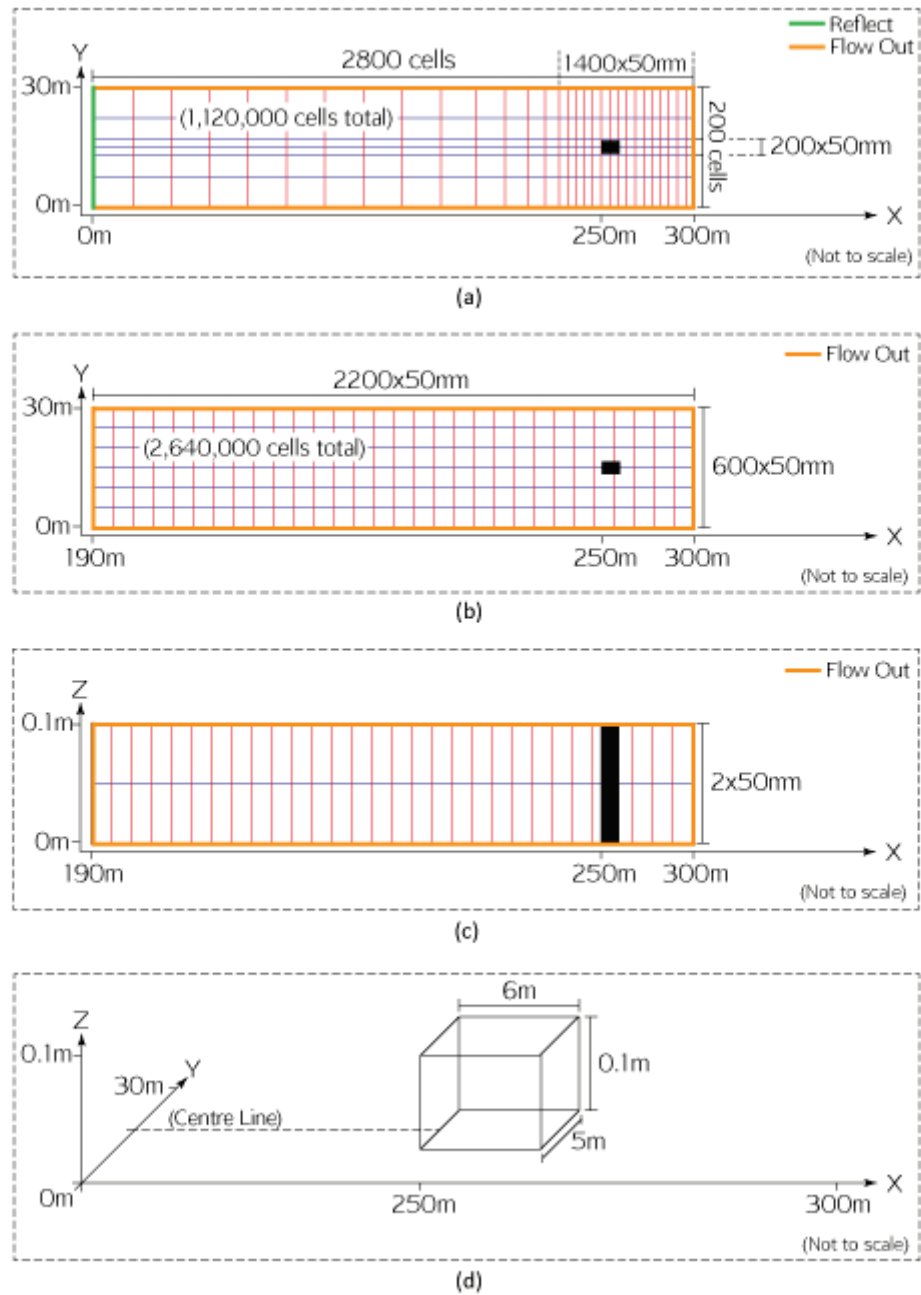


Figure A.6: CFD domain discretization schematic in 2D at 50mm: (a) AUTODYN X-Y; (b) Air3D X-Y; (c) Air3D X-Z; (d) Air3D test structure dimensions.

Table A.9 compares CFD solutions with the mean trial data from the reflected blast environment. Initial examination reveals the Air3D result for reflected overpressure to be +7% greater than the trial result versus +8% in AUTODYN. Further analysis shows the Air3D and AUTODYN results for positive phase duration to be -22% and -14% shorter than the trials respectively. Examination of the specific impulse results reveals the Air3D solution to be -5% below the trial result versus -10% in AUTODYN.

Table A.9: CFD reflected blast environment results, 50mm cell size versus ABT trials.

	P_r (kPa)	t^+ (ms)	i_r (kPa-ms)
Air3D	32.7	90	851
AUTODYN	33.3	100	802
Trial ($\bar{X}$)	30.7	116	894

Comparisons of the 2D results at 50mm cell size versus those at 100mm shows the Air3D model to widen its disparity with the mean trial result for reflected overpressure from +2% to +7%. Similarly, positive phase duration results in Air3D can be seen to reduce in accuracy from -14% to -22%. In contrast, the Air3D result for specific impulse can be seen to improve from -8% to -5% versus the trial. Similar analyses for the AUTODYN models at 50mm and 100mm cell sizes indicates an improvement in accuracy for reflected overpressure with the trial disparity reduced from +11% to +8%. In contrast, examination of the positive phase duration results reveals a minor in accuracy from -13% to -14% of the trial result. The largest difference is however visible with the specific impulse results with a loss of accuracy from +0.3% to -10%.

Figure A.7 compares an experimental pressure-time history with the CFD results where it can be seen that the CFD solutions decay at a slower rate than the trial result, resulting in a greater application of impulse for the majority of the positive phase. As stated previously however, it is possible that these differences may be due to the utilisation of a 2D analysis to model the effect of blast wave clearing around a 3D object with finite height. Closer examination of the AUTODYN result for impulse shows a lower amplitude than Air3D throughout the duration of the positive phase. This contrasts with figure 5.5 where the two CFD solutions can be seen to correlate more closely for the first ~50ms of the positive phase. It should be noted however that the reliability of the AUTODYN solution may have been affected by the non-conservation of energy. Consequently, the energy loss may have been amplified over a longer run-time versus the 100mm cell size model.

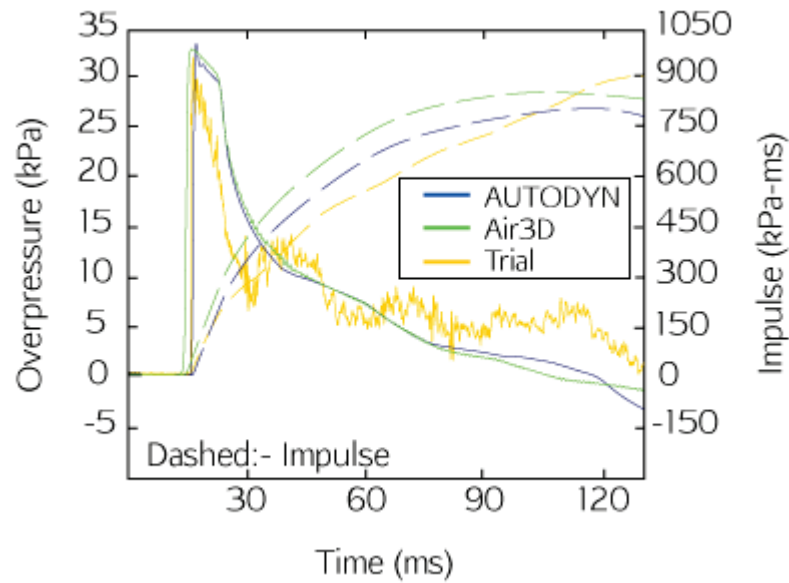


Figure A.7: CFD reflected overpressure & impulse at 50mm versus ABT trials.

Table A.10 details the computational overhead of the CFD solutions with 50mm cell size. Initial inspection reveals a sizably lower run time of 121 minutes in Air3D versus 840 minutes in AUTODYN. This represents an increase of 594% in AUTODYN which is approximately double the relative disparity seen with the 100mm cell size in 2D. Table A.10 also shows a lower memory requirement in Air3D with the AUTODYN model requiring an additional 303% increase in memory. This further demonstrates a lower memory overhead per cell in Air3D despite the additional $\sim 1.52 \times 10^6$ cells to accommodate the third dimension in the pseudo 2D scenario. Table A.10 also shows a higher CPU utilisation for the AUTODYN model with an average value of 85% versus 19% in 1D and 63% in pseudo 2D for Air3D.

Table A.10: Computational overhead summary for CFD solutions in 2D, 50mm cell size.

	Cells	Memory (MB)	CPU (%)	Run Time (mins)
Air3D - 1D	8.0E+03	3.1	19	1
Air3D - 2D	2.64E+06	230	63	120
Total				121
AUTODYN Parallel - 1D	8.333E+03	221	85	47
AUTODYN Parallel - 2D	1.12E+06	833	85	793
Total				840

An unexpected side-effect of utilising 85% CPU for 840 minutes to execute the AUTODYN model in parallel was the near-overheating of the CPU cores. These were

monitored individually to reveal mean temperatures of $\sim 95^{\circ}\text{C}$ throughout. Whilst the AUTODYN literature indicates a larger gain in efficiency for parallel solutions versus serial as the problem size increases, it may not be possible to run these due to possibility of irreversibly damaging the CPU hardware. Consequently, this indicates that AUTODYN may not be suitable to model long-duration blasts in 3D as these will require substantially more cells and therefore greater execution times. An alternative approach may however be the utilisation of a serial execution in 3D to reduce CPU overhead, but this is likely to incur a severe penalty in run time.

As a means of further quantifying the influence of cell size on the Air3D solution accuracy in 'pseudo 2D', an additional simulation was designed to utilise 25mm cells in 3D. This revealed a similar cell count limitation as demonstrated previously for 2D. This threshold was established by initially determining the extent limit for 25mm and 50mm cell sizes by trial and error with the results detailed in table A.11. It was then possible to determine that each model was restricted to a total of 3000 cells in each direction in 3D before encountering the stack overflow error. Thereby indicating that the maximum number of cells in a 3D simulation is 27×10^9 or 3000^3 .

Table A.11: Air3D domain extent limits in 3D.

Cell Size (mm)	25	50
Domain Limit in X (m)	75	150
Number of Cells	3000	3000

Consequently, it can be seen that the 25mm cell size requires a reduction of the X domain extents in absolute terms to 75m. This limit is less than twice the notional blast wave 'thickness' of $\sim 40\text{m}$ at the 250m stand-off, indicating insufficient space in the X direction to accommodate the blast wave on either side of the test structure. Which may prevent an accurate model of blast clearing. This limitation therefore highlights the need to calculate the blast thickness as a means of determining the minimum acceptable cell size to accommodate blast wave clearing around a structural geometry. This can be accomplished by utilising the

expression in equation A.1 which multiplies positive phase duration by the shock front velocity at the stand-off of interest.

$$\text{blast thickness} = U_s \cdot t^+ \quad (\text{A.1})$$

Where:

U_s = Shock front velocity (m/ms)

t^+ = Positive phase duration (ms)

This comparative study has focussed on the blast environment recorded in the 10.2m section of the UK Air Blast Tunnel (ABT) during a series of previously conducted trials for 15 tonnes of TNT at 250m stand-off (Adams et. Al, 2014). Future experimentation within the ABT will however emphasise its capabilities at full power within the 4.9m diameter sections. The narrower of these two represents a power envelope with 450t TNT equivalence at 235m stand-off to produce a 110kPa free-field overpressure and a blast thickness of ~106m when utilising the expression above. The utilisation of this section at 50% power can also be shown to represent a blast event with 200t of TNT equivalence at 250m stand-off which results in 55kPa free-field overpressure and blast thickness of ~85m.

These results indicate that the events would require at least ~212m and ~170m respectively to accommodate the blast wave on either side of a structure. This value may however increase as the overall X domain extent value will also be a function of the structural geometry in the X direction. Utilising these provisional values with the 3000 cell limit, it is possible to derive the smallest possible cell sizes of ~70mm and ~55mm for these two scenarios respectively.

A.2 Summary

CFD models were produced within Air3D and AUTODYN to replicate the free-field and reflected blast environments recorded in the 10.2m diameter section of the UK Air Blast Tunnel (ABT) during a series of previously conducted long-duration blast trials. 1D simulations revealed similar levels of overall accuracy within +/-10% of the trial results in the free-field. Air3D was found however to possess a significant speed advantage in terms of solution

execution versus AUTODYN in both serial and parallel modes. 2D simulations were subsequently produced to model the reflected blast environment of the ABT trials. CFD results were once again found to represent comparable levels of overall accuracy within $\pm 15\%$ of the trial data. The utilisation of a 2D analysis to model blast clearing around a finite structure may however limit the ability to improve this accuracy level. Computational overhead comparisons in 2D revealed Air3D to possess a significant speed advantage versus AUTODYN whilst also utilising a lower overall memory overhead per cell.

Additional 2D analyses were also conducted with a finer mesh resolution to reveal similar levels of overall accuracy within $\pm 22\%$ of reflected trial data. The design phase of the 2D simulations revealed a 3800 cell count limitation within Air3D in any direction, restricting the ability to conduct finer resolution models at large stand-offs. 3D models were subsequently produced with the minimum Z domain extents to closely replicate a 2D analysis. These solutions produced comparable levels of accuracy with the AUTODYN models in true 2D. Analysis of the computational overhead results also showed a significant speed advantage for Air3D whilst AUTODYN was found to introduce the potential of CPU overheating when utilising parallel execution for long run times.

Finally, additional attempts to utilise a finer mesh resolution in the pseudo 2D state for Air3D revealed a cell count limitation of 3000 in each direction in 3D. This demonstrated the need to calculate the notional blast wave thickness to ensure adequate space to accommodate a blast wave on either side of a structure in 3D model. Indicating a limit on the finest possible cell size as a function of blast wave thickness.

Appendix B

Journal accepted by Engineering Structures, 2019

The abstract shown below outlines the journal paper which has been accepted by Engineering Structures. The current version of the full paper is unformatted as per the journal's submission guidelines. The final version of the paper is currently with the editing department of the journal prior to its full, formatted release. This paper can be accessed online via <https://doi.org/10.1016/j.engstruct.2019.109727>

Investigating the Scaling of Masonry Structures in a Blast Environment

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Abstract

Full-scale experimental testing of masonry response to blast can be challenging with the requisite need for significant measurement area in conjunction with high construction and material costs. This paper investigates the use of dynamic similitude to produce reduced-scale masonry structures which model the damage state and debris distribution of a full-scale counterpart due to blast loading. An investigation into the fundamental physical components of masonry response to blast loading facilitated the development of a new scaling methodology which maintains the ratio of lateral and vertical force components as prescribed by dynamic similitude. It is shown that this can be accomplished by using a reciprocal scale factor for the reduced-scale structure's density. Computational models of full and reduced-scale masonry response to blast loading were produced with the Applied Element Method (AEM) to verify the underlying theory of the proposed scaling methodology. These utilised single-storey cuboid structures with non-responding roofs and half-thickness stretcher bond construction. Computational Fluid Dynamics (CFD) models of blast wave interaction with the masonry structures were defined at a range of peak overpressures, enabling a remap procedure into AEM. Importantly, AEM models utilised a constant 1:2 scale factor with masonry material parameters for commercially available units, demonstrating the practical applications of this scaling methodology for blast trials. Analysis of the AEM models demonstrated close qualitative agreement in damage state for full and reduced-scale structures at 55kPa and 110kPa peak free-field overpressure. These results also indicated agreement for a range of failure modes with the 110kPa model showing front-panel collapse versus the 55kPa model which indicated partial front-panel deflection. Chi-square analysis of the resultant debris distribution at 110kPa indicated quantitative agreement for the relative quantities of bricks found within the rubble pile as a function of their original panel wall location.

Keywords: Masonry, Structures, Blast, Scaling, Dynamic Similitude, Damage, Rubble

